UNIVERSITY OF MINNESOTA
ST. ANTHONY FALLS HYDRAULIC LABORATORY
LORENZ G. STRAUB, Director

Technical Paper No. 12, Series B

Hydraulics of Closed Conduit Spillways

Part 1. Theory and Its Application

by

Fred W. Blaisdell, Hydraulic Engineer
USDA, ARS

January 1952
Revised February 1958

Prepared by

UNITED STATES DEPARTMENT OF AGRICULTURE
AGRICULTURAL RESEARCH SERVICE
SOIL AND WATER CONSERVATION RESEARCH DIVISION

in cooperation with the

Minnesota Agricultural Experiment Station
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ABSTRACT

The closed conduit spillway is any conduit having a closed cross section through which water is spilled. The inlet and outlet may be of any type. The barrel may be of any size or shape and may flow either full or partly full. Also, the barrel may be on any slope. This broad definition includes the smallest culvert as well as the largest morning glory spillway.

The basic theory of the flow is the same for each of the many forms which the spillway may take. This paper discusses the control of the flow through closed conduit spillways by weirs, the barrel exit, tailwater, pipe, orifice, and short tube, since each of these controls may govern, at some time or other, the rate of flow through the spillway. The effect of these various controls on the performance of the spillway is explained. A means of developing a composite head-discharge curve is given. Pressures within the closed conduit spillway must sometimes be determined, so the methods for this determination are presented. A selected bibliography useful to the understanding and for the design of closed conduit spillways concludes this technical paper.

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NOMENCLATURE

A area, in square feet

B length of rectangular drop inlet, in feet

B' effective length of rectangular drop inlet crest, in feet

C discharge coefficient in Eq. I-1 and I-2, in English units

C_o discharge coefficient in Eq. I-7

C_s discharge coefficient in Eq. I-8

d_c critical depth, in feet

D conduit diameter, in feet

D_r drop inlet (riser) diameter, in feet

D_{rc} effective diameter of drop inlet crest, in feet

f Darcy-Weisbach friction factor

g acceleration due to gravity, in feet per second per second

h_f friction head loss, in feet

h_n local pressure deviation from the friction grade line, in feet

h_p pressure head = p/w, in feet

h_{vp} velocity head in conduit = $V_p^2/2g$, in feet

H head on crest, in feet

H_o head on orifice, in feet

H_s head on short tube, in feet

H_t total head from water surface to point at which the hydraulic grade line pierces the plane of the outlet = $H + Z + (\frac{D}{2} - \beta D) \cos^{-1}(\sin S)$, or to the tailwater surface, in feet

K_e entrance loss coefficient

K_o outlet loss coefficient

ℓ length of conduit, in feet

L crest length, in feet

n Manning roughness coefficient, in $(\text{feet})^{1/6}$

N number of observations

o subscript ordinarily denoting a reference point but also used in connection with the inlet orifice and the outlet

p pressure, in pounds per square foot

q discharge per foot of width, in cubic feet per second

Q discharge, in cubic feet per second

r subscript denoting riser or (drop) inlet

R hydraulic radius = area ÷ wetted perimeter, in feet

S conduit slope (sine)

S' slope of dam face (cotangent: one vertical on S' horizontal)

t thickness of anti-vortex wall, in feet

V velocity at any point, in feet per second

V_p velocity in conduit, in feet per second

w unit weight of water = 62.4 pounds per cubic foot

W width of rectangular drop inlet, in feet

W' effective width of rectangular drop inlet crest, in feet

z elevation, in feet

Z difference in elevation between inlet crest or conduit invert at inlet and centerline of outlet, in feet

Z_1 height of drop inlet, crest to invert of conduit at entrance, in feet

β ratio of the distance above the invert at which the projected hydraulic grade line pierces the plane of the conduit exit to the conduit diameter

HYDRAULICS OF CLOSED CONDUIT SPILLWAYS

Part I

Theory and Its Application*

INTRODUCTION

The theory of the hydraulics of closed conduit spillways and a method of applying the theory are presented in this paper. Although methods for the design of the closed conduit spillway are explained, this paper is not complete by itself since the required quantitative values of the constants are not given. This outline of the theory is intended to be Part I of a series. Subsequent parts will present information on the hydraulic characteristics for the various forms which the spillway may take. In addition, loss coefficients and pressure coefficients will be given for the use of the designer.

Closed conduit spillway is a general term that covers many specific types of spillways. In the Soil Conservation Service it is known as a "drop inlet culvert," "drop inlet spillway with pipe conduit (pipe drop inlet spillway)," "trickle tube," "pipe outlet," "tube outlet," "tile outlet," etc. The ordinary highway culvert is a closed conduit spillway. In large sizes the closed conduit spillway is known as a "morning glory," "bell mouth," "vertical shaft," etc., spillway. The closed conduit spillway assumes the characteristics of a siphon spillway when the spillway operates with pressures that are less than atmospheric. As used here, a closed conduit spillway is taken to mean a pipe or box culvert including the entrance and outlet. The barrel may be on either a flat or a steep slope. The inlet and outlet may be of any type. The outlet may be submerged or may discharge freely into the atmosphere. The conduit may be designed to flow either partly full or full.

The analysis described here is the product of an investigation conducted by the staff of the Soil and Water Conservation Research Division of the Agricultural Research Service, U.S. Department of Agriculture, located at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis. There the Agricultural Research Service, the Minnesota Agricultural Experiment Station, and the St. Anthony Falls Hydraulic Laboratory cooperate in the solution of problems concerning conservation hydraulics. This study was originally made under the immediate direction of Lewis A. Jones, Chief, Division of Drainage and Water Control, Soil Conservation Service Research, U. S. Department of Agriculture. Mr. Jones has now retired. The study is continuing under the direction of Austin W. Zingg, Chief, Watershed Technology Research Branch, and Dr. C. H. Wadleigh, Director, Soil and Water Conservation Research Division, Agricultural Research Service. Thanks are due Robert V. Keppel for critically reviewing this paper.

CAPACITY OF THE SPILLWAY

A number of different equations are used to compute the flow through closed conduit spillways. That part of the spillway controlling the head-discharge relationship may be a weir at the conduit entrance, an orifice at the conduit or the barrel entrance, a section of the conduit acting as a short tube, or the conduit flowing as a full pipe. These flow conditions are sketched in Fig. I-1. It may be necessary to use four types of equations to determine the capacity of some forms of closed conduit spillways, but ordinarily the weir and pipe equations will suffice. No attempt will be made here to explain which equations to use for any particular spillway design. This will be left for the papers describing the performance of each form which the spillway takes. Here the various equations that may apply will be described and their use will be outlined.

Weir Flow

The first flow that passes through the spillway will be controlled by a weir at the entrance, as in Fig. I-1a. For the weir to exercise the control over the head-discharge relationship, the flow depth must be greater than the critical depth upstream of the weir and must decrease through the critical depth in the vicinity of the weir. This will be taken as a definition of "weir control."

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*Agricultural Research Service Report No. 41-505-48.

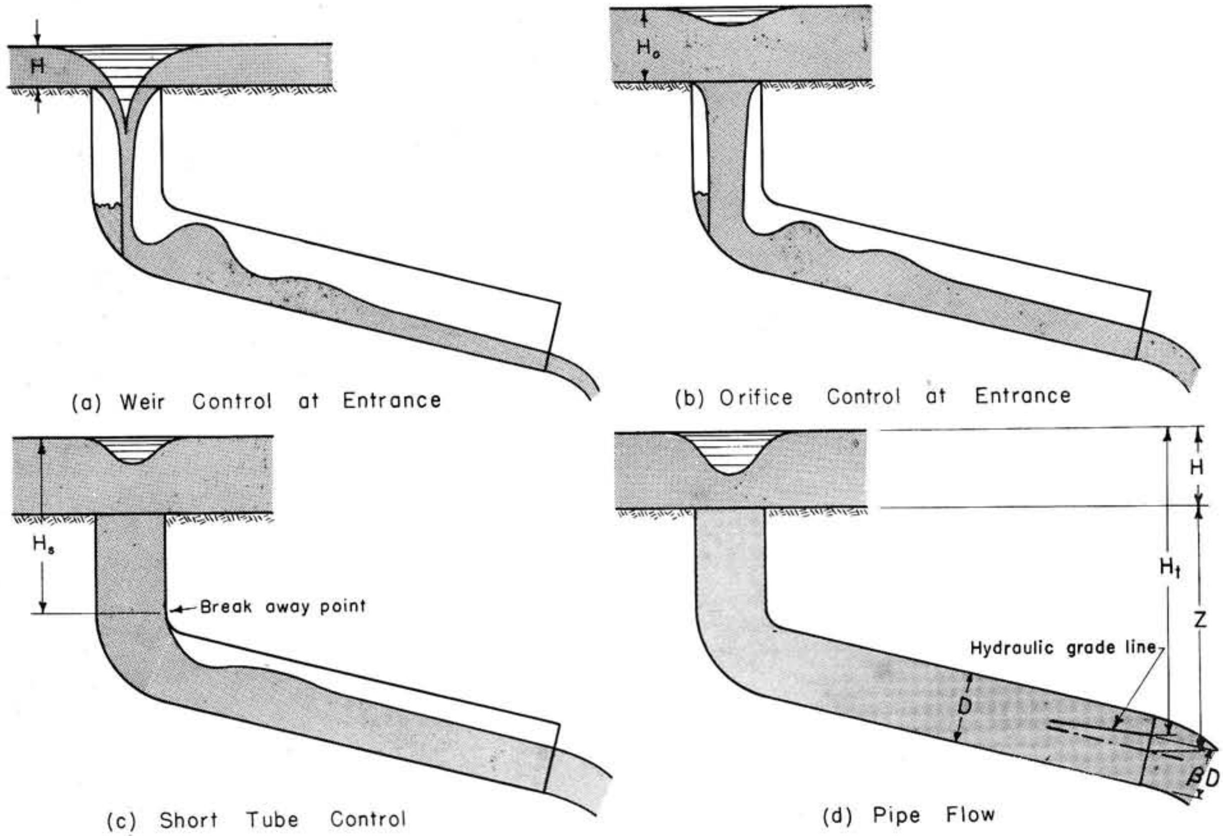

Fig. I-1 - Types of Flow Control

The weir at the entrance to the spillway may take any one of a number of forms. The entrance to a box culvert may form a rectangular weir. The entrance to a pipe may form a circular weir. A drop inlet crest may be horizontal and form a weir, either rectangular or circular in plan. Although the weir may take other forms, the discussion here will be limited to the three types mentioned, these being the types that are most commonly used.

Rectangular Weir

The general equation for flow over a rectangular weir is

$$Q = C L H^{3/2} \tag{I-1}$$

where Q is the discharge in cubic feet per second, C is the coefficient of discharge, L is the length of the crest in feet, and H is the head on the crest in feet. The symbols H and L are also defined in Figs. I-1a and I-2. In Fig. I-2 critical depth must occur near the culvert entrance if Eq. I-1 is used to determine the discharge.

It may be more convenient in analyzing the data and easier to apply the results in certain instances if Eq. I-1 is presented in a semi-dimensionless form; the equation is written

$$\frac{Q}{D^{5/2}} = C \frac{L}{D} \left(\frac{H}{D}\right)^{3/2} \tag{I-2}$$

where D is the conduit diameter. To convert to the form of Eq. I-1, it is necessary only, for any given conduit size, to substitute a numerical value for D in Eq. I-2.

Eq. I-2 is valid for any spillway which is geometrically similar to the spillway for which C is evaluated, no matter what the size of the spillway may be. The use of D restricts Eq. I-1 to geometrically similar circular conduit (pipe) spillways. However, Eq. I-2 can be used for other conduit shapes if D is replaced by some dimension which characterizes the proportions of the spillway. The advantage of Eq. I-2 is that because it acts like a dimensionless expression, one determination of C permits the computation of the discharge for any number of spillways that are geometrically similar in form.

The discharge coefficient C varies from about 2.5 to 5.7. Things which affect C are the velocity of approach, the elevation of the weir crest above the approach channel, the width of the weir crest, etc. It is therefore imperative that the value of C used in Eq. I-1 be carefully selected and that the selection be based on tests made as closely similar as possible to those which exist at the weir under consideration. Reference can be made to a number of standard texts to determine C. A few of these texts are listed in the Bibliography.

Circular Weir

The circular weir is mentioned here, not because the weir itself is used as the entrance to a closed conduit spillway, but because the entrance to a pipe is circular.

The theoretical equation for the discharge of a circular weir is quite complicated and no attempt will be made to present it here. If it were presented, it would be necessary to vary the discharge coefficient with the approach and exit conditions and with the form of the weir lip itself. From a practical standpoint, then, it is preferable to present rating curves for the different conditions which it is anticipated will arise. These curves can be presented--as Mavis [I-37],* Manohar [I-36], Straub, Anderson, and Bowers [I-50], and the author in Part X have done--in units which are independent of the pipe size; the coordinates H/D and $Q/D^{5/2}$ will permit the determination of the head-discharge curve for similar conditions no matter what the actual size of the pipe may be. Here H is the head on the pipe invert in feet and D is the pipe diameter in feet. Curves of H/D versus $Q/D^{5/2}$ will be included in the papers describing the results of tests on various types of inlet structures.

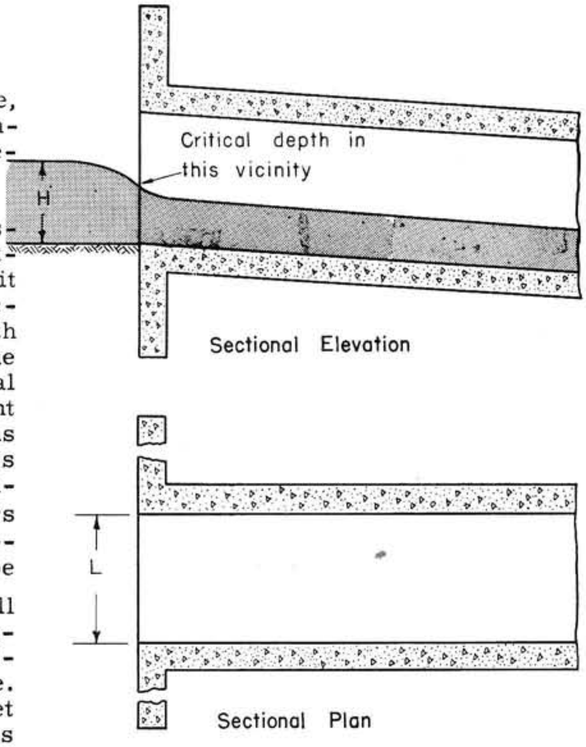

Fig. I-2 - Rectangular Weir at a Box Culvert Entrance

Drop Inlet Weir

A drop inlet is used as the entrance to many closed conduit spillways. For this reason a discussion of weir flow over the drop inlet crest is pertinent.

The drop inlet may be either rectangular or circular in cross section. In addition a headwall may or may not be used, depending possibly on the hydraulic design or on the willingness of the designer to take a chance that vortices will not form or that those vortices which do form will not reduce the anticipated capacity of the structure.

The head-discharge relationship for weir flow is given by Eq. I-1. If a headwall is used, the crest length is reduced by the length occupied by the headwall. If the crest is square-edged its effective length is measured on the inside of the drop inlet as in Fig. I-3a. If the crest is rounded--a better practice--the crest length may be measured, for all practical purposes, at the point of tangency with the horizontal, as in Fig. I-3b. If the upstream corners of

*Numbers in brackets refer to Bibliography listed on p. 17.

the diametrical headwall are square-edged, they can cause contractions that reduce the effective length of the crest. However, the relatively thin wall is insufficient to secure full contraction and the effect of what contraction exists will be taken into account in determining the discharge coefficients of those crests reported in subsequent parts of this report series. If other sources are utilized to determine C in Eq. I-1 and full contraction exists, Francis* suggests that the crest length be reduced by 0.1H for each contraction or, for the two contractions here, by 0.2H. If a tangent wall is used with a circular drop inlet, radial access of the flow to the drop inlet is partly cut off. For this condition, shown in Fig. I-3d, the effective crest length is taken as $D_r + \pi D_r/2$. The reasoning is apparent from the streamlines sketched in Fig. I-3d, which shows that the flow is radial for about one-half the circumference and parallel to the headwall for the other half.

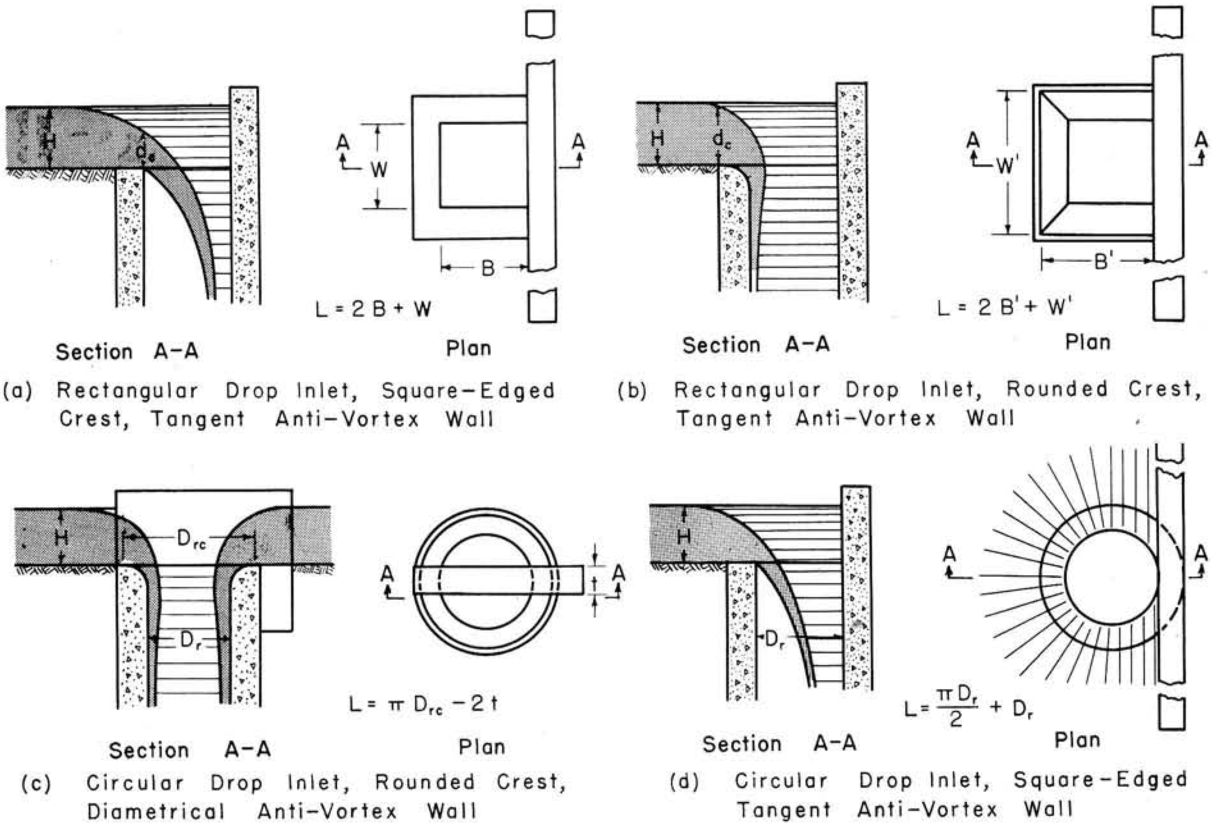

Fig. I-3 - Crests of Drop Inlets

When the conduit is flowing full of a mixture of water and air, there is a suction created in the drop inlet which serves to increase the effective head on the weir. This increase in effective head is usually small and ordinarily may be neglected and considered as an additional safety factor. It will be discussed in subsequent research reports for those drop inlets where it can be detected.

Barrel Exit Control

The exit of the barrel may exercise primary control over the head-discharge relationship. Three criteria are important in this connection: (1) The barrel slope must be less than the friction slope; (2) the flow along the entire length of the barrel is at a velocity less than the critical velocity; and (3) critical depth must occur at the barrel exit. The inlet does not control the head-discharge relationship, although the entrance loss will influence this relationship. The friction slope is h_f/ℓ--the friction head loss per foot of conduit.

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*Reference [I-31], p. 82.

The head at the barrel inlet when the control is at the barrel exit is determined by adding the friction loss in the barrel, the entrance loss, and the velocity head to the critical depth at the barrel exit.

The critical depth in a rectangular section is given by the equation

$$d_c = \sqrt[3]{\frac{q^2}{g}} \tag{I-3}$$

Where d_c is the critical depth in feet and q is the discharge per foot of width in cubic feet per second. The critical depth depends on the water surface width. Because the surface width varies with the depth of flow for circular barrels, the determination of the critical depth in a circular section is not as easy as for a rectangular section. Woodward and Posey [I-54] give a table that facilitates the determination of the critical depth in circular sections. Critical depths in trapezoidal sections are given in Reference [I-49]. Critical depths in other cross-sections less commonly used may be found using the criterion that the velocity head is equal to half the average depth of flow, the average depth of flow in this case being the area of flow divided by the surface width.

Tailwater Control

The tailwater level will affect the head-discharge relationship if the barrel is only partly full and the depth of flow is greater than the critical at every point within the spillway. Such conditions may be found frequently for highway culverts in regions having flat ground slopes. If the tailwater level affects the head-discharge relationship it will be necessary to compute a backwater curve to determine the headwater elevation. The rate of flow through the conduit is determined using open channel flow procedures. Methods for the computation of water surface profiles and rates of flow may be found in References [I-44] and [I-49].

Pipe Flow

The head-discharge relationship is governed by pipe flow equations when the conduit is completely full as in Fig. I-1d. For this condition the head H_t causing flow is the difference in elevation between the headwater surface and the point where the hydraulic grade line pierces the plane of the conduit exit, which is usually assumed as the center of the conduit at the plane of its exit. If the outlet is submerged, H_t is measured from the headwater surface to the tailwater surface. This head is entirely consumed in causing water to flow through the spillway. The head consumed is given by the equation

$$H_t = \left[K_e + K_o + \ldots + f\frac{\ell}{D} + f_r\frac{\ell_r}{4R_r}\left(\frac{A}{A_r}\right)^2\right]\frac{V_p^{\,2}}{2g} \tag{I-4}$$

and the discharge is given by the equation

$$Q = A\,V_p = \frac{\pi D^2}{4}\sqrt{\frac{2g\,H_t}{K_e + K_o + \ldots + f\frac{\ell}{D} + f_r\frac{\ell_r}{4R_r}\left(\frac{A}{A_r}\right)^2}} \tag{I-5}$$

where D is the pipe diameter for circular pipe, K_e is the loss coefficient for the drop inlet and/or the entrance to the barrel, K_o is the exit loss coefficient, f is the Darcy-Weisbach friction coefficient for the barrel, ℓ is the length of the barrel, R is the hydraulic radius of the drop inlet, A is the area of the conduit, and V_p is the velocity in the conduit. The subscript r refers to the drop inlet or riser.

A few of the terms used in Eqs. I-4 and I-5 require elaboration. Strictly, H_t is measured to the point at which the hydraulic grade line pierces the plane of the pipe outlet when the outlet is not submerged. See Fig. I-1d. Since the exact location of this point is still an open question (see Part X and References [I-19], [I-45] and [I-33]), it has been assumed to be at the center of the pipe in Parts II to IX inclusive, except for the work of Ree [I-42] reported in Part IX. In Part X the hydraulic grade lines used in the computation K_e did not necessarily pass through the center of the conduit exit but were in their correct positions. Also, K_e is the loss in the entrance in excess of any friction loss, the last term in the brackets of Eq. I-4 being the inlet friction loss; K_e includes the losses at the inlet which are not susceptible of determination otherwise. K_e will be evaluated for each entrance on which tests are made and will be given in subsequent reports. The hydraulic radius of the drop inlet R_r is used in place of the diameter to take care of drop inlets that do not have a circular cross-section. It should be substituted also for the conduit diameter D if its cross section is not circular. The term 4R is equivalent to D, as can be seen from the following derivation of the hydraulic radius for a circular pipe.

$$R = \frac{\text{area}}{\text{perimeter}} = \frac{\pi D^2/4}{\pi D} = \frac{D}{4}$$

or

$$D = 4R$$

Therefore, the term $4R_r$ is the equivalent diameter of the drop inlet. If the area of the drop inlet is different from that of the barrel, the average velocity in the drop inlet will also be different. Since the barrel velocity is used in the subsequent computations, a correction must be applied. This correction is $(A/A_r)^2$.

The friction coefficient f is the Darcy-Weisbach value and not the more familiar Manning n. It is suggested that f for smooth concrete pipe and plain corrugated metal pipe be taken from the paper reporting tests conducted at the St. Anthony Falls Hydraulic Laboratory [I-51]. Ree [I-42] gives values of the friction coefficient for rougher concrete pipe and for paved invert corrugated metal pipe. The friction coefficient for large, corrugated metal pipe was determined at the Bonneville Hydraulic Laboratory of the Corps of Engineers [I-14]. Rouse [I-43, I-44] also presents curves that may be used to determine f for a number of different pipe materials. For those who are more familiar with the Manning coefficient and prefer to use it, the equation for converting from the Manning n to f is

$$f = \frac{185\,n^2}{D^{1/3}} \tag{I-6}$$

The conversion from n to f is given in Table I-1 for a number of pipe diameters and values of n. Also, values of $K_p = f/D$ and $K_c = f_r/4R_r$ computed for various conduit sizes and Manning's n values may be found in Reference [I-49], section 5.5.1, drawing ES-42 for those who are more familiar with these coefficients.

It should be especially noted that the Manning n and the Kutter n are different and are not interchangeable. For smooth surfaces the values are almost identical, but for corrugated metal surfaces the Manning n is larger. According to Straub and Morris [I-51], the Manning n recommended in some handbooks is too low, the indication being that the lower Kutter n has been substituted for the higher Manning n assuming that, as Manning originally intended, the two n values are interchangeable.

If there are bends, changes in pipe size, two or more kinds of pipe, etc. used in the conduit, it will be necessary to add terms to the brackets of Eq. I-4 and to the denominator under the radical of Eq. I-5 to take these additional sources of loss into consideration.

Orifice at Entrance

The head-discharge relationship sometimes may be controlled by the entrance acting as an orifice. For some forms of the inlet this is not a desirable condition. It is especially undesirable when the orifice control extends over a considerable range in head where orifice

TABLE I-1

CONVERSION OF MANNING'S n TO DARCY-WEISBACH FRICTION FACTOR f

D in.	n 0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.0233	0.0282	0.0336	0.0394	0.0457	0.0524	0.0596	0.0673	0.0755	0.0841	0.0932	0.1028	0.1128	0.1233	0.1342	0.1456
8	.0212	.0256	.0305	.0358	.0415	.0476	.0542	.0612	.0686	.0764	.0847	.0933	.1024	.1120	.1219	.1323
10	.0197	.0238	.0283	.0332	.0385	.0442	.0503	.0568	.0637	.0710	.0786	.0867	.0952	.1040	.1132	.1229
12	.0185	.0224	.0266	.0313	.0363	.0416	.0474	.0535	.0599	.0668	.0740	.0816	.0895	.0979	.1066	.1156
15	.0172	.0208	.0247	.0290	.0337	.0386	.0440	.0496	.0557	.0620	.0687	.0758	.0831	.0909	.0989	.1074
18	.0162	.0196	.0233	.0273	.0317	.0364	.0414	.0467	.0523	.0583	.0646	.0713	.0782	.0855	.0931	.1010
21	.0154	.0186	.0221	.0259	.0301	.0345	.0393	.0444	.0497	.0554	.0614	.0677	.0743	.0812	.0884	.0960
24	.0147	.0178	.0211	.0248	.0288	.0330	.0376	.0424	.0476	.0530	.0587	.0648	.0711	.0777	.0846	.0918
30	.0136	.0165	.0196	.0230	.0267	.0307	.0349	.0394	.0442	.0492	.0545	.0601	.0660	.0721	.0785	.0852
36	.0128	.0155	.0185	.0217	.0251	.0289	.0328	.0371	.0416	.0463	.0513	.0566	.0621	.0679	.0739	.0802
42	.0122	.0147	.0175	.0206	.0239	.0274	.0312	.0352	.0395	.0440	.0487	.0537	.0590	.0645	.0702	.0762
48	.0117	.0141	.0168	.0197	.0228	.0262	.0298	.0337	.0378	.0421	.0466	.0514	.0564	.0617	.0671	.0729
54	.0112	.0136	.0161	.0189	.0220	.0252	.0287	.0324	.0363	.0405	.0448	.0494	.0542	.0593	.0645	.0700
60	.0108	.0131	.0156	.0183	.0212	.0243	.0277	.0313	.0351	.0391	.0433	.0477	.0524	.0572	.0623	.0676
66	.0105	.0127	.0151	.0177	.0205	.0236	.0268	.0303	.0340	.0378	.0419	.0462	.0507	.0554	.0604	.0655
72	.0102	.0123	.0147	.0172	.0200	.0229	.0261	.0294	.0330	.0368	.0407	.0449	.0493	.0539	.0586	.0636
78	.0099	.0120	.0143	.0168	.0194	.0223	.0254	.0287	.0321	.0358	.0397	.0437	.0480	.0524	.0571	.0620
84	.0097	.0117	.0139	.0163	.0190	.0218	.0248	.0279	.0313	.0349	.0387	.0426	.0468	.0512	.0557	.0604
90	.0095	.0114	.0136	.0160	.0185	.0213	.0242	.0273	.0306	.0341	.0378	.0417	.0458	.0500	.0545	.0591
96	.0093	.0112	.0133	.0156	.0181	.0208	.0237	.0267	.0300	.0334	.0370	.0408	.0448	.0489	.0533	.0578
102	.0091	.0110	.0131	.0153	.0178	.0204	.0232	.0262	.0294	.0327	.0363	.0400	.0439	.0479	.0522	.0567
108	.0089	.0108	.0128	.0150	.0174	.0200	.0228	.0257	.0288	.0321	.0356	.0392	.0430	.0470	.0512	.0556

control alternates with full conduit flow. When the control can alternate between the orifice and the full conduit, it is possible to have two discharges at an identical head and it is not possible to predict which section exerts the control at any one time. However, since orifice control may exist for some forms of closed conduit spillways, their hydraulic characteristics will be outlined.

The orifice may be horizontal, as at the crest of a drop inlet (Fig. I-1b); or it may be nearly vertical, as at the entrance of a barrel having a simple inlet or no inlet at all (Fig. I-4) or a drop inlet sufficiently large so that the controlling orifice is at the barrel entrance rather than at the drop inlet crest. For the former case the head H_o is measured from the crest and for the latter case it is measured from the center of the entrance. The discharge equation in either case is

$$Q = C_o A H_o^{1/2} \qquad \text{(I-7)}$$

where C_o is a discharge coefficient that will vary with the inlet conditions. It, like the weir coefficient, should be selected carefully since it can vary between rather wide limits.

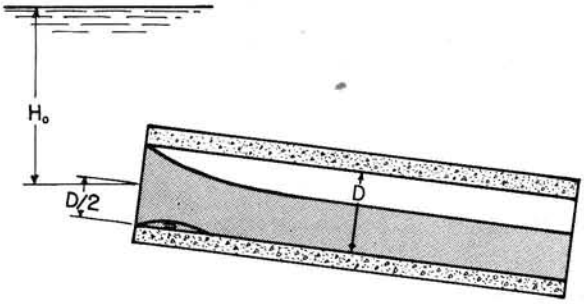

Fig. I-4 - Orifice Control at Conduit Entrance

It should be recognized that it is possible to have orifice control at the inlet of the barrel even though the barrel is on a slope less than the friction slope for uniform flow. Another way of saying the same thing is: The barrel may flow partly full with the inlet submerged even though the barrel slope is such that normally the barrel would be expected to flow full. The form of the inlet is important in this connection. If the inlet form is such that the stream contracts sufficiently so the velocity in the contracted section is greater than both the normal velocity and the critical velocity in the barrel, then the excess velocity head energy must be expended by friction and/or turbulence, and/or converted to depth energy. It usually takes some distance to accomplish this. The distance required to insure a depth of flow equal to the barrel height can be determined by computing the water surface profile. The velocity in the contracted section can be used as a starting point for these water surface computations if the orifice contraction coefficient is known or can be estimated. Methods given in References [I-44] and [I-49] may be used to determine the water surface profile. If the barrel is so short that the depth in the barrel never equals the barrel height, then the barrel will not flow full and the entrance acting as an orifice controls the head-discharge relationship. Straub, Anderson, and Bowers [I-50] have shown how the barrel length affects the capactiy of the culvert. The reasoning given in this paragraph does not apply if the velocity in the barrel is less than the critical velocity.

Short Tube at Entrance

When the drop inlet is full but the barrel is only partly full, as in Fig. I-1c, the condition is that of a short tube control. As observed to date, short tube control is a residual type of flow occurring after the headpool level has risen under orifice control and has been drawn down under pipe flow. It is not a desirable condition since it is possible to have three different discharges at identical heads--that is, orifice controlled discharge, short tube controlled discharge, and pipe or weir flow controlled discharge. In fact, with certain constant rates of flow to the spillway, the control shifts from point to point with accompanying fluctuations in head and discharge and it is not possible to predict the outflow with certainty at any given time.

The equation for short tube flow is

$$Q = C_s A H_s^{1/2} \qquad \text{(I-8)}$$

where C_s is a discharge coefficient which varies with the various forms of short tube. The head H_s is assumed to be the difference between the water surface and the point in the drop inlet where the water breaks away from the drop inlet wall. This is shown in Fig. I-1c. Eq. I-8 tacitly assumes that the short tube is not so long that friction losses are important.

The Composite Head-Discharge Relationship

The head-discharge curve for the spillway can be computed now that the equations governing the four types of flow have been established. Curves representing these equations are shown in Fig. I-5.

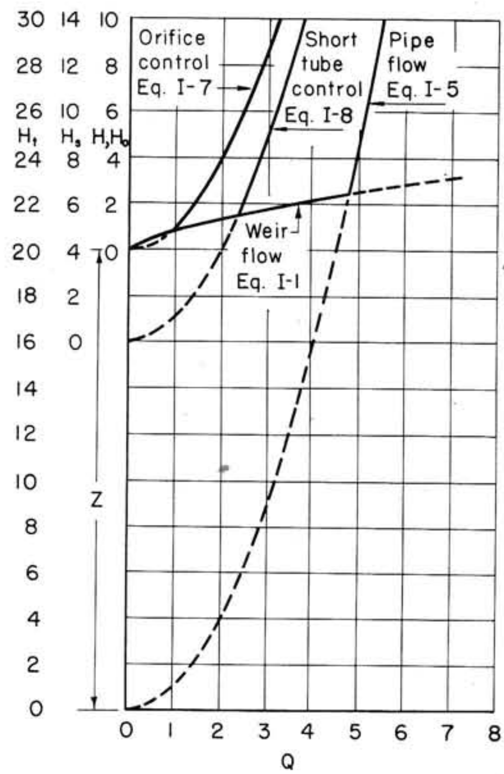

Fig. I-5 - Typical Head-Discharge Curves (See Fig. I-1 for typical sectional elevations of types of flow control.)

It will be noticed in Fig. I-5 that the origin of heads has been shifted downward for the short tube and pipe curves so that the headpool level is always measured from the same point on the plot. In the case of the short tube curve the shift downward is equal to the effective height of the drop inlet. In the case of the pipe flow curve the shift downward is equal to the difference between the crest of the inlet and the point where the hydraulic grade line passes through the plane of the conduit exit, since the outlet is not submerged. This shift in the origin is important in determining the point at which the control shifts from one curve to the other. The orifice in this example is at the entrance, as it is in Fig. I-1b, and in this case $H_o = H$.

Hypothetical Portions of the Head-Discharge Curve

It will be noticed that some portions of the head-discharge curves presented in Fig. I-5 are dashed. These are computed portions of the curves which never actually govern the head-discharge relationship. Explanation of this may be found in the following paragraphs.

The section of the orifice curve below the weir curve never determines the head-discharge relationship because there is insufficient flow to fill the orifice. Similarly, the section of the short tube curve below the weir curve does not control the head-discharge relationship because its demand for water is greater than can be supplied through the inlet with the head available over the inlet crest. These sections of the curves are hypothetical and imaginary, and so have been dashed.

The portion of the pipe flow curve below the weir curve is dashed; the pipe cannot control the flow because there is insufficient water to completely fill the pipe. In fact, there can be no flow at all through the pipe until H_t exceeds $Z + (\frac{D}{2} - \beta D) \cos^{-1} (\sin S)$. The capacity of the pipe is greater than the flow to it until the pipe flow curve crosses the weir flow curve.

The portion of the weir curve to the right of the pipe curve cannot control the flow because for these high discharges the weir is flooded out. The portion of the weir curve which is hypothetical and imaginary is dashed.

The remaining solid portions of the rating curves will control the flow through the closed conduit spillway. However, the intersections of the curves may be rounded rather than abrupt as indicated in Fig. I-5. Also, the highest heads at which the orifice and short tube curves apply are indeterminate. This is because the disturbances caused at the elbow shown in Fig. I-1b or Fig. I-1c by the falling stream seal off the barrel at the upper end and the barrel rapidly fills. The discharge then abruptly jumps over to the pipe flow curve. The head at which this occurs is indeterminate analytically and cannot be predicted accurately from tests because it may be different each time it occurs. The control exercised by the pipe curve is definite and this curve can be extended upward indefinitely.

The indefinite upper limits of the orifice and short tube curves make it impossible to predict the reservoir level at intermediate flows. The fact that three different rates of flow through the spillway can be obtained for the same head also is undesirable. It is possible to design closed conduit spillways so that the orifice and the short tube can exercise no control over the head-discharge relationship. This is certainly a desirable situation, for then the two sections of the head-discharge curve representing the orifice and the short tube controls are eliminated and the remaining two sections, representing the weir and the pipe, serve to control the flow through the spillway. Then for any given reservoir pool elevation the computation of the discharge capacity can be made with confidence since it is possible under these conditions to have only one rate of discharge for any reservoir level. The principal reason for including the orifice and short tube curves in Fig. I-5 is to show what can happen if these sections do control the flow.

Appearance of the Flow

When the weir controls the head-discharge relationship there is always a definite drawdown in the inlet, as is shown in Fig. I-1a. Up to the point where orifice control takes over, the nappe usually falls free down the drop inlet, but at low heads it may cling to the sides of the drop inlet. Between the points where the orifice and short tube curves intersect the weir curve of Fig. I-5, the water is still well drawn down at the inlet but the drop inlet is full of a mixture of air and water. Between the points where the short tube and pipe flow curves intersect the weir curve, the water surface in the inlet has the same appearance as previously but both the drop inlet and barrel are full of an air-water mixture. Thus, it is possible in the field to recognize weir control by the pronounced dropdown of the water surface over the inlet. Air will be sucked through the spillway, sometimes noisily, when the weir controls the flow.

For orifice, short tube, or pipe flow there may be a slight depression of the water surface over the inlet but not the deep depression characteristic of weir control. Normally, no air passes through the spillway when the control is orifice, short tube, or pipe but some air may be sucked in through vortices. For orifice flow the drop inlet will be partly full, as in Fig. I-1b, unless the orifice is at the barrel entrance at the base of the drop inlet; for short tube flow the drop inlet will be full, as in Fig. I-1c; and for pipe flow the conduit will be completely full, as in Fig. I-1d.

Change of Control

It is of interest to describe how the control changes from one section to another. First, the four types of flow shown in Fig. I-5 will be assumed to exist. Second, flow through a better type of spillway where only the weir and the pipe control the flow will be described.

According to the curves of Fig. I-5 the weir will control the flow when the discharge is less than about 0.9. This control is definite and is not shared with any other section of the spillway.

When the flow to the spillway is greater than 0.9, the head will increase first along the weir curve to its intersection with the orifice curve then along the orifice curve. At some unknown pond or reservoir level the outflow rate will jump abruptly from the orifice curve to the pipe curve. The pipe may control the discharge if the rate of inflow to the pond is great enough. If the outflow rate exceeds the inflow rate, the reservoir is drawn down first along the pipe flow curve to its intersection with the weir curve then along the weir flow curve. The flow may stabilize on the weir flow curve or it may jump to either the short tube or orifice curves, possibly resulting in a further buildup of the headpool level and a repetition of the cycle. As observed to date, the jump across is always to the pipe curve from either the orifice or short tube curves and never to the short tube curve from the orifice curve.

While the weir can control the flow to the right of the intersection of the orifice and short tube curves of Fig. I-5, the head and discharge along these sections of the weir flow curve cannot be reached directly. The pond level must first rise along the orifice curve, the flow must jump to the pipe flow curve, and the reservoir level must fall along the pipe flow curve to reach these sections of the weir curve. Similarly, the short tube control is not reached directly but is a residual condition that may exist only after pipe flow and the subsequent drawdown of the reservoir through the upper section of the weir curve.

In contrast to these unsteady and unpredictable flows just described, a spillway design which insures that orifice and short tube flow will not exist also insures that a definite head-discharge relationship can be established. This relationship is given by the weir and pipe curves of Fig. I-5. With increasing flow to the spillway the discharge is determined first by the weir curve and then, after the capacity of the weir is exceeded, by the pipe curve. The transfer of control between the two curves is gradual and reversible--not abrupt and irreversible as for the orifice and short tube controls. The head-discharge relationship is dependable for both increasing and decreasing flows to the spillway.

It should be recognized that the flow into the reservoir can be either greater or less than the flow through the closed conduit spillway; the difference between inflow and outflow is stored in or withdrawn from the reservoir and serves to increase or decrease its level. However, for any given constant rate of inflow, the pond level will eventually stabilize if the weir and the pipe alone control the flow. In contrast, when orifice and short tube control are also present, the reservoir level and the spillway flow may never stabilize but will alternately rise and fall as the control shifts from one section to another.

PRESSURES WITHIN THE SPILLWAY

The pressures in the closed conduit spillway and in its inlet depend on the type of flow in the spillway as well as on the discharge and on the form of the spillway. It is convenient to divide the ensuing discussion into sections dealing with the conduit partly full, the intermediate flow conditions, and the conduit completely full.

Conduit Partly Full

When the conduit is partly full, the normal pressures on the invert should be approximately equal to the depth of flow. Improperly aligned joints or similar disturbances may cause local increases or decreases in the pressure. Ordinarily, these pressure changes will be unimportant, but the possibility of cavitation and its resulting damage should not be overlooked where the velocity is high. The remedy for this is in better construction, and it should not be considered a hydraulic problem within the scope of this paper.

Water falling down a drop inlet may cause high local pressures at the foot of the drop inlet and on the barrel invert at its entrance. For the usual case these pressure increases over the normal pressures will not be sufficient to cause concern.

Pressures at any point in a conduit flowing partly full can be expressed in terms of the conduit diameter or of some other pertinent dimension (for example in terms of h_p/D) and can be converted into prototype pressures by multiplying this ratio by the proper dimension--in this case, the pipe diameter.

Intermediate Flow Conditions

For intermediate flow conditions the conduit is alternately partly full and completely full, or is continuously full of a water-air mixture. In other words, intermediate flow conditions cover the range between continuous open channel flow and continuous full conduit flow. Air is carried along with the water either intermittently or continuously when flow conditions are intermediate.

Particularly when the conduit is on a slope steeper than the friction slope, there is a range of discharge where the conduit alternates between part-full flow and where occasional hydraulic jumps form near the barrel entrance, completely fill the barrel cross section, and travel down and out the conduit. The traveling hydraulic jumps suck considerable air along with them. The pressure fluctuates for these conditions. The range of fluctuation of the pressures is from a normal maximum equal to the depth of flow, to a minimum which is greater than the minimum observed later on and which therefore does not influence the design. With increasing discharges the frequency of the traveling hydraulic jumps increases until the conduit is flowing completely full of a water-air mixture. The pressures are low but are steady for this flow condition. With further increases in the discharge, the air flow decreases until the conduit flows completely full of water alone. The minimum pressure occurs at this latter discharge. Methods of determining the pressures for full pipe flow are given in the following section.

Water-air mixture flow also occurs for drop inlet spillways when the barrel is on a slope that is less than the friction slope but the traveling hydraulic jumps may not occur.

Conduit Completely Full

A knowledge of the pressures for the condition of full pipe flow is of considerable importance because for some closed conduit spillway designs it is possible to have pressures that are close to absolute zero. Pressures close to absolute zero imply that cavitation may take place which, if sufficiently severe, can result in the complete destruction of the spillway.

The analysis of the pressures is made in such a way as to separate the pressure effects due to conduit length, total fall, and friction from those pressure effects caused by local disturbances. Local disturbances may originate at the entrance, bends, etc. They cause local deviations of the hydraulic grade line from its normal position. Local pressure deviations are determined experimentally. The method of analysis given here simplifies their application by the designer. In other words, because the local pressure effects are given independently of the total fall or roughness they can be applied to any conduit length, to any total fall of the conduit, or to any conduit roughness. Therefore, the pressure which can be anticipated under field conditions is determined by adding algebraically the local pressure effects to the pressure effects resulting from the conduit length, fall, and friction. Also, the analysis is made in such a way as to compensate for the actual size of the conduit and the actual quantity of water flowing in the conduit, the only restriction being that the conduit is full. Thus, one relative local pressure value can be used to compute, at any one point in the spillway, the actual local pressure for any conduit size and any rate of flow provided the conduits are geometrically similar.

The pressure at any point in the conduit is determined by computing the unknown pressure at the point under consideration from the known pressure at some reference point. The Bernoulli equation

$$\frac{V_o^2}{2g} + \frac{p_o}{w} + z_o = \frac{V^2}{2g} + \frac{p}{w} + z + h_f + \ldots \qquad \text{(I-9)}$$

is used for this purpose where V is the velocity in feet per second, p is the pressure in pounds per square foot, w is the unit weight of water in pounds per cubic foot, z is the elevation of the point in feet, and h_f is the friction head loss between the two points in feet. The subscript o denotes a reference point in a zone of uniform flow. The reference point is here taken at the point where the hydraulic grade line pierces the plane of the conduit exit, or at the center of the exit if the hydraulic grade line position is unknown. To the right side of Eq. I-9 should be added any additional losses that occur between the conduit exit and the point in question. These losses may be due to bends, expansions, contractions, etc.

Eq. I-9 may be reduced to a dimensionless basis. This both simplifies the experimental work and broadens the subsequent application to design problems. Dividing by the reference velocity head and rearranging, Eq. I-9 becomes

$$\frac{\frac{p}{w} + z - \frac{p_o}{w} - z_o + h_f + \ldots}{\frac{V_o^2}{2g}} = 1 - \left(\frac{V}{V_o}\right)^2 \tag{I-10}$$

In this paper the reference velocity is assumed to be the mean velocity in the barrel, and the reference point for elevation and distance is the point where the hydraulic grade line pierces the conduit at its exit end; at this point z_o, p_o, and ℓ are zero. The length ℓ is measured upstream along the axis of the barrel and is negative in sign--a fact that must be remembered when determining the sign of h_f. Replacing $V_o^2/2g$ by the symbol h_{vp}, p/w by h_p, and substituting the values of z_o and p_o in Eq. I-10, its form becomes

$$\frac{h_p + z + h_f + \ldots}{h_{vp}} = 1 - \left(\frac{V}{V_o}\right)^2 \tag{I-11}$$

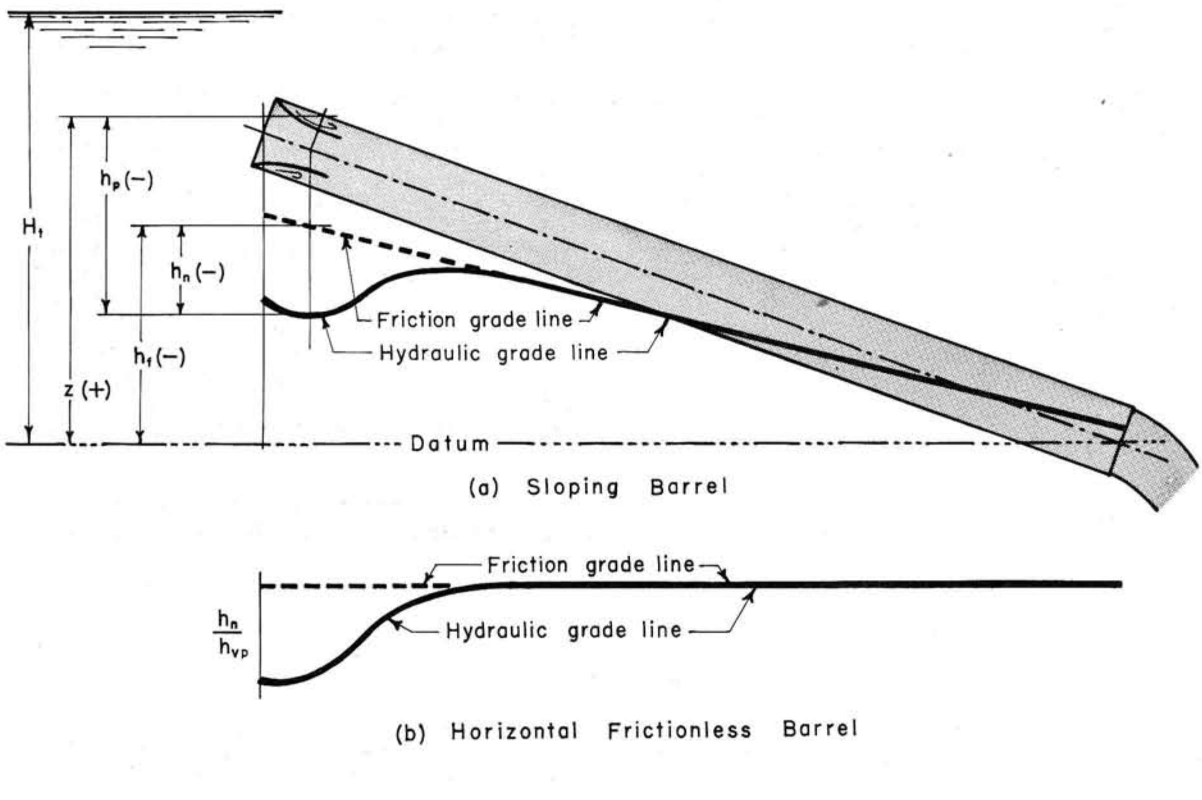

Fig. I-6 - Typical Hydraulic Grade Line

The pressure head h_p, the elevation z, and the friction head h_f are defined in Fig. I-6a. There it can be seen that z is the elevation of a point in the conduit above the datum plane and that h_p is measured from the elevation of that point to give the elevation of the hydraulic grade line $h_p + z$. The friction head h_f is measured upward from the datum plane but is negative in sign because the point being considered is upstream from the reference point.

The sum of (+ or -) h_p, (+ or -) z, and (-) h_f is symbolized by (+ or -) h_n or

$$h_n = h_p + z + h_f + \ldots \tag{I-12}$$

The term h_n represents the departure of the hydraulic grade line from the friction grade line; h_n also represents the local pressure variation from the fricition grade line resulting from some disturbance such as, for example, the conduit entrance; and h_n can be considered to give the elevation of the hydraulic grade line for a horizontal frictionless conduit since it represents the pressure remaining after the effects of elevation and friction (and bends, etc., if applicable) are taken into account.

Combining Eqs. I-11 and I-12,

$$\frac{h_n}{h_{vp}} = 1 - \left(\frac{V}{V_o}\right)^2 \tag{I-13}$$

In words, Eq. I-13 states that the local pressure head for a hypothetical horizontal frictionless conduit h_n relative to the mean velocity head in the pipe h_{vp} is a function only of the ratio of the local velocity V to the mean velocity in the conduit V_o. For flow in any dynamically similar spillway the ratio V/V_o is identical for any similarly located point within the spillway. It follows from Eq. I-13 that the ratio h_n/h_{vp} is also identical for any similarly located point within a dynamically similar spillway. Therefore, if h_n/h_{vp} is determined for any closed conduit spillway, it is identical to h_n/h_{vp} for any other dynamically similar spillway no matter whether it be small enough to test in a hydraulic laboratory or large enough to pass a major flood.

Once h_n/h_{vp} has been determined in the laboratory, the value of h_n can be computed for any size spillway if the discharge is known so that h_{vp} can be computed. Also, since h_n/h_{vp} is the same no matter what the flow through the full conduit may be, the value of h_n for any flow may be determined by computing the value of h_{vp} for that discharge and multiplying the result by h_n/h_{vp}. The interesting and valuable deduction is that a single determination of h_n/h_{vp} will permit the computation of h_n for any size of geometrically similar spillway as well as for any rate of full conduit flow through the spillway.

Rouse [I-43, pp. 80 to 85] has explained these relationships in slightly different terms. Reference may be made to this text for further information.

Values of h_n/h_{vp} for the hydraulic grade line of Fig. I-6a are given in Fig. I-6b. There it will be noticed that h_n/h_{vp} is zero for most of the conduit length and only near the entrance--a local disturbance--does h_n/h_{vp} deviate from zero. A zero value of h_n/h_{vp} indicates that there is no local disturbance to affect the pressure and that the hydraulic grade line can be drawn by plotting the friction and other losses. Near the entrance or in the vicinity of other disturbances it will be necessary to take h_n/h_{vp} into account and add it to the friction and other losses. Whether the conduit is under pressure or whether a partial vacuum exists depends on whether the conduit is below or above the hydraulic grade line.

The designer is interested in determining the pressure head h_p at a number of points in the structure with which he is working. This is accomplished by rearranging Eq. I-12 to the form

$$h_p = \frac{h_n}{h_{vp}} \times h_{vp} - z - h_f - \ldots \tag{I-14}$$

and substituting the proper values of h_{vp}, z, h_f, etc., for his particular structure together with values of h_n/h_{vp} selected from the results of tests on a spillway which is dynamically similar to his. Values of h_n/h_{vp} for various types of closed conduit spillways will be given in subsequent papers in this series.

If the computed value of h_p is less than about -25 ft to -28 ft of water, the designer should be concerned about the possibility of cavitation damage and should give consideration to a change that will raise h_p. This may be accomplished by using rougher pipe, by lowering the pipe, etc. The minimum value of h_p will ordinarily occur near the conduit entrance or at some other disturbance that serves to lower the pressure and at the discharge which exists when the conduit just flows completely full of water, that is, at the beginning of pipe flow.

APPLICATION OF THEORY

The first step in applying the theory as outlined here is a determination of the head-discharge relationship--or a determination of the capacity of the spillway if the complete head-discharge curve is not needed. It is presumed here that the preliminary computations have determined the form and dimensions of the spillway.

The capacity of the spillway, if it is known that only weir and pipe control are present, is determined by computing the discharge for the design head assuming both weir and pipe control. The lesser discharge is the correct one. In making these computations the weir head is, of course, measured from the weir crest and the pipe head from the tailwater level or from the center of the outlet or, more precisely, from the point where the hydraulic grade line pierces the outlet plane. The discharge coefficient of the weir is taken from some later part of this report series or from some other reliable source. The coefficients in the pipe formula (Eq. I-5) also are taken from some reliable source wherein the spillway form is the same as is to be used in the spillway under consideration. The friction factor for the pipe is also obtained from some reliable source. A few sources of information are given in the Bibliography.

If a rating curve for the spillway is desired, it will be necessary to compute the discharge for a number of heads. The head and discharge should be plotted as in Fig. I-5 with the origin of head for pipe flow offset so that the plotted heads always refer to the crest of the weir. The lesser discharge at any given head computed using both the weir and the pipe formulas is what actually passes through the spillway, the greater indicated discharges being purely imaginary since only one section of the conduit controls the flow at any one time.

Whether or not orifice and short tube flow are possible in a spillway can be determined by reference to the part of this report series which covers the form of the spillway being considered. If these controls exist their capacity should also be determined. A minimum head at which each control becomes effective is determined from a plot similar to Fig. I-5 or by equating the weir and orifice or short tube equations. The maximum head at which each control is effective can be approximated by referring to the pertinent part of this report series. If there is uncertainty as to whether orifice or short tube control is present in some design which has not been tested, these controls can be assumed and the discharges can be computed. It is safe to suppose that the control is not as assumed if the computed discharge at any head is greater than weir or pipe flow. Conversely, lesser computed discharges indicate the possibility of orifice or short tube control, as the case may be.

The pressures for part-full flow are low and ordinarily will be of no concern. Usually it will not be necessary to compute them. If their determination becomes necessary they can be computed from the common equations dealing with open channel flow. These equations should give satisfactory results except in the vicinity of the inlet, or at other locations where there are disturbances that affect the pressures.

Pressures for intermediate flow conditions fluctuate widely and cannot be reliably determined at any instant. Since the range of pressure is between those observed for part-full flow and those observed for full flow, it is not necessary to actually determine them. The sucking of air and the possible vibration of the structure make intermediate flow spectacular. The air flow and noise are presumably of little practical importance. The vibration may or may not be important.

Minimum pressures will ordinarily occur for the discharge where weir control changes to pipe control, that is at the minimum discharge for full pipe flow. The pressure in the conduit is determined from Eq. I-14. Except in the vicinity of disturbances, h_n/h_{vp} is zero. If there are no disturbances in the spillway the pressures can be computed without recourse to model studies. If disturbances exist, the pressures in their vicinity must ordinarily be determined experimentally. Values of h_n/h_{vp} for each spillway tested will be given in subsequent parts of this report series and reference should be made to them to determine h_n/h_{vp}. Harris [I-23] has shown that h_n/h_{vp} can be as low as -1.27. It is therefore important to determine the minimum values of h_p to make sure that the pressure is not so close to absolute zero that the water will vaporize, cavities form, and subsequently collapse. If the collapse takes place next to the wall of the spillway the concrete and the steel may be eaten away and eventually the structure may be destroyed. This is the process known as "cavitation."

SUMMARY

The theory of closed conduit spillways may be summarized as follows:

1. The capacity of the spillway, when the control is

   a. a rectangular weir, is given by the equation

   $$Q = C L H^{3/2} \qquad \text{(I-1)}$$

   or, in the semi-dimensionless form, by the equation

   $$\frac{Q}{D^{5/2}} = C \frac{L}{D}\left(\frac{H}{D}\right)^{3/2} \qquad \text{(I-2)}$$

   b. a circular weir, is obtained from experimentally determined curves of the dimensionless head-discharge relationship;

   c. a drop inlet weir, is given by Eqs. I-1 or I-2, the proper value of L being determined from the geometry of the crest and the presence or absence of head walls;

   d. the barrel exit, is determined by the critical depth at the barrel exit plus the losses to that point;

   e. the tailwater level, is determined using the methods for the determination of the rates of flow in open channels;

   f. pipe flow, is given by the equation

   $$Q = \frac{\pi D^2}{4}\sqrt{\frac{2g H_t}{K_e + K_o + \ldots + f\frac{\ell}{D} + f_r \frac{\ell_r}{4R_r}\left(\frac{A}{A_r}\right)^2}} \qquad \text{(I-5)}$$

   g. an orifice at the entrance, is given by the equation

   $$Q = C_o A H_o^{1/2} \qquad \text{(I-7)}$$

   h. a short tube at the entrance, is given by the equation

   $$Q = C_s A H_s^{1/2} \qquad \text{(I-8)}$$

2. Orifice and short tube control of the flow are ordinarily undesirable and should be avoided by proper design. Examples of proper and improper design are given in subsequent parts of this report series.

3. The head and discharge at which the control changes from one point to another can be determined from the head-discharge curve.

4. The pressure head in the conduit for

   a. part-full flow is approximately equal to the depth of flow except near the conduit entrance;

   b. intermediate flow fluctuates between the pressure for part-full flow and the pressure for full pipe flow (considerable air is sucked through the structure under these conditions);

   c. pipe flow is given by the equation

$$h_p = \frac{h_n}{h_{vp}} \times h_{vp} - z - h_f - \ldots \qquad \text{(I-14)}$$

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