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Bulletin 286

Drainage Structures

Design and Performance, 1960

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Contents

Laboratory Study of Spur Dikes for Highway Bridge Protection

USUMU KARAKI, Assistant Research Engineer, Civil Engineering Department,
'olorado State University, Fort Collins, Colorado

This research study of spur dikes was sponsored by the State Highway Departments of Mississippi and Alabama in cooperation with the Bureau of Public Roads, Washington, D. C. It was conducted for the purpose of determining the value of spur dikes as protection for bridge abutments and to determine the relationships between the various geometric parameters.

The investigation was made in two stages; first, the effectiveness of the spur dike for reducing scour was demonstrated, the location and shape determined, and second, criteria were established for determining the length of dike required at a particular location. The results are qualitative and restricted by the limitations of the study.

PROTECTION of bridge abutments and piers from scour during floods has long posed problem to bridge engineers. Bridge failures by scour could be prevented if bridges ere constructed to span the entire channel with no obstruction in the channel. This ethod, of course, would be impractical and expensive. Bridges could also be proected if the foundations extended to sufficient depths to avoid undermining by scour. lthough this is perhaps a better solution in most instances, knowledge of scour phenoenon has not yet advanced to a stage where reliable predictions of scour depths can e made.

Scour at bridge abutments is caused primarily by flow concentrations and turbulence. has been found that flow concentrations at the abutment can be reduced by streamning the approach to the bridge opening with spur dikes located at the abutments. Spur ikes are guides to direct the flow properly through the bridge while distributing the ow across the opening and making the entire passage a more efficient waterway.

Spur dikes have been used in a number of states. Some, as in Georgia have been onstructed of timber; others, as in Pennsylvania, have been constructed with rockll embankments. In Missouri, Mississippi and Alabama, they have been constructed ith earth fill embankments. In all cases, the chief purpose of the spur dikes is to rotect the bridge foundations from scour by reducing high local velocities and preventg excessive turbulence and eddy formation.

Despite the numerous and varied types of construction of spur dikes, there is still n apparent lack of adequate criteria to be used as guides to proper design. It is perps for this reason that spur dikes are not more frequently used, for certainly the ost of spur dikes in most cases is small compared to the total cost of the bridge or e entire highway project. To establish criteria for design of spur dikes, the sponsors rranged for a research study to be conducted at the Hydraulics Laboratory of Colorado ate University. The study was conducted in two stages: The first stage was to determine the effectiveness of spur dikes and the important variables to be considered in detailed study. The second stage was to establish criteria, however tentative, as a guide to design. The entire study was primarily qualitative in nature; that is, the models show where scour will probably occur but cannot be scaled to indicate how dee the scour might go for prototype conditions.

Recognizing that wide stream channels consist of two parts, a main channel and flood plains for overbank flow, this research was limited to study of spur dikes for abutments on the flood plain away from the main channel. This paper is a report on these model studies, and the results can be used as a guide for design where distribu-tion of flow on the flood plain is reasonably uniform.

LABORATORY EQUIPMENT

Flume

The laboratory study was conducted in a flume 16 ft wide and 84 ft long (Fig. 1). I consisted of two sections of flume, each 32 ft long, separated by a recessed section 4 ft deep and 20 ft long for the purpose of providing scour depth at the test section. The bed of the flume consisted of sand to form an erodible bed with a fixed slope of 0.0003 Water was supplied to the flume by a 14-in. pump and recirculated. Discharge mea-surements were made with a flat plate orifice and a standard differential air-water manometer.

As the study progressed it became desirable to establish concentrated flow along th roadway embankment. This was accomplished by constructing a separate inflow to th flume at one side of the test section. Water was supplied to the side box by an 8-in. pump connected to the same recirculating system.

Models

Highway embankment models were made 1 ft wide at the top and the roadway was placed 0.6 ft above the flume bed. The embankment side slopes were 1½:1. The spu dikes were of both erodible and non-erodible types. For the initial and latter part of the study involving riprap, erodible dikes were used. All dikes were 3 in. wide at th top and constructed to the same height as the roadway embankment. Side slopes of th dikes were 1½:1 except for the riprap studies where 2:1 slopes were studied to obser effects of undercutting. Riprap for the dikes consisted of ⅜-in. median size gravel

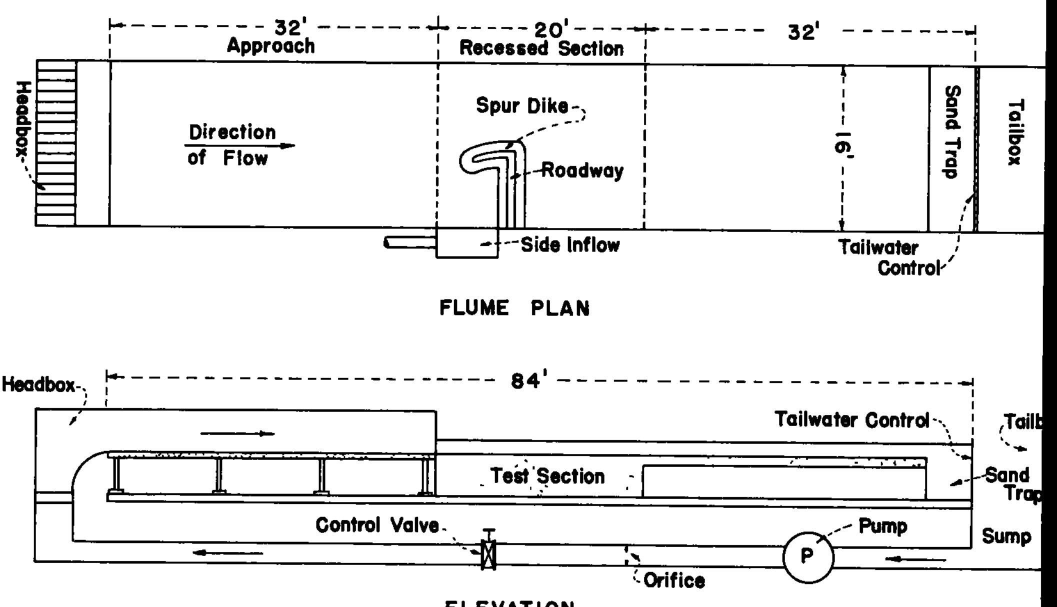


Figure 1. Schematic drawing of test equipment.

ith gradation in size from 1/4 in. to 1/2 in. The gravel was placed at random on the ıce of the dike.

TEST PROCEDURES

The procedure used for all runs was the same after certain pilot runs were made. he entire study was limited to clear water (no upstream or recirculating supply of ediment) with flow quantities varying with the size of the flume constriction. Pilot ıns were first made to determine the flow discharge in the flume which, at about 4-ft depth, would not develop ripples or dunes on the sand bed but the shear force ı the bed would be near the critical tractive force of the bed material. This test as made with no roadway constriction in the flume. The desirable discharge was und to be 4.8 cfs which gave an average velocity in the flume of 0.75 ft per sec. easurement of velocity in the flume was made with a Pitot tube and adjustments were ade in the head box so that a uniform distribution of flow was obtained across the idth of the flume.

The length of roadway embankment necessary to develop measurable scour depth as determined by trial. At a contraction of 0.5, scour depth reached a value of 0.75 ft a period of 5 hr and increased very little after that time. Since sediment was not ıpplied in the flow, equilibrium scour conditions could not be expected within a relavely short period of time. Therefore, it was decided to standardize test time rather an to proceed to equilibrium conditions because the study was primarily qualitative d it was desired to avoid an unnecessary amount of time for each run. Flow depth 0.4 ft was used in all tests measured at a section 4 ft upstream from the tailgate ntrol.

In tests involving flow from the side, the total discharge with a given bridge opening as held constant for comparative purposes and to avoid transport of sand in the flume. ıus, the discharge from the head box was reduced by the amount of the side inflow. this procedure, a longer roadway embankment was simulated by assuming that the de flow essentially represented an additional width of the flume. The additional ngth of embankment was computed by dividing the total side discharge by the unit scharge from the headbox.

rocedure for Each Test

The channel bed was leveled before each run and the same bed slope was used for l tests. Water was introduced into the flume from both the upstream and downstream ds to prevent scour at the test section before proper flow conditions were established in the flume. After filling the flume to the proper depth, the downstream pump ıs shut off and the upstream discharge increased to the proper amount. The water pth was controlled by the tailgate to 0.4-ft depth at the downstream end of the flume. ter 5 hours' run, the upstream discharge was shut off and as the water receded in the st section, the scour hole which formed at the bridge and spur dikes was contoured 0.1-ft intervals. The water surface in the scour hole was measured with a point ge.

ıta Taken

The results of all tests were recorded by photographs in both motion pictures and ill photographs, and most of the measured data were obtained directly from the photoaphs.

RESULTS

otation

The following is a list of definitions for symbols used in this paper. Terms are so defined in Figure 2 and where they first appear in the text.

L_o = Length of bridge opening in the flume (ft).

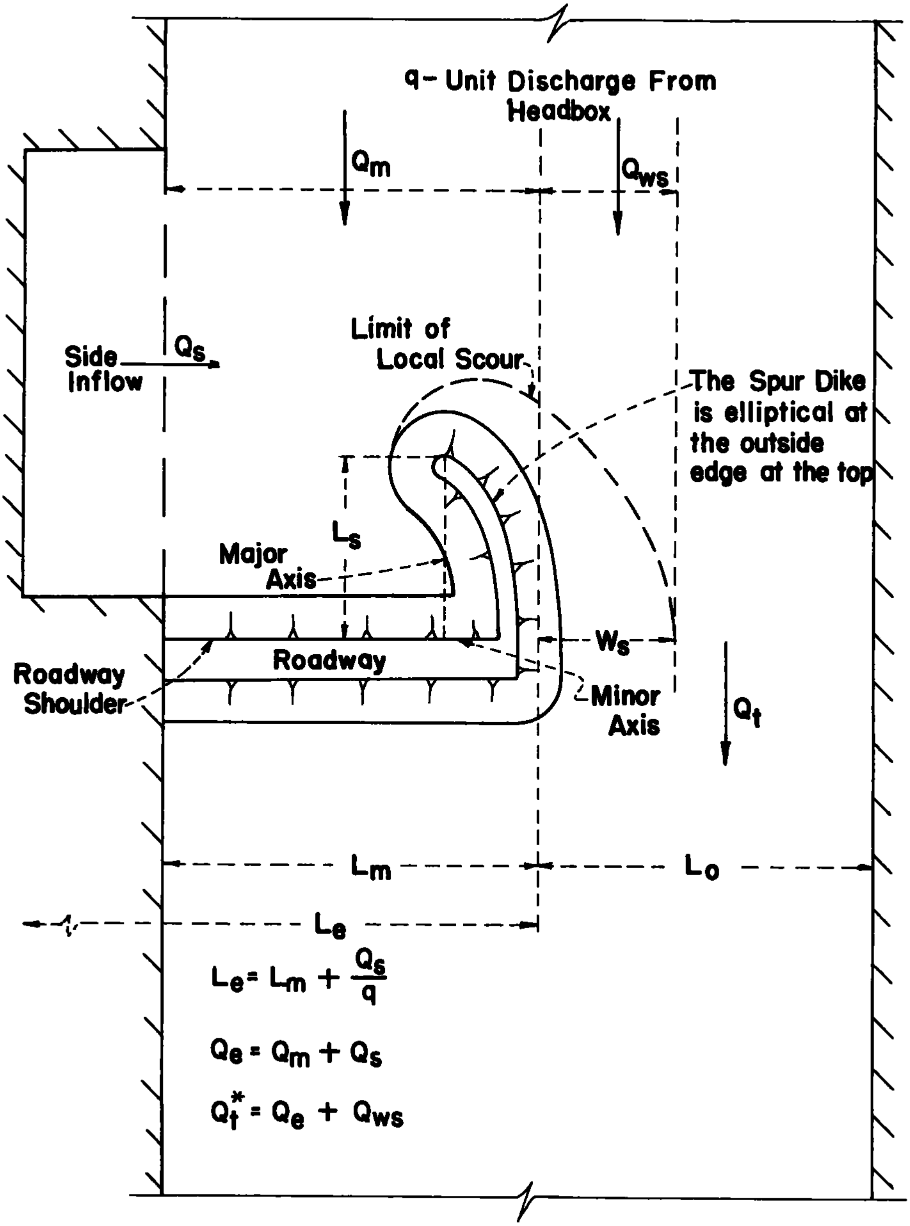


Figure 2. Definition sketch for symbols.

L_s = Length of spur dike measured along the major axis of the ellipse, normal to the roadway (ft).

L_e = Equivalent length of roadway embankment projecting into the stream channe normal to the direction of flow (ft).

λ = Ratio of the major axis to the minor axis of the elliptical spur dike.

W_s = The width at the bridge section, measured from the abutment, through whic the embankment flow Q_e is concentrated (ft).

d_s = Depth of scour measured at the bridge section (ft).

Q_m = Quantity of flow in the flume obstructed by the roadway (cfs).

Q_t = Total discharge through L_o of the flume (cfs).
$Q_t{}^*$ = Total discharge through the length W_s, equal to $Q_s+Q_m+Q_{ws}$ (cfs).
Q_{ws} = Quantity of flow approaching W_s normally (cfs).
Q_s = Quantity of flow entering from the side of the flume (cfs).
Q_e = Quantity of flow obstructed by the embankment equal to Q_s+Q_m (cfs).
q = Unit discharge per width of flume from head box.
m = Contraction ratio of the flume equal to $\frac{16-L_o}{16} = \frac{L_m}{16}$.

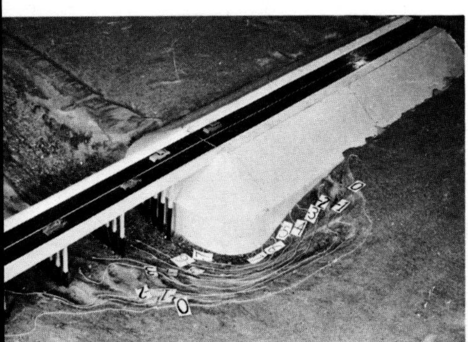
ffect of Spur Dike Shape and Location

The initial stage of the study was conducted to demonstrate the effectiveness of spur kes to control scour at the bridge foundation and to develop a better understanding the important variables involved. The results of the study are assumed to be com- rative, except for those otherwise designated.

Figure 3 shows scour that can occur at a bridge abutment which in most instances uld probably cause undermining of the abutment with possible failure of the first w spans of the bridge. Contour interval of the scour hole is 0.1 ft. The scour hole caused by large velocities due to flow concentration, which develop shear forces eater than the bed material can withstand. This is augmented by the development turbulence due to merging flow near the abutment. The effectiveness of a spur dike reduce scour at the bridge is shown in Figure 4. Although there is evidence of scour the end of the dike, actual scour at the bridge section is reduced, demonstrating that e bridge of Figure 3 would probably have failed, but the bridge of Figure 4 would not ve been threatened severely for the same flood condition.

The importance of spur dike location is shown in Figures 5 and 6. As the dike is fset from the abutment, there is increasing scour at the bridge section, and when the ke becomes sufficiently displaced from the abutment two distinct scour holes form, e at the abutment and another at the tip of the dike. It was demonstrated by these sts that the spur dike should be located at the abutment to be most effective.

When a channel is constricted by a roadway, the obstructed flow is forced to flow ound the constriction. Under this condition, the flow lines are usually curved near e bridge abutments. Because of this natural curvature, it would seem logical for a rved dike to develop better streamlining than a straight dike. There are a multiplicity curved shapes that could be used: parabolic, hyperbolic, spiral, elliptical, and rcular. Of these, the elliptical is probably the simplest geometrical shape and the e to be considered because of the adaptability to field layout. A convenient reference established by locating the minor axis of the ellipse along the roadway shoulder and ranging the side slope of the spur dike so that it becomes tangent to the abutment the case of spill-through abutments).



gure 3. Scour at a spill-through abut- nt. $L_s = 0$ $Q_t = 4.8$ cfs $Q_s = 0$ $L_o = 8.0$ ft.

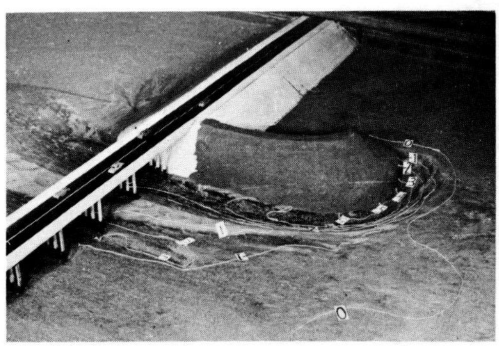


Figure 4. Scour at the bridge is reduced although there is scour at the end of the spur dike. $\lambda = 2\frac{1}{2}$ $L_s = 3.0$ ft $Q_t = 4.8$ cfs $Q_s = 0$ $L_o = 8.0$ ft.

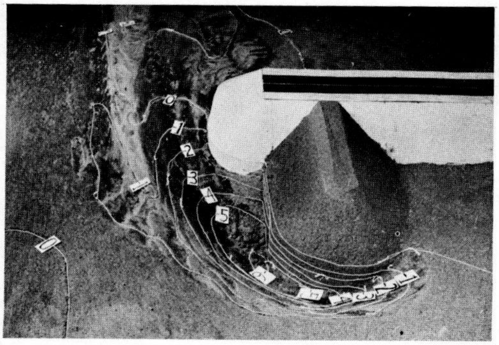


Figure 5. Straight spur dike is offset from the abutment a distance of $0.4\ L_s$. $L_s = 2.28$ ft $Q_t = 4.8$ cfs $Q_s = 0$ $L_o = 8.0$ ft.

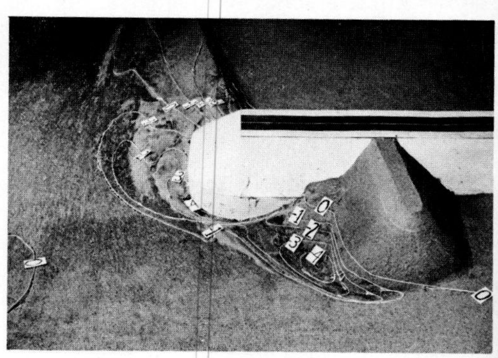


Figure 6. Straight dike is offset from the abutment a distance of L_s. $L_s = 2.28$ ft $Q_t = 4.8$ cfs $Q_s = 0$ $L_o = 8.0$ ft.

Figure 7 shows the results of tests conducted for two spur dike lengths of various elliptical shapes with the major axis normal to the roadway and the minor axis along the roadway shoulder line. As the shape of the dike becomes more nearly circular, there is an increase of d_s, the scour depth at the bridge. This is reasonable, for as the dike assumes a greater degree of curvature, the concentration of flow is greater along the dike. The results also show that another important variable in designing spur dikes is the length, L_s, along the major axis. For the two lengths, 2.27 ft and 3.41 ft, tested d_s decreases with an increase in L_s.

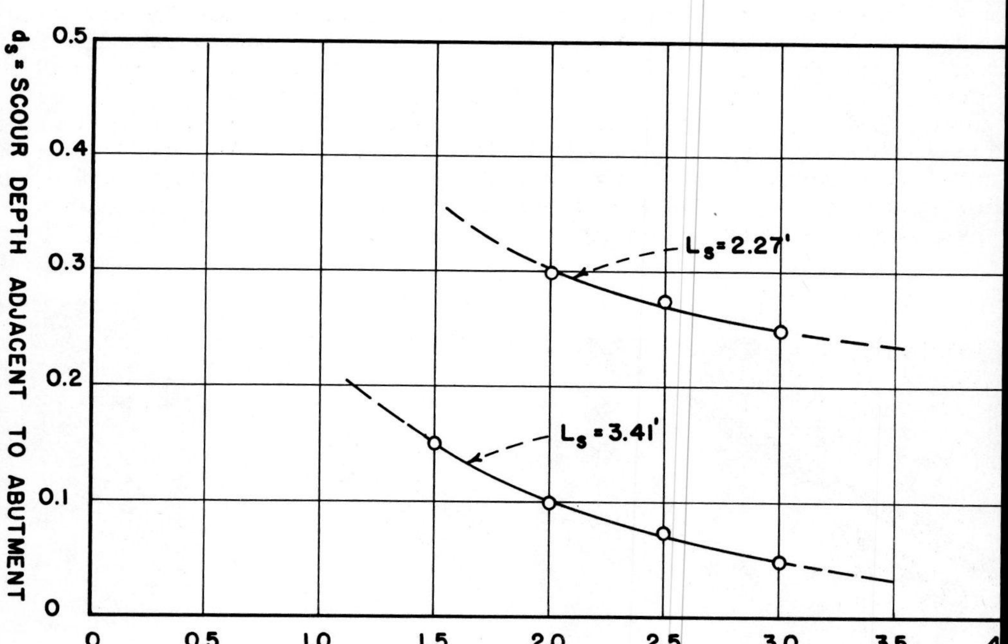


Figure 7. Relationship between the curvature of the ellipse and scour at the bridge section.

Observations made during these tests indicated that although the 3:1 elliptical dike ppears to be best from the standpoint of least scour, the flow did not follow the boundary f the dike. As a consequence, the total bridge opening was not fully effective. This s indicated by deposition of sand adjacent to the abutment (Fig. 8). Figure 9 shows ne test results with a $2\frac{1}{2}$:1 elliptical dike of the same length showing no deposition. he latter indicates better utilization of the bridge opening. The more efficient bridge pening with $\lambda = 2\frac{1}{2}$ offsets the slightly greater depth of scour, therefore, the $2\frac{1}{2}$:1 lliptical dike was selected as standard in the remainder of the tests.

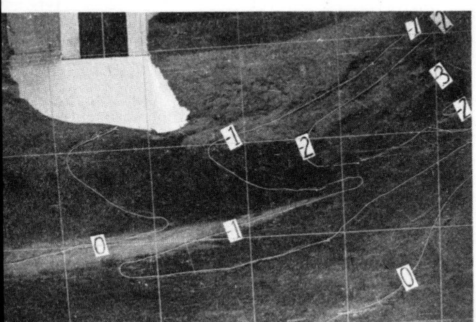
ffect of Spur Dike Length

The preliminary study has demonstrated the effectiveness of spur dikes to protect idge abutments from scour. In designing a spur dike it is necessary to consider its incipal functions. These are (a) to distribute the concentrated flow at the abutment s uniformly as possible through the bridge opening, and (b) to reduce the mean velocity ljacent to the abutment and decrease the turbulence. The dikes can be made to perrm these functions by choosing proper shape, location and length. The dike at the utment was shown to be the desirable location and an elliptical spur dike with a $2\frac{1}{2}$:1 ajor-to-minor axis ratio to be most effective. The length requirement of the dike mained to be established.

Results of tests made with normal embankments, and $\lambda = 2\frac{1}{2}$ are given in Table 1. nese tests were made to determine the effect of embankment length, L_e, and disarge on the spur dike length. Although values of L_e varied, there were basically ree sizes of clear bridge openings, L_o, tested in the flume. Values of L_o were 4.8, and 11.2 ft. The various parameters are shown in Figure 2. In these tests, it as assumed that the wall of the flume in the bridge opening approximated a flow line d that the wall had little or no influence on the scour pattern around the dikes and e abutment. This was not found to be true for all of the tests with the small opening 4.8 ft. The larger openings of 11.2 ft were not included in the results, because they quired such large discharges (in order to be comparable to the other tests) that neral movement of the bed was developed in the flume.

For each value of L_o, data from spur dike lengths of 1.5, 2, 3, and (where possible) t were obtained. To simulate longer roadway embankments, a side discharge, Q_s, rying to a maximum of 1.5 cfs was used. The discharge was converted to equivalent ditional flume width using the assumption of uniform approach flow. Because L_o reained constant, the additional flume width meant increased embankment length.

The results plotted dimensionally (Fig. 10) show the influence of spur dike length on



gure 8. Note the 0 contour is midway ang the embankment. There is deposition wnstream from this point. $\lambda = 3$ L_s = 41 ft Q_t = 4.80 cfs Q_s = 0 L_o = 8.0 ft.

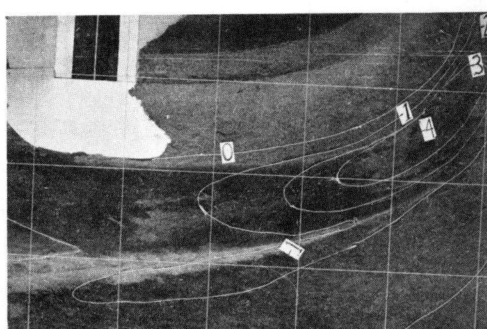


Figure 9. No deposition along abutment. $\lambda = 2\frac{1}{2}$ L_s = 3.41 ft Q_t = 4.80 cfs Q_s = 0 L_o = 8.0 ft.

TABLE 1

MEASURED AND CALCULATED DATA FOR NORMAL EMBANKMENTS

Q_s	Q_t	Q_t-Q_s	$q=\frac{Q_t-Q_s}{16}$	$m=\frac{L_m}{16}$	$m(Q_t-Q_s)=Q_m=L_mq$	$Q_e=Q_m+Q_s$	W_s	$Q_{ws}=q \times W_s$	$Q_t^*=Q_e+Q_{ws}$	$\frac{Q_e}{Q_t^*}$	$L_e=\frac{Q_e}{q}$	L_s	$\frac{L_s}{L_e}$	d_s	$\frac{d_s}{L_s}$	$\frac{W_s}{L_e}$
0	3.0	3.0	.188	0.7	2.10	2.10	3	0.56	2.66	.790	11.2	0	0	0.55	- -	.268
0	3.0	3.0	.188	0.7	2.10	2.10	4	0.75	2.85	.736	11.2	1.5	.134	0.30	.200	.357
.75	3.0	2.25	.141	0.7	1.58	2.33	4	0.56	2.89	.806	16.55	1.5	.091	0.33	.220	.242
1.50	3.0	1.50	.094	0.7	1.05	2.55	4	0.38	2.93	.870	27.2	1.5	.055	*	- -	.147
0	3.0	3.0	.188	0.7	2.10	2.10	4.5	0.85	2.95	.712	11.2	2.0	.178	0.30	.150	.401
0.5	3.0	2.5	.156	0.7	1.75	2.25	4.5	0.70	2.95	.763	14.4	2.0	.139	0.15	.075	.312
1.5	3.0	1.50	.094	0.7	1.05	2.55	4.5	0.42	2.97	.859	27.2	2.0	.074	0.35	.175	.165
0	3.0	3.0	.188	0.7	2.10	2.10	4.6	0.86	2.96	.709	11.2	3.0	.268	0.20	.067	.410
0.4	3.0	2.6	.163	0.7	1.82	2.22	4.6	0.75	2.97	.747	13.68	3.0	.219	0.22	.073	.345
0.75	3.0	2.25	.141	0.7	1.58	2.33	4.6	0.65	2.98	.782	16.55	3.0	.181	0.25	.083	.285
0	4.80	4.80	.300	0.5	2.400	2.40	3.0	0.90	3.30	.727	8.0	0	0	0.65	- -	.375
0.25	4.80	4.55	.284	0.5	2.28	2.53	3.0	0.85	3.38	.749	8.90	0	0	0.65	- -	.337
0.75	4.80	4.05	.253	0.5	2.03	2.78	3.0	0.76	3.54	.785	10.54	0	0	0.55	- -	.284
1.25	4.80	3.55	.222	0.5	1.78	3.03	3.0	0.67	3.70	.819	13.62	0	0	0.55	- -	.220
1.5 ft Spur Dike																
0.75	4.80	4.05	.253	0.5	2.03	2.78	4.0	1.01	3.79	.733	10.54	1.5	.146	0.25	.167	.380
1.00	4.80	3.80	.238	0.5	1.90	2.90	4.0	0.95	3.85	.754	12.20	1.5	.123	0.30	.200	.328
1.25	4.80	3.55	.222	0.5	1.78	3.03	4.0	0.89	3.92	.773	13.62	1.5	.110	0.30	.200	.294
1.50	4.80	3.30	.206	0.5	1.65	3.15	4.0	0.82	3.97	.793	15.30	1.5	.098	0.25	.167	.262
2.0 ft Spur Dike																
0	4.80	4.80	.300	0.5	2.400	2.40	4.5	1.35	3.75	.640	8.0	2.0	.250	0.25	.125	.562
0.25	4.80	4.55	.284	0.5	2.28	2.53	4.5	1.28	3.81	.665	8.90	2.0	.225	0.35	.175	.506
0.50	4.80	4.30	.269	0.5	2.15	2.65	4.5	1.21	3.86	.686	9.85	2.0	.203	0.23	.115	.457
0.75	4.80	4.05	.253	0.5	2.03	2.78	4.5	1.14	3.92	.710	10.54	2.0	.190	0.20	.100	.427
1.00	4.80	3.80	.238	0.5	1.95	2.90	4.5	1.07	3.97	.730	12.20	2.0	.164	0.25	.125	.369
1.25	4.80	3.55	.222	0.5	1.78	3.03	4.5	1.00	4.03	.751	13.62	2.0	.147	0.35	.175	.330
3.0 ft Spur Dike																
0	4.80	4.80	.300	0.5	2.40	2.40	5.0	1.50	3.90	.615	8.0	3.0	.375	0.22	.073	.625
0.25	4.80	4.55	.284	0.5	2.28	2.53	5.0	1.42	3.95	.640	8.90	3.0	.337	0.12	.04	.562
0.50	4.80	4.30	.269	0.5	2.15	2.65	5.0	1.34	3.99	.664	9.86	3.0	.304	0.12	.04	.507
0.75	4.80	4.05	.253	0.5	2.03	2.78	5.0	1.26	4.04	.688	10.54	3.0	.285	0.15	.05	.474
1.00	4.80	3.80	.238	0.5	1.90	2.90	5.0	1.19	4.09	.710	12.20	3.0	.246	0.20	.07	.410
1.25	4.80	3.55	.222	0.5	1.78	3.03	5.0	1.11	4.14	.732	13.62	3.0	.220	0.20	.07	.367
1.50	4.80	3.30	.206	0.5	1.65	3.15	5.0	1.03	4.18	.754	15.30	3.0	.196	0.20	.07	.327
4.0 ft Spur Dike																
0	4.80	4.80	.300	0.5	2.40	2.40	6.0	1.80	4.20	.571	8.0	4.0	.500	0.13	.025	.750
.25	4.80	4.55	.284	0.5	2.28	2.53	6.0	1.70	4.23	.598	8.90	4.0	.450	0.05	.012	.675
.50	4.80	4.30	.269	0.5	2.15	2.65	6.0	1.61	4.26	.622	9.86	4.0	.405	0.07	.018	.609
.75	4.80	4.05	.253	0.5	2.03	2.78	6.0	1.52	4.30	.647	10.54	4.0	.379	0.20	.050	.570

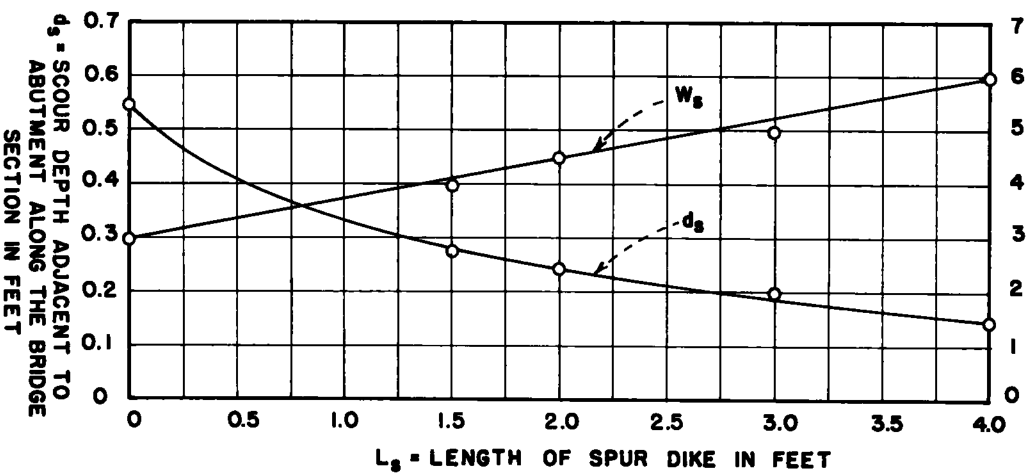


Figure 10. Effect of spur dike length on scour depth and width of spread.

cour depth at the bridge and distribution of the concentrated flow through the bridge pening. As the length of spur dike increases, there is an increase in the width of pread of the concentrated flow. This leads to a reduction in local velocity which reults in smaller depths of scour.

Based on the results in Figure 10 and the limited data from the study, a tentative uide for determining the length of spur dike is shown in Figure 11. A trial and error ethod must be used. At any given stream crossing it is assumed that the length of the oadway embankment and flood discharge are known. It is further assumed that a disribution of flow in the channel can be determined. The chart should be applied to conitions where distribution of flow is fairly uniform over the entire width $L_e + W_s$ (Fig.) and for normal embankments. Since it is in the interest of economy to construct e shortest length of dike necessary, the minimum value of L_s/L_e of 0.15 will be ied. With this value, calculate L_s. From d_s/L_s given from the selection line coresponding to the value of L_s/L_e, calculate d_s. If d_s appears excessive, a larger lue of L_s/L_e should be tried. When an acceptable value of d_s is determined, the lue of W_s/L_e on the abscissa corresponding to the selected L_s/L_e is read from the election line. The width of spread W_s, is calculated and Q_{ws} is determined. The lue of Q_{ws} is the quantity of flow which is approaching W_s normally. Knowing L_e, e is estimated. $Q_t{}^*$, the sum of Q_e and Q_{ws}, is determined and the ratio $Q_e/Q_t{}^*$ is omputed. This value is then compared to the value of the abscissa given along the p of the chart. If $Q_e/Q_t{}^*$ is greater than, or equal to, the value given, the trial ngth of spur dike is satisfactory.

There is a limit of L_e, the roadway embankment length, to which this chart should e applied. Since the tentative minimum spur length ratio, L_s/L_e, is 0.15 roadway ngths of about 1 mile would give an impractically long spur dike. Generally it is

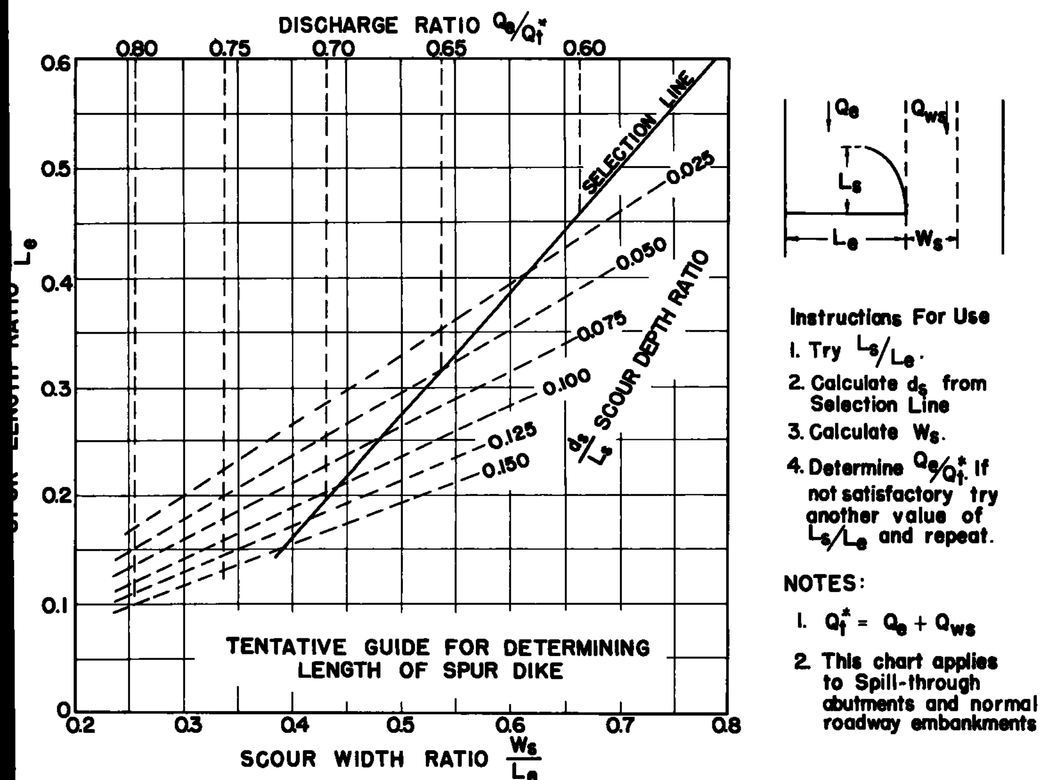


Figure 11.

not good design practice to construct a road embankment longer than 2,000 or 3,000 ft on a flood plain without providing a relief bridge. For L_e of 2,000 ft, L_s would be 300 ft—which is not excessive. Consideration of the discharge ratio will somewhat offset this limitation.

Figures 12 and 13 show the effect of an earth embankment spur dike with a 45° wing wall abutment. Because the abutment is vertical, there is a discontinuity of the flow boundary from the spur dike to the abutment. A partial transition is formed by the wi wall but it is insufficient to effect smooth flow conditions, and a secondary flow disturbance is created at the intersection of the wing wall with the abutment. The effectiveness of the spur dike is nevertheless clearly demonstrated. The principal require ment in construction is that the toe of the spur dike should be tangent to the vertical face of the abutment.

A limited number of tests were made with full bridge models to determine the effec of spur dikes on small bridges. These tests were conducted by installing two roadway embankments of equal length on opposite sides of the flume. The roadway lengths were increased successively so that the scour holes which formed at the abutments were made to overlap. As expected, when the scour holes overlapped, there was an increase in depths of scour at the bridge. This indicated that the bridge opening was too small to convey the total discharge. The results also indicated that so long as the bridge was sufficiently longer than the added W_s at both abutments, Figure 11 could be used to determine required spur dike lengths. However, it was observed that when th length of bridge was approximately equal to the sum of W_s at both abutments as determined from Figure 10, the actual W_s which occurred in the flume was less than that originally estimated. This was attributed to the influence of flow from the opposing side which tended to streamline the flow in a shorter width. The smaller W_s resulted in greated d_s. Thus it was necessary to increase L_s to offset the smaller W_s and to reduce d_s. The additional increase in L_s required for short bridges of the latter category could not be established in the form of criteria because of the limited data.

Frequently road alignments are set to cross stream channels at a skew. This may be necessitated by a number of things, highway alignment standards, economics of right-of-way, and cities. Whatever the reason for the skew, the hydraulics of flow will necessitate an adjustment in the spur dike length as determined for normal crossings. Some tests were conducted to give general indications of the skew effects. Only 45° skews upstream and downstream were tested with various spur dike lengths and with a contraction ratio of 0.50.

Figure 14 shows that shorter spur dikes can be used for abutments skewed downstream and longer spur dikes are necessary at abutments skewed upstream than

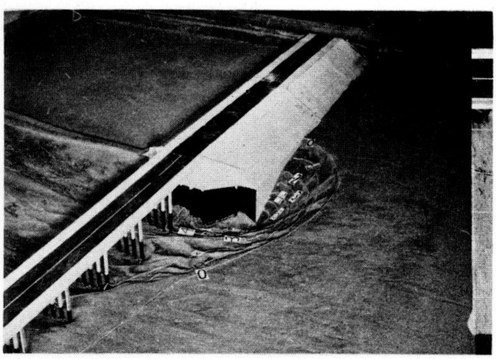


Figure 12. Scour at a 45° wing wall abutment. Q_e = 2.40 cfs Q_t = 4.80 cfs L_o = 8.0 ft. The top of the black painted area is the original stream bed. Contour interval is 0.1 ft.

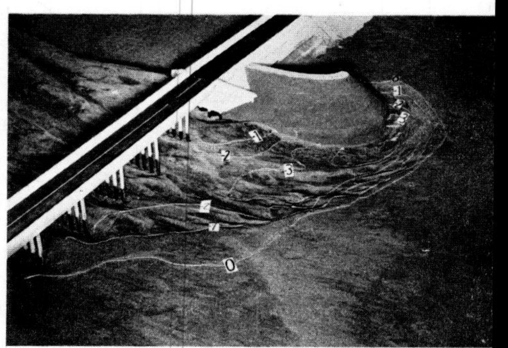


Figure 13. Effect of spur dike on reduc tion of scour at the bridge with a 45 wing wall abutment. $\lambda = 2\frac{1}{2}$ L_s = 2.0 f Q_e = 2.78 cfs Q_t = 4.80 cfs L_o = 8.0 ft

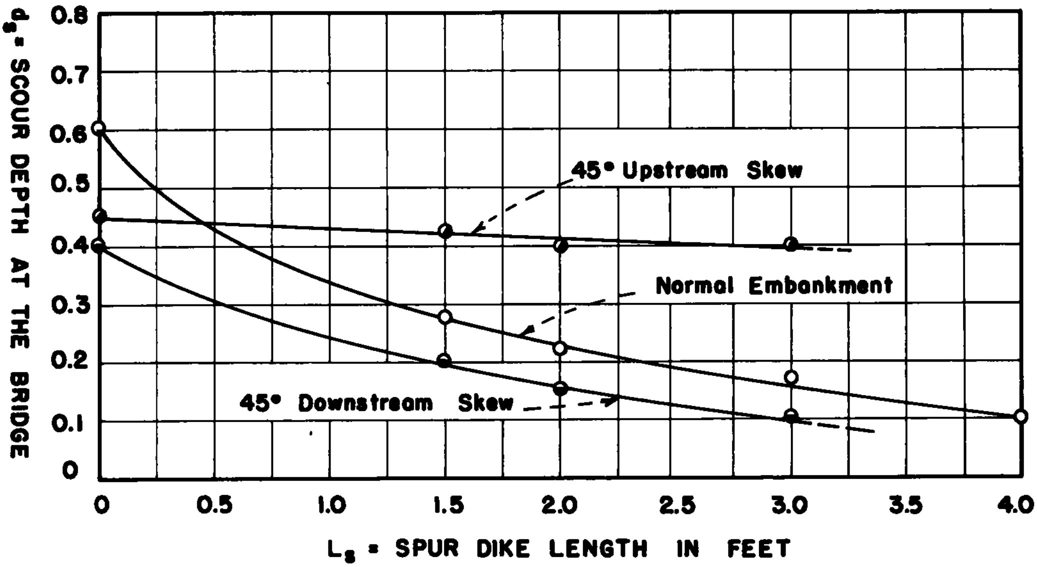


Figure 14. Effectiveness of spur dikes for different embankment skews.

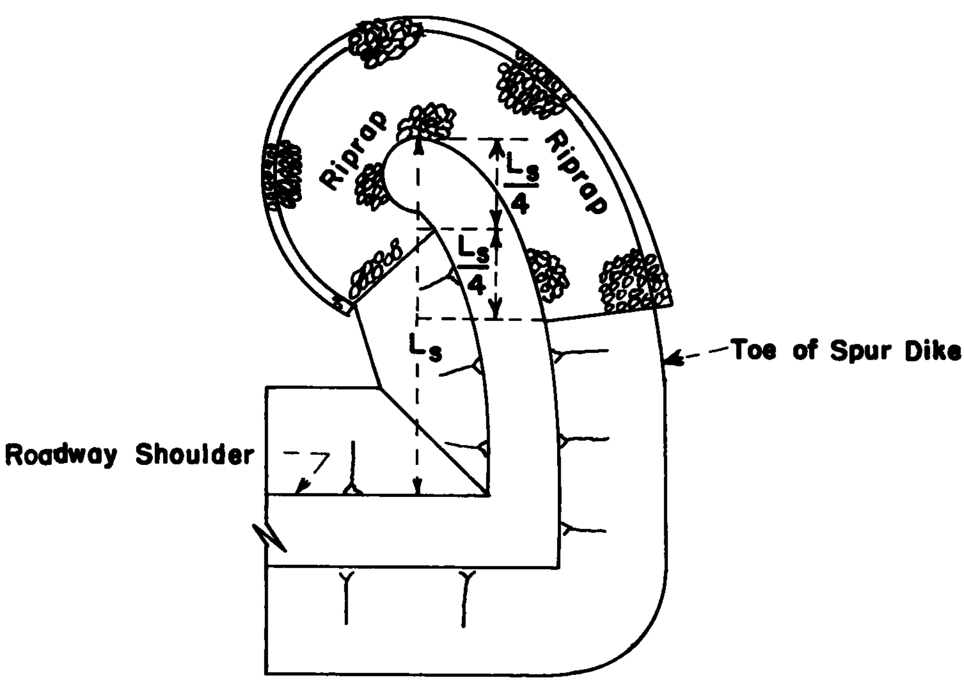


Figure 15. Diagram to show extent of riprap protection on spur dike.

required for normal bridges. Within the limits of the test, where 3-ft spur dikes showed significant reduction of scour both for normal and downstream skews little reduction of scour is noted for the upstream skew condition. The effect of spur dikes on scour reduction for normal embankments is sudden and significant while for downstream skews the effect is rather gradual.

The results of tests with skewed embankments are not incorporated with the tentative design chart because of the limited data collected.

Spur dikes constructed of earth embankment will normally require riprap protectio to prevent scour of the dike itself. The laboratory study was made to determine wher riprap was required. It was found that about one-half of the spur dike length from the end of the dike on the front or bridge side and about one-fourth on the back side required protection (Fig. 15). The riprap should be extended out from the toe of the dike on the flood plain so that as the scour hole forms, the riprap will fall into place on the side of the scour hole to prevent undermining of the spur dike.

RESEARCH NEEDED

The study has served to point out many aspects of the total problem which needs further investigation. There is a conspicuous need to determine the time relationship between small-scale movable bed studies conducted in the laboratory and the prototyp counter parts. Without specific knowledge of this time scale, it is difficult to relate quantitatively certain model phenomena to field behavior. This relationship can perhaps be established by experimentation with larger scale models and eventually correlating with prototype data.

Additional studies are required to determine the length requirements of spur dikes to protect small bridges. The problem of skewed bridges was only touched upon in this study. Additional information is needed to indicate the effect of skew angle on the increase or decrease in the spur dike length. A very important consideration in any scour problem is the effect of sediment in the flow. Although this research was limited to clear water, in the actual case it is probable that floods have a large concentration of suspended sediment in the flow. It is desirable to know whether the suspended sediment increases or decreases the amount of scour at the abutment. The effect of bed movement is another aspect of the problem which needs investigation. With general movement of the bed, the scour hole may not extend as deeply as it does for conditions involving no bed movement. Studies should also be made to determine the effects of routing complete flood hydrographs through the bridge opening to includ effects of suspended sediment and bed load movement. This study will involve knowledge of the time scale to conduct properly the laboratory studies. These few suggestions show that this study on spur dikes is only the beginning; much additional researc is needed for a better understanding of the total problem.

CONCLUSIONS

The study of spur dikes has resulted in tentative guides for design. Although speci fic guides were developed only for normal embankments, a general guide is presente for skewed conditions. It was also indicated that small bridges designed with minimu openings required longer dikes than bridges with longer openings.

The limitations of the laboratory study prevent explicit use of the design curve. The study has served to determine the following conclusions:

1. Spur dikes are effective measures to reduce scour at bridge abutments.
2. The effectiveness of spur dikes is a function of the geometry of the roadway em bankments, flow on the flood plain, and size of bridge opening.
3. The proper location for an earth embankment spur dike is at the abutment with the slope of the spur dike tangent to the slope of the abutment.
4. The curved spur dikes are more efficient than straight spur dikes because of the smoother streamlining of the flow.
5. Additional research is necessary to establish better criteria for design.

Culvert Inlet Failures—A Case History

OY C. EDGERTON, Research Engineer, Oregon State Highway Department

Bent-up ends have been experienced on three large structural plate culverts installed with the upstream ends square and projecting to the fill toe.

The paper describes the installations and failures and presents one explanation of the cause.

INLET FAILURES on three structural plate culverts on new construction on the regon Coast Highway in Curry County, Oregon, occurred in January 1959. The int ends were bent up, apparently by the buoyant force resulting from the difference water surface inside and outside the culverts.

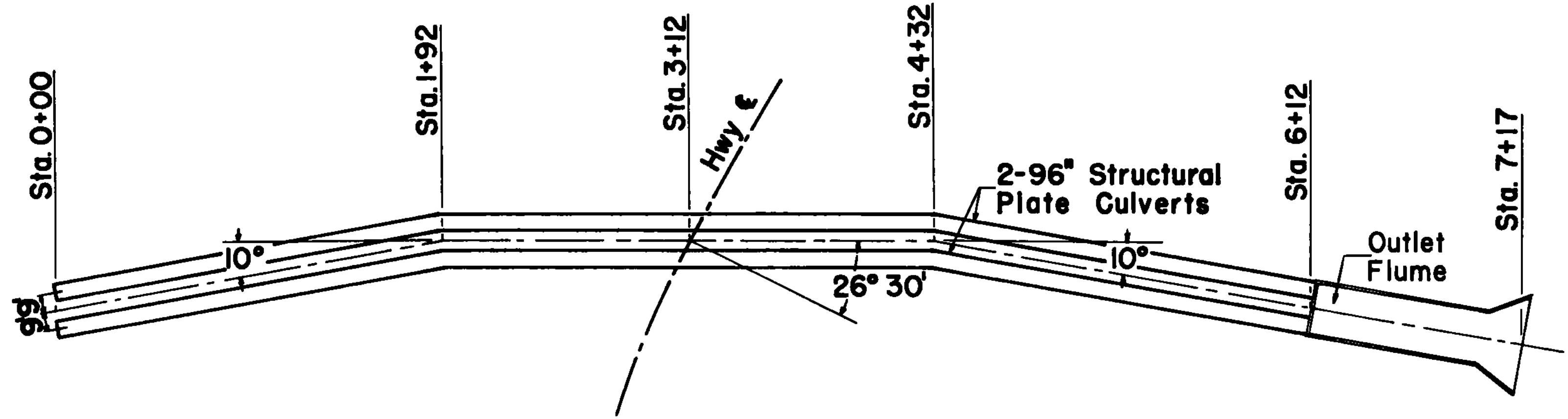
Figure 1 shows the Burnt Hill Creek installation in plan and section. The two 96-in. ructural plate culverts vary from 1 gage at the center to 10 gage at the ends with 1-ge inverts throughout. The length is 612 ft. The upstream ends are square and exnd to the fill toe; the entrances are thus of the projecting type. The culverts outlet to a concrete flume. The culverts were staked according to the plan which called r a 1.2 percent over-all slope with 1-ft camber at the center. From profiles taken the undisturbed portions of the culverts after failure, it seems probable that at the ne of failure the slopes of the inlet sections were steeper than the over-all design ope and supercritical.

The fill slope was 2:1 normal to the centerline. Since the upstream leg of the culrts was skewed about 30° with the normal, the actual slope in the direction of the lverts was about 2.3:1. The culvert inverts were placed above the channel bottom d the channel upstream was raised to invert elevation by a random fill. The relting approach channel was approximately level for about 300 ft upstream from the lverts.

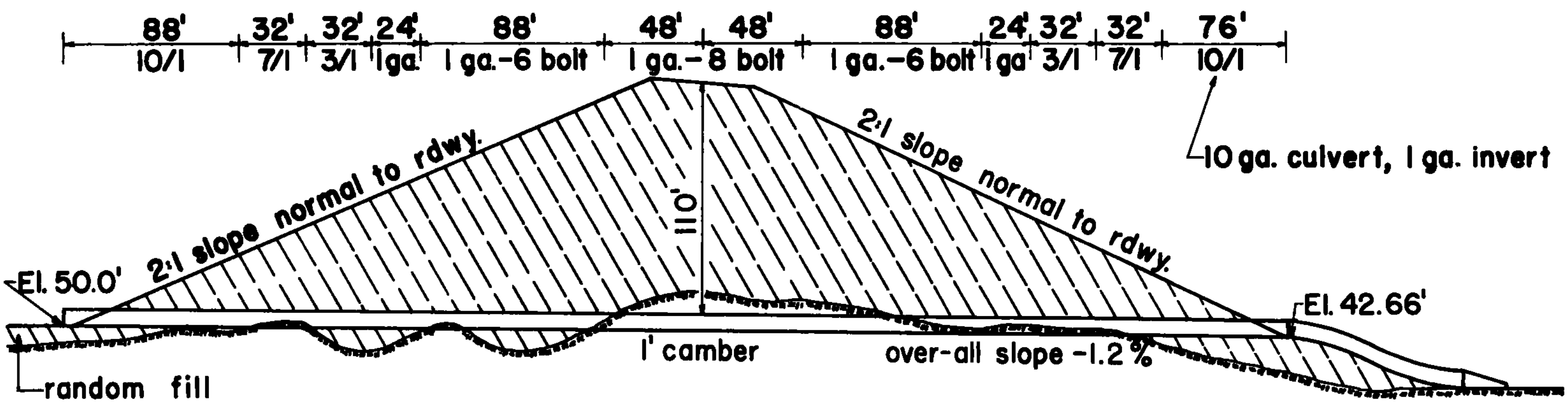
The Whiskey Creek installation is a single 90-in. structural plate culvert. Again, e gage varies from 1 to 10 with 1-gage invert. The culvert is straight, 411 ft long, d has a concrete outlet flume. The culvert grade situation is similar to that at rnt Hill Creek except that there is no question that the entrance section was on a percritical slope at the time of failure. The fill slope is 2:1 and the culvert is so arly normal that the same slope applies to the line of the culvert. A level random l extends about 400 ft upstream from the culvert entrance.

On the morning of January 9, following an intense storm, the situation shown in gure 2 was found at the Burnt Hill Creek site. Similar conditions were found at iiskey Creek. In all cases, the bends were smooth, well formed elbows with straight, listorted sections of pipe extending upward from the bends.

The Whiskey Creek culvert and the south culvert at Burnt Hill Creek were blown off the bend with ring charges to allow the ponds to drain. Figure 3 shows the Burnt ll Creek installation after the blast. Although the blasts destroyed or distorted most the bend area of two culverts, there was an opportunity to examine the north culvert Burnt Hill Creek before and after disassembly. The corrugations in the 1-gage inrt plates that formed the outside of the bend were extended quite uniformly from the iginal 6 in. to about 6.6 in. The corrugations in the 10-gage plates that formed the side of the bend vary from 1 to 3 in., apparently depending on the amount and nae of fill material trapped in the folds. The curvature was not uniform, being arpest at the center of the bend. There was about 25 ft of straight, undistorted pipe ending upward from the bend. The bends in the Burnt Hill Creek culverts occupied



PLAN



DEVELOPED ELEVATION

some 18 ft of pipe. The distances from the centers of the bends to the original upstream ends were about 44 ft for the south culvert and 34 ft for the north culvert.

There were no slides in either area. Other than minor fill settlement, the only displacement of fill material was that lifted out by the raising culverts. The amount of debris in the area could not have materially affected culvert operation. The random fill channel approaches eliminated any bed load problem at the culvert entrances.

The known facts point to buoyancy as the cause of the failures. The verification of this would require that the moment applied to a culvert by external forces be compared with the resisting moment of the culvert. This has not been possible since no data on the longitudinal bending properties of structural plate culverts have been found. External moment calculations were made for a 96-in. culvert of the type used at Burnt Hill Creek. A headwater depth of 96 percent of the diameter was used as preliminary calculations indicated that this produced the highest moment of any unsubmerged depth. The assumptions for the moment calculations are shown in Figure 4. A discharge of 340 cfs was used.

Moments were calculated for fill slopes from 0.5:1 to 2.5:1. The magnitudes and locations of the maximum moments are listed in Table 1. Within this range, the maximum moment more than doubles for each half unit flattening of the slope. This might explain how large structural plate culverts with projecting ends have been used successfully with steep fill slopes. The locations of the maximum moments can be considered to agree with the locations of the bends at Burnt Hill within the accuracy of the basic assumptions.

The culverts were replaced as originally installed except that the fill slopes in the vicinity of the entrances were steepened to about 0.5:1 with a facing of large rock riprap.

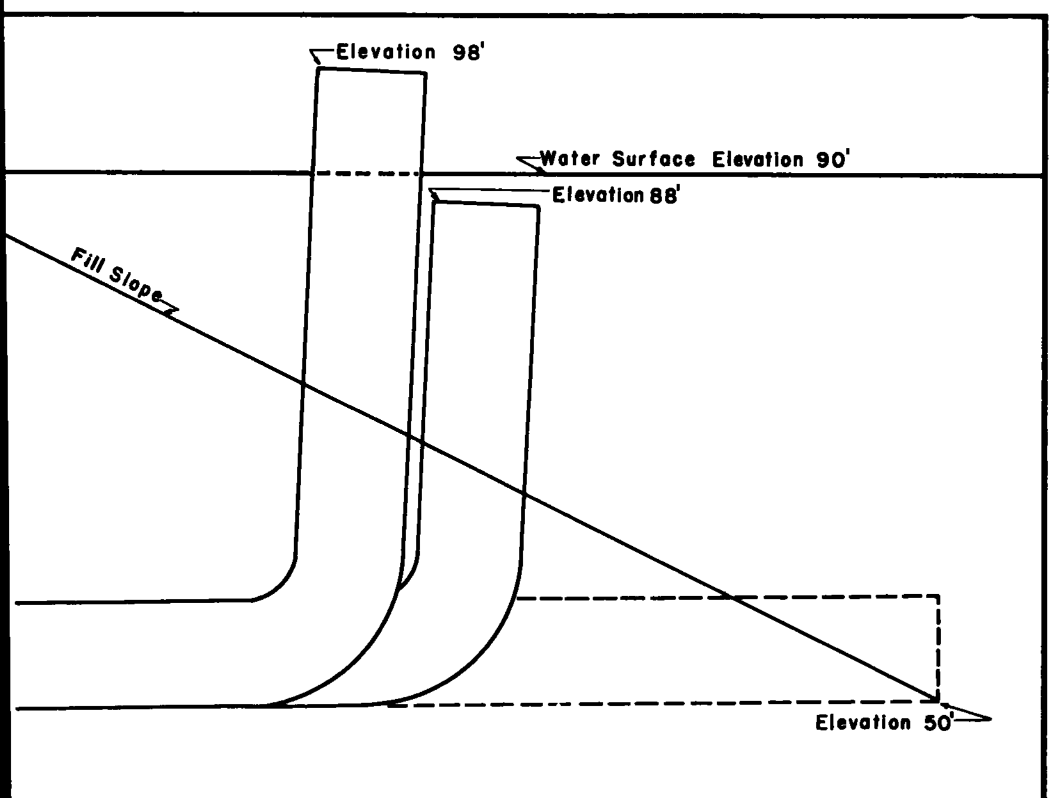


Figure 2. Schematic sketch of Burnt Hill culverts failure.

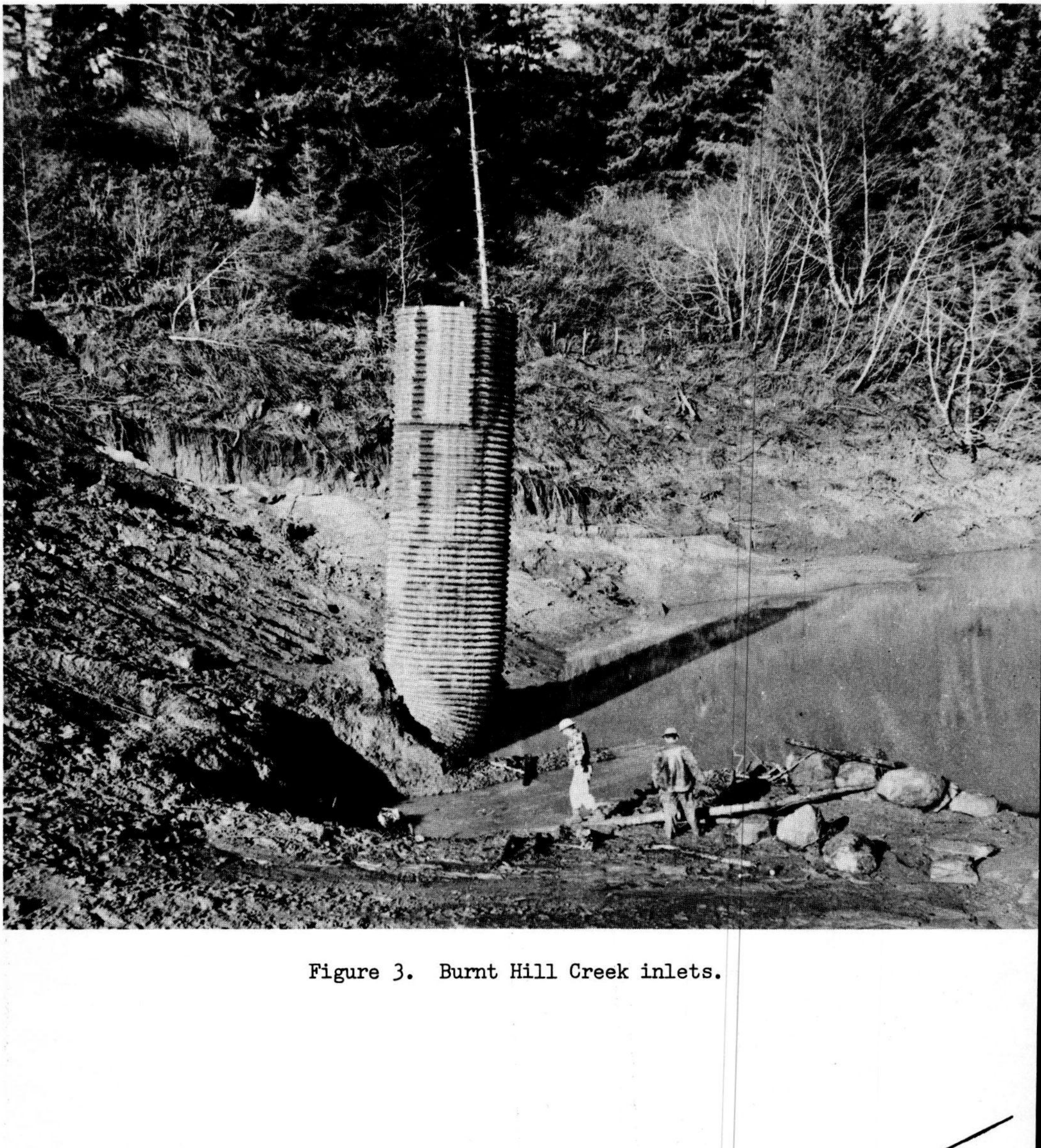


Figure 3. Burnt Hill Creek inlets.

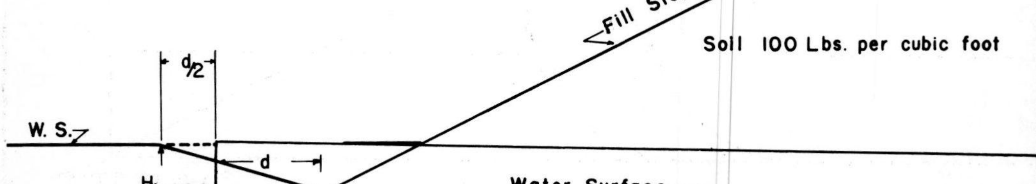


Figure 4. Assumed entrance conditions.

TABLE 1

RELATION OF BUOYANT MOMENT TO FILL SLOPE

Fill Slope	Maximum Moment (ft-lb)	Location (ft from entrance)
0.5:1	3,200	6.3
1:1	25,300	12.5
1.5:1	81,800	19.5
2:1	172,500	26.5
2.5:1	398,000	40.1

There are two general approaches to the prevention of future failures of this kind. One of these is to determine the longitudinal bending properties of the culvert and design within safe limits. The other is to adopt designs that avoid unbalanced uplift moment.

The first approach would require a series of full-scale tests of culverts for bending moment. The second would involve the employment of culvert end treatments now commonly used such as beveled or step-beveled ends or concrete head walls. Concrete head walls offer an advantageous solution in that they eliminate pipe projection, furnish weight to the pipe end, and improve the entrance condition.

In conclusion, the culverts are considered to have failed by buoyancy. Future failures of this kind can be avoided by determining the longitudinal bending properties of culverts and designing within these limits or by avoiding unbalanced uplift moments.

Discussion

. ROBINSON ROWE, Principal Bridge Engineer, California Division of Highways, acramento, Calif.—Edgerton's contribution is noteworthy because of the dearth of eliable data on failures, particularly of small structures which are so frequently re-laced or covered up quickly to salve the reputation of the designer of the reliability the material. When faithfully reported, failures are better teachers than successes nd the lessons are more eloquent.

The writer will present brief accounts of other failures due to buoyancy of culverts, omment on the determinacy of uplift and ballast, and express his views on economy of lternative safeguards.

The most common failure due to buoyancy in California has been due to installation small metal pipes in pervious embankment with shallow overfill, particularly in w-standard highways and detours during construction. A typical case is a 24-in. pipe stalled in a pervious gravel embankment covered by a 6-in. overfill and a 3-in. oil-ke slab. Water rises to the culvert crown, saturating the gravel but flowing only in. deep in the pipe. Per foot of pipe, uplift is 200 lb, but surcharge is 340 (20 for pe, 140 for water, 120 for fill and 60 for slab). If drift reaches and clogs the cul-rt reducing the surcharge to 200, just balancing the uplift, failure is imminent—the oximate cause being one of the following:

1. The uplift is greater at the downstream (lower) end of the pipe, which floats rst.
2. Ponding behind blocked pipe increases uplift at upstream end of the pipe, which ats first.
3. Shear resistance of slab prevents flotation of pipe, but ponding overtops road, ats the slab, and the pipe rises throughout.

All three types of failure have been observed. No doubt many complete washouts llowed this pattern, judging from stranded location of pipe and slab, but were not ported as such for lack of conclusive evidence. Elastic bending of pipe may have ntributed to failure, but recovered pipes appeared straight except for those swept oadside against a tree or power pole.

Next in frequency is the flotation-flexure of downstream ends of tide-gated metal pipe. Unlike flexure of the upstream end which backs water higher so as to increase uplift, flexure of the downstream end reduces uplift until pressures are balanced. Hen flexure may not produce failure, the deflection being (a) elastic, (b) tolerably plastic, or (c) correctable instead of (d) beyond repair.

An instance of the latter was reported by William A. White who had been Resident Engineer for the Corps of Engineers on the Deer Island Project along the Columbia River in Oregon. A series of CM pipe culverts 24 to 48 in. in diameter carried runof from the hills under a project dike into a canal. Passing the site in 1942 he observed that all the gated outfall ends had been bent vertical by high water in the canal. He estimated the head could not have exceeded 3 ft from the canal itself, but considered it possible that the river had overflowed the site at a higher stage. There were 6 to 8 projecting above the canal and possibly many others bent below the water and hence n visible.

Flotation is not limited to metal pipes. During the flood of December 1955, an 8- by 8- by 48-ft RC box culvert, built in 1926 at the mouth of Cold Canyon along Merce River, jammed with drift and floated out intact, wingwalls included. The structure weighed 90 tons displaced 132 tons and carried an overfill of 34 tons so that net buoya was 8 tons less what little water was still inside. Overlapping the highway increased the uplift. It came to rest 15 ft nearer the river and 5 ft downstream (Fig. 5).

Also, on a number of occasions the downstream end of RC pipe culverts projecting into a large channel below high water have been displaced in a manner strongly sugges ing flotation rather than erosion. Runoff from such small tributaries may be far dow on the falling stage before the main stream peaks, at which time flow past the projecting pipe entrains water from the pipe to create uplift.

Recognizing situations which might induce uplift failures is important in design, bu predicting uplift pressures is quite speculative. In dam design authorities are in wid disagreement. For example, some speculate that intensity of pore pressure varies uniformly from full hydrostatic at the heel to none at the toe and combine that pressu with a coefficient proportioned to the ratio of pore area to base. This latter coefficie has been set at 0.2 to 0.4 by some who then use 0.5 or 0.67 for safety. Others set it as high as 0.9 and allow for lifting of the heel so that full pressure penetrates under the dam for some distance.

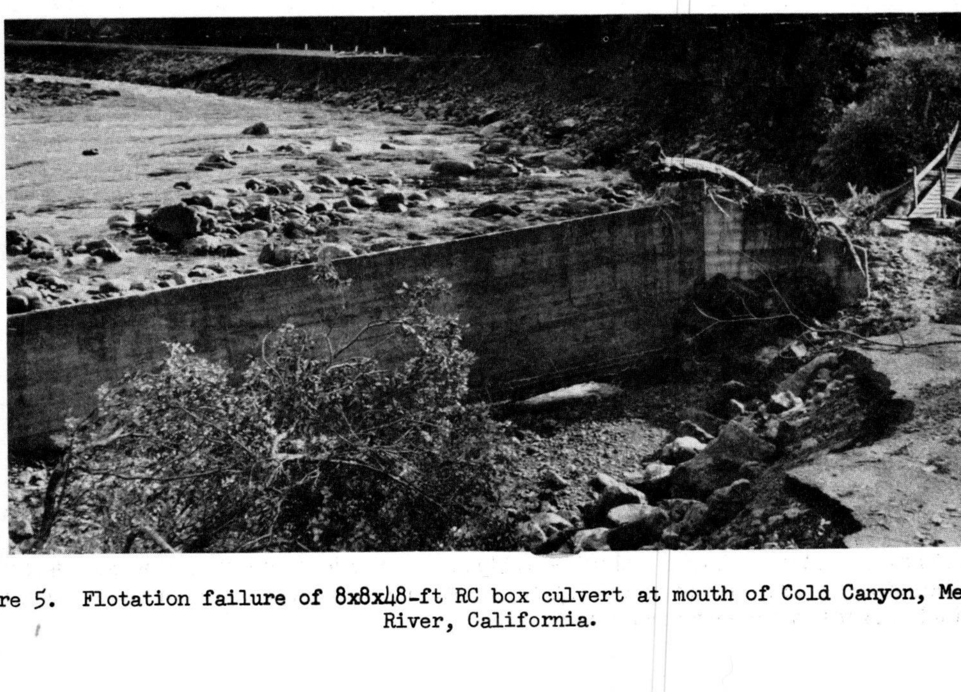


Figure 5. Flotation failure of 8x8x48-ft RC box culvert at mouth of Cold Canyon, Merc River, California.

Pipe bedding is generally much more pervious than dam foundations, so that coefficients should be higher. Elastic deflection of the projecting end will admit a wedge of water at full pressure. Some saturated embankments may become semi-fluid so that uplift is magnified by the specific gravity of the soil-water mixture.

The hydrostatic head is just as speculative. Edgerton has computed a critical value for a clear entrance and rapid percolation. The head may be much greater for culverts blocked by drift at the entrance or by a gate at the outfall. Percolation may be much less for a tight fill or a flashing stream. Deflection of the culvert may be progressive, each increment adding to the head as water is impounded above the entrance.

Just as elusive is the weight of soil over the pipe effective in ballast. At first the pressure is active, but as the pipe begins to rise it becomes passive and much greater, at least until the soil shears to the surface. The weight, passive pressure and shear are sensitive to saturation when stage rises above the pipe; probably for granular fills the unit weight increases but the passive pressure decreases.

Once a pipe has floated, these factors affecting uplift and ballast can be estimated with some confidence, since the combined effect is known. Prediction for a new installation is futile.

In considering the economy of safeguards against flotation, the infrequency of failure must be given great weight. A good general rule might be: use cheap safeguards for ordinary hazards and expensive measures when the risk or consequence of failure is very great.

As an example, interpolating Table 1 for a slope of 2.3:1 the buoyant moment on the Burnt Hill Culvert would have been 280,000 ft-lb, 35 ft from the entrance. This could have been ballasted by 8,000 lb at the entrance or 16,000 lb distributed along the pipe. Even a small headwall or cutoff at the entrance or paving of the pipe invert would have sufficed at a nominal cost.

Comparable moments for larger pipe would be much greater, varying about as the cube of the diameter. Even for a 15-ft pipe, entrance ballast could be provided with wingwalls or other transition amounting to 13 cu yd of concrete at a cost of $1,000. Good hydraulic design ordinarily warrants a much heavier structure.

Summarizing, the failure cited by Edgerton is a spectacular example eloquently reminding engineers that culverts are light structures for which buoyancy cannot be neglected. The hazard is greatest for metal pipes, increasing rapidly with diameter, but concrete structures are not immune. The hazard has been increased by modern standards of milder slopes for highway embankments. Ordinary entrance transition structures will provide assurance of stability. Special ballast may be required for gated outlets and for culverts under low fills subject to overflow; if strategically located such ballast would be relatively light and inexpensive.

B. LE MÉHAUTÉ and J.F. FULTON, Queen's University, Kingston, Ontario—Edgerton has reported a very interesting occurrence which graphically illustrates the damage which can result due to neglecting uplift pressures, which will develop under certain conditions dependent upon how culverts are laid.

From calculations based on the assumed conditions, it is felt that the culverts would not have failed had the fill slope existed as shown in Figure 4. It is obvious that the pressure created by the headpond level acts around the culvert's circumference near its entrance and throughout its length by infiltration into the bank, the seepage line decreasing very slowly downstream almost parallel to the gradient line inside the culvert (Fig. 6). The uplift force is, therefore, equal to the weight of the volume of water which would occupy that volume of the culvert between the outside water level and the inside water level. This difference in levels is greater than, but roughly equal to the velocity head of the flow in the culvert $V^2/2g$. This uplift force, although appreciable, is not sufficient to lift the culvert, a wedge of the embankment ballast, and simultaneously overcome the culvert's resistance to bending properties.

If, on the other hand, the culvert was laid so that its end projected upstream of the embankment heel, or if scour effects around the culvert entrance were extensive enough to cause an embankment slope failure and thereby uncover an appreciable length of

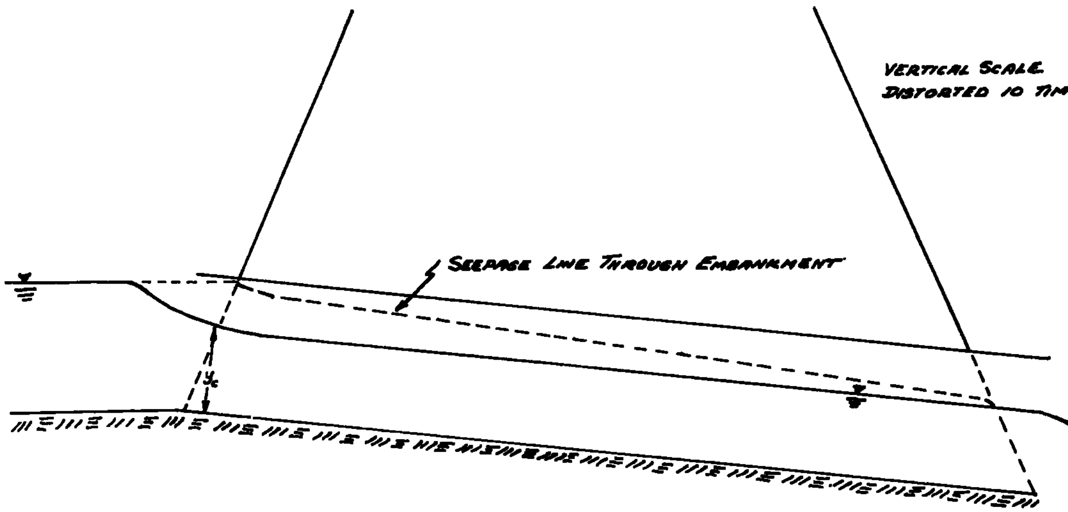


Figure 6.

culvert (at Burnt Hill Creek this length would appear to have been in the order of 25 ft), then the uplift force is sufficient to raise the culvert slightly. Once the pipe has lifted slightly, this phenomenon increases in magnitude. This is due to the fact that the headpond level is forced to rise (in order to discharge into the culvert), while the depth inside the bent section tends to decrease (since the velocity tends to increase with the increased slope); these two effects thereby increasing the difference between the inside and outside water levels.

It is the opinion of the writers that the failure was caused due to the combined fact that the culvert projected too far upstream of the fill slope and that the culvert was laid to too steep a grade.

The grade shown causes the critical depth to develop very close to the entrance, b if it developed farther down the culvert the intervening depths would form a backwate curve thereby reducing the magnitude of the uplift force.

One solution to this uplift problem would be to start the fill slope at the entrance of the culvert. A simple headwall (constructed of any material from hand placed roc to a complete concrete header) should also be included to prevent scour around the culvert entrance and guard against a slope failure of the embankment. A second solu tion would be to lay the culvert to a much gentler slope. Indeed, if the culvert were laid so that the critical depth developed at the downstream end, the difference betwee inside and outside water levels near the culvert entrance would be very small. In the case of flood the culvert would flow full (thereby causing buoyancy effects to disappea the discharge of the culvert depending only on the difference between the upstream a downstream water levels. In installations where this head difference is large, additional factors such as air entrainment, cavitation, and stability pose further difficult At Burnt Hill Creek this head difference was only of the order of 40 ft when the culve were out of operation. It is felt that under normal operation this head difference wo not be duplicated, but even if it were these additional factors would not likely create serious problems. If it is found that stability difficulties are encountered, these ma easily be corrected by simply pushing down the top of the downstream end of the culvert until the diameter is reduced to 0.9 D. Under flood conditions, the critical dep forming downstream will increase until this bent lip is encountered. The culvert wi then fill up from the downstream end and will force any trapped air out the upstream end thereby eliminating the possibility for serious instability to develop. This bent section is shown in Figure 7.

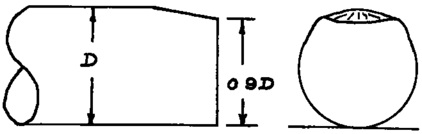


Figure 7.

The writers feel that either one of these solutions, or a combination of them, would provide a more economical answer than fixing the embankment slope at 0.5:1.

OY C. EDGERTON, Closure—This paper was presented to call attention to the need r consideration of buoyancy in culvert installation and to encourage the release of eports of similar failures. Rowe's discussion furthers the second of these aims, ontributing more to the subject than did the original paper.

Most of the points raised by Le Méhauté and Fulton are considered to have been overed in sufficient detail in the paper; however, a few points might warrant amplifica- on.

As with photographs in general, Figure 3 was not intended for quantitative interpre- tion. The skew of the culvert with respect to the fill slope and the level of water in e pond makes this impossible. Figures 1, 2, and 4 were expected to serve the quanti- tive function. It should be noted that with a 96-in. culvert and an effective fill slope 2.3:1 the slope line would intersect the top of the culvert about 18 ft from the end of e culvert.

The method of calculation of the water surface inside the culvert incorporated an trance head loss of about 0.7 $Vc^2/2g$. It was assumed that by the time of failure, the ll upstream head would be effective on the outside of the culvert to the area of the nd.

In the mind of the author there is not "too steep a grade" for a culvert except as it fects culvert wear and the control of outlet velocity. In general, culverts are placed ere and as required. There is no particular interest in solutions that materially duce culvert capacity.

The description of the restoration seems to have been misread. The culverts were placed as originally installed. Large rock riprap was then placed around the culvert trances. The slope of the riprap face was about 0.5:1. Since then a combination of ep-bevel culvert ends and riprap slope facing have been adopted as standard for ure construction.

New Developments for Erosion Control at Culvert Outlets

GEORGE L. SMITH and DASEL E. HALLMARK, Respectively, Hydraulic Research Engineer, Colorado State University, Fort Collins; Highway Research Engineer, Division of Hydraulic Research, U.S. Bureau of Public Roads, Washington, D.C.

● A PROBLEM which frequently confronts the highway engineer is excessive erosion or gullying at the outlet end of culverts. Nature's method of keeping erosion under control on mild slopes is to spread the runoff over a wide area of the watershed so th flow depths are shallow and resistance to flow rather high. Typically a highway embankment crossing a watershed disrupts the drainage plan of nature in that the runoff from a wide area is funneled to culverts where the flow is concentrated within a small area. This concentration of flow into a culvert greatly increases the erosive ability of the flowing water at the culvert outlet, since the flow has been accelerated (Fig. 1) The accelerated flow must travel some distance before it can fan out again over a wid area after it has passed through the culvert. As time elapses, erosion or gullying increases, and scour control becomes more difficult and expensive. It is no longer realistic to consider only the initial cost of a culvert and ignore future maintenance.

The basic design problem is the dissipation or proper control of the kinetic energy of the jet of water issuing from the culvert outlet. It should be pointed out that not all culverts require special structures to control erosion; for example, where the natura stream channel exhibits a high resistance to erosion, or where the downstream chann control provides adequate depth of flow for energy dissipation of the impinging jet and thereby minimizes erosion of the channel. However, when proper erosion or scour control is needed, it is accomplished by means of an energy dissipator constructed at the downstream end of the culvert. In the past, where this has been done, concrete or stone structures have been employed. Considering the number of culverts involve this type of construction is expensive, both in initial costs and maintenance costs. Therefore, there is a need for a stilling basin that is more economical and yet efficie for use as an energy dissipator. Such a basin, to be economical, should be of simple design, low in construction cost, and require a minimum amount of maintenance. Maintenance in this case would be a measure of the efficiency of the basin.

The purpose of this paper is to present information on a very promising and inexpensive method of controlling erosion at culvert outlets. Sufficient testing has been performed in the laboratory and in the field to demonstrate the effectiveness of the method. In general, the method consists of simply excavating a hole downstream fro the culvert outlet and lining it with a graded layer of protective material consisting o coarse sand, gravel and boulders up to a size that will resist erosion at the peak flow In this report, this method will be referred to as a pre-shaped, armorplated stilling basin. Limitations on applications of the results are presented. Future research ne conclude this paper.

FUNDAMENTAL CONCEPTS OF ENERGY DISSIPATION AND SCOUR CONTROL

Kinetic energy of a jet of water can be dissipated in a stilling basin by one of the following methods (Fig. 2):

1. In the horizontal direction by the hydraulic jump or artificial roughness in the channel;
2. In the vertical direction by jet diffusion and dispersion by the drop structure o manifold, or;
3. In a combination of both directions.

To be effective, the hydraulic jump
nust be stabilized by a combination of
locks or sills placed on the basin floor
nd a minimum tailwater depth must be
naintained. It is very difficult when the
'roude number of the flow is less than
vo to eliminate completely the waves and
ome high-velocity flow downstream from
ie basin. The waves and high-velocity
ow as will be demonstrated in another
ction, are factors which erode the banks
nd bed at the basin outlet. As the cul-
ert outflow becomes sub-critical, no
mp will form but there still may be suf-
cient energy to create erosion problems
the culvert outlet.

Effective dissipation of energy in the
ertical direction requires that the jet
water be diffused by the surrounding
ow or tailwater. Also, it is essential
at a minimum depth of tailwater be
aintained in the stilling basin in order
at the jet diffusion will be sufficient to
event high energy waves from emanating
om the stilling area. However, this
ethod has the distinct advantage of con-
ing the energy dissipation to a relative-
small area, compared to the hydraulic
mp, by quickly bringing the water into
state of high-intensity turbulence, trans-
rming the bulk of the flow energy into
at and fine-grain turbulence.

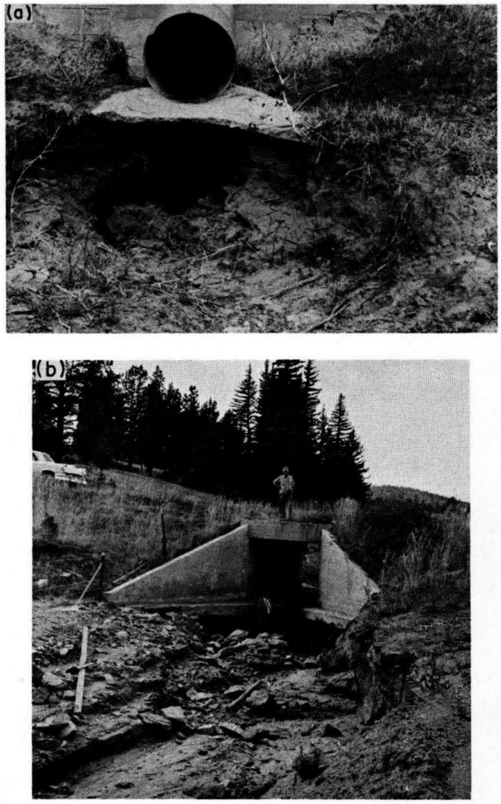


Figure 1. Erosion at culvert outlets caused by accelerated flow; (a) pipe culvert, and (b) box culvert.

Recent hydraulic laboratory tests by
llmark (1) and Smith (2) at Colorado
ate University have demonstrated that
e kinetic energy of a freely falling jet
water can be effectively dissipated in the vertical direction by means of a pre-shaped,
morplated stilling basin. For either the box culvert outlet (Fig. 3) or the pipe cul-
rt outlet (Fig. 4) the basin in the alluvial bed is an excavated hole which is lined with
graded layer of protective material consisting of coarse sand, gravel, and boulders
to a size that will resist erosion at the peak flow.

The function of the pre-shaped stilling basin is to provide a large enough stilling
ol so that the kinetic energy of the freely falling jet is dissipated by diffusion within
e pool. The function of the graded gravel is to act essentially as an armorplate lin-
; for the pre-shaped basin controlling erosion. The armorplate material must be
such size that the velocity of the deflected and diffused jet leaving the stilling basin
ll not carry the armorplate particles from the scour hole. Armorplate effectiveness
pends on its gradation; that is, it must be uniformly graded from the maximum size
the stream bed material up to that of the largest size armorplate material. With
iform gradation, each particle of armorplate is protected by smaller particles un-
rneath so that it will not be undermined. For example, the armorplating material
ts in a manner similar to the "Terzaghi" graded filter (3). Separation and place-
ent of separate size material is not required for the armorplate material to be ef-
tive.

A criterion for maximum size and gradation has not been determined either ex-
rimentally or theoretically. However, Peterka (4) gives an empirical expression
sed on the bottom velocity of channel flow for determining the maximum size of
morplate that will resist erosion. Lane (5) in his stable channel design theory

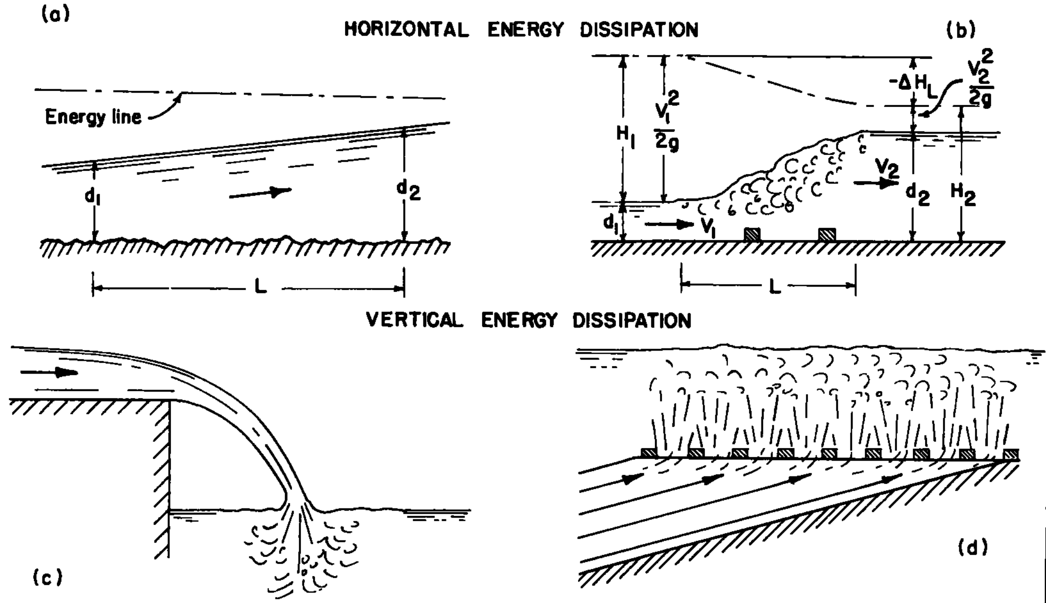


Figure 2. Methods of dissipation of kinetic energy; (a) channel resistance, (b) hydraulic jump and form resistance, (c) downward—vertical jet diffusion downstream of a drop structure, and (d) upward—vertical jet diffusion above a manifold outlet.

provides a trial and error method of estimating a maximum size of armorplate for bank protection. In the laboratory tests 1-in. maximum material was used in varying amounts and according to Figure 5 a small percentage of armorplate gave adequate erosion control.

These studies demonstrated that erosion control depends not only on the use of

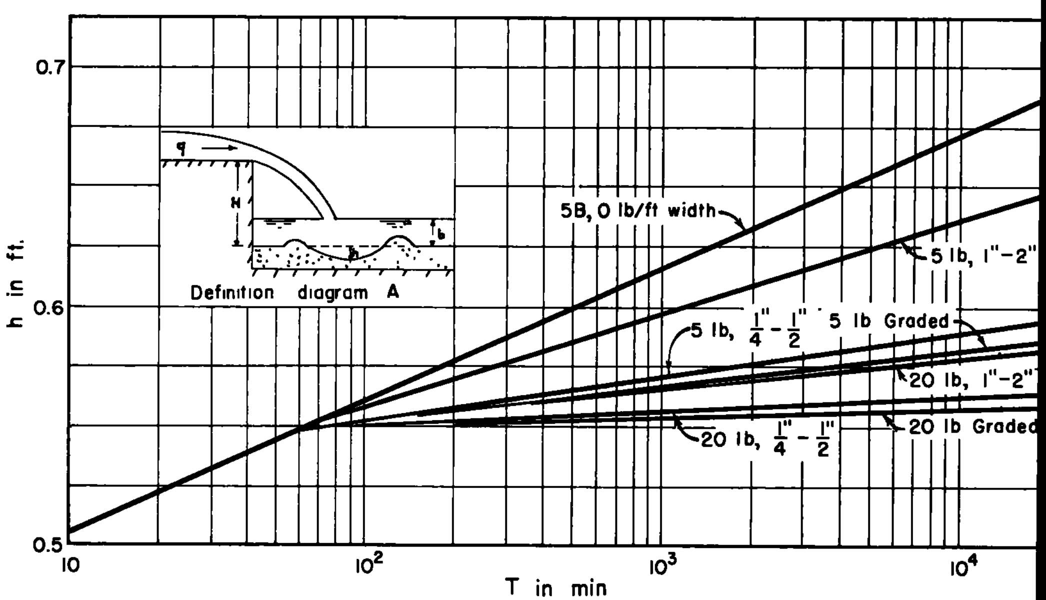
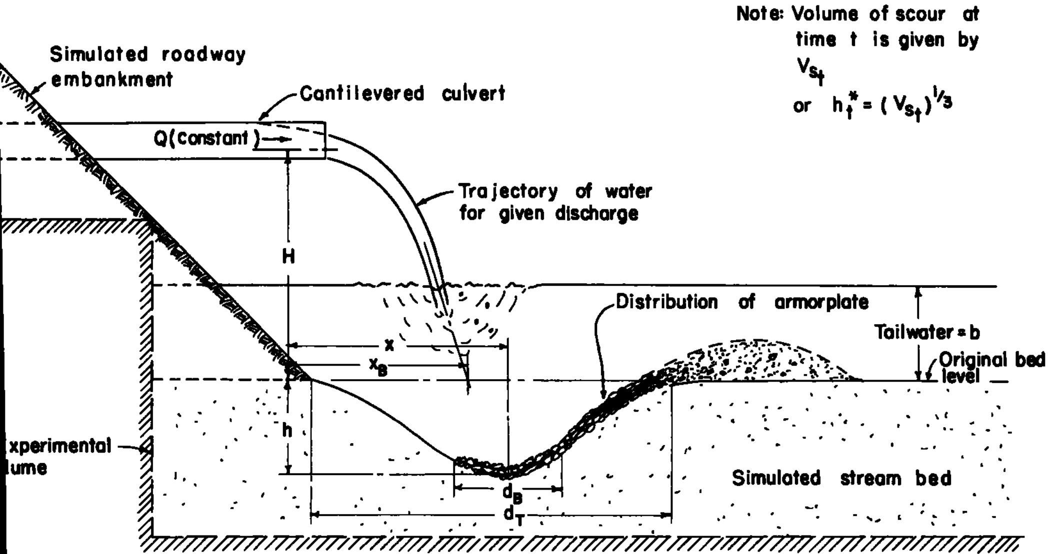
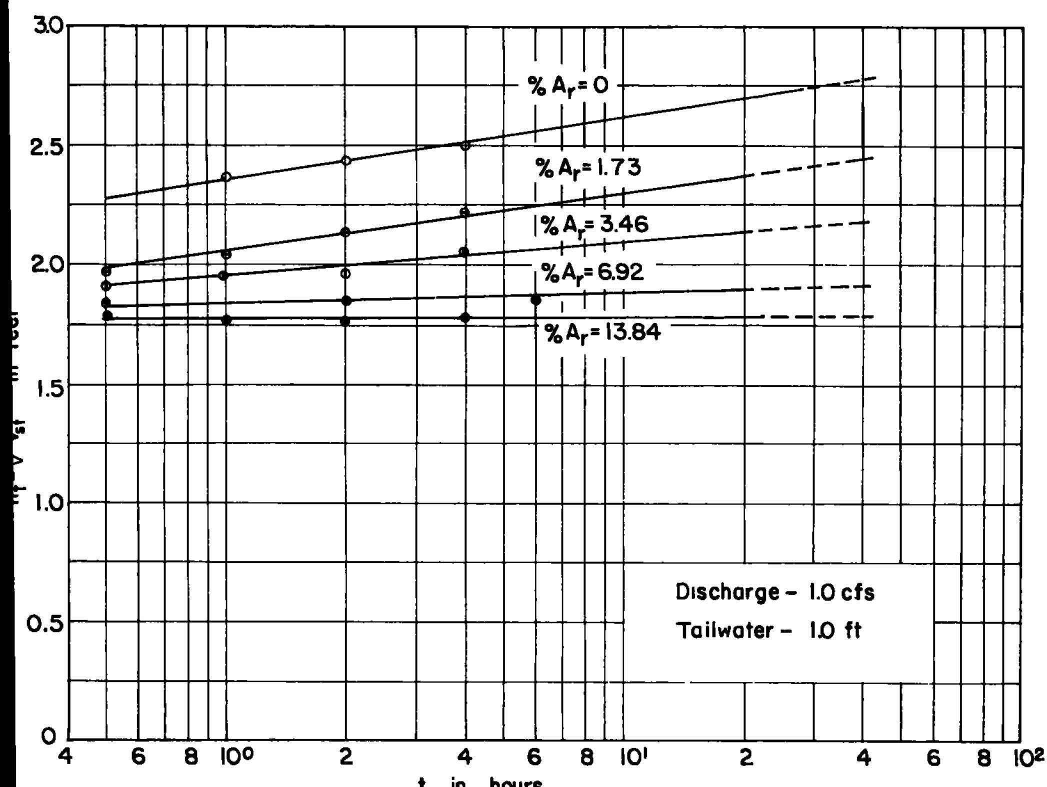


Figure 3. Variation of rate of scour with size and quantity of armorplate.



gure 4. Schematic diagram showing some of the variables that can affect the rate of our caused by outflow from a cantilevered culvert into a rectangular channel with gid sides and an alluvial bed. (Note: Geometry of pre-shaped basin is included.)



ure 5. Variation in rate of scour as influenced by quantity of armorplate, discharge and tailwater depth.

armorplating a pre-shaped stilling basin and along the channel banks but also on the proper control of the energy line of the channel downstream of culvert outlets. Control of the energy line as related to tailwater depth may be accomplished by the use of a series of drops, whose crest is higher than the upstream lip of the stilling basin. However, in most cases the position of the total energy line of the downstream channe in relation to the culvert outlet will permit the use of a stilling basin at the culvert outlet without requiring controls further downstream in the channel.

DEVELOPMENT OF AN ARMORPLATED, PRE-SHAPED STILLING BASIN

To develop design criteria for a pre-shaped, armorplated stilling basin, a researc program was conducted at Colorado State University (6). The hydraulic laboratory tests were made for a limited range of flow conditions, boundary geometry, and sediment characteristics. The sediment characteristics for armorplating were selected to be representative of those materials found in sand-gravel-cobble deposits. The research program was conducted as follows:

1. An experimental investigation was made of the scour phenomenon caused by a freely falling jet of water impinging into an alluvial bed.
2. The geometry of a standard scour hole (pre-shaped stilling basin) was determined for the three-dimensional jet by an analysis of the scour data.
3. The effect of quantity of graded gravel (armorplate) on the rate of scour was investigated.
4. The most effective point of placement and the minimum effective quantity of armorplate needed for the pre-shaped basin were obtained by an analysis of experimental data.

In making an investigation of the problem of scour in alluvial stream bed caused b a freely falling jet of water, it is important first to consider the various factors affecting such scour. Besides the energy of the jet, which at the pool surface can be e pressed in terms of discharge, density and height of fall or velocity, there is the extent of jet disintegration and jet diffusion before it impinges upon the erodible bed. Thus the distance the jet travels in the air, and the depth of tailwater, as well as lat extent of the pool are important. Finally, the sediment characteristics of the alluvia bed must be considered. Rouse (7) had demonstrated that for completely alluvial ma terial, the geometric mean fall velocity and the standard deviation of this fall velocit about the mean adequately characterize the erodibility of granular sediment. These variables together with the sediment density, completely describe the sediment.

Because of the many factors that can have an effect on scour (Fig. 4) the experimental investigations were limited in the following manner:

1. The jet issued from a source sufficiently close to the bed so that air resistanc was negligible ($H = 4.0$ ft, $Q = 0.5$ to 2.0 cfs).
2. The depth of tailwater above the original bed was varied in discrete increment ($b = 0.5$ to 1.5 ft).
3. One size of bed material ($d_m = 2.6$ mm) having a narrow size range ($\sigma_d = 1.35$ mm) was used.
4. During any single test run, the discharge from the jet was held constant and th elevation of the tailwater remained fixed.
5. The channel at the culvert outlet was rectangular in cross-section with rigid sides and alluvial bed.
6. The size of the pre-shaped basin used for testing the armorplate was based on the size of scour hole that would be developed by a given discharge for a given tailwater depth and time of scour, which for both cases was approximately one hour.

In order to design an armorplated, pre-shaped stilling basin downstream from a culvert outlet, it is necessary to have criteria for determining: (a) the design dimensions of the pre-shaped basin; (b) the orientation of the basin relative to the culv outlet; (c) the minimum size of armorplate that will resist erosion at peak design flo

) the location or point of placement of the armorplate; and (e) the quantity size dis-ibution of armorplate needed.

From an analysis of experimental data for the three-dimensional case (cantilevered :lvert), the following criteria were obtained for a pre-shaped basin in a rectangular annel with rigid sides and alluvial bed:

$$d_t = 2.38\ h_t^* \tag{1}$$

$$h = 0.50\ h_t^* \tag{2}$$

$$d_B = 0.65\ h_t^* \tag{3}$$

which

d_t = the diameter of the pre-shaped basin at the surface of the stream bed,
h = the maximum depth of the pre-shaped basin,
d_B = the diameter of the pre-shaped basin at the depth h, and
h_t^* = the cube root of the volume (V_{st}) of the basin at time t, = $(V_{st})^{1/3}$

For example, if as in the laboratory the discharge is 1 cfs and the tailwater depth ll be maintained at 1.0 ft, then from Figure 5 for 1 hr the value for $h_t^* = 2.35$. In gure 5, A_r is the armorplate expressed as a percent of the total volume of the pre-aped basin. The dimensions of the pre-shaped basin would be $d_T = (2.38)(2.35) =$ 5 ft; $h = (0.50)(2.35) = 1.2$ ft; and $d_B = (0.65)(2.35) = 1.5$ ft.

No criterion for maximum size of armorplate has been determined. However, ex-rimental tests showed that the effective point of armorplate placement is at a distance ıal to X_B from the outlet. The quantity of armorplate in cubic feet is determined by

$$V_{ar} = 0.19\ (h_t^*)^2 D \tag{4}$$

which D is the maximum diameter of armorplate in inches.

Also of significance to the field engineer would be the quantity of graded armorplate pressed as a percent of the total volume of the pre-shaped basin. Eq. 4 can be itten as

$$\%Ar = \frac{V_{ar}}{V_{st}} \times 100 = \frac{19\ D}{h_t^*} \tag{5}$$

For the experimental tests D = 1 in. was used; therefore, the percent of armorplate this example is

$$\%Ar = \frac{19}{2.35} = 8.1$$

$$V_{ar} = 1.05\ ft^3$$

As indicated by Figure 5, the %Ar as determined by Eq. 5 would be sufficient to ovide for an effective armorplated, pre-shaped stilling basin for this particular ex-ple. The 8.1 percent line would also lie between the 6.92 and 13.84 percent lines h nearly a zero increase in size basin, h_t^* as time continues.

To make the design criteria applicable to field conditions such as illustrated in ure 1, it was necessary to investigate scour as influenced by boundary geometry the culvert outlet.

INFLUENCE OF BOUNDARY GEOMETRY ON SCOUR

Any consideration of the effect of boundary geometry on scour in the pre-shaped st start with the simplest boundary form. A channel that is rectangular in cross-tion is of this type. By keeping the sides rigid, the effect of channel width, B, on rate of scour was investigated.

The results of these tests indicated that as the channel becomes wider, the rate of scour increases for any given increment of time. This increase in scour is due in part to the standing eddy which develops between the impinging jet and the channel sides. As the channel becomes wider, the eddy increases in both magnitude and stability.

The standing eddy exerts a shear force along the bed surface and with a decreasin pressure gradient normal to the bed, lifts into suspension the fine sediment particles As it rotates, the suspended particles are carried into the jet flow and eventually to a point of deposition downstream.

The effect of the channel sides is to destroy the energy of the eddy by providing frictional resistance to the rotating fluid. As the channel sides approach the impingi jet, there is an increase in the energy transfer to small scale turbulence. This smal scale turbulence is then dissipated in the form of heat.

Another important factor to be studied was the effect of side slopes of the channel bank on scour. For this case the channel cross-section was of a trapezoidal shape. In the field, rigid channel boundaries are seldom encountered at culvert outlets;

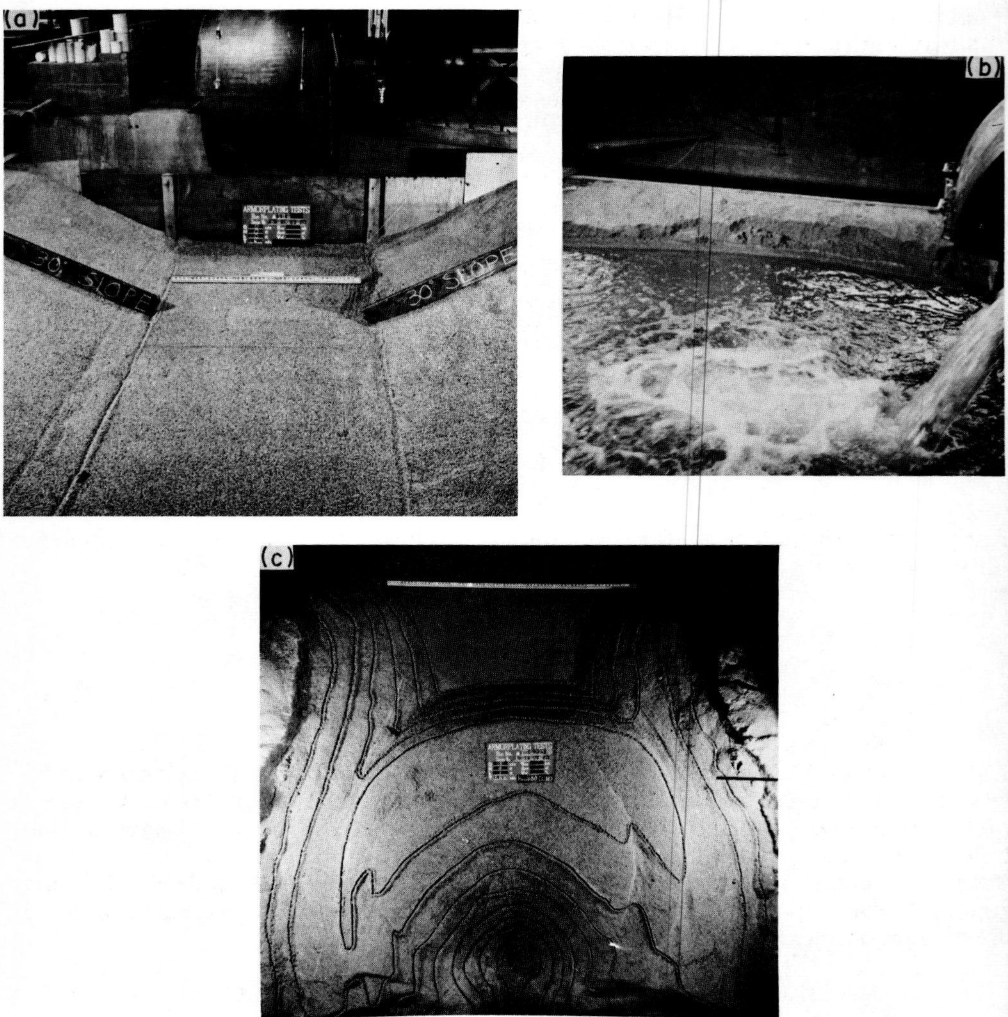


Figure 6. Scour in an alluvial channel at a culvert outlet without armorplate pro- tection. Discharge 1 cfs, tailwater 1 ft, and time of scour 8 hr; (a) initial be condition, (b) flow pattern, and (c) final scour pattern.

erefore for this study the alluvial banks, whose slopes were made equal to the natural gle of repose of the material composing the banks, were studied.

To stabilize an alluvial bank, it is necessary that those flow characteristics causing s erosion be determined. Keeping the channel width constant (Fig. 6), a study was ade of the flow phenomenon causing disintegration of an alluvial bank.

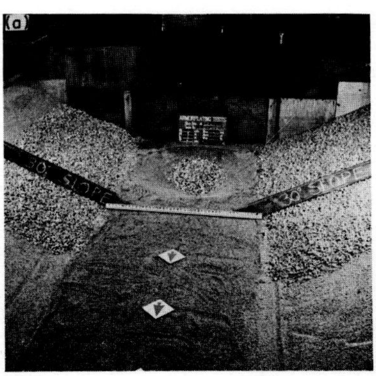
During the laboratory tests it was noted that the waves emanating from the stilling ol area constantly attacked the channel banks. The impact of the wave on the bank slodged the fine soil particles, which were then carried away by the receding or re-cted wave as suspended material. This suspended material was carried into the ain stream flow by the action of the standing eddy between the bank and the center of e stilling pool. As the soil particles were removed, sloughing of the bank occurred der the action of gravity and wave forces, and the exposed raw bank provided a fresh urce of fine soil particles. As the banks eroded, the stilling area increased in diam-er. There was a decrease in the amplitude of the waves reaching the bank, and an crease in size of the standing eddy. Kinetic energy in one form or another was con-ntly attacking the channel banks. From the results of this study it can be inferred t stability of the channel at the culvert outlet would require not only an armorplated, e-shaped basin for protection of the channel bed against erosion by the energy of e impinging jet of water, but also armorplating of the channel banks against erosion wave action. This is demonstrated in Figure 7 which shows the effect of armorplate a means of protection against erosion of an alluvial channel of a trapezoidal shape wnstream of a cantilevered culvert outlet.

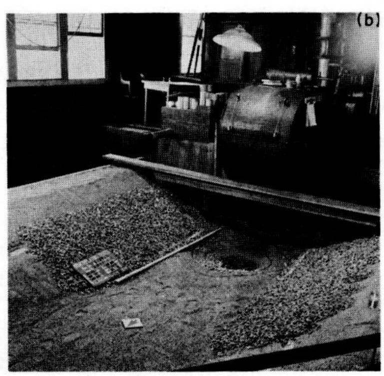
SUMMARY

The problem of erosion control at culvert outlets depends on dissipation of kinetic ergy in the horizontal direction, vertical direction, or combination of both directions. ssipation of kinetic energy may be controlled in the vertical direction by use of a e-shaped armorplated stilling basin.

Systematic experiments upon the rate of scour and scour control in an alluvial bed a jet of freely falling clear water have indicated the following essential facts:

1. A graded gravel has proved to be extremely effective as protection against e-sion from high-velocity flow and waves. It is important, however, that the gravel graded so that the larger size material is protected from undermining by the smaller e material underneath. The maximum size of the gravel material must be sufficient resist movement from the stilling basin.





ure 7. Scour in an alluvial channel at a culvert outlet with armorplate protec-n. Discharge 1 cfs, tailwater 1 ft, and time of scour 8 hr; (a) initial bed condi-tion, and (b) final scour pattern.

2. Wave action and standing eddies at culvert outlets are two important factors in causing erosion of alluvial banks. An increase in channel width near the culvert outl causes an increase in the rate of scour.
3. Armorplate protection must be provided to the channel banks within the stilling basin area to provide protection from waves and eddies.

Under the laboratory test conditions the characteristics of a standard pre-shaped stilling basin in terms of h_t^* are given by

$$d_T = 2.38\ h_t^*$$
$$h = 0.5\ h_t^*$$
$$d_B = 0.65\ h_t^*$$

in which h_t^* is the cube root of the volume of scour, $(V_{st})^{1/3}$ developed by a jet of wa in t hours of scouring action. Furthermore, for the laboratory conditions a minimu of 10 percent armorplate based on the volume of scour hole at one hour of scour effectively controlled the erosion process. In the report (6) an analysis is made to determine h_t^* and size of armorplate material for a range of flow and stream bed mate characteristics.

ADDITIONAL STUDY REQUIRED

Practical results of importance to highway engineers will result from fundamenta research investigations regarding the dynamics of the kinetic energy of a jet of fluid and its proper control. In general, in order to obtain improved design criteria for stilling basins at culvert outlets, it will be necessary to study and understand the mechanics of scour and energy dissipation more thoroughly. Although research advances made in the last few years have been very fruitful, some of the studies that w still need to be conducted are:

1. The influence of various types of natural bed materials—cohesive and non-cohesive soils—on the performance of an armorplated, pre-shaped stilling basin.
2. The effect of the degree of overlap of material size between armorplating material and bed material on the rate and control of scour.
3. To determine maximum size and gradation of armorplate for varied flow conditions to control scour.
4. The effect of discharge and height of fall of the jet on the scour phenomenon i the stilling basin.
5. The influence of channel boundary conditions as they relate to channel width a slope of channel sides downstream from the culvert outlet.
6. Investigation of the influence of a rising and falling hydrograph on the design criteria for a pre-shaped basin.
7. A research study using results previously obtained, on low cost armorplating scour control for culverts laid on the natural grade of the stream bed.

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