ANALYSIS OF RIGID OUTFALL BASINS WITH HIGH TAILWATER

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Diffusion characteristics of jets from circular pipes discharging into basins lined with stones were measured under conditions of tailwater either slightly above or slightly below the crown of the pipes. These data together with data from a previous study on culvert outlet protection and with data from orifice jet diffusion studies by others are incorporated into a method for designing stable energy-dissipating basins at culvert outlets where high tailwater exists.

•HIGH TAILWATER is defined as the condition where the water surrounding the high-speed, jet-like core of water discharging from the culvert outlet is as high as or higher than the elevation of the crown of the pipe. This situation occurs at culvert outlets where downstream channel constrictions create backwater or where the culvert discharges into a narrow, low-gradient channel with high banks and a large normal depth.

Unknowns that confront the engineer faced with the problem of designing a stable energy-dissipating basin where high tailwater conditions prevail are the rate of decay of the high-speed velocity core, the rate of lateral expansion of the core, and the probability of the core being diverted off to one side, thus imperiling the banks.

The problem of 2- and 3-dimensional jets discharging into a large volume of quiescent ambient fluid has been studied in detail (1, 2, 3). The purpose of this study was to determine the diffusion characteristics of a jet bounded on the top by a free surface and on the bottom by a rough (rock-lined) essentially rigid boundary. Data obtained during this study correlated well with data presented in another report (1) for the 3-dimensional orifice flow field. The remainder of this report describes the tests conducted and the data collected, presents a comparison of these data with results presented elaswhere (1, 2, 3), and illustrates the application of these results to the solution of practical problems.

EXPERIMENTAL APPARATUS AND PROCEDURE

Three arrangements of culvert and basin configurations were examined. All basins were constructed within a large 185 ft by 20 ft by 8 ft deep outdoor flume equipped with a movable overhead instrument carriage. A smooth circular approach pipe, 1.45 ft in diameter, was used for runs F44 and F45. The basin was approximately 25 ft long with a horizontal floor 6 ft wide and side berms 1 ft high parallel to the centerline and sloping 1 on 2. This condition allowed bank over flow. The basin was constructed of river-rounded rock ranging in size from 4 to 10 in. in diameter with a D_{50} of 7 in. The floor of the basin was placed at approximately the elevation of the pipe invert.

Measurements were taken at 2 discharges: 22.5 ft^3/sec for run F44 and 14.6 ft^3/sec for run F45. The water surface in the basin was maintained about 1.57 ft above the pipe invert; i.e., the crown of the pipe was about 0.12 ft below the average water surface.

Velocity profiles were measured along the centerline of the basin at stations 0.0, 5.0, 10.0, and 20.0. Station number indicates the distance in feet downstream of the culvert outlet. Additional velocity measurements were obtained at stations 15 and 20 for the purpose of constructing isovel plots. This information is shown in Figures 1 and 2.

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Point velocities were measured with an Ott minor meter. A reliable mean value was obtained by sampling each point for a period of 50 sec. The meter, supported on a point gage, was mounted horizontally with the axis parallel to the longitudinal centerline of the basin. In all runs, the basin was slowly filled to the specified height; the flow was then increased until the desired discharge was obtained.

For runs G56 and G57, a 3-ft diameter smooth pipe was used. The basin was 35 ft long and 20 ft wide with parallel vertical walls 6 ft high. The bed of the basin was constructed with the same rounded rock material as was used for runs F44 and F45. The discharges tested were 65.4 ft^3/sec in run G56 and 84.0 ft^3/sec in run G57. The water surface was maintained at a level 3.05 ft above the pipe invert. A centerline velocity profile was obtained at the outlet, station 0.0. Sufficient velocity data were taken at stations 5.0, 10.0, 15.0, 20.0, and 25.0 to construct the isovel plots shown in Figures 3 and 4.

For runs K70 (20.9 ft^3/sec) and K71 (13.9 ft^3/sec), the 1.45-ft diameter smooth steel approach pipe was used. The basin berms were removed, and the floor was lowered approximately 0.3 ft below the invert. The horizontal floor of the basin was 20 ft wide and 35 ft long, with parallel vertical walls. The rounded material used to construct the basin was much smaller than that used for the previous 4 runs. The D_{50} was slightly under 5 in. with a D_{max} of 7 in.; i.e., the D_{50} rock in this series weighed about one-fourth as much as the rock used for the previous series.

Instead of the tailwater being held at crown elevation or higher as had been the case for the previous runs, the surface was maintained at an elevation approximately 0.2 ft below the crown of the pipe. Velocities were measured at stations 2.5, 5.0, 7.5, 10.0, 12.5, and 15.0, and for run K71 at station 20.0. During run K70, deposition downstream of the scoured region distorted the flow field and, therefore, measurements were not completed at station 20.0. The information collected is shown in Figures 5 and 6.

PRESENTATION OF DATA

Figures 1 through 6 show data collected for the 6 runs. All plots are to scale, with the appropriate scales appearing on the drawings. A section along the longitudinal centerline of the basin is shown on the upper portion of each figure. The water surface elevation, centerline profile of the bed, and the vertical distribution of velocity at the centerline are shown for the various sections. Plots of isovels were constructed from the measured data. The small filled circles indicate points where measurements were taken. The large dotted circle shows the position of the approach pipe relative to the section. The velocity profile in a horizontal plane at an elevation D/2 above the pipe invert is plotted as a solid line directly above each isovel section. The theoretical velocity profile based on the mean exit velocity and the approach pipe diameter is shown as a dashed line. This profile was computed by using data shown in Figures 9 and 10 and the measured mean velocities at each station. Three facts are apparent from the various plots.

1. Lowering the tailwater only one-seventh of the approach pipe diameter allowed the jet to plunge in such a manner as to cause significant scour. How much of this scour resulted from the plunging effect and how much resulted because of the smaller rock are not known; however, the slope of the water surface indicated the jet was directed toward the floor.

2. Where the jet discharged into the low tailwater basin, the location of the core of maximum velocity is at the surface, whereas the location is at mid-depth or lower for the high tailwater basins.

3. The theoretically predicted velocity profiles are in good agreement with measured values for both tailwater conditions. Thus it is apparent that data shown in Figures 9 and 10 used in conjunction with those shown in Figure 8 are adequate criteria for computing transverse velocity distribution.

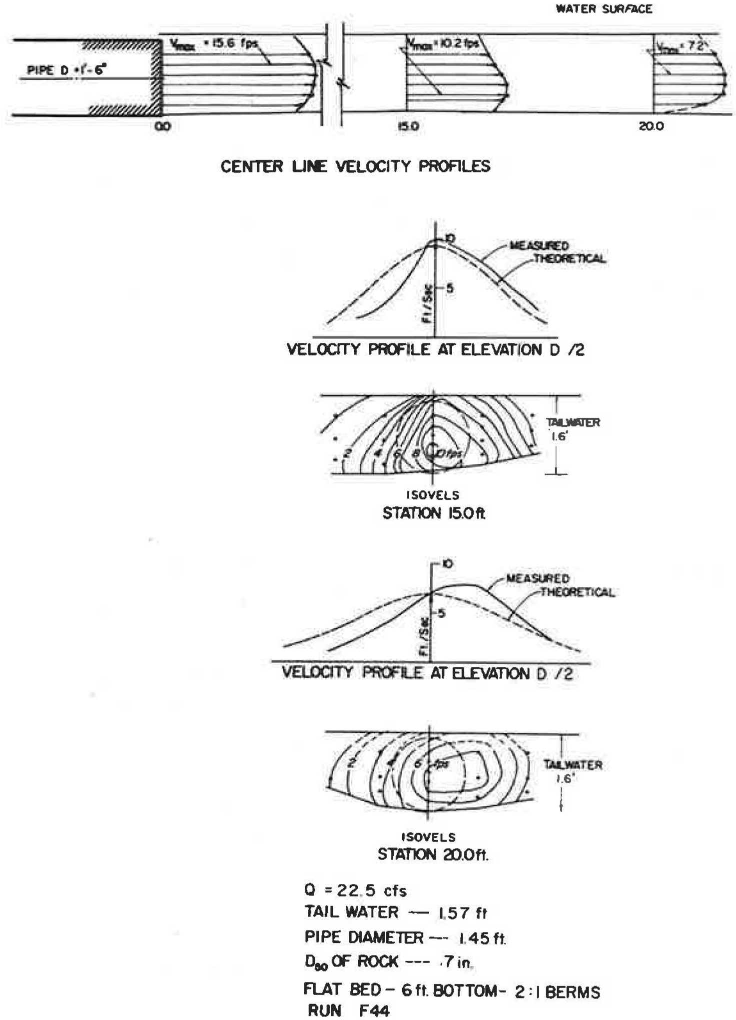


Figure 1. Centerline velocity profiles, isovels, and transverse velocity profiles for run F44.

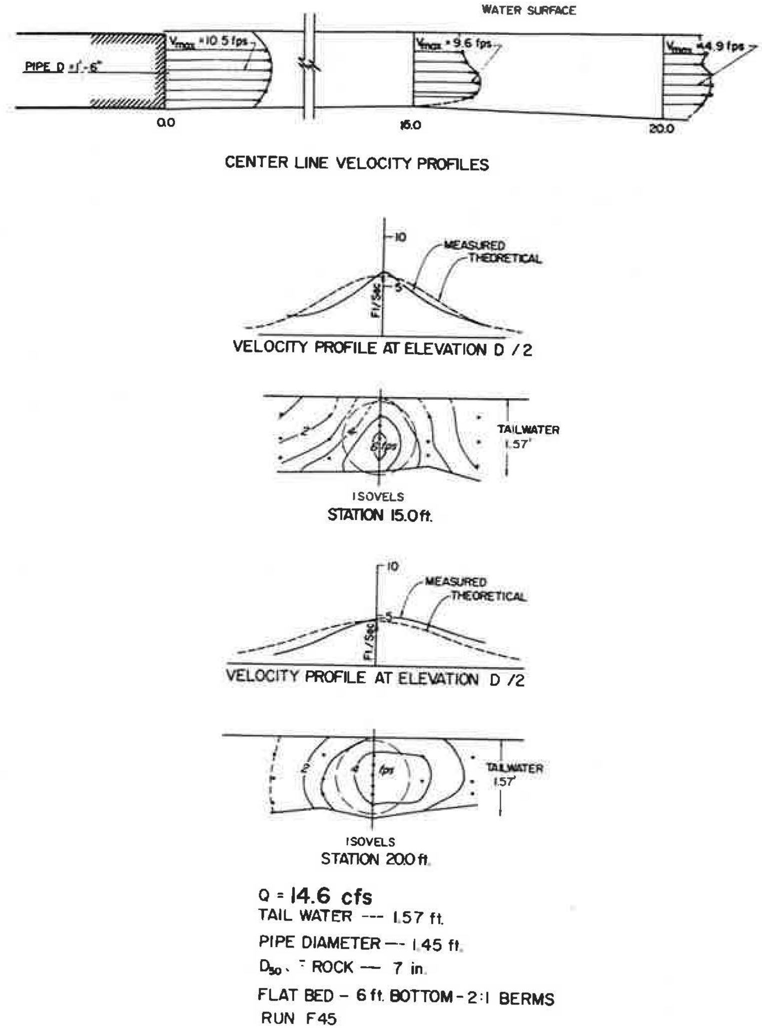


Figure 2. Centerline velocity profiles, isovels, and transverse velocity profiles for run F45.

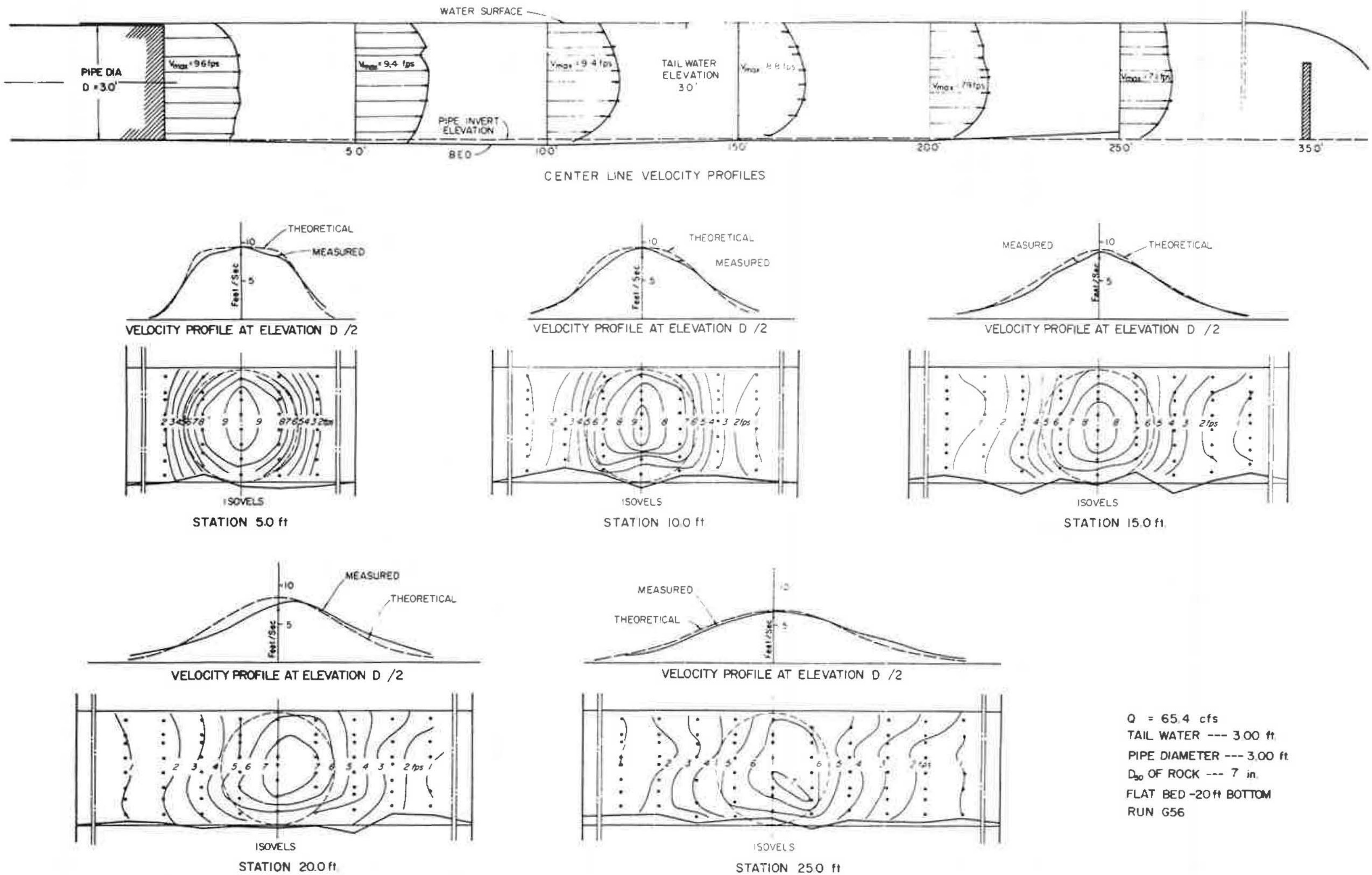


Figure 3. Centerline velocity profiles, isovels, and transverse velocity profiles for run G56.

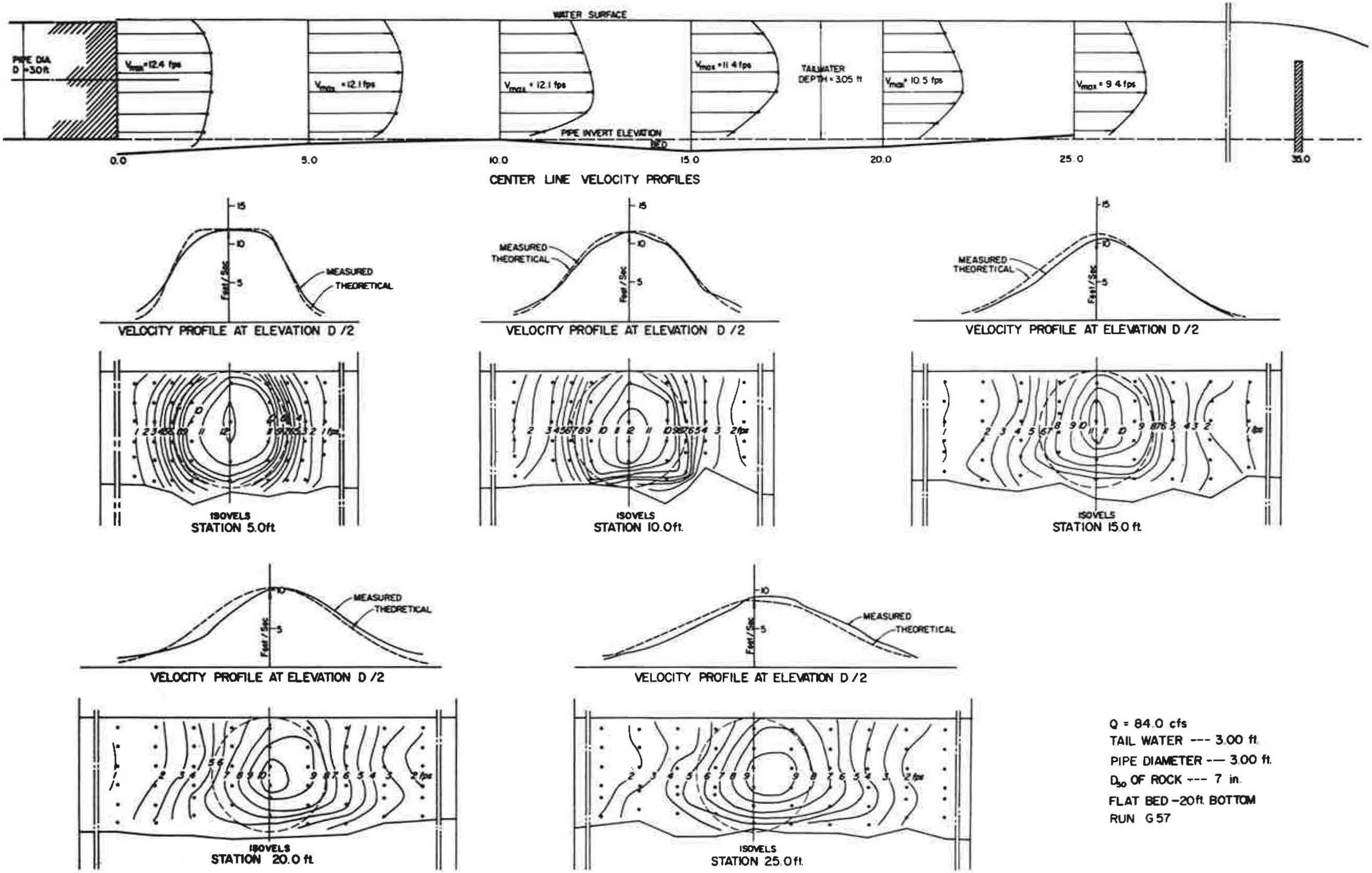
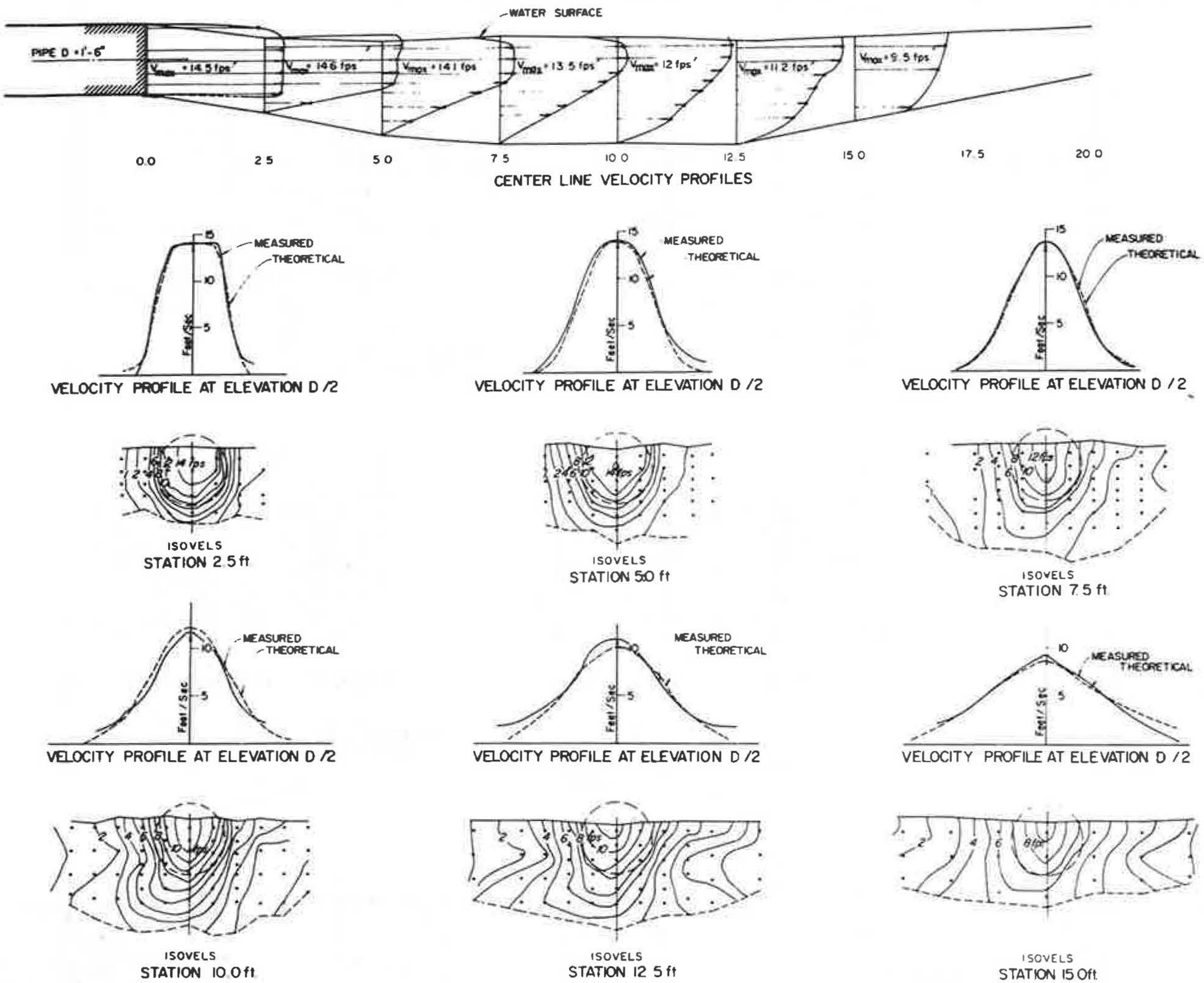


Figure 4. Centerline velocity profiles, isovels, and transverse velocity profiles for run G57.



Q = 20.9 cfs
TAIL WATER --- 1.57 ft
PIPE DIAMETER --- 1.45 ft
D_{50} OF ROCK --- 7 in
FLAT BED – 6 ft BOTTOM-2:1 BERMS
RUN K70

Figure 5. Centerline velocity profiles, isovels, and transverse velocity profiles for run K70.

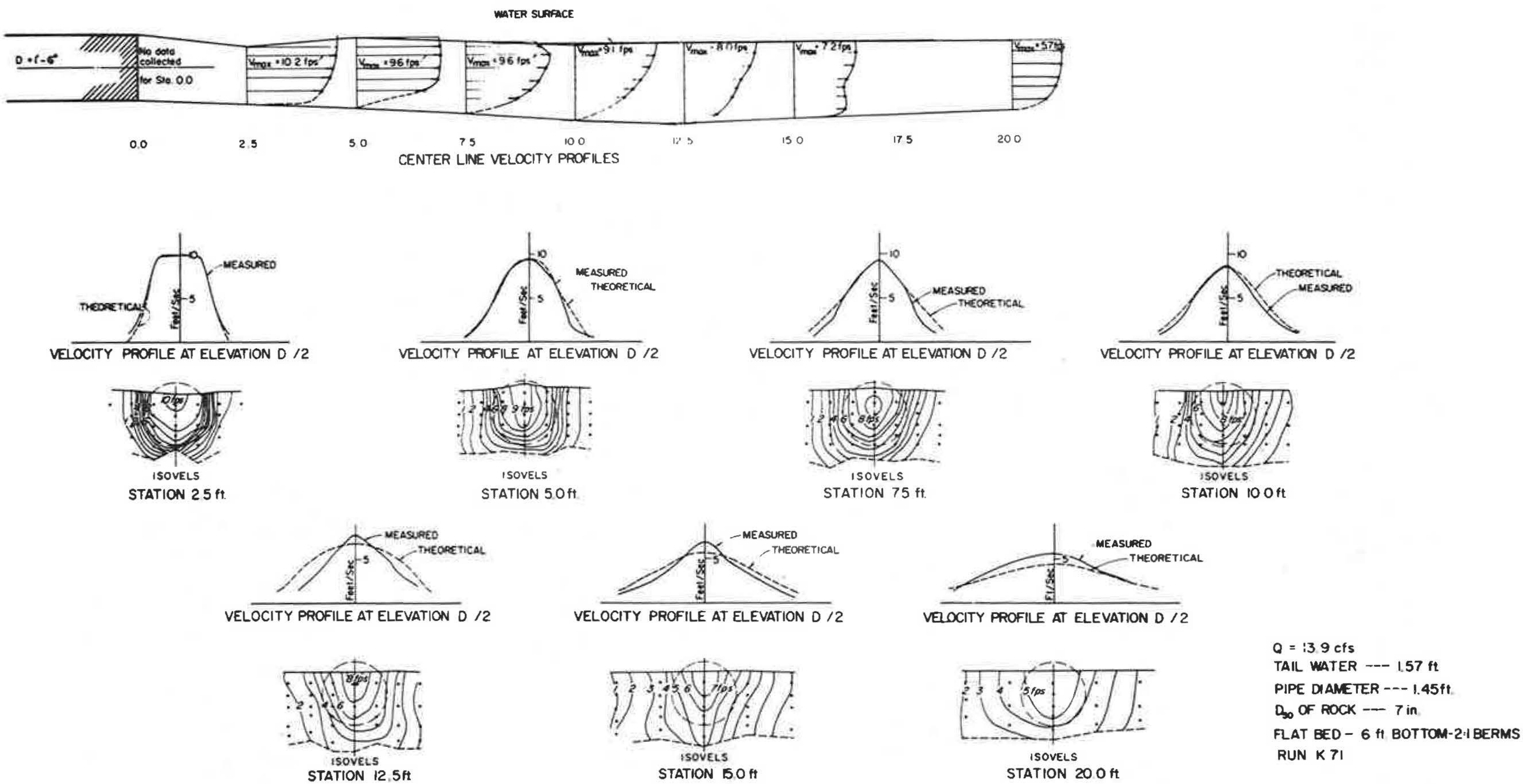


Figure 6. Centerline velocity profiles, isovels, and transverse velocity profiles for run K71.

RESULTS

The data collected downstream of the culverts appeared to correlate closely with the data presented by Albertson et al. for the 3-dimensional orifice flow field (1). The 3-dimensional orifice flow field was divided into 2 zones: the zone of flow establishment adjacent to the outlet and the zone of established flow (Fig. 7). For each of the zones, Albertson et al. presented the following relationships (1):

(a) Distribution of centerline velocity for flow from orifice,

$$V_{max}/V_o \text{ versus } X/D$$

where

V_{max} = maximum longitudinal velocity at a normal section,
V_o = mean velocity at the outlet section,
X = distance downstream from the outlet, and
D = diameter of the outlet pipe.

(b) Distribution of longitudinal velocity in zone of establishment of flow from orifices,

$$(r - D/2)/X \text{ versus } V_x/V_o$$

where

r = radial distance normal to the longitudinal centerline of the basin, and
V_x = longitudinal velocity at point (X, r).

(c) Distribution of longitudinal velocity in zone of established flow from orifices,

$$r/X \text{ versus } (V_x/V_o)\ (X/D)$$

Other significant plots were presented, but only those that relate to the problem at hand are mentioned here. The reader is referred to another report (4) for additional plots and analysis.

Figure 8 shows a comparison of the data collected downstream of the culvert outlets (6 runs) and those plotted in the other reports (1, 2).

Because the velocity distribution at the culvert outlet is nonuniform in contrast to the uniform distribution for the orifice, it seemed more reasonable to compare the arithmetic mean of the velocities measured along a centerline vertical at station X, $V_{x\,ave}$, with an arithmetic mean of the velocities measured along a centerline vertical at the outlet, $V_{o\,ave}$. The maximum velocity for an orifice is equal to the mean velocity, which is not the case for usual pipe flow. The data collected during this study are superimposed over the prediction curve shown in Figure 8. In the range X/D < 8.0, the prediction curve is conservative with the exception of the data for the low tailwater runs. For the range X/D > 8, the culvert data follow the prediction curve.

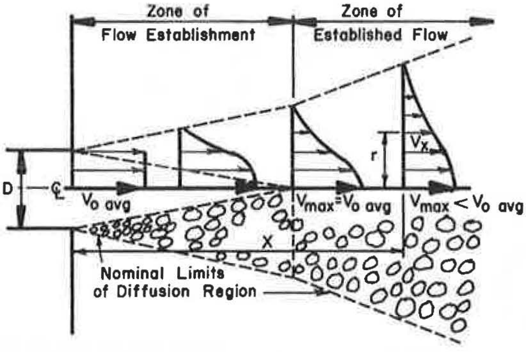
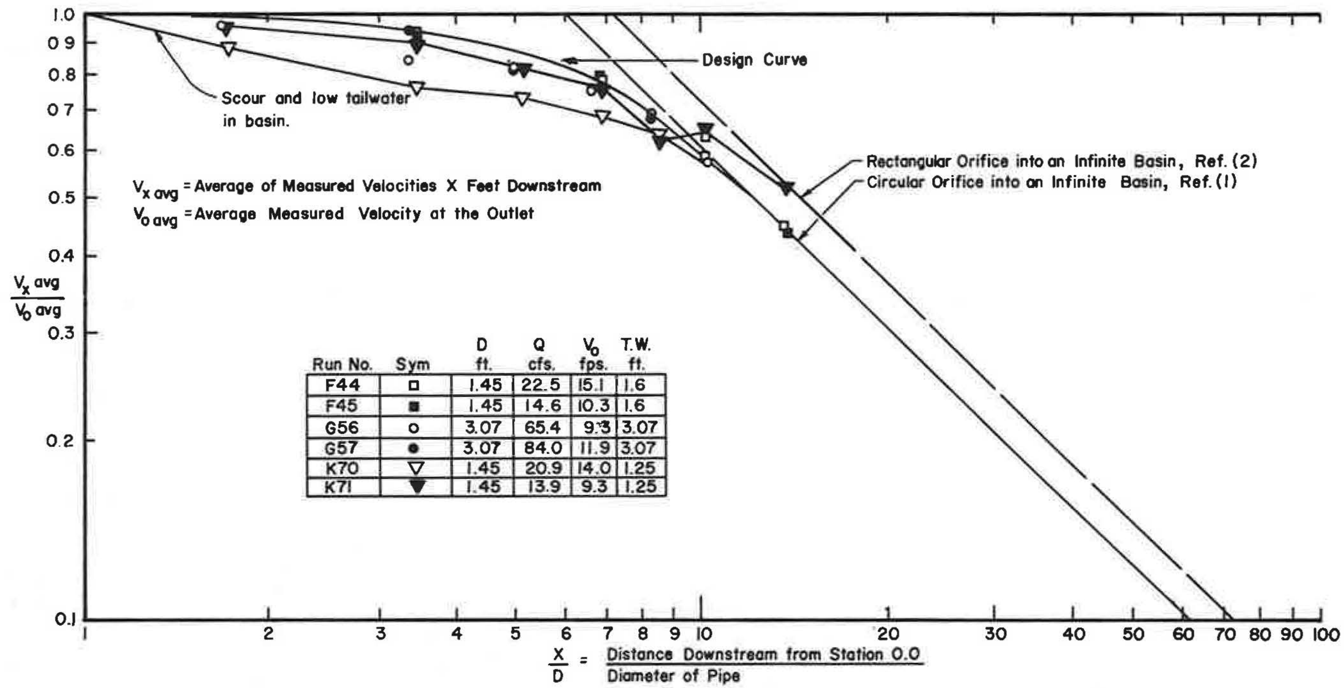


Figure 7. Zone of flow establishment and zone of established flow.

DESIGN CONSIDERATIONS

The curves recommended for design purposes are those shown in Figure 8 used in conjunction with those shown in Figures 9 and 10. The $V_{o\,ave}$ to be used with data shown in Figure 8 for basin design can be obtained by using the formula

$$V_{o\,ave} = KQ/A \qquad (1)$$



Run No.	Sym	D ft.	Q cfs.	V_o fps.	T.W. ft.
F44	□	1.45	22.5	15.1	1.6
F45	■	1.45	14.6	10.3	1.6
G56	○	3.07	65.4	9.3	3.07
G57	●	3.07	84.0	11.9	3.07
K70	▽	1.45	20.9	14.0	1.25
K71	▼	1.45	13.9	9.3	1.25

Figure 8. Distribution of centerline velocity from submerged outlets.

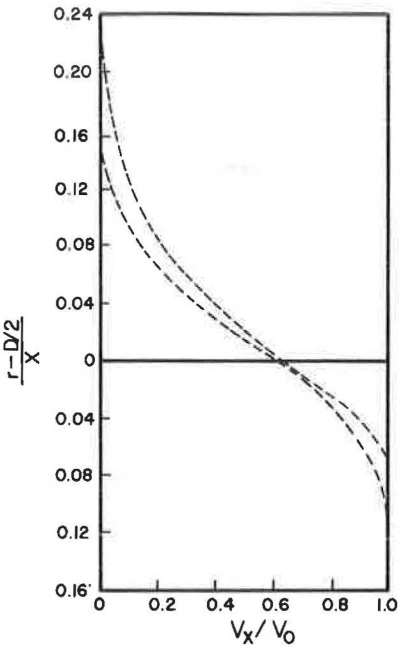


Figure 9. Distribution of transverse velocity for X/D < 6.

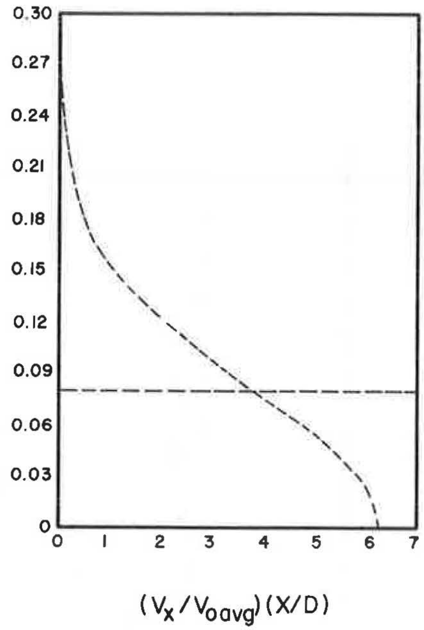


Figure 10. Distribution of transverse velocity for X/D > 6.

where Q is the design discharge, A is the gross cross-sectional area of the culvert, and K is a constant relating Q/A to the arithmetic mean of the vertical velocity profile. Values of K, obtained from 34 runs with smooth pipe having 18-in. and 36-in. diameters, ranged from 0.96 to 1.16 with an arithmetic mean of 1.07. It was not possible to correlate these values with Froude number or other dimensionless parameters; therefore, the value K = 1.1 is suggested for design purposes for smooth pipe.

Only 2 sets of data were available for corrugated pipe. The values of K were 1.14 and 1.21 with the former value associated with a typical maximum design discharge and the latter value with a Q well over the usual design discharge. It is suggested that K = 1.15 be used for corrugated pipe.

Whether the core of the jet is diverted to one side of the basin seems to depend on the ratio of the basin width, W_b, to the pipe diameter. With a large ratio, there is little danger of such an occurrence, but when $W_b/D \leq 4$ jet attachment to a bank or wall is a possibility. Data from this study do not adequately define the ratio where jet attachment will first occur.

High Tailwater Basins

There are 2 solutions to the scour problem for the high tailwater cases. One is to riprap the banks for a sufficient distance downstream, and the other is to increase the cross-sectional area of the culvert so that the exit velocity is tolerable and little scour occurs downstream of the outlet. If culvert flare is sufficiently gradual, the entire section will be occupied by the flow, and this will result in a low exit velocity; with large flare angles, the flow will separate from one wall and a large eddy in the basin will hold the flow against the other wall. The following example illustrates design techniques.

For a high tailwater basin, discharge, Q = 330 ft^3/sec; tailwater, d_t = 7 ft; and smooth pipe diameter, D = 6 ft. The task is to compute (a) the rock size required to prevent scour and (b) the maximum velocity in the channel 60 ft downstream of the outlet.

The design parameters are as follows: $Q/D^{2.5} = 330/6^{2.5} = 330/88.3 = 3.74\ ft^{1/2}/sec$; $d_t/D = 7/6 = 1.16$; and $y_o/D = 6/6 = 1.00$.

The rock size, d_m, required to prevent scour below culverts is given elsewhere (6, 7). In this case $d_s/D \simeq 0$, where d_s is the depth of scour, for $d_m/D = 0.1$ (smallest recommended size). Therefore, $d_m = 0.1 \times 6 = 0.6$ ft is used. With a smooth pipe, K = 1.1 and $V_{o\,ave} = K(Q/A) = 1.10 \times (Q/D^{2.5}) \times [D^{2.5}/(\pi/4)D^2]$ or $V_{o\,ave} = 1.4(Q/D^{2.5}) \times \sqrt{D} = 1.4 \times 3.74 \times 2.45 = 12.8\ ft^3/sec$. $X/D = 60/6 = 10$. Figure 8 shows that $V_{x\,ave}/V_{o\,ave} = 0.6$ when $X/D = 10$. Therefore, at a distance 60 ft downstream, mean velocity on the centerline vertical is given by $V_{x\,ave} = 0.6 \times 12.8 = 7.7\ ft^3/sec$. At a distance $D/2 = 3$ ft above the bed, the velocity distribution can be estimated by using data shown in Figure 10. $V_{o\,ave}D/X = (12.8 \times 6)/60 = 1.28$.

Values of $(V_x/V_{o\,ave})(X/D)$, such at 6, 5, and 4, are used to obtain values of r/X from data shown in Figure 10, and then r and V_x are computed. The following results are obtained.

$(V_x/V_{o\,ave})(X/D)$	r/X	r = X(r/X)	$V_x = (V_x/V_{o\,ave})(X/D)(V_{o\,ave}D/X)$
6	0.03	1.8	7.7
5	0.06	3.6	6.4
4	0.075	4.5	5.1
3	0.100	6.0	3.8
2	0.13	7.8	2.5
1	0.17	10.3	1.8
0	0.24	14.4	0

The velocity profile at X = 60 ft and at a distance D/2 = 3 ft above the bed is shown in Figure 11. In general when $V_{x\,ave}$ (mean velocity at centerline vertical) has been decreased to a value 1.5 times the velocity that is compatible with the downstream channel, the basin can be terminated. In the example, the basin could be terminated at 60 ft if the downstream channel could withstand a velocity of $7.7/1.5 = 5.1\ ft^3/sec$.

A 6- by 6-ft box culvert carrying 420 ft^3/sec would have the same velocity at the submerged outlet and the same $V_{x\,ave}$ 60 ft downstream if it is assumed that $X/W_o \simeq X/D = 60/6 = 10$. To predict the size of rock needed to prevent scour below box culverts, one can use the data published in another report (7).

Nonscouring, Low Tailwater Basins

The problem of describing expanding jets on rigid rock floors is similar in many respects to that of jets discharging into an infinite basin of fluid. However, with the rough floor there is a large decrease in fluid momentum in the downstream direction caused by the dynamic force of the fluid on the rock. With low tailwater, the force should be even more pronounced.

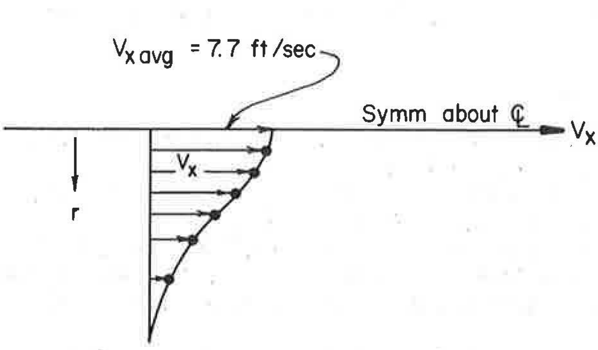


Figure 11. Transverse velocity profile.

The Colorado State University study does not adequately describe the downstream decay of velocity within nonscouring rock basins when the tailwater is below the pipe invert. However, some observations are given in order to supply a design criterion until more detailed studies are undertaken.

When the tailwater level is below the crown of the culvert outlet, the flow plunges onto the bed and spreads laterally very rapidly. The lateral expansion can be described by the angle θ (Fig. 58, 6). Watts (4) shows that θ is approximately 3 deg when $d_t/D \geq 1$ and the bed slope is zero. The data were crude and scattered around the curve that is shown. Moreover, the expansion of such a jet is too complex to be accurately described by only 3 variables: θ, slope, and d_t/D.

However, continuing with this model, at any point downstream of the outlet, the width of the jet is $W_j = 2X \tan \theta + D$, and the average velocity of the jet is

$$V_{ave} = Q/[2X \tan \theta + D)y]$$

where y is the difference between the elevation of the bed and the elevation of the tailwater at point X. When the bed is horizontal, $y = d_t$ for all X.

When V_{ave} is reduced to a level compatible with the downstream channel, the rock riprap can be terminated. V_{ave} will be about $\frac{2}{3}$ of the maximum velocity on the centerline at any section X.

The following example is given.

For a low tailwater, nonscouring basin, discharge, $Q = 50$ ft^3/sec; barrel diameter, $D = 3$ ft; tailwater depth, $d_t = 1$ ft; brink depth, $d_b = 2.0$ ft; barrel slope, $S = 1$ percent; and available rock size, $d_m = 1.3$ ft.

The task is to design a nonscouring basin for a plain outlet. The design parameters are as follows: $Q/D^{2.5} = 50/3^{2.5} = 50/15.6 = 3.2$ ft$^{1/2}$/sec; $d_t/D = \frac{1}{3} = 0.33$; $d_b/D = \frac{2}{3} = 0.66$; $d_m/D = 1.3/3 = 0.43$; and $d_t/d_b = \frac{1}{2} = 0.50$.

According to Simons et al. (Figs. 51 and 52, 6), no scour will occur when this riprap is used. $\tan \theta = 0.18$ for $S = 1$ percent and $d_t/\bar{d_b} = 0.50$ (Fig. 58, 6).

The depth of flow at any section X downstream is $Y = d_t + SX/\overline{100} = 1 + X/100$ and so $V_{ave} = Q/\{(2X \tan \theta + D) [1 + (X/100)]\} = 50/\{(0.36 X + 3) [1 + (X/100)]\}$. If V_{ave} is to be reduced to 2 ft^3/sec, $X = 41$ ft. The width of the jet for $X = 41$ ft is $W_j = (0.36 \times 41 + 3) = 18$ ft. Thus, the minimum basin dimensions are $L = 41$ ft and $W_b = 18$ ft.

The same procedure may be employed with the metal end section except that D would be replaced by the width of the end section, 2D, in the equation. That is, $V_{ave} = Q/[(2X \tan \theta + 2D)y] = Q/[2(X \tan \theta + D)y]$.

The new expression for V_{ave} is valid provided $d_t/D \leq 0.4$; otherwise, with higher tailwater, the jet will not expand in the end section, and the computations would be carried out as for a plain outlet.

CONCLUSIONS

A method for the analysis and design of energy basins at culvert outfalls where the high tailwater prevails is presented. Velocity-predicting equations used in conjunction with experimentally derived design aids (5, 6, 7) can be used to proportion a satisfactory basin.

ACKNOWLEDGMENTS

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REFERENCES

1. Albertson, M. L., Dai, Y. B., Jensen, R. A., and Rouse, H. Diffusion of Submerged Jets. ASCE Trans., Vol. 115, 1950, pp. 639-697.
2. Yevjevich, V. M. Diffusion of Slot Jets With Finite Orifice Length-Width Ratios. Colorado State Univ., Fort Collins, Hydraulics Paper 2, 1956.
3. Murota, A., and Muraoka, K. Turbulent Diffusion of the Vertically Upward Jet. Proc., Twelth Congress of IAHR, Colorado State Univ., Fort Collins, Vol. 4, Part 1, 1967.
4. Watts, F. J. Hydraulics of Rigid Boundary Basins. Colorado State Univ., Fort Collins, PhD dissertation, 1968.
5. Stevens, M. A. Scour in Riprap at Culvert Outlets. Colorado State Univ., Fort Collins, PhD dissertation, 1969.

6. Simons, D. B., Stevens, M. A., and Watts, F. J. Flood Proctection at Culvert Outlets. Colorado State Univ., Fort Collins, Publ. CER 69-70 DBS-MAS-FJW4, 1970.
7. Stevens, M. A., Simons, D. B., and Watts, F. J. Riprapped Basins for Culvert Outlets. Paper presented at the 50th Annual Meeting and published in this RECORD.