DESIGN CRITERIA FOR CONTROLLED SCOUR AND ENERGY DISSIPATION AT CULVERT OUTLETS USING ROCK AND A SILL

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This study establishes the criteria for the effective design of rock-basin energy dissipators for flow from culverts without or with a transverse sill. Design tables have been prepared on the basis of laboratory studies with 175-, 3-, and 6-in. diameter culvert models on a zero slope with a low tailwater. Models of standard end flares were used to simulate the culvert outlet conditions. Stable rock sizes and basin geometry can be determined by using the design tables developed in the study. The design tables provide data for flows up to a discharge factor, $Q/D^{2.5}$, of 13.5 and are applicable for angular rock as well as rounded rock. The tables are used in examples for design of rock basins for no-scour situations and controlled depths of scour. The study concludes that the rock basin should have a width of at least 3 pipe diameters and divergence angles of 1:3 when no sill is used and 1:1.75 when a sill is used. The length of the basin is dependent on the culvert discharge, culvert diameter, size of rock, extent of permissible scour, and use of a sill. The sill of this study was placed at the flared end and was 1 diameter long and 0.3 diameter high. The tables, by means of dimensionless parameters, make it possible to select the proper sized rock to realize a selected velocity reduction and a degree of scour control.

•THE CONTROL of erosion at the outlet of culverts under highways is a demanding prob lem. The complex interplay between the many involved parameters puts limitations on a complete solution. Economy of construction, designer's time, aesthetics, and safety factors were considerations prompting the use of rock and a simple sill for this study.

The objectives of this study are to determine the following by means of model simulation of flow from circular culverts onto basins of rock riprap:

1. Proper size of rock for stability to movement and ability to dissipate the flow energy,
2. Effect of the shape of the rock (rounded or angular),
3. Proper geometry of the rock basin with regard to width and length and the expansion angle from the culvert outlet, and
4. Effect and proper dimensions of sills at the end of the flared end.

Pertinent to this study is the state of the flow just as it leaves the circular section of the culvert and enters the end transition. In other words, the designer by his analysis has brought the flow through the culvert and now wonders what to do with it at the brink of the circular outlet. Variables necessary to describe the flow at the brink are as follows:

Q = volumetric rate of discharge,
D = inside diameter of culvert,
d_b = depth of flow at the brink,
ρ = mass density of the water,
μ = dynamic viscosity of water,
g = gravitational acceleration, and
V_b = average velocity of flow at the brink.

After leaving the circular culvert, the flow enters a flared end. The flared end has dimensions proportioned to the diameter of the culvert according to a standard concrete flared end used by the South Dakota Department of Highways (Fig. 1). After leaving the flared end, the flow discharges onto a rock basin that has the following characteristics:

d = size of riprap (an average diameter determined by passing and retention on specified square-opening screens),
ρ_s = mass density of rock riprap,
d_1 = depth of flow over the rock basin at the outlet end of the flared end,
d_2 = depth of flow at the downstream end of the rock basin,
ℓ_1/ℓ_2 = divergence ratio of the sides of the basin (Fig. 2),
W_b = width of the basin at the flared-end outlet,
d_s = depth of scour hole,
L_s = length of scour measured from flared-end outlet,
L_d = length to downstream edge of dune measured from flared-end outlet,
W_s = width of scour hole,
L = length of basin to which velocity is reduced to v, measured from flared-end outlet, and
v = average velocity of flow at a specified distance L downstream from the flared-end outlet.

Preliminary laboratory observations demonstrated that the best location for a simple, rectangular sill would be downstream from the flared-end outlet a distance of about 1 to 2 pipe diameters. However, because such a location would present a safety hazard on the right-of-way and would present a costly construction problem, it was decided to
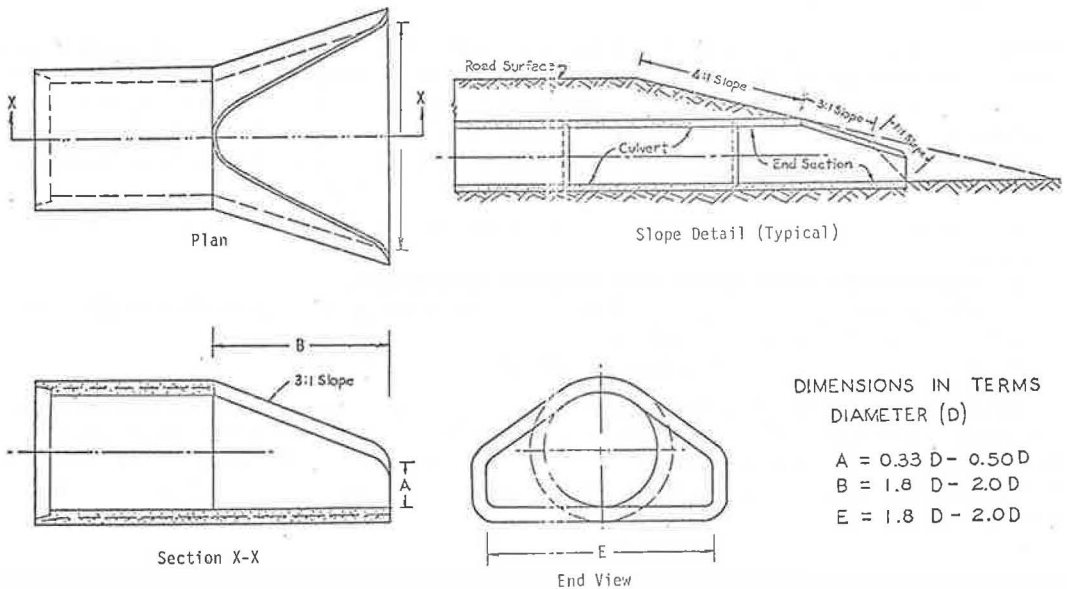

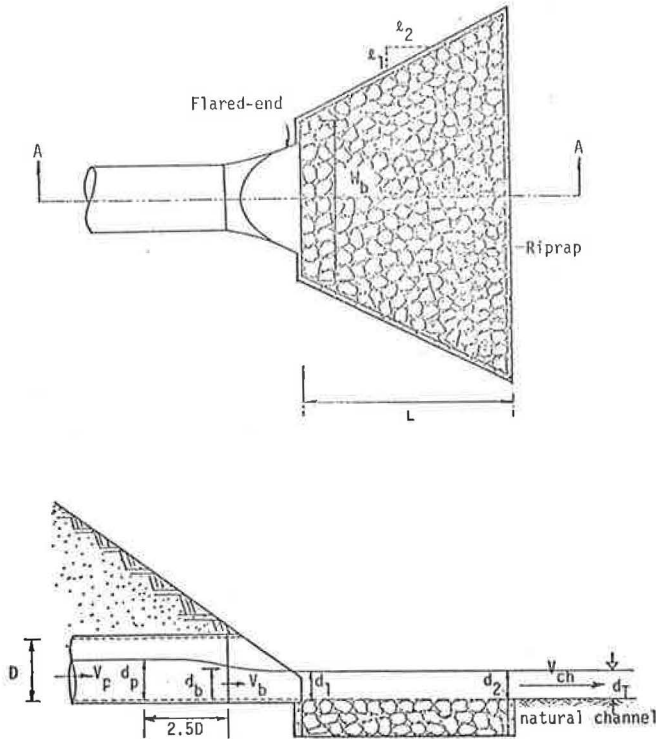Figure 1. Dimensions of the standard concrete flared end.

Figure 2. Definition of parameters.

confine this study to a sill location immediately downstream from the flared-end outlet. This location has a definite advantage in that the sill can be incorporated in an end-wall construction. The sill dimensions to be considered are as follows (Fig. 3):

W = length,
P = height, and
t = thickness of the sill.

Many forces, such as pressure, gravitation, viscosity, traction, lift, and drag, influence the energy dissipation efficiency of a bed of rock riprap. In addition, the many variables of the kinematics and geometry involved preclude a rigorous theoretical analysis. This necessitates recourse to model studies to obtain quantitative relationships necessary for design.

Parameters combining the many variables can best be established by having in mind conventional force ratios, useful scaling ratios, and meaningful comparison relations.

The dimensionless parameter $\rho Q/\mu D$ indicates the influence of fluid viscosity and can be shown to be related to the Reynolds number. On the basis of previous research (5, 6) this parameter can be considered to have no significant influence on the energy dissipation by a bed of rock riprap. Subsequent experiments of this study confirm this assumption.

Study of a dimensionless form Froude number, $\rho Q^2/[g(\rho_s - \rho)D^5]$, would disclose that, all other factors being constant in comparisons, a relatively large variation in ρ_s would be required to bring about a significant change in the Froude number. Small variations in specific gravities of rock will not be important.

A meaningful form of the Froude number is complicated by 2 possible regimes of flow at the brink of the culvert: full-pipe flow and partly full flow. For full-pipe flow, $Q/D^{2.5}$ is a commonly accepted term; and, because it can be related to partly full flow through
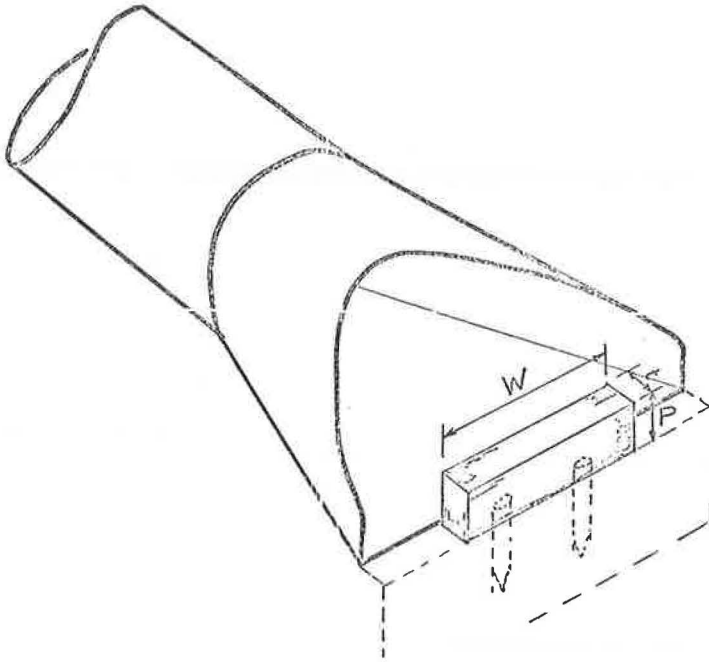
Figure 3. Dimensions and location of sill.

dimensionless ratios, it will be used for this study. This term has dimensions of ft$^{1/2}$/ sec because $\sqrt{g}$ has been dropped from the denominator.

This study is limited to a culvert with a zero slope. This should present no difficulty in making comparisons with sloped culverts by using $Q/D^{2.5}$ (sometimes called the discharge factor) as long as the pipe is flowing full. However, for partly full and a given discharge, a pipe with a steep slope would have a jet of a higher $V/d_b^{0.5}$ value than that which would occur for the jet of this same rate of flow in a pipe with a zero slope. Just what effect the higher momentum jet in the steep pipe would have will not be determined. As long as the culvert slope is "mild" the difference would probably be slight.

The study is confined to the use of rock with a Corey shape factor of approximately one (i.e., the length, breadth, and thickness of a rock are quite comparable). Nonscour or controlled scour of the riprap bed is required as a precondition for design. This sets the parameter, d_s, equal to 0 or 1 or 2 times d. For this study, the slope of the riprap bed is to be considered to be horizontal.

From the review of literature (1, 3, 4, 8) and field and laboratory observations, a limitation can be made regarding tail-water without putting too significant a restriction on the usefulness of the data gathered from this study. For the large majority of situations in the field the natural channel, downstream from the rock-basin energy dissipator, is wider than the basin itself. This results in a channel depth of flow less than that in the rock basin. As long as the rock in the basin is fully innundated (i.e., rock is then being lifted by full buoyant force), the most critical case of scour within the rock basin is being realized. This is so because an increase in depth of backwater on the rocks immediately begins to reduce the jet velocity and provides a cushion for energy dissipation.

However, because the jet, when so submerged, $d_T > D$, does not diffuse so readily as when free, the subsequent velocities in the downstream section of the basin and consequently in the following channel can be greater than when there is no tailwater. This would be a less frequently occurring situation. Therefore, the only tailwater condition considered in this study was less than the depth of flow on the rock basin. A barrier

was placed at the end of the rock basin to ensure that the rock was fully submerged during a test.

EXPERIMENTAL STUDIES

Laboratory Model

Model studies were conducted in a 12-ft long, 6-ft wide and 2-ft high wooden flume. The culvert models were of smooth plastic pipe with 1.75-, 3-, and 6-in. inside diameters. Fiberglass models of the standard concrete end flare used by the South Dakota Department of Highways were used to simulate the culvert end conditions (Fig. 1).

Rock of the following sizes (ASTM standard sieve sizes) was used as riprap:

Size (in.)	Passing Sieve (in.)	Retained on Sieve (in.)
0.500	0.500	0.375
0.525	0.525	0.44
0.750	0.750	0.625
0.875	0.875	0.75
1.000	1.000	0.875
1.500	1.500	1.25
2.000	2.000	1.75
3.000	3.000	2.50

Separate tests were made by using quarried rock (crushed, angular limestone with an average apparent specific gravity of 2.69) and field rock (rounded conglomerate shales, quartz, and occasionally sandstone, with average apparent specific gravity of about 2.63). The rock riprap of a selected size was dumped in a basin to form a horizontal riprap bed level with the pipe invert. Placing of some of the rock by hand was required to attain a level bed.

Flows

The rock bed was subjected to different flow conditions. Average velocities were determined at required transverse sections by dividing the total flow by measured cross-sectional areas. Where possible, the velocity of flow was checked with an Ott current meter. Water elevations and bed elevations were determined with a point gage. The flows at which movement of rocks was imminent (i. e., incipient motion) along with those for 1-d and 2-d scour to occur were noted. (Scour depths equal to the size of the riprap were referred to as 1-d scour, and scour depths equal to twice the size of the riprap were referrred to as 2-d scour.)

Observing the flow patterns made it possible to determine the limits of the boundaries of the basin required for no erosion outside of the basin. The effectiveness of these selected boundaries was checked by placing fine sand (passing ASTM sieve no. 30) along the outside of these boundaries and by directing the flow over the basin again. The erosion pattern on this sand confirmed the proper basin boundaries.

Sills

Because of the long testing procedure adopted with any one size of rock, it was decided to select just one size of sill and compare its performance with the various angular and rounded rocks. The selection was based on observations using the 6-in. pipe and sills of lengths W equal to 2-D, 1¼-D, 1-D, and ¾-D. The 2-D length stretched completely across the end of the flared-end outlet.

Each length was observed with 5 different heights, P, equal to 0.1-D, 0.2-D, 0.25-D, 0.3-D, and 0.4-D. It was discovered that the thickness of the sill was of no concern from a hydraulic standpoint because, even at the lowest flows, the water sprung clear from the sides and top of the sill at the upstream edges. The sills used had a thickness of 0.1-D.

ANALYSIS OF DATA

Model Similitude

Whether reliable similitude could actually be attained with rocks was of great concern. However, comparisons of data taken clearly indicated the similarity of performance of the 1.75-, 3- and 6-in. models. When the reliability of this similitude was established, subsequent tests were conducted by using the 6-in. model.

Geometry of the Basin

Observations during the numerous tests made it clear that the width of the basin at the flared-end outlet must be at least equal to 3-D. This was necessary to prevent scouring by the eddies on either side of the jet rotating in opposite directions. A divergence ratio of 1:3 was needed to contain any permitted scour from the culvert jet at high as well as at low flows. This shape of basin also prevented the scouring of any fine sand placed outside the basin. The desirable depth of the rock basin is equal to two times the maximum rock size of the armor plus the depth of the filter blanket required below. Filter blanket design procedures are well defined elsewhere (7, 9).

Dimensionless Scour and Energy-Dissipating Relations

Some of the main considerations of this study can be reduced to an equation relating dimensionless parameters and a conventional expression of Froude number.

$$L/D = f[(Q/D^{2.5}), (D/d), (v/V_p), (d_s/D)]$$

When the data are reduced to this dimensionless form and plotted in the manner shown in Figure 4 and points of incipient motion, 1-d scour, and 2-d scour are noted, a great deal is revealed about the relationship of these variables.

Plots of this type were made for angular rock, with and without sills, and for rounded rock, with and without sills. These types of plots could be used for design purposes. However, because they present so much information in such detailed form they are not considered the most convenient manner of data presentation for design criteria. Ultimately, tabulated information was extracted from these plots and is given later in this report.

Length of the Basin

The proper length of a rock basin, as considered in this study, would (a) reduce the velocity of pipe flow to a tolerable or noneroding velocity on the downstream channel and (b) be long enough to contain the scour hole and dune material if any scouring is to be permitted.

The dimensionless plots shown in Figure 4 were made for reductions of velocity of flow to $0.5V_p$, $0.4V_p$, and $0.3V_p$ when no sill was used and to $0.3V_p$ and $0.2V_p$ when the sill was used. V_p is the average velocity in the pipe calculated from measured discharge Q, and the depth of flow, d_p, measured 2.5-D upstream from the brink of the circular pipe.

Further reductions in velocities were limited because the jet had diverged and contacted the sides of the flume. Use of a wider flume or a smaller culvert model would have permitted further velocity reduction studies. However, because the velocity reductions are based on the average velocity in the pipe 2.5-D upstream from the brink and because this velocity is less than the brink velocity or critical velocity, the percentage of velocity reduction is greater for partly full pipes than one might think if accustomed to working with brink or critical velocities.

The use of the design tables for partly full pipe flow when the brink of critical velocity is prescribed (which probably is the more usual situation) is facilitated by curves relating depth of flow in pipe, brink depth, and critical depth dimensionlessly to the discharge factor (Fig. 5). Figure 5 used with the following data for partly filled and standard pipes should enable the designer to enter the design tables based on V_p. Area ratios of partly filled pipes for depth-diameter ratios are as follows:

d_a/D	a/A
0.0	0.000
0.1	0.052
0.2	0.143
0.3	0.252
0.4	0.373
0.5	0.500
0.6	0.626
0.7	0.748
0.8	0.858
0.9	0.950
1.0	1.000

Areas and $D^{2.5}$ for standard pipe sizes are as follows:

Diameter (in.)	$D^{2.5}$ $(ft^{2.5})$	A (ft^2)
12	1.00	0.785
18	2.76	1.77
24	5.65	3.14
30	9.88	4.91
36	15.6	7.06
42	22.9	9.62
48	32.0	12.6
54	43.0	15.9
60	55.9	19.6
66	70.9	23.7
72	88.2	28.3

Examples of various applications of these criteria to design will be presented later.

If no scouring is to be permitted within the basin, then the length required to attain a desired velocity reduction governs the length of the basin. If a scour hole is to be allowed, however, the velocity just downstream from the dune may be sufficiently small that no additional length of basin is needed. In this case the length that is needed to contain the scour hole and dune governs the required length of the rock basin. These lengths to contain holes of controlled depth of scour can be generalized as follows:

1. For 1-d scour, a length of 4-D is needed to contain the scour hole and dune (an L_s of about 2-D and an L_d of about 4-D); and

2. For 2-d scour, a length of 6-D is needed to contain the scour hole and dune (an L_s of about 3-D and an L_d of about 6-D).

These criteria are reflected in the data given in the design tables. The dimensionless lengths, L/D, for no-scour are given below the single underlined values and above the double underlined values. If a 2-d scour is to be allowed for a given velocity reduction, the proper L/D values for a given discharge factor are given below double underlined values and above triple underlined values. These dimensionless lengths are given for various rock sizes expressed in a dimensionless form, D/d, equal to from 2 to 8.

Depth of Flow on Basin

The depth of the backwater was, for all practical purposes, identical with the depth of flow of the jet itself. This depth of flow was greatest at the culvert outlet, d_1, and diminished slightly toward the end of the basin, d_2. Because this change in downstream depth was so small, only the maximum upstream depth d_1 is given in the tables for design considerations. These depths varied from about 0.2-D for low flows without sills to 0.5-D for high flows with sills.
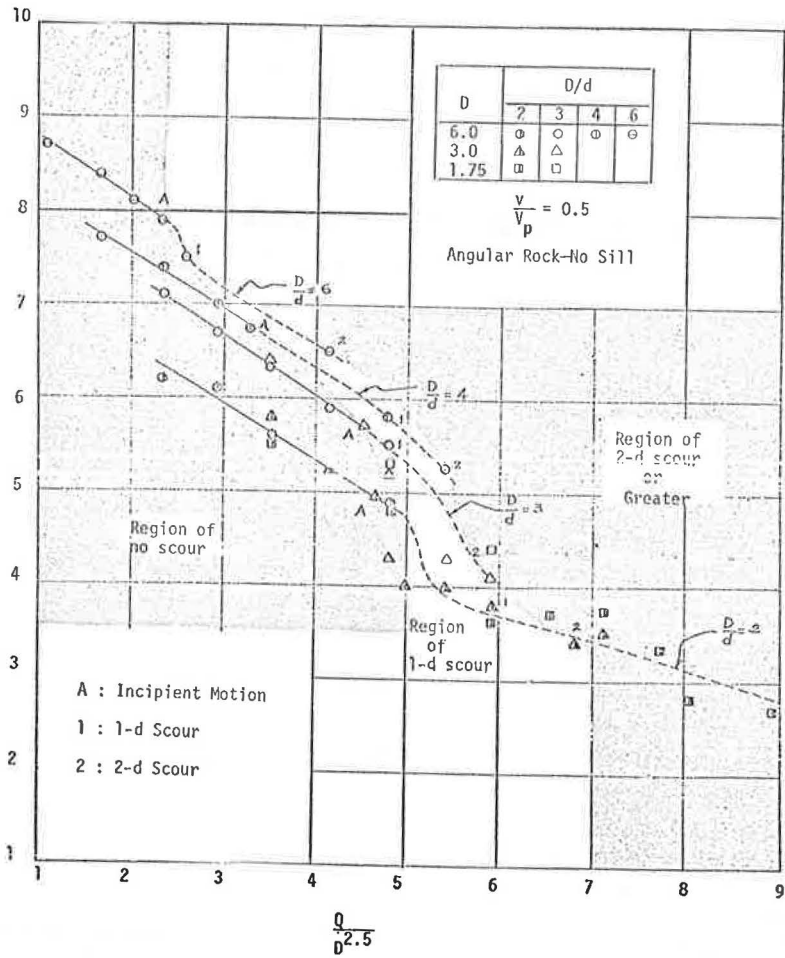
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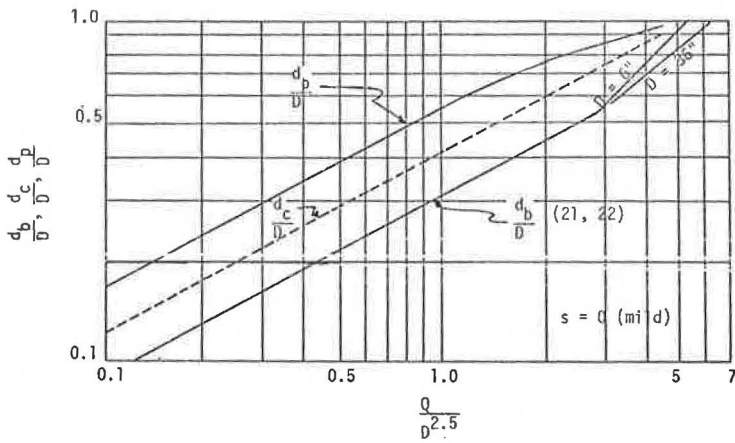Figure 4. Dimensionless scour and energy-dissipating relations.

Figure 5. Dimensionless depths of flow versus discharge factor.

TABLE 1

LENGTH RATIO OF BASIN FOR VELOCITY
REDUCTION RATIO OF 0.5, NO SILL

$Q/D^{2.5}$	d_1/D_{max}	L/D for $v/V_p = 0.5$				
		8 D/d	6 D/d	4 D/d	3 D/d	2 D/d
1.00	0.25	9.10	8.65	8.00	7.75	6.90
1.25	0.25	8.95	8.50	7.85	7.60	6.75
1.50	0.25	8.80	8.35	7.70	7.45	6.65
1.75	0.25	8.70	8.25	7.60	7.30	6.50
2.00	0.25	8.50	8.15	7.50	7.20	6.40
2.25	0.25	7.70	8.00	7.30	7.00	6.25
2.50	0.25	7.45	7.85	7.15	6.90	6.10
2.75	0.25		7.60	7.05	6.75	5.95
3.00	0.25		7.30	6.90	6.60	5.85
3.25	0.25		7.05	6.00	6.45	5.70
3.50	0.25		6.90	6.65	6.30	5.60
3.75	0.25		6.70	6.45	6.20	5.45
4.00	0.25		6.55	6.25	6.05	5.35
4.25	0.25		6.40	6.10	5.95	5.20
4.50	0.25			6.00	5.70	5.05
4.75	0.25			6.00	6.00	4.95
5.00	0.25			6.00	6.00	4.80
5.25	0.25			6.00	6.00	4.20
5.50	0.25			6.00	6.00	4.00
5.75	0.25			6.00	6.00	4.00
6.00	0.25				6.00	4.00
6.25	0.25				6.00	4.00
6.50	0.25					4.00
7.00	0.25					4.00
7.50	0.25					6.00
8.00	0.25					6.00

Note: W_b = 3D and ℓ_1/ℓ_2 = 1:3.

TABLE 2

LENGTH RATIO OF BASIN FOR VELOCITY
REDUCTION RATIO OF 0.4, NO SILL

$Q/D^{2.5}$	d_1/D_{max}	L/D for $v/V_p = 0.4$				
		8 D/d	6 D/d	4 D/d	3 D/d	2 D/d
1.00	0.25	10.95	10.35	9.60	9.40	9.05
1.25	0.25	10.85	10.15	9.45	9.20	8.90
1.50	0.25	10.70	10.00	9.35	9.10	8.75
1.75	0.25	10.55	9.85	9.20	9.00	8.55
2.00	0.25	10.30	9.75	9.10	8.85	8.40
2.25	0.25	9.75	9.55	9.00	8.70	8.25
2.50	0.25	9.60	9.40	8.85	8.60	8.05
2.75	0.25		9.30	8.75	8.45	7.95
3.00	0.25		9.20	8.60	8.35	7.75
3.25	0.25		9.15	8.50	8.15	7.55
3.50	0.25		9.05	8.40	8.05	7.40
3.75	0.25		9.05	7.85	7.90	7.20
4.00	0.25		8.95	7.50	7.75	7.05
4.25	0.25		8.85	7.25	7.65	6.90
4.50	0.25			7.35	7.00	6.75
4.75	0.25			6.80	7.05	6.55
5.00				6.65	6.80	6.40
5.25	0.25			6.45	6.30	6.10
5.50	0.25			6.35	6.10	5.85
5.75	0.25			6.30	6.00	5.70
6.00	0.25				6.00	5.45
6.25	0.25				6.00	5.30
6.50	0.25					5.15
7.00						4.80
7.50	0.25					6.00
8.00	0.25					6.00

Note: W_b = 3D and ℓ_1/ℓ_2 = 1:3.

TABLE 3

LENGTH RATIO OF BASIN FOR VELOCITY
REDUCTION RATIO OF 0.3, NO SILL

$Q/D^{2.5}$	d_1/D_{max}	L/D for $v/V_p = 0.3$				
		8 D/d	6 D/d	4 D/d	3 D/d	2 D/d
1.00	0.25	12.80	11.90	11.20	10.85	10.65
1.25	0.25	12.65	11.75	11.05	10.90	10.45
1.50	0.25	12.50	11.55	10.95	10.75	10.30
1.75	0.25	12.35	11.45	10.85	10.65	10.10
2.00	0.25	12.10	11.25	10.70	10.50	10.00
2.25	0.25	11.85	11.05	10.60	10.35	9.80
2.50	0.25	11.80	10.95	10.45	10.25	9.65
2.75	0.25	11.75	10.80	10.30	10.10	9.50
3.00	0.25		10.70	10.20	9.95	9.30
3.25	0.25		10.65	10.10	9.80	9.10
3.50	0.25		10.60	10.00	9.65	9.00
3.75	0.25		10.60	9.40	9.50	8.80
4.00	0.25		10.50	9.10	9.45	8.65
4.25	0.25		10.40	8.85	9.30	8.50
4.50	0.25			8.60	9.00	8.30
4.75	0.25			8.40	8.80	8.15
5.00	0.25			8.20	8.35	7.95
5.25	0.25			8.05	7.90	7.65
5.50	0.25			7.95	7.75	7.40
5.75	0.25			7.90	7.60	7.20
6.00	0.25				7.50	7.05
6.25	0.25				7.40	6.90
6.50	0.25					6.75
7.00	0.25					6.40
7.50	0.25					6.00
8.00	0.25					6.00

Note: W_b = 3D and ℓ_1/ℓ_2 = 1:3.

TABLE 4

LENGTH RATIO OF BASIN FOR VELOCITY
REDUCTION RATIO OF 0.3, WITH SILL

$Q/D^{2.5}$	d_1/D_{max}	L/D for $v/V_p = 0.3$				
		8 D/d	6 D/d	4 D/d	3 D/d	2 D/d
1.00	0.4	9.20	8.75	7.80	7.65	7.20
1.25	0.4	9.00	8.50	7.65	7.50	7.00
1.50	0.4	8.75	8.25	7.50	7.35	6.80
1.75	0.4	8.50	8.00	7.30	7.10	6.60
2.00	0.4	8.30	7.80	7.25	7.05	6.50
2.25	0.4	6.00	7.50	7.00	6.85	6.30
2.50	0.4	6.00	6.70	6.80	6.65	6.15
2.75	0.4		5.75	6.75	6.50	5.85
3.00	0.4		4.75	6.60	6.35	5.80
3.25	0.4		4.05	5.20	6.20	5.60
3.50	0.4		4.00	5.05	6.00	5.40
3.75	0.4		4.00	5.00	5.70	5.05
4.00	0.4		6.00	6.00	5.70	5.05
4.25	0.4		6.00	6.00	5.50	4.90
4.50	0.4		6.00	6.00	4.90	4.70
4.75	0.4			6.00	4.70	4.50
5.00	0.4			6.00	4.60	4.35
5.25	0.4			6.00	4.55	4.25
5.50	0.4			6.00	4.50	4.10
5.75	0.4				6.00	4.05
6.00	0.4				6.00	4.00
6.25	0.4				6.00	4.00
6.50	0.4				6.00	4.00
7.00	0.4				6.00	4.00
7.50	0.4				6.00	4.00
8.00	0.4				6.00	4.00
10.00	0.5				6.00	4.00
11.00	0.5					4.00
13.50	0.5					6.00

Note: W_b = 3D and ℓ_1/ℓ_2 = 1:1.75.

Comparison of Rounded and Angular Rock

In this study the "hydraulic" performance of one type of rock compared to another is based on its stability to movement and ability to dissipate the flow energy.

The angular rock, as would be expected, is slightly more effective in dissipating energy, but the difference is so slight as to be insignificant. At any rate, when the tables were prepared for design purposes, the length specified for a proper velocity reduction is sufficient that rounded rock will accomplish this task.

Considering the superior ability of angular rock to interlock, one would expect the rounded rock to move before equivalent-sized angular rock. However, this was not noticeable in this study. The complicated interplay of lift, drag, gravity, and other body forces gives the rounded rock as good an advantage as the angular. This was partly explained when the specific gravity of the rock was determined. Even though the angular rock had a greater specific gravity, it weighed less per rock than the rounded. It follows that, if rock is selected to perform according to size, as in this study, rounded field rock will be as stable as angular quarried rock.

When the begin-scour or incipient motion of a rock-bed material as a function of $Q/D^{2.5}$ was considered as a criterion, the values of this study compare favorably with those given by Laushey (3). When compared on a velocity basis, the point of incipient motion agrees with the Isbash formula (2).

TABLE 5

LENGTH RATIO OF BASIN FOR VELOCITY REDUCTION RATIO OF 0.2

$Q/D^{2.5}$	d_1/D_{max}	L/D for $v/V_p = 0.2$				
		8 D/d	6 D/d	4 D/d	3 D/d	2 D/d
1.00	0.4	12.95	12.00	11.40	10.40	9.25
1.25	0.4	12.90	11.90	11.20	10.20	9.15
1.50	0.4	12.85	11.80	11.00	10.05	9.00
1.75	0.4	12.80	11.75	10.85	9.90	8.90
2.00	0.4	12.75	11.65	10.65	9.75	8.80
2.25	0.4	10.90	11.60	10.50	9.60	8.65
2.50	0.4	8.70	11.40	10.30	9.45	8.55
2.75	0.4		11.15	10.10	9.30	8.40
3.00	0.4		10.50	9.90	9.15	8.30
3.25	0.4		8.90	9.75	9.00	8.15
3.50	0.4		6.75	9.55	8.85	8.00
3.75	0.4		6.45	9.45	8.65	7.85
4.00	0.4		6.15	9.30	8.50	7.75
4.25	0.4		6.00	9.20	8.25	7.60
4.50	0.4		6.00	9.10	7.80	7.45
4.75	0.4			8.75	7.50	7.35
5.00	0.4			7.50	7.35	7.20
5.25	0.4			6.75	7.20	7.00
5.50	0.4			6.25	7.05	6.60
5.75	0.4				6.75	6.25
6.00	0.4				6.45	6.00
6.25	0.4				6.10	5.75
6.50	0.4				6.00	5.55
7.00	0.4				6.00	5.30
7.50	0.4				6.00	5.20
8.00	0.4				6.00	5.00
10.00	0.5				6.00	5.00
11.00	0.5					5.00
13.50	0.5					6.00

Note: $W_b = 3D$ and $\ell_1/\ell_2 = 1:1.75$.

Figure 4 shows that the larger rock size appears to dissipate the energy of the flow better than the smaller rock size. The spreading of the flow is more efficient with the larger rock size. A particular rock size cannot be expected to be stable beyond a certain value of $Q/D^{2.5}$. Beyond this value, some structure (such as a sill) is necessary for satisfactory stability and energy dissipation. This is significant for the use of graded rock. The design must be based on the smaller rock size when graded rock is used.

Sill Performance

When flow patterns were compared for various combinations of lengths and heights of sills, the final choice to be studied was a 6-in. sill length (W = 1-D) with a height of 1.8 in. (P = 0.3-D).

The experimenters were reluctant not to pursue further the study of some of the other sills. For example, a 9-in. sill with a height of 1.8 in. developed the most efficient energy-dissipating action in the form of a jump within the flared-end transition. However, with this type of action went some scouring effects on the highway side-slope area.

The 12-in. sill (W = 2-D), although also effective in producing a confined jump, was not studied in detail because there was no way silt and debris could be swept away around the sides at low flows.

The sill causes the jet to diverge more rapidly. The divergence angle that will encompass the jet streams and any resulting scour pattern with the sill is $\ell_1/\ell_2 = 1:1.75$. $W_B = 3$-D is still sufficient.

Although the sill was not effective in advancing the discharge factor at which incipient motion occurred, it did retard scouring with respect to a given flow without a sill. For a given flow the sill reduced the length of the distance to the end of scour and to the end of the dune. The sill was effective in reducing the length required to decrease the culvert velocity to a specified percentage when compared with riprap action and no-sill condition.

The 1-d scour hole requires a length of 4-D to contain it. The 2-d scour hole requires a length of 6-D. These are the same general lengths as were specified for scour with no sill. However, these holes with the sill are wider than without. The sill spread the energy out over a larger area. Even though the length of the scour holes had not changed considerably, slightly more volume of rock had been removed. The energy involved in producing a wider hole along with energy lost in the sill-defected jet results in an overall shorter basin for a given velocity reduction.

Comparisons of the amount of required riprap show that more than one-third less riprap is needed with a sill as compared with the same protection offered by riprap without a sill. The other advantage of the sill, of course, is that, for a given size of rock with sill, a greater flow can be allowed through a culvert for a given allowable scour.

DESIGN OF ROCK BASINS

Within the limits of the experimental results of this report, a basin formed of rock or rock and a sill can be designed to control scour and dissipate the flow to a specified average velocity tolerable by the downstream channel. Using the design tables and Figure 5 and knowing the culvert discharge and diameter and the desired downstream channel velocity, the designer can determine the proper length of the basin, divergence angle, size of rock, approximate basin depth of flow, whether a sill will be required, and whether a no-scour basin or a basin of controlled depth of 1-d or 2-d can be attained.

The depth of flow, as measured in the model culvert, and the subsequent velocity ratios v/V_p are referenced to a position 2.5-D upstream from the brink of the culvert (d_p, v_p). Studies were also limited to the model culvert in a horizontal (s = 0) position. Figure 5 has been prepared to enable the designer to enter the design tables for partly full pipe flow. Figure 5 also allows the designer to compare the brink depth of flow for the horizontal laboratory model pipe with the actual depth of flow at the brink of the culvert of the field case under study.

The following examples will best illustrate how the tables and Figure 5 are used to arrive at basic dimensions of a rock basin.

Example 1

Assume that, for a culvert, D = 3 ft, Q = 60 ft^3/sec, v_{ch} = 3 fps, and $Q/D^{2.5}$ = 60/15.6 = 3.85.

Figure 5 shows that the pipe flows partly full. $d_p/D \sim 0.9$, a/A = 0.95, A = 7.06 ft^2, A_p = (0.95) (7.06) $\sim$ 6.7 ft^2, V_p = Q/A_p = (60/6.7) $\sim$ 9 fps, and v_{ch}/V_p = 3/9 = 0.33. For $Q/D^{2.5}$ = 4.00, L/D = 9.45 for D/d = 3 (Table 3).

Therefore, with 12-in. rock the basin could have a length of about 30 ft, W_b = 9 ft, ℓ_1/ℓ_2 = 1:3, depth of flow on the basin is 9 in. (0.25-D), and no scour is expected. If a 9-in. rock is used (D/d = 4), 1-d scour would occur, but the basin would not have to be quite so long (L/D = 9.1). The action within the scour hole and over the dune would dissipate some energy. With 6-in. rock (D/d = 6), 2-d scour would occur, and the basin should be a little longer (L/D = 10.5). If 18-in. rock were available (D/d = 2), the basin could be shorter (L/D = 8.65), and no scour would occur.

A study of Figure 4 and the tables (around the regions between incipient motion and 1-d scour) shows that until a scour hole has developed the contribution of the hole and dune to dissipate energy is not available. Within these regions of flow, a lower discharge can actually require a longer basin than the design tables show for a higher discharge. This could be true until the higher discharge had formed the hole and dune.

Example 2

If the 60 ft ^{3}/sec in example 1 is for a 1- or 2-year occurrence and if a recurrence factor of 2 is assumed, then Q = 120 ft 3/sec, D = 3 ft, v_{oh} = 3 fps, and $Q/D^{2.5}$ = 120/15.6 = 7.7.

Figure 5 shows the pipe flows full. A_p = 7.06 ft^2, V_p = 120/7.06 = 17 fps, and v_{oh}/V_p = 3/17 ~ 0.18, say 0.2. The design tables show that, without a sill, 18-in. rock (D/d = 2) or larger would be needed to control the flow within the limits of 2-d scour or less. For $Q/D^{2.5}$ = 8.00, L/D = 6.00 for D/d = 3 (Table 5).

With a sill and 12-in. rock, the scour could be kept at 2-d. The sill with 18-in. rock could control the scour to 1-d depth, and the basin might be shortened to a length of 5.00-D. The maximum expected depth of flow on these basins would be 0.4-D. The divergence angle with the sill is 1:1.75. The sill has a length of 1-D (3 ft) and a height of 0.3-D (0.9 ft); W_b = 3D = 9 ft. If the rock in this example is larger than available or the scour greater than desired, then a larger culvert would be needed.

Example 3

Assume that, for a culvert, Q = 120 ft ^{3}/sec, D = 4 ft, v_{oh} = 3 fps, and $Q/D^{2.5}$ = 120/32 = 3.75.

Figure 5 shows that the pipe flows partly full. d_p/D = 0.9, a/A = 0.95, A = 12.6 ft^2, A_p = (0.95) (12.6) = 11.95, V_p = 120/11.95 ~ 10 fps, and v_{oh}/V_p = 0.3.

For $Q/D^{2.5}$ = 3.75 and with a sill for 1-d scour and 8-in. rock (D/d = 6), L/D = 4.00; for no scour and 16-in. rock (D/d = 3), L/D = 5.70 (Table 4). d_1 = 0.4D, W_b = 3D, ℓ_1/ℓ_2 = 1:1.75, and W = 4 ft.

For $Q/D^{2.5}$ = 3.75 and with a sill for 1-d scour and 8-in. rock (D/d = 6), L/D = 4.00; for no scour and 16-in. rock (D/d = 3), L/D = 5.70 (Table 4). d_1 = 0.4D, W_b = 3D, ℓ_1/ℓ_2 = 1:1.75, and W = 4 ft.

For $Q/D^{2.5}$ = 3.75 and without a sill for 2-d scour and 8-in. rock, L/D = 10.60; for 1-d scour and 12-in. rock, L/D = 9.40; for no scour and 16-in. rock L/D = 9.50 (Table 3). d_1 = 0.25D, W_b = 3D, and ℓ_1/ℓ_2 = 1:3,

Comparing the various rock sizes used in the basin of this flow, with and without a sill, shows the effectiveness of the sill in dispersing the jet and reducing the required length of basin.

SUMMARY AND CONCLUSIONS

Relevant to this study is the attempt to simplify the design procedure as much as possible and still keep within the realm of significance of the governing experimentally determined data. The following conclusions are made on the basis of the laboratory investigations with quarried angular rock and rounded field rock, in basins without a sill and in basins with a sill, acting at the outlet of 1.75-, 3- and 6-in. diameter circular culverts on a zero slope with a standard flared end.

1. The upstream width of the rock basin should be at least 3-D;

2. The divergence angle of the rock basin downstream from the standard flared end should be 1:3 when no sill is used and 1:1.75 when a sill is used;

3. The proper length of the basin is governed by criteria given in Tables 1 through 5 [this length is dependent on the culvert discharge, culvert diameter, size of rock, extent of permissible scour (no scour, 1-d scour, or 2-d scour), and whether a sill or no sill is to be used in conjunction with the basin at the end of the culvert flared end];

4. If rock is selected according to size by reference to square openings, and if the rock length is about equal to its width and thickness, then there is no significant difference between rounded field rock and angular quarried rock when scour stability and energy-dissipation ability are compared;

5. For given high flows, a sill of a length equal to D and a height of 0.3-D proves more effective in reducing scour and culvert velocities in a length of basin shorter than when no sill is used; and

6. Similitude of performance of rock basins can be attained with small culver models (1.75, 3, and 6 in. and rock sizes from ½ to 3 in (this similitude is apparently independent of Reynolds number).

REFERENCES

1. Argue, J. R. New Structure for Roadway Pipe Culverts. Jour. Institution of Engineers, Vol. 132, No. 6, June 1960.

2. Isbash, S. V. Construction of Dams by Depositing Rock in Running Water. Trans. Second Congress on Large Dams, Vol. 5, 1936, pp. 123-136.

3. Laushey, L. M. Design Criteria for Erosion Protection at the Outlet of Culverts., Univ. of Cincinnati, Sept. 1966.

4. McPherson, M. B., and Meredith, D. D. Factors in the Design of Culvert Energy Dissipators. Univ. of Illinois, 1963.

5. Mehrotra, S. C. Scale Effects in Model Tests of Rock Protected Structures. Univ. of Iowa, MS thesis, June 1967.

6. Parkin, A. K., Trollope, D. H., and Lawson, J. D. Rockfill Structures Subject to Water Flow. Jour. Soil Mech. and Found. Div., Proc. ASCE, Vol. 94, No. SM4, July 1968, p. 1036.

7. Sherard, J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A. Earth and Earth-Rock Dams. John Wiley and Sons, New York, 1963.

8. Stevens, M. A. Scour in Riprap at Culvert Outlets, Colorado State Univ., Fort Collins, PhD dissertation, 1969.

9. Design of Roadside Drainage Channels. U. S. Bureau of Public Roads, Hydraulic Engineering Circular 6, April 1962.