HYDRAULIC MODEL STUDY OF SPUR DIKES FOR HIGHWAY BRIDGE OPENINGS

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FOREWORD

This research study of spur dikes for highway bridge openings was conducted in the Hydraulics Laboratory of Colorado State University under the joint sponsorship of the State Highway Departments of Mississippi and Alabama in cooperation with the Hydraulics Research Division of the U. S. Bureau of Public Roads.

The preliminary studies were made in the period July to December of 1958; the results of which are reported in "Progress Report on Hydraulic Model Study of Spur Dikes for Highway Bridge Openings," January 1959 by S. S. Karaki. Further laboratory research was conducted in the period January to September 1959. The results of the research during the latter period is contained in this report.

The writer wishes to express grateful appreciation to the State Highway Departments of Mississippi and Alabama for the opportunity to inspect spur dike installations and many bridge sites in both states.

The laboratory study was conducted with the assistance of Messrs. F. Videon, M. Poreh, B. Bryner, and other staff members. General supervision and advice was given by A. R. Chamberlain, Acting Dean, College of Engineering, Colorado State University. Special acknowledgments are due C. F. Izzard, J. N. Bradley, and D. Hallmark of the Hydraulic Research Division of the U. S. Bureau of Public Roads for their valuable technical assistance.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES - Continued

LIST OF FIGURES - Continued

LIST OF FIGURES - Continued

LIST OF TABLES.

SYNOPSIS

A laboratory study of spur dikes for highway bridges was conducted in the Hydraulics Laboratory of Colorado State University. From this study, a tentative guide for design of spur dikes was developed for roadway crossings normal to the stream channel. The design chart is shown in Fig. 23. The results were qualitative and additional studies are needed for refinement and adjustment. There were also other areas of the problem encountered in this study which need further investigation. Despite the limitations of the study, the results should be applicable to a wide range of actual field installations.

I. INTRODUCTION

Scour at bridge abutments during floods has long been a problem to bridge engineers. Where a highway crosses a river with a wide flood plain it is necessary, for economic reasons, to project the highway fill onto the flood plain so that a minimum bridge length is constructed. During floods the roadway embankment forces the flow along the embankment to the bridge opening. The flow is usually sufficiently large to develop locally high velocities and eddies at the abutment of the bridge and the ensuing scour causes the bridges to fail.

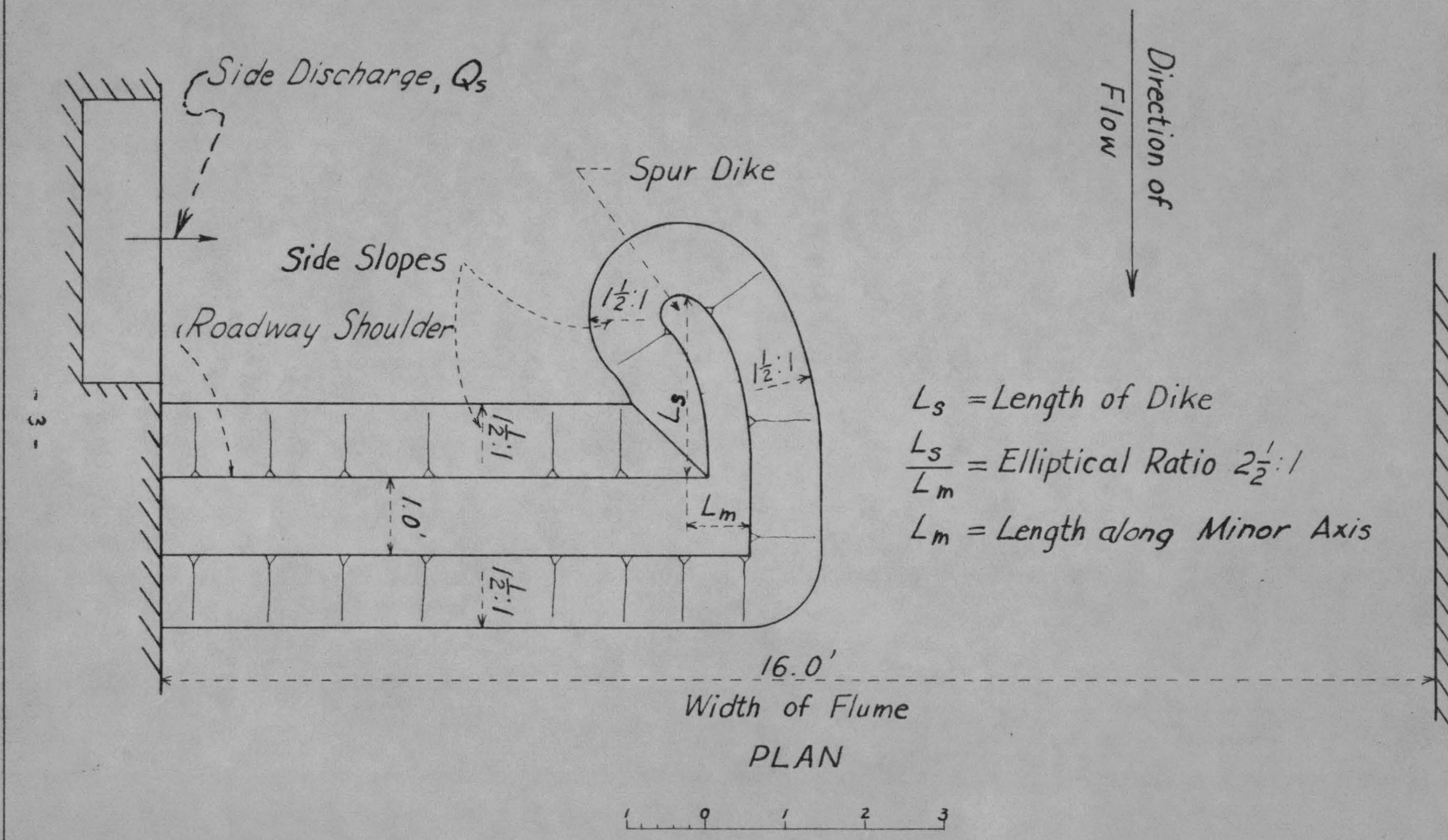
Flow on the flood plain is not always uniform. In some states, such as Mississippi and Alabama, entire stream channels are usually very thickly wooded. Occasionally, however, the woods are cleared on the flood plain and developed into productive agricultural areas. These cleared areas constitute reduced resistance to flow, and this concentrates the flow. The highway right of ways are also usually cleared to facilitate construction. As the flow on the flood plain meets the cleared right of way the water follows the path of least resistance and flows along the cleared area. Thus at the bridge, the flow must make an acute turn. The momentum of the flow however does not permit an abrupt change in direction; and a zone of separation is created at the abutment. This separation zone not only creates scouring eddies but reduces the effective bridge opening and hence the efficiency of the waterway.

Spur dikes have been used by a number of states to eliminate separation and scour at the abutment, and to increase the efficiency of the bridge opening. Generally described, a spur dike is a projection constructed near a bridge abutment to streamline the flow through the bridge opening. The dikes may be either permeable or nonpermeable. Loose rock-fill timber cribs, rock-fill embankments, and open timber pilings are examples of permeable dikes, while earth embankments and solid timber sheeting can be classed as non-permeable dikes. The study reported herein was limited to non-permeable earth embankment spur dikes.

The purpose of this laboratory study was to develop or formulate criteria for field designs of spur dikes based upon the hydraulics of flow observed in the laboratory flume. Despite numerous and varied construction of spur dikes both in type and location by the various states, there is an apparent lack of uniform criteria to guide highway engineers in the design of spur dikes.

The initial phase of the study determined that spur dikes function most effectively if constructed at the bridge abutment. For spill-through abutments, the side slope of the spur dike should be tangent to the abutment to avoid discontinuity of the flow boundary. It was also determined that curved dikes were superior to straight dikes, essentially because they conformed more closely to the flow lines. Various elliptical dikes were studied, and for the conditions of flow in the flume, a dike with a major to minor axis ratio of 2-1/2: 1 was most satisfactory, see Fig. 1. for a definition sketch.

The study was conducted in a movable bed flume but was limited to clear water flows with no bed movement except in the contracted zone. Only one size of bed material was used. The sieve analysis of the sand is shown in Fig. 2. From the results of the preliminary studies, a spur dike with a major to minor axis ratio of 2-1/2: 1 was used. Although the major portion of the study was concerned with conditions simulating bridge abutments on a flood plain, a limited study was made to determine the effect of the proximity of the opposing abutment on the length of spur dike. Limited studies were also made to determine the effect of bridges skewed both upstream and downstream with respect to the flow. River channels were not included in any of the studies nor were any attempts made to simulate wooded and cleared areas.



Definition Sketch
Elliptical Spur Dike

FIG. 1

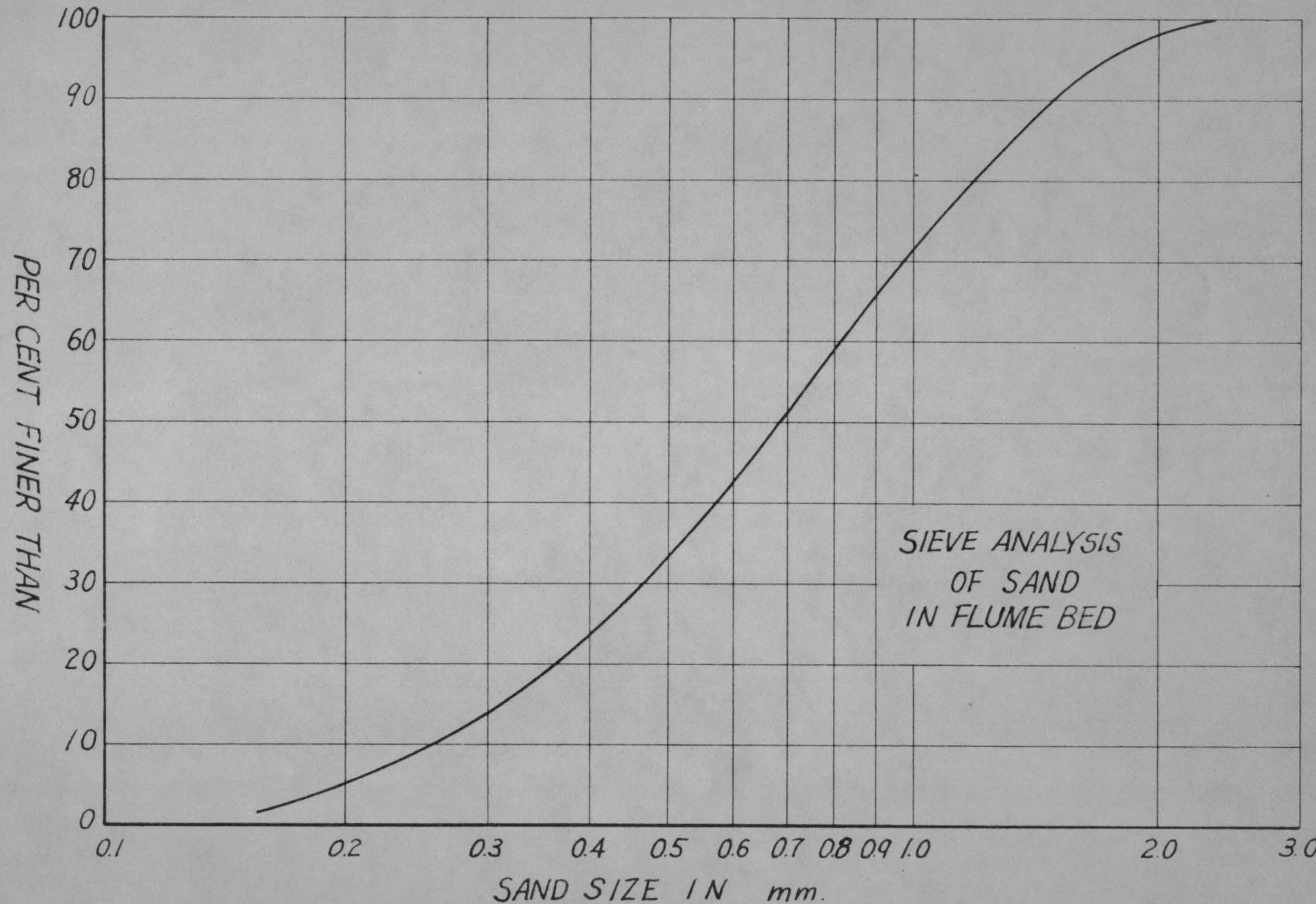

SIEVE ANALYSIS
OF SAND
IN FLUME BED
PER CENT FINER THAN
100
90
80
70
60
50
40
30
20
10
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2.0
3.0
SAND SIZE IN mm.

FIG. 2

II. LABORATORY EQUIPMENT

Flume

The laboratory study was conducted in a flume 16 feet wide and 84 feet long with a 20-foot long test section. The test section had a recessed floor about 4 feet deeper than the normal flume bottom to provide for scour depth at the bridge section. The slope of the flume bed was the same (0.0003) for all runs. Provision was made for separate inflow to the test section from one side of the flume and upstream from the highway embankment. This facility enabled simulation of longer roadway embankments than that normally possible in the 16 foot flume width.

The entire flume bed consisted of sand to enable determination of scour pattern around the bridge piers, abutments, and the spur dikes. See Fig. 2 for a graphical presentation of the sieve analysis for the sand.

Models

The highway embankment models were made one foot wide at the top and the roadway was placed 0.6 foot above the flume bed. Side slopes were 1-1/2: 1. The spur dikes were both erodible and nonerodible. For the preliminary studies and the latter part of the study which involved riprap, erodible dikes were used. All dikes were 3 inches wide at the top and constructed to the same height as the roadway embankment. Dimensions of the dikes are referenced to the shoulder of the roadway as shown in Fig. 1.

The side slopes of the spur dikes were 1-1/2: 1 except for the riprap studies where 2:1 slopes were used to determine if undercutting occurred if the riprap did not slide down the face of the dike as the scour hole formed. Riprap for the dikes consisted of 3/8 inch median size gravel with gradation in size from 1/4 inch to 1/2 inch. The gravel was placed at random on the face of the dike. For field construction, a gravel blanket or graded riprap consistent with good engineering practice, should be used to prevent the underlying materials from filtering through the riprap.

III. TEST PROCEDURE

Test Constants

The entire study was limited to clear water (no upstream or recirculating supply of sediment), with flow quantities varying in magnitude with the constriction of the embankment in the flume. The normal discharge for the flume without a constriction and at 0.4 foot depth used in the preliminary studies was 4.8 c.f.s. The average velocity at this discharge and depth was 0.75 ft/sec. An embankment which provided a contraction ratio of 0.50 at the test section was found to produce a scour hole about 0.75 foot deep in a time of 5 hours. This was considered satisfactory flume conditions and all tests were made comparable. Contraction ratio is the length of embankment divided by the total flume width. The flow depth was maintained at 0.4 foot for all tests.

When side flow was used, the total discharge through the bridge opening was maintained constant for the series to enable comparisons. Thus, the total discharge from the head of the flume was decreased by the amount of the side flow.

Procedure for Each Test

The channel bed was leveled before each run and the same bed slope was used for all of the tests. Water was introduced into the flume from both the upstream and downstream ends 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 was shut off and the upstream discharge increased to the proper amount. The water depth was controlled at the downstream end of the flume to 0.4 ft. depth.

After 5 hours, the upstream discharge was shut off and as the water receded, the scour hole was contoured at 0.1 foot intervals. The water surface was measured with a point gage.

Data Taken

For all tests, the results were recorded by photographs. Both motion pictures and still photos were used. A motion picture film was assembled and accompanies this report. Some of the still photographs are shown in this report, while others are included in the preliminary report. Because of the great number of photographs taken, not all of the runs are included in the reports. A complete file has been made at the Washington Office of the U. S. Bureau of Public Roads.

Notation

The following is a list of definitions for symbols used in this report. When convenient, terms are also defined where they are first used.

- L_o - length of bridge opening tested in the flume.
- L_s - length of spur dike measured along the major axis of the ellipse, normal to the roadway.
- L_e - length of embankment projected into the stream channel, normal to the direction of flow.
- W_s - the width at the bridge section, measured from the abutment through which the embankment flow Q_e is concentrated. This is identified by the limit of local scour.
- d_s - depth of scour measured at the bridge section.
- Q_e - quantity of flow obstructed by the roadway embankment in c.f.s.
- Q_t - total discharge through L_o of the flume measured in c.f.s.

Q_t^* - total quantity of flow through W_s . The value is equal to Q_e plus Q_{ws} .

Q_{ws} - quantity of flow approaching W_s normally.

Q_s - quantity of flow entering from the side of the flume.

M - contraction = $\frac{16 - L_o}{16}$

IV. PRESENTATION AND DISCUSSION OF RESULTS

The results of the study are qualitative, and all tests are assumed to be comparable, except for those otherwise specifically designated. Because of the limited time for the study, it was decided to use only the 2-1/2: 1 elliptical dike for this phase of the study. This ratio was determined as being a satisfactory shape from the preliminary study. The time of run was also maintained at 5 hours as a base for comparison of each test result. Several runs were made for a period of time longer than 5 hours, and the results are shown graphically in Fig. 3.

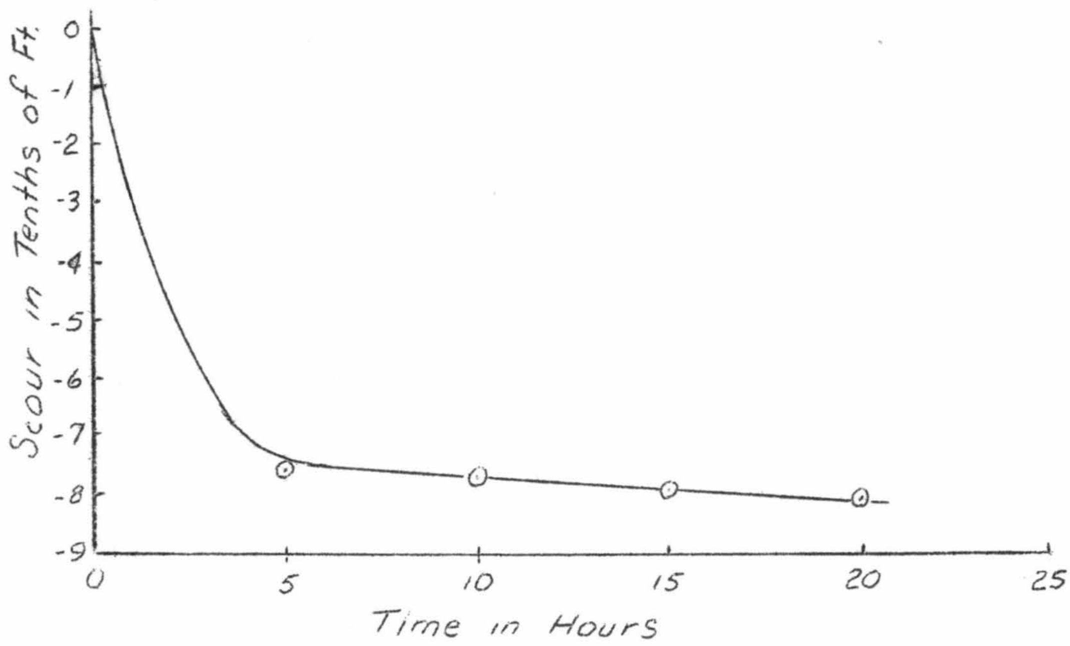


Fig. 3. Time rate of scour at the abutment with no spur dike.

This figure shows that although the time was limited to 5 hours, somewhat arbitrarily, this period contained the interval where the greatest portion of the scour occurred.

Normal Embankments

Embankments normal to the flume walls were studied to determine the effect of embankment length, L_e , on the spur dike length L_s. Although values of L_e varied, there were basically three sizes of clear bridge openings, L_o , tested in the flume. Values of L_o were 4.8 , 8 and 11.2 ft. In this series, it was assumed that the flume wall in the bridge opening approximated a flow line. It was further assumed that the wall had little influence on the scour pattern around the abutments and the dikes.

The total discharge, Q_t , for tests with L_o = 8 ft was 4.8 c.f.s., and the flow depth was 0.4 ft at the downstream tailgate control. Discharge from the side of the flume, Q_s , was introduced to simulate longer embankment, using the assumption of uniform approach flow. ? When side flow was introduced, the inflow from the head of the flume was reduced by the same amount so that the total discharge through the contracted section remained the same. If the discharge had varied, comparisons with the base tests would not have been possible.

The initial test run was made with no spur dike and no side flow. The deepest point of scour occurred at the upstream corner of the abutment, and in the 5-hour period, reached a depth of 0.75 feet. Other test runs were made with side flows of 0.25, 0.75, and 1.25 c.f.s. to increase the embankment flows, Q_e , to 2.53, 2.78, and 3.03 c.f.s. respectively. Q_e is defined as the quantity of flow obstructed by the roadway embankment projecting into the stream channel and will be described as embankment flow. The results did not indicate substantial differences in the scour pattern or depth, nor were any discernible differences noted in the flow pattern. Results of tests with Q_e = 2.40 and 3.03 c.f.s. are shown in Figs. 4 and 5. In these and subsequent figures, the white lines are contours of scour depth with intervals of 0.1 ft. The symbols L_o , Q_t

Fig. 4. $L_o = 8.0$ ft. $Q_e = 2.40$ c.f.s.
$Q_t = 4.80$ c.f.s. Contour interval 0.1 ft.

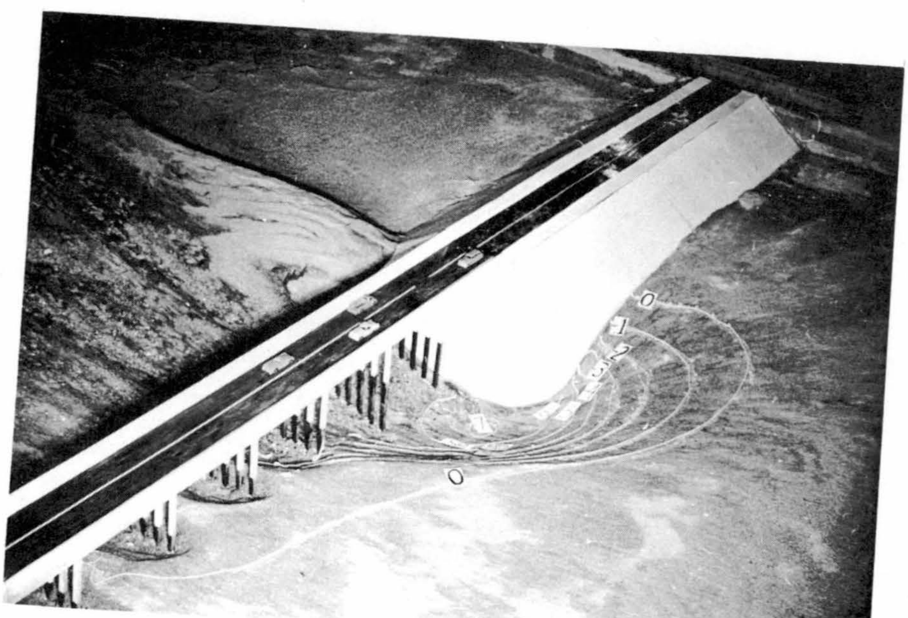


Fig. 5. $L_o = 8.0$ ft. $Q_e = 3.03$ c.f.s.
$Q_t = 4.80$ c.f.s.

and Q_e is as explained above. There was a tendency for the deepest point of scour to develop away from the abutment as Q_e became greater. This was due to the greater momentum of flow from the side and parallel to the roadway.

It appears, therefore, in general, that the greater the momentum of flow along the embankment, the less effective will be the total bridge opening; thus, greater is the need of a spur dike to break up the flow and redirect it efficiently through the bridge opening. The geometry of the spur dike is thus a function of the momentum of flow obstructed by the roadway embankment, which in turn, assuming uniform flow distribution upstream, would be a function of the roadway embankment length, L_e.

Spur dike lengths, L_s , of 1.5, 2, 3, and 4 feet were tested with various quantities of Q_e embankment discharge. L_s is measured normal to the roadway embankment along the major axis of the ellipse. The results were recorded photographically in motion pictures, and still photos, some of which are shown in Figs. 6 to 13. The data obtained from the photographs along with the analyzed results are given in Table 1.

In the tests with the various spur dike lengths, the maximum scour depth was a direct function of the embankment flow. That is, as Q_e was increased, the maximum depth of scour increased. However, the safety of the bridge is not predicated on the maximum scour depth, but on the scour at the bridge section, which will be designated d_s . As Q_e increased, d_s decreased for, as the momentum of flow from the side increased, the spur dike distributed or "spread" the flow more widely through the bridge opening. Compare Figs. 6 and 7, which show the results with a 1.5-foot spur dike and embankment flows of 2.78 and 3.15 c.f.s. respectively. Compare also Figs. 8 and 9 with the 2-foot spur dike; Figs. 10 and 11 with the 3-foot spur dike; Figs. 12 and 13 with the 4-foot spur dike.

Fig. 6. L_o = 8.0 ft. L_s = 1.5 ft.
Q_e = 2.78 c.f.s. Q_t = 4.80 c.f.s.

Fig. 7. L_o = 8.0 ft. L_s = 1.5 ft.
Q_e = 3.15 c.f.s. Q_t = 4.80 c.f.s.

Fig. 8. $L_o = 8.0$ ft. $L_s = 2.0$ ft.
$Q_e = 2.40$ c.f.s. $Q_t = 4.80$ c.f.s.

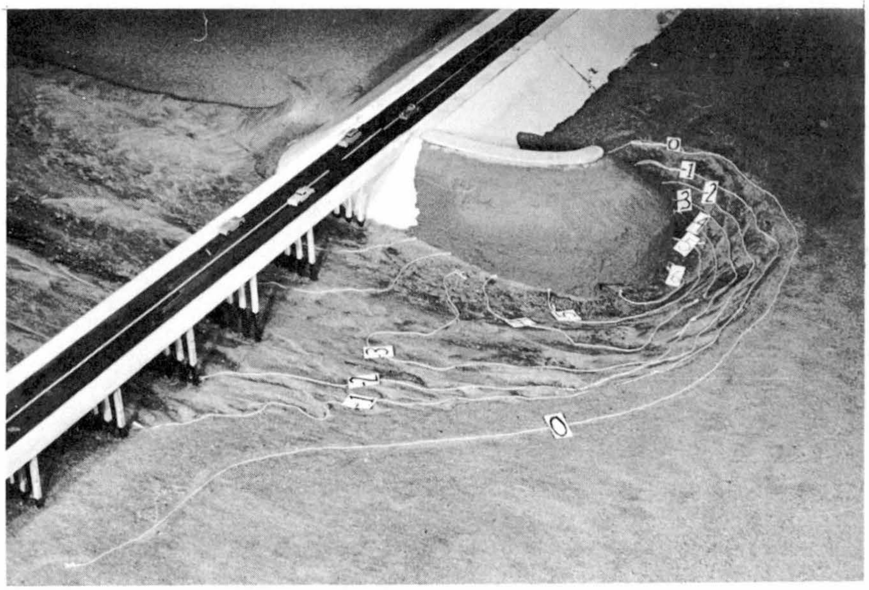


Fig. 9. $L_o = 8.0$ ft. $L_s = 2.0$ ft.
$Q_e = 2.90$ c.f.s. $Q_t = 4.80$ c.f.s.

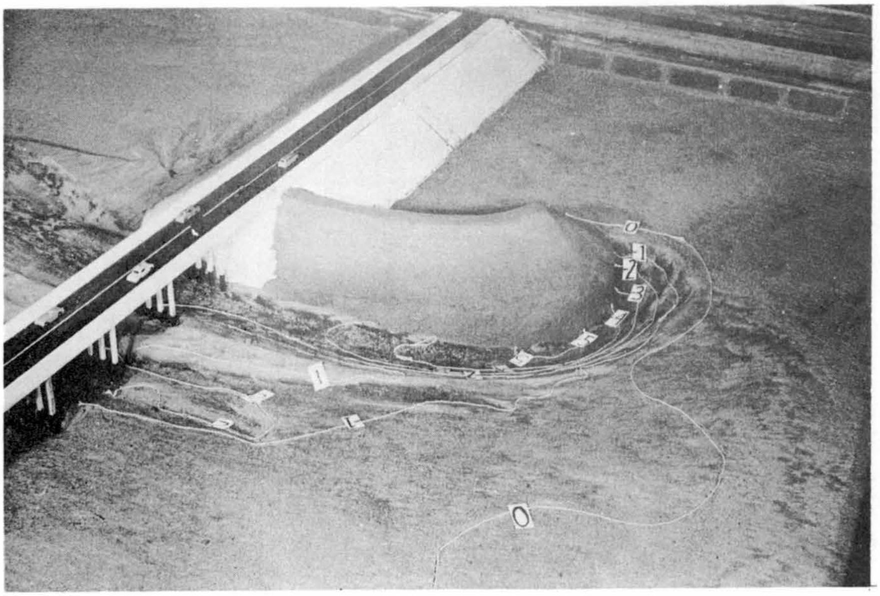


Fig. 10. $L_o = 8.0$ ft. $L_s = 3.0$ ft.
$Q_e = 2.40$ c.f.s. $Q_t = 4.80$ c.f.s.

Fig. 11. $L_o = 8.0$ ft. $L_s = 3.0$ ft.
$Q_e = 3.15$ c.f.s. $Q_t = 4.80$ c.f.s.

Fig. 12. $L_o = 8.0$ ft. $L_s = 4.0$ ft.
$Q_e = 2.40$ c.f.s. $Q_t = 4.80$ c.f.s.

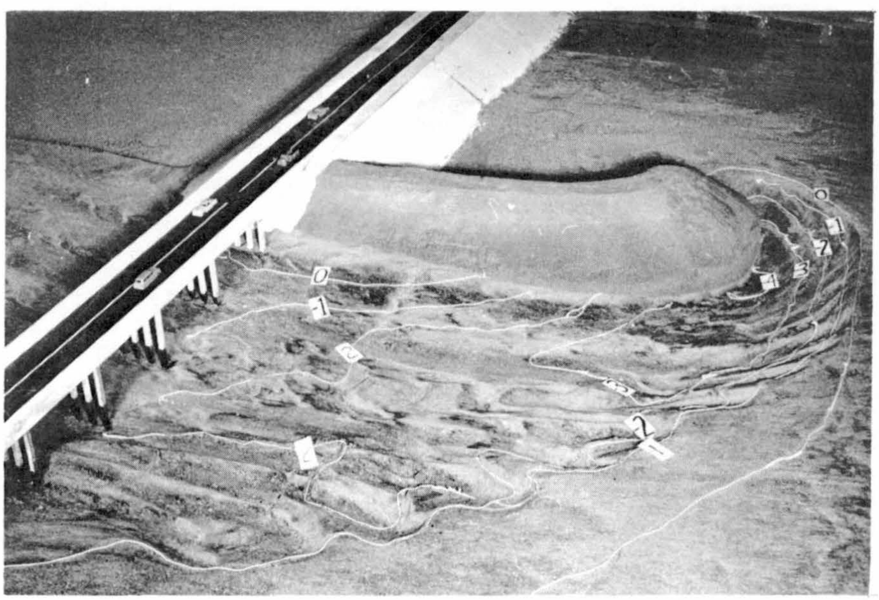


Fig. 13. $L_o = 8.0$ ft. $L_s = 4.0$ ft.
$Q_e = 2.78$ c.f.s. $Q_t = 4.80$ c.f.s.

It will also be noted, in the foregoing figures, that both the maximum depth of scour and d_s are inversely proportional to the length of the spur dike. As L_s increases in length from 1.5 ft to 4 feet, the maximum scour depth decreases from about 0.7 ft to 0.4 ft, and d_s reduces from 0.3 ft. to about 0.1 ft. Also, the longer the spur dike, the greater is the width of spread of the flow. Fig. 14 shows the relationship graphically. Width of spread, W_s, is defined here as the width at the bridge section, measured at the normal bed level, through which the embankment flow is concentrated. Photographically this is identified as the edge of the scour pattern under the bridge.

The roadway embankment length was increased in the flume to reduce the bridge opening, L_o, to 4.8 feet. If the discharge of 4.8 c.f.s. were made to flow through the smaller opening, there would be a greater unit discharge through the opening, which would result in a different scour rate from the previous tests. Thus, in a time of 5 hours, the total scour would not be comparable to other tests. Either the time or the discharge, Q_t, could be adjusted. For this study, the discharge, Q_t, was adjusted by trial and set at 3.0 c.f.s., for at this discharge, with no side flow and no spur dike, the scour at the abutment was approximately the same as the scour produced with L_o of 8.0 ft, and Q_t of 4.8 c.f.s.

Spur dikes 1.5, 2, and 3 feet long were tested at the abutment with various embankment discharges. Figs. 15 to 20 indicated similar results to those obtained with spur dikes at the 8.0 ft opening. There is an exception for the test condition shown in Fig. 20 where the proximity of the flume wall affected the flow lines, resulting in a smaller width of spread through the bridge opening.

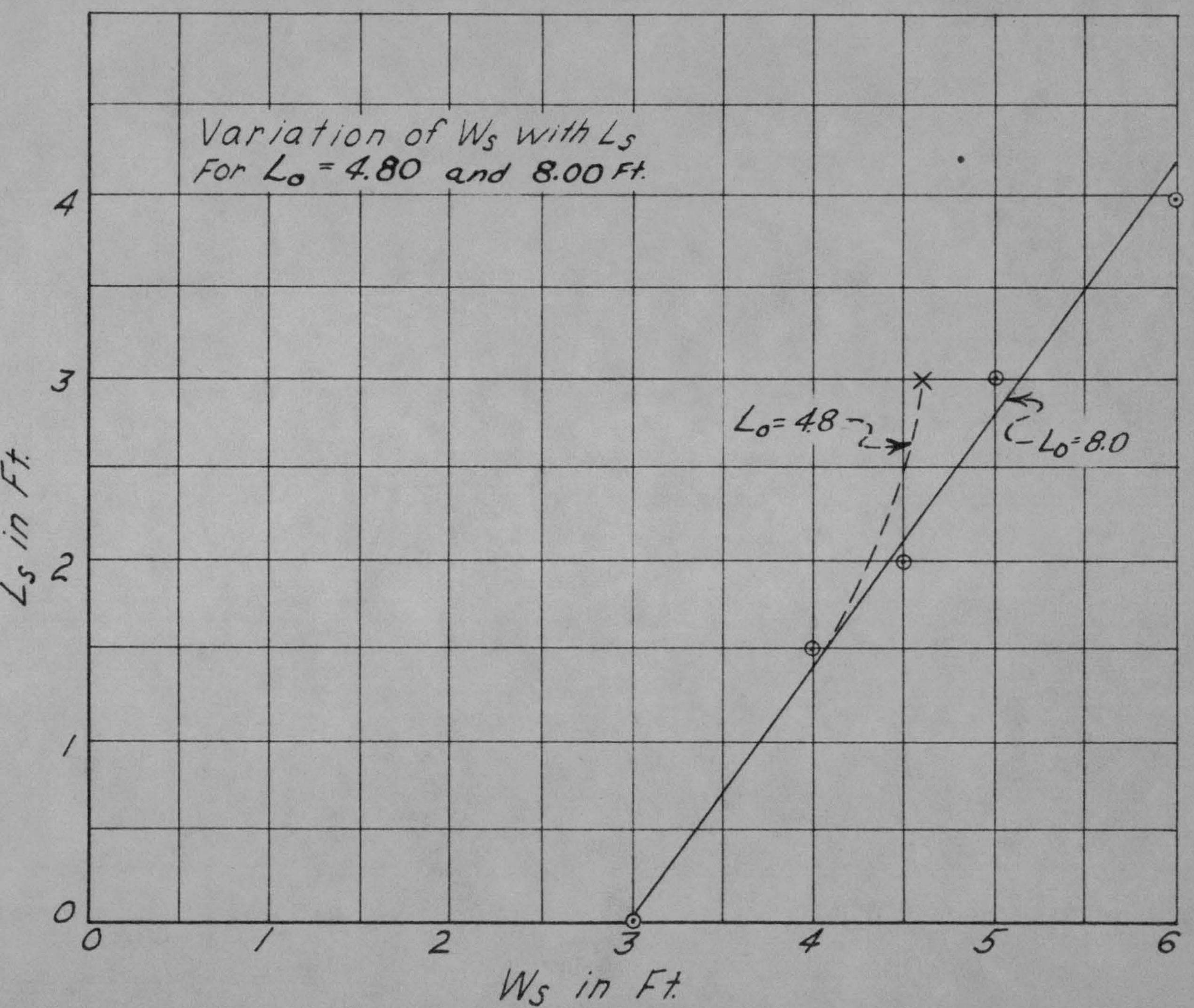

Variation of Ws with Ls
For Lo = 4.80 and 8.00 Ft.
Lo = 4.8
Lo = 8.0
Ls in Ft.
Ws in Ft.
0
1
2
3
4
5
6

FIG. 14

Fig. 15. L_o = 4.8 ft. L_s = 1.5 ft.
Q_e = 2.10 c.f.s. Q_t = 3.00 c.f.s

Fig. 16. L_o = 4.8 ft. L_s = 1.5 ft.
Q_e = 2.33 c.f.s. Q_t = 3.00 c.f.s.

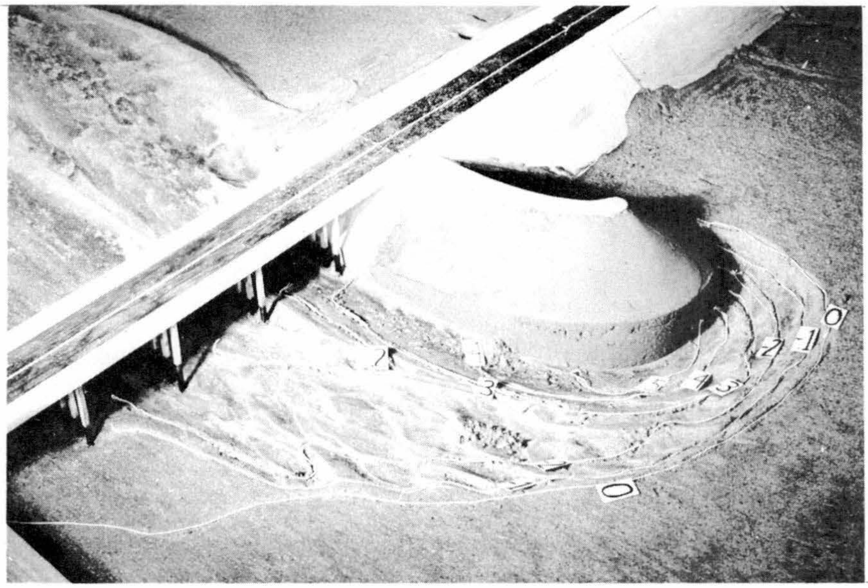


Fig. 17. L_o = 4.8 ft. L_s = 2.0 ft.
Q_e = 2.10 c.f.s. Q_t = 3.00 c.f.s.

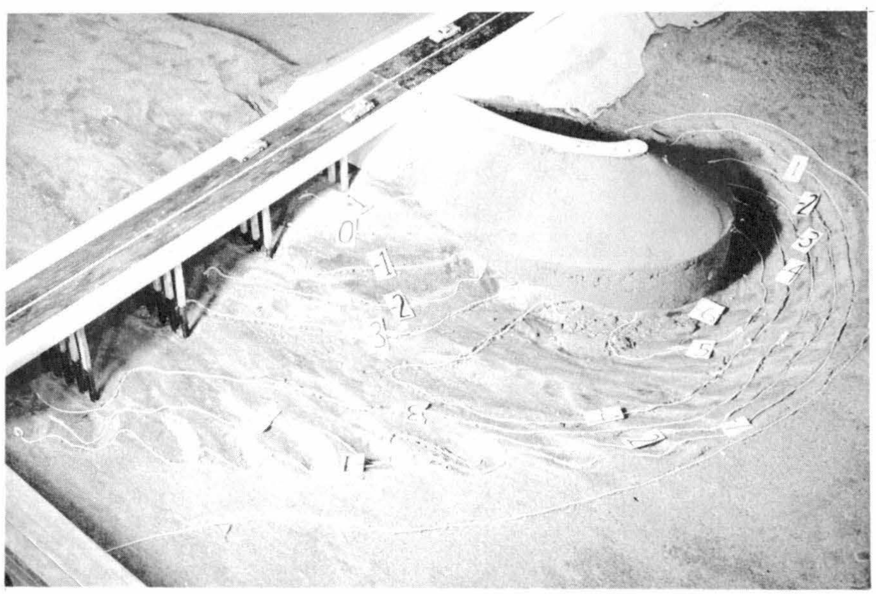


Fig. 18. L_o = 4.8 ft. L_s = 2.0 ft.
Q_e = 2.55 c.f.s. Q_t = 3.00 c.f.s

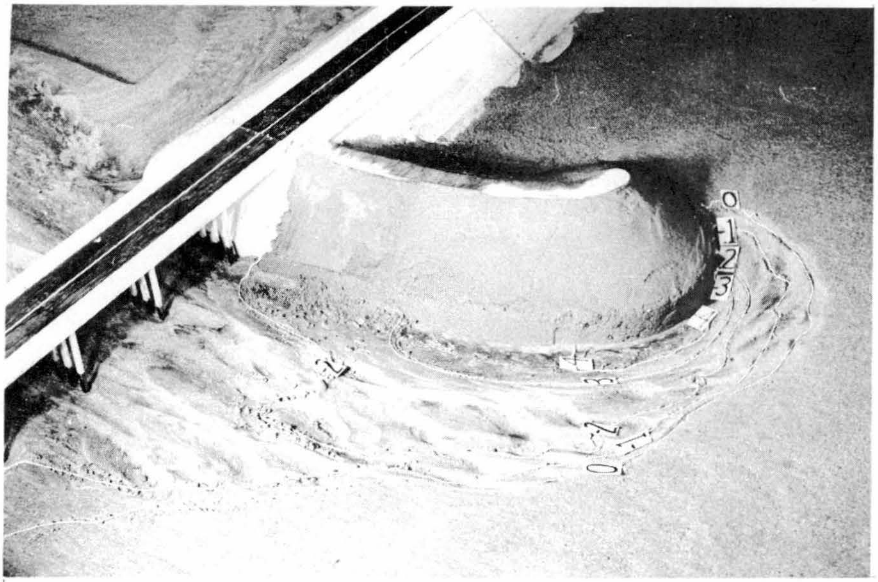


Fig. 19. $L_o = 4.8$ ft. $L_s = 3.0$ ft.
$Q_e = 2.10$ c.f.s. $Q_t = 3.00$ c.f.s.

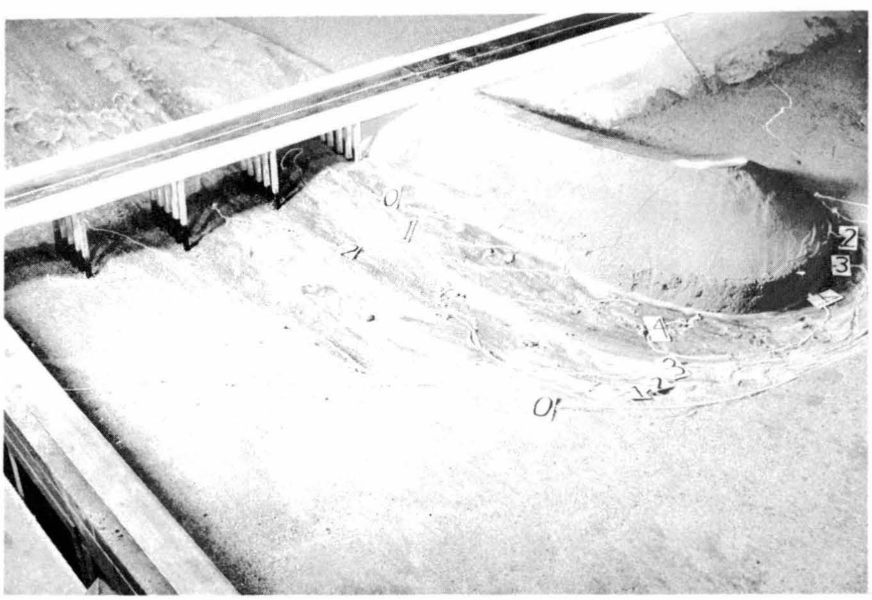


Fig. 20. $L_o = 4.8$ ft. $L_s = 3.0$ ft.
$Q_e = 2.33$ c.f.s. $Q_t = 3.00$ c.f.s.

The same procedure described above was used to test a long bridge opening of 11.2 feet. An attempt was made to increase Q_t sufficiently to develop 0.7 ft scour at the abutment in a period of 5 hours but with the flow depth maintained at 0.4 ft, ripples began to form on the bed with discharges greater than about 5 c.f.s. Thus, it was not possible to maintain both comparable scour depths and time with respect to the previous tests. However, in order to observe if the same general trend prevailed as with the other bridge lengths, tests were made with a total discharge of 6.0 c.f.s., flow depth at 0.4 foot, and test time of 5 hours. With no spur dike, the maximum depth of scour at the abutment was about 0.5 foot. Side discharge did not affect the scour depth measurably. Tests with the 2-foot and 3-foot spur dikes indicated that the observations made with the two other bridge openings applied essentially to this opening also. Figs. 21 and 22 show the results with the 2-foot spur dike. The scour depths shown in these figures are not comparable to other tests, however, for the reason already discussed.

Data from photographs of the results of the foregoing tests were taken and analyzed in the manner shown in Table 1, and plotted as shown in Fig. 23. It was reasoned that the length of the spur dike influenced the effective width of spread of the flow obstructed by the roadway embankment, and the spur dike length was in turn a function of the roadway embankment with uniform approach flow. In order to generalize the results, the parameters were made dimensionless. The results also show that with a given flow condition, the depth of scour is a function of the spur dike length. Thus, the parameter, d_s/L_s was used. Explanation of the notations used are given on page 8.

Fig. 21. L_o = 11.2 ft. L_s = 2.0 ft.
Q_e = 1.80 c.f.s. Q_t = 6.00 c.f.s.

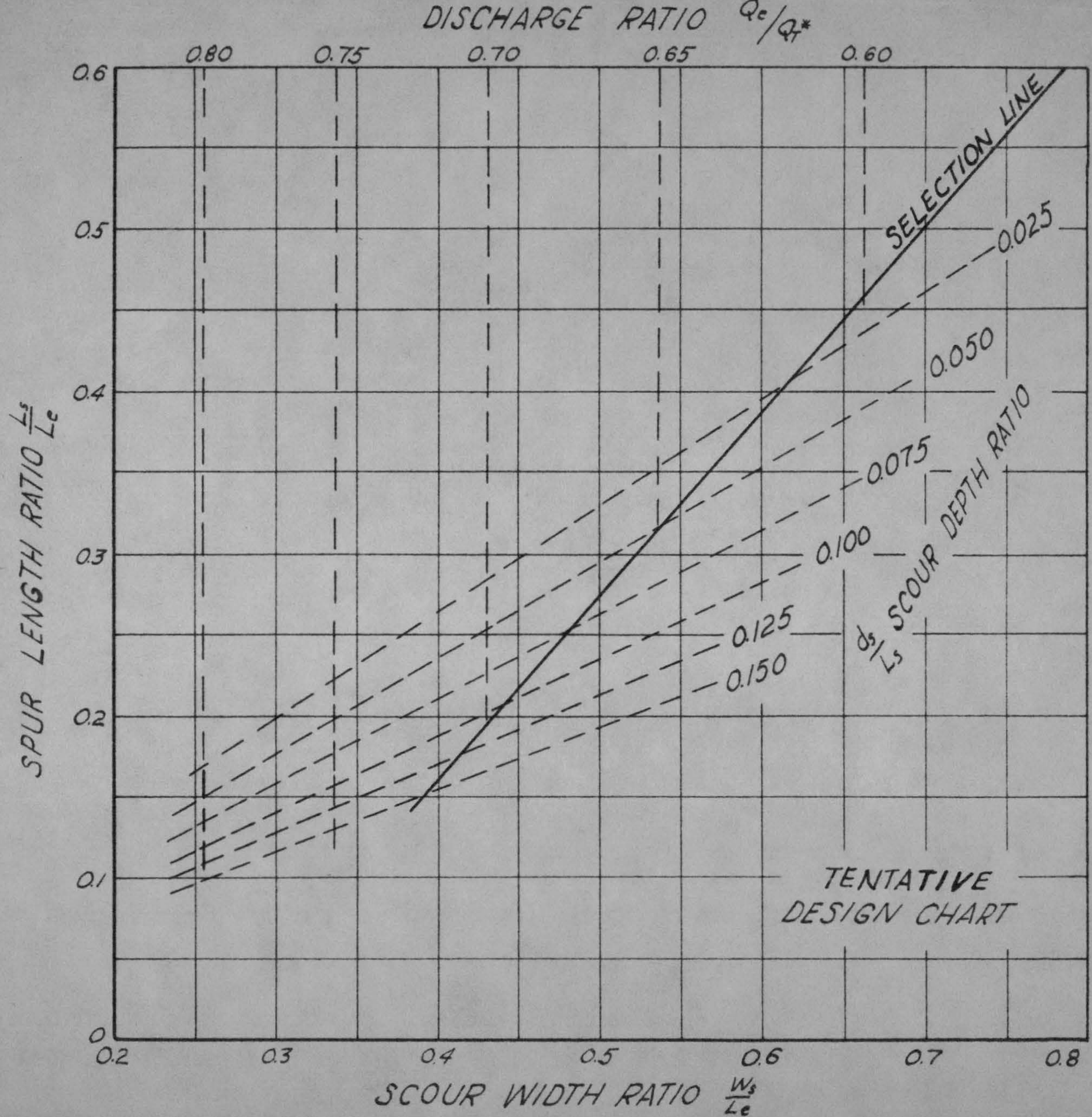
Fig. 22. L_o = 11.2 ft. L_s = 2.0 ft.
Q_e = 2.5 c.f s. Q_t = 6.00 c.f.s.

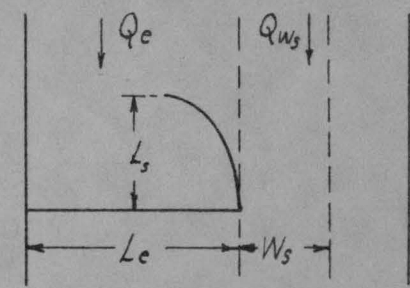
TABLE I

NORMAL EMBANKMENTS
MEASURED AND CALCULATED DATA

Q_s	Q_t	$Q_t - Q_s$	$\overset{*}{M}(Q_t-Q_s)$	Q_e	$M(16)+W_s$	$(Q_t-Q_s)\times\frac{L_e+W_s}{L_t}$	Q_t*	$\frac{Q_e}{Q_t*}$	q	$L_e = \frac{Q_e}{q}$	L_s	$\frac{L_s}{L_e}$	d_s	W_s	$\frac{W_s}{L_e}$	$\frac{d_s}{L_s}$
0	3.0	3.0	2.10	2.10	14.0 11	2.62	2.62	.80	.188	11.2	0	0	0.55	3	.268	----
0	3.0	3.0	2.10	2.10	15.0 11	2.81	2.81	.747	.188	11.2	1.5	.134	0.30	4	.357	.200
.75	3.0	2.25	1.58	2.33	15.0 11	2.11	2.86	.813	.141	16.55	1.5	.091	0.33	4	.242	.220
1.50	3.0	1.50	1.05	2.55	15.0 11	1.406	2.91	.879	.094	27.2	1.5	.055	*	4	.147	----
0	3.0	3.0	2.10	2.10	15.5 11	2.91	2.91	.722	.188	11.2	2.0	.178	0.30	4.5	.401	.150
0.5	3.0	2.5	1.75	2.25	15.5 11	2.42	2.92	.770	.156	14.4	2.0	.139	0.15	4.5	.312	.075
1.5	3.0	1.50	1.05	2.55	15.5 11	1.45	2.95	.865	.094	27.2	2.0	.074	0.35	4.5	.165	.175
0	3.0	3.0	2.10	2.10	15.6 11	2.92	2.92	.720	.188	11.2	3.0	.268	0.20	4.6	.410	.067
0.4	3.0	2.6	1.82	2.22	15.6 11	2.54	2.94	.755	.163	13.68	3.0	.219	0.22	4.6	.345	.073
0.75	3.0	2.25	1.58	2.33	15.6 11	2.20	2.95	.789	.141	16.55	3.0	.181	0.25	4.6	.285	.083
0	4.80	4.80	2.400	2.40	11.0 8	3.30	3.30	.727	.300	8.0	0	0	0.65	3.0	.375	----
0.25	4.80	4.55	2.28	2.53	11.0 8	3.13	3.38	.747	.284	8.90	0	0	0.65	3.0	.337	----
0.75	4.80	4.05	2.03	2.78	11.0 8	2.78	3.53	.786	.253	10.54	0	0	0.55	3.0	.284	----
1.25	4.80	3.55	1.78	3.03	11.0 8	2.44	3.69	.820	.222	13.62	0	0	0.55	3.0	.220	----
1.5 ft. Spur Dike																
0.75	4.80	4.05	2.03	2.78	12.0	3.04	3.79	.732	.253	10.54	1.5	.146	0.25	4.0	.380	.167
1.00	4.80	3.80	1.90	2.90	12.0	2.85	3.85	.754	.238	12.20	1.5	.123	0.30	4.0	.328	.200
1.25	4.80	3.53	1.78	3.03	12.0	2.66	3.91	.773	.222	13.62	1.5	.110	0.30	4.0	.294	.200
1.50	4.80	3.30	1.65	3.15	12.0	2.47	3.97	.793	.206	15.30	1.5	.098	0.25	4.0	.262	.167
2.0 ft. Spur Dike																
0	4.80	4.80	2.400	2.40	12.5	3.75	3.75	.640	.300	8.0	2.0	.250	0.25	4.5	.562	.125
0.25	4.80	4.55	2.28	2.53	12.5	3.56	3.81	.662	.284	8.90	2.0	.225	0.35	4.5	.506	.175
0.50	4.80	4.30	2.15	2.65	12.5	3.36	3.86	.686	.269	9.85	2.0	.203	0.23	4.5	.457	.115
0.75	4.80	4.05	2.03	2.78	12.5	3.16	3.91	.710	.253	10.54	2.0	.190	0.20	4.5	.427	.100
1.00	4.80	3.80	1.95	2.90	12.5	2.97	3.97	.730	.238	12.20	2.0	.164	0.25	4.5	.369	.125
1.25	4.80	3.55	1.78	3.03	12.5	2.78	4.03	.750	.222	13.62	2.0	.147	0.35	4.5	.330	.175
3.0 ft. Spur Dike																
0	4.80	4.80	2.40	2.40	13.0	3.90	3.90	.615	.300	8.0	3.0	.375	0.22	5.0	.625	.073
0.25	4.80	4.55	2.28	2.53	13.0	3.70	3.95	.640	.284	8.90	3.0	.337	0.12	5.0	.562	.04
0.50	4.80	4.30	2.15	2.65	13.0	3.49	3.99	.664	.268	9.86	3.0	.304	0.12	5.0	.507	.04
0.75	4.80	4.05	2.03	2.78	13.0	3.29	4.04	.687	.253	10.54	3.0	.285	0.15	5.0	.474	.05
1.00	4.80	3.80	1.90	2.90	13.0	3.09	4.09	.710	.238	12.20	3.0	.246	0.20	5.0	.410	.07
1.25	4.80	3.55	1.78	3.03	13.0	2.88	4.13	.732	.222	13.62	3.0	.220	0.20	5.0	.367	.07
1.50	4.80	3.30	1.65	3.15	13.0	2.68	4.18	.754	.206	15.30	3.0	.196	0.20	5.0	.327	.07
4.0 ft. Spur Dike																
0	4.80	4.80	2.40	2.40	14.0	4.20	4.20	.571	.300	8.0	4.0	.500	0.13	6.0	.750	.025
.25	4.80	4.55	2.28	2.53	14.0	3.98	4.23	.596	.204	8.90	4.0	.450	0.05	6.0	.675	.012
.50	4.80	4.30	2.15	2.65	14.0	3.76	4.26	.622	.269	9.86	4.0	.405	0.07	6.0	.609	.018
.75	4.80	4.05	2.03	2.78	14.0	3.54	4.29	.685	.253	10.54	4.0	.379	0.20	6.0	.570	.050

* $M = \frac{16-L_o}{16}$





Instructions For Use

1. Try L_s/L_e.
2. Calculate d_s from Selection Line.
3. Calculate W_s.
4. Determine Q_e/Q_T^*. If not satisfactory, try another value of L_s/L_e and repeat.

NOTES:

1. $Q_T^* = Q_e + Q_{Ws}$
2. This chart applies to spill-through abutments.

FIG. 23

The criteria presented in Fig. 23 is tentative. A trial and error method is to be used for design. At any given crossing, it is assumed that the length of the embankment L_e and flood discharge is known. It is desired to determine the length of spur dike to construct at the bridge abutment. First, the minimum value of $\frac{L_s}{L_e}$ of 0.15 will be tried. With this value the length of dike L_s will be calculated. From $\frac{d_s}{L_s}$ given on the selection line corresponding to the value of $\frac{L_s}{L_e}$, d_s is calculated. If this value of d_s is excessive, that is, if it is considered uneconomical to extend the foundations of the piers or abutment, another value of $\frac{L_s}{L_e}$ will be tried. When an acceptable value of d_s is determined, the value of W_s/L_e on the abscissa corresponding to the selected $\frac{L_s}{L_e}$ is read from the selection line. W_s is calculated and Q_{W_s} determined. The value of Q_{W_s} is the quantity of flow which is approaching W_s normally. Knowing L_e, Q_e is estimated. Q_t^*, which is the sum of Q_e and Q_{W_s} is then determined and the ratio $\frac{Q_e}{Q_t^*}$ is computed. This value is then compared to the value of the abscissa given along the top of the chart. If $\frac{Q_e}{Q_t^*}$ is greater than or equal to the value given, the trial length of spur dike is satisfactory.

It is not always a simple matter to determine the distribution of discharge in the stream channel at a given roadway crossing. There are many factors which will affect the flow distribution, such as topography of the area, geometrical alignment of the stream channel, and alterations to the natural surroundings. These factors were not considered in the laboratory, but are important in the field situation.

The limit of the length of roadway embankment L_e to which this chart applies is approximately 2,000 ft. Tentatively, the minimum value of $\frac{L_s}{L_e}$ recommended from the laboratory studies is 0.15. For values of L_e greater than, say 2,000 feet, the computed length of spur dike, practically speaking, becomes excessively long. Other criteria not determined in this study must apply to longer embankments.

45^0 Wing-Wall Abutments

A limited study was made to demonstrate the use of earth embankment spur dikes with 45^0 wing-wall abutments. The wing-wall abutment model was installed in the flume with L_0 of 8.0 ft. The total discharge was 4.8 c.f.s. at a flow depth of 0.4 foot. Tests were made without a spur dike and with spur dikes constructed so that the toe of the spur dike was tangent to the abutment wall. This arrangement represented a discontinuity in the flow line along the spur dike which created secondary scour at the abutment, but the effect does not appear serious.

Results of tests without a spur dike and with 2 and 3-foot spur dikes are shown in Figs. 24 to 26. The spur dikes are essentially as effective as they are for the spill-through abutments.

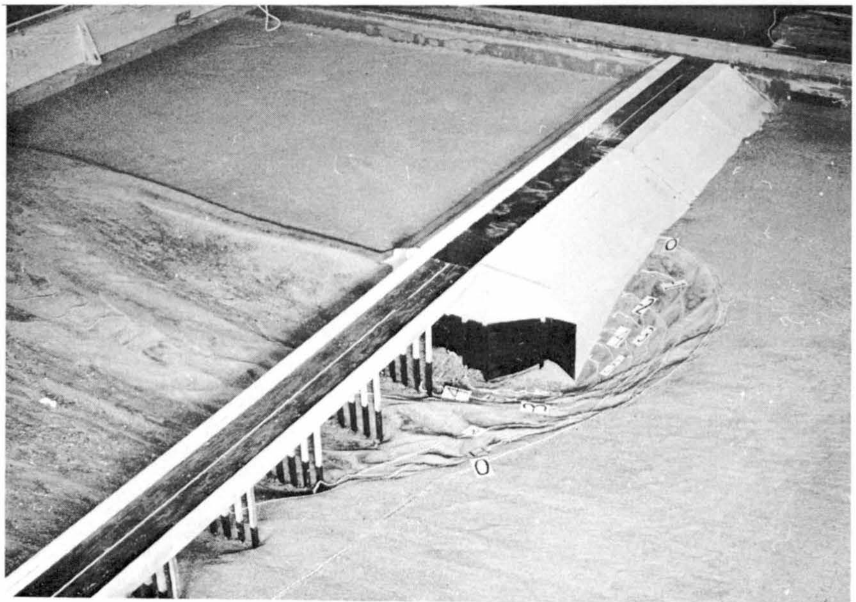


Fig. 24. 45^0 Wing wall. $Q_e = 2.40$ c.f.s. $Q_t = 4.80$ c.f.s.

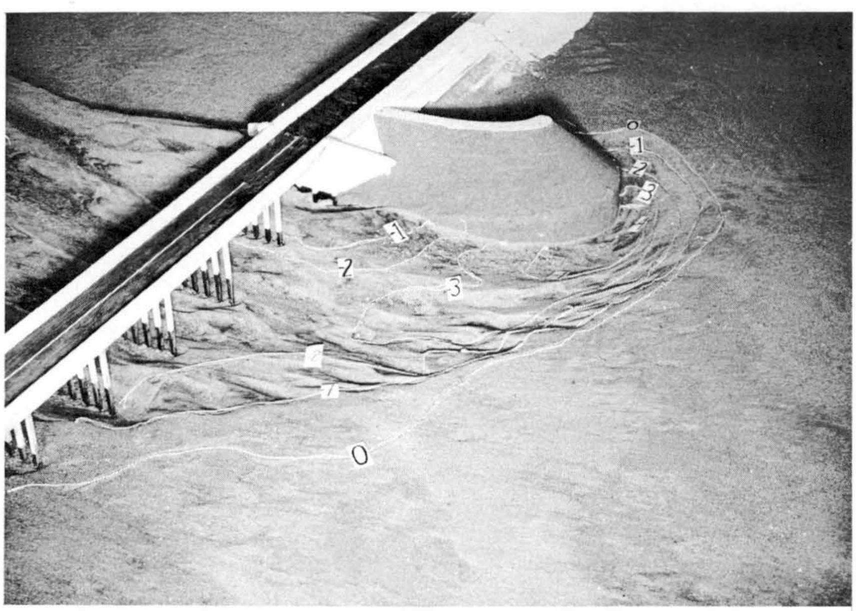


Fig. 25. 45^0 Wing wall. $L_s = 2.0$ ft.
$Q_e = 2.78$ c.f.s. $Q_t = 4.80$ c.f.s.

Fig. 26. 45^0 Wing wall. $L_s = 3.0$ ft.
$Q_e = 2.65$ c.f.s. $Q_t = 4.80$ c.f.s.

Full Bridge Models

Tests with full bridge models were made to determine the effect of spur dikes on small bridges, and to determine, in general, if the hydraulic behavior of flow around spur dikes was the same for small bridges as for large bridges. In many instances small relief bridges constitute as much a problem as do the large main bridges.

The results of the tests are recorded in Figs. 27 to 30. Note that Figs. 28, 29, and 30 are test results with riprapped sand embankments. Despite this, the scour pattern is fairly comparable to Fig. 27. In Fig. 27 the abutments are not sufficiently close together to influence the scour development; consequently, independent scour holes are formed, not unlike the result of tests with the single roadway embankment. In the tests of Figs. 28 to 30, however, the abutments are sufficiently close together that the scour holes overlap, resulting in degradation of the entire bridge waterway. Use of 2 and 3-foot spur dikes improved the efficiency of the opening so that there was not as much scour under the bridge. This trend was similar to that found with the tests using only one embankment.

Results of these few tests are inconclusive. Sufficient time and funds were not available to make a more comprehensive study. Nevertheless, it appeared that the flow was more narrowly concentrated adjacent to the bridge abutment for the smaller opening as compared to larger opening, with the result that with a given spur dike length, more scour can be expected at the bridge than for comparable flow conditions at a larger bridge opening. The test results show that the longer spur dikes make a more efficient waterway which reduces the scour below the bridge. It would seem, therefore, that for a given condition of obstruction by the roadway embankment, a longer spur dike would be required for a small opening than would be necessary for a larger opening. Additional studies are necessary to determine quantitatively if this hypothesis is correct and then to determine the amount of additional spur dike length required.

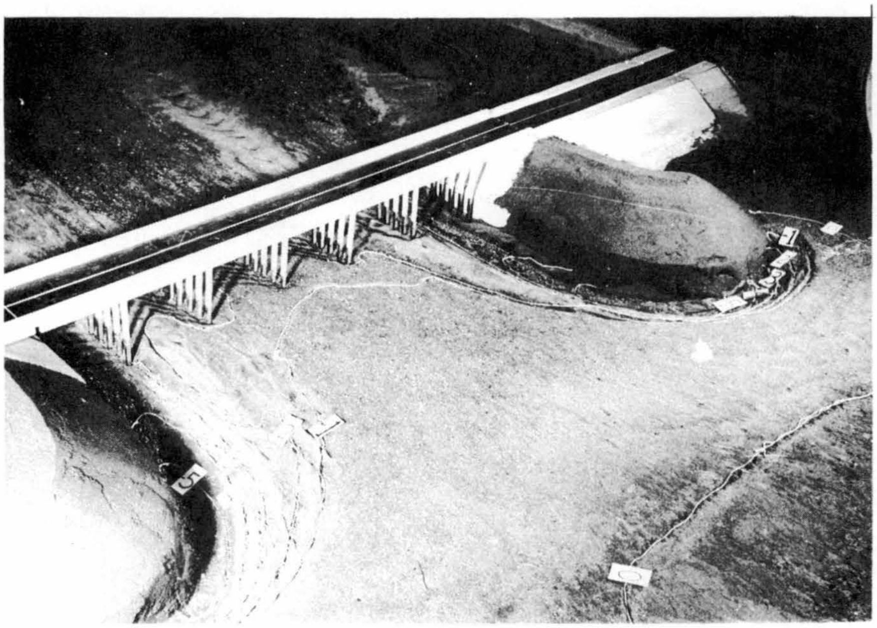


Fig. 27. Full Bridge Model. L_s = 2.0 ft.
Q_t = 5.0 c.f.s. Symmetrical Flow.

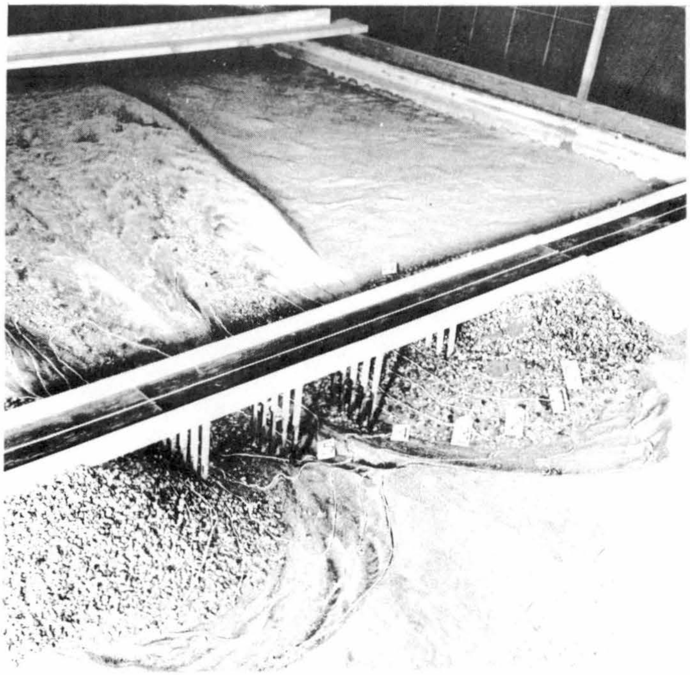


Fig. 28. Full Bridge Model. Q_t = 3 c.f.s.
Symmetrical Flow.

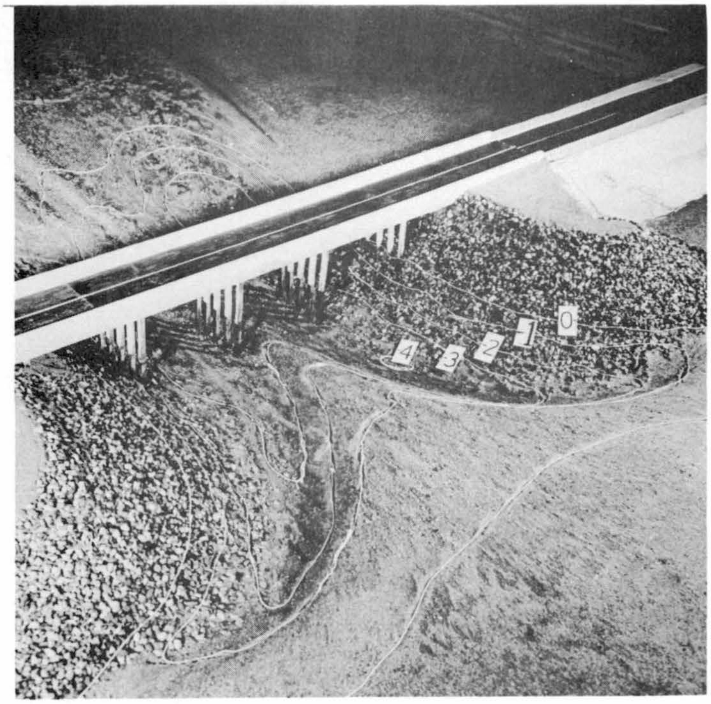


Fig. 29. Full Bridge Model. $L_s = 1.5$ ft.
$Q_t = 3.0$ c.f.s. Symmetrical Flow.

Fig. 30. Full Bridge Model. $L_s = 3.0$ ft.
$Q_t = 3.0$ c.f.s. Symmetrical Flow.

Riprap Requirement

In the states of Mississippi and Alabama, Mississippi in particular, riprap is scarce and, therefore, an expensive item of construction. Most of the riprap used in the state is transported by rail from outside of the state. It is important to the designers, therefore, to know how much riprap is required to protect the spur dike from destruction by erosion.

Accordingly, tests were made in the laboratory to determine riprap requirements on the face of the spur dike. Because the tests were qualitative, an unprotected spur dike constructed of sand only was tested in the laboratory. A 2-foot spur dike on a 4.8 ft. normal roadway embankment and a full symmetrical bridge model with a discharge of 5.0 c.f.s. lasted only 25 minutes before being completely eroded. Although the test conditions were severe in using sand without a mixture of cohesive material, subsequent riprapping would readily demonstrate the protection to the dike.

Riprap requirements on the front and back face and along the toe of the spur dike were then studied. The results as shown in Figs. 31 to 40 indicate that as long as the stream channel is narrowly confined, or the roadway embankment is short, only the front face of the spur dike needs protection. However, if there is considerable flow along the embankment, the spur dike must be riprapped on the back side also. Fig. 32 shows the results with one-fourth of the length of the spur dike protected and Fig. 33 with one-half the length riprapped. Both results are for a test period of 3-1/2 hours. Fig. 34 shows the effect of concentrated flow along the embankment while Fig. 35 shows the results after adequate protection was made to the back side of the dike. Fig. 36 is the result with no riprap at the toe of the dike, while Fig. 37 shows results with 4 inches of riprap at the toe. Tests of Figs. 31 to 35 were conducted with 2 inches of riprap at the toe.

Toe riprap refers to protection extended onto the flood plain adjacent to the spur dike embankment.

Studies were also made on a flatter side-sloped spur dike, 2:1. At this slope, the riprap would not easily roll down the face of the dike, and the study was made to determine if the riprap would undercut from the toe and result in rapid erosion of the spur dike. Figs. 38, 39, and 40 show the results with 2:1 side slopes and various quantities of side discharges.

The riprap studies indicated that protection will probably be necessary for spur dikes where estimated velocities are in excess of scour velocities. If the velocities are low, probably normal highway practices of establishing vegetation would be sufficient to control erosion of the dikes. The risk exists, of course, that a flood will occur before vegetation is developed on the side slopes or during the nongrowing season. If, however, the expected velocities are large, the ends and sides of the spur dikes should be protected at least one-half the length of the spur dike on the front face and one-quarter the length of the spur dike from the end on the back face, with some riprap extended at the toe on the flood plain to protect the dike in the event of scour adjacent to the dike.

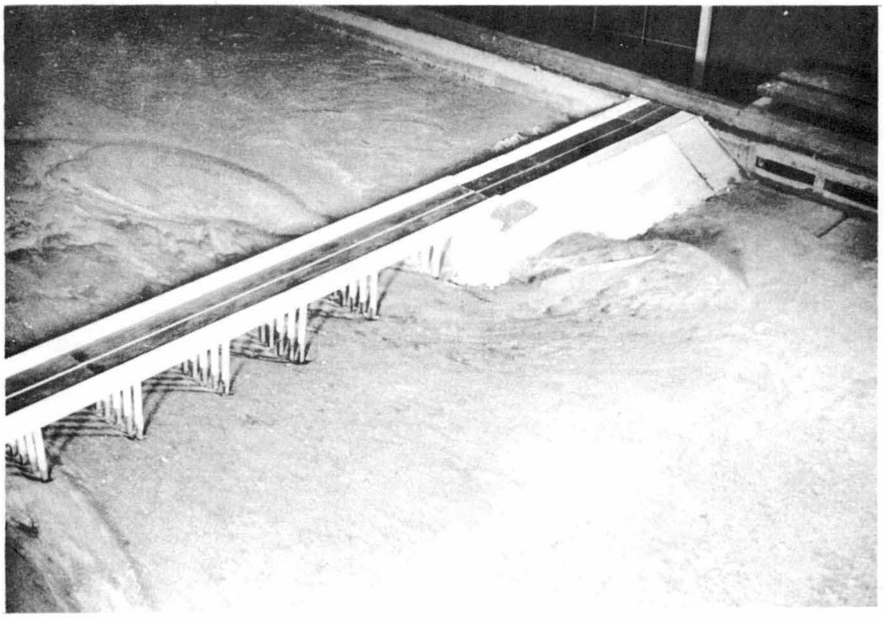


Fig. 31. Spur dike with no protection. $L_s = 2.0$ ft.
Time of run = 25 min. $Q_t = 5.0$ c.f.s.
$Q_e = 1.5$ c.f.s.

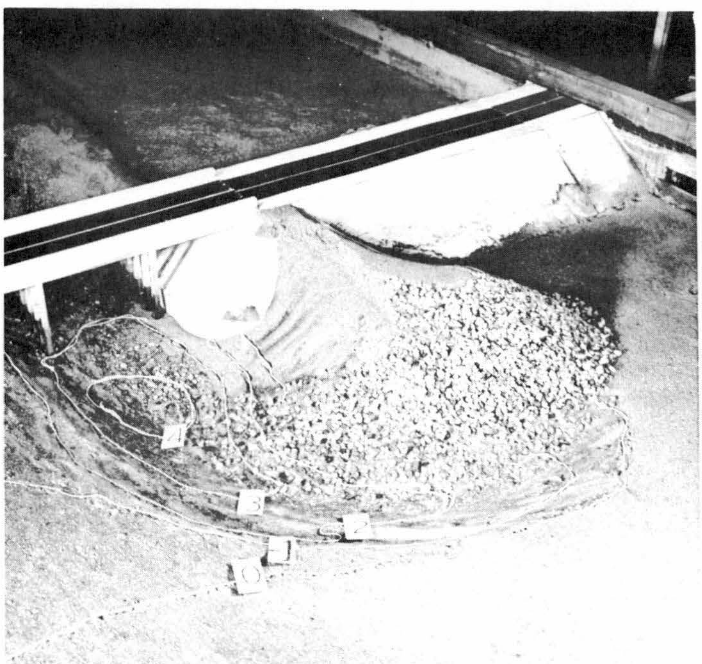


Fig. 32. Spur dike with 0.25 L_s on front face protected.
2 in. Toe riprap. $L_s = 2.0$ ft. Time of run = 3 hrs. 30 min.
$Q_t = 5.0$ c.f.s. $Q_e = 1.5$ c.f.s.

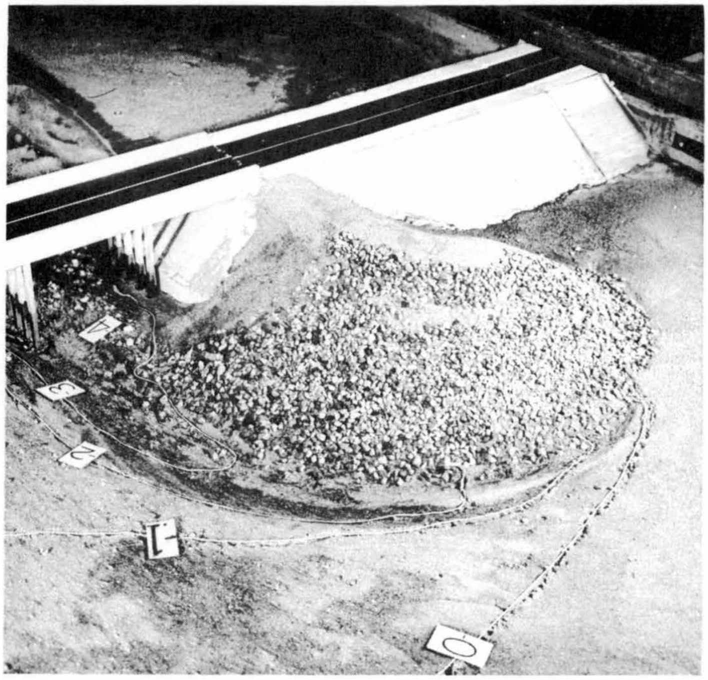


Fig. 33. Spur dike with 0.50 L_s on front face protected. 2 in. Toe riprap. L_s = 2.0 ft. Time of run 3 hrs. 30 min. Q_t = 5.0 c.f.s. Q_e = 1.5 c.f.s.

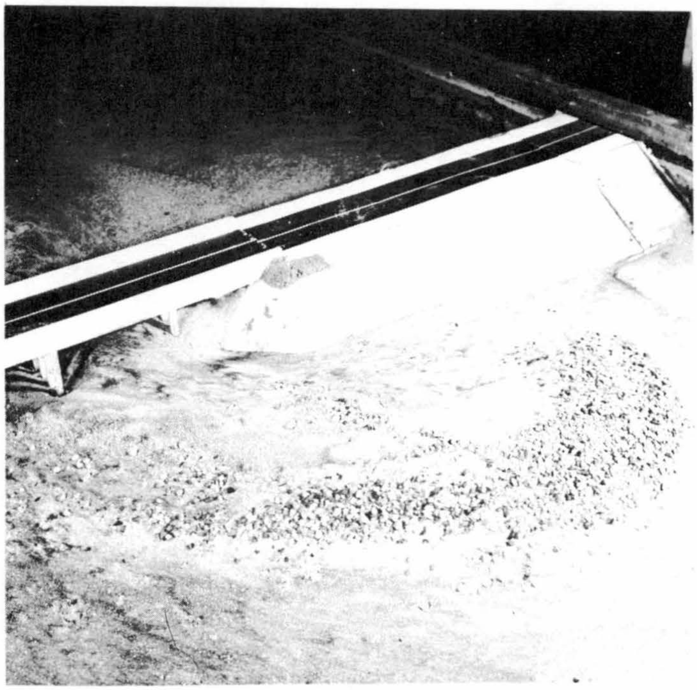


Fig. 34. Spur dike with 0.50 L_s on front face protected. Little back face protection. 2 in. Toe riprap. L_s = 2.0 ft. Time of run = 30 min. Q_t = 5.0 c.f.s. Q_e = 2.55 c.f.s.

Fig. 35. Spur dike with 0.50 L_s on front face protected. 0.25 L_s on back face protected. 2 in. Toe riprap. L_s = 2.0 ft. Time of run = 5. hrs. Q_t = 5.0 c.f.s. Q_e = 2.55 c.f.s.

Fig. 36. Spur dike with 0.50 L_s on front face protected. 0.25 L_s on back face protected. No Toe riprap. L_s = 2.0 ft. Time of run = 5 hrs. Q_t = 5.0 c.f.s. Q_e = 1.5 c.f.s.

Fig. 37. 0.50 L_s on front face protected. 0.25 L_s on back face protected. 4 in. Toe riprap. L_s = 2.0 ft. Time of run = 5 hrs. Q_t = 5.0 c.f.s. Q_e = 1.5 c.f.s.

Fig. 38. 0.50 L_s on front face protected. 0.25 L_s on back face protected. No toe riprap. 2:1 side slopes. Time = 5 hrs. Q_t = 5.0 c.f.s. Q_e = 1.5 c.f.s.

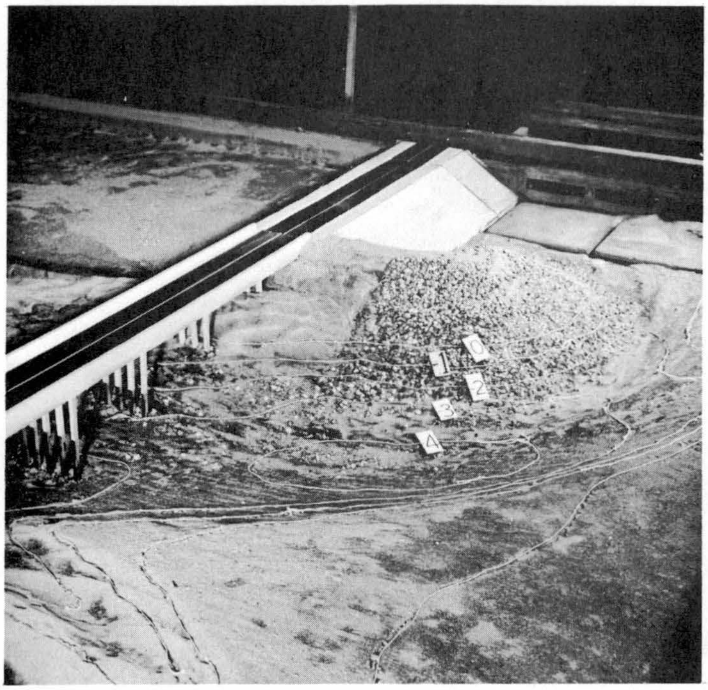


Fig. 39. 0.50 L_s on front face protected. 0.25 L_s on back face protected. No toe riprap. Q_t = 5.0 c.f.s. Q_e = 2.55 c.f.s. Side slope = 2:1.

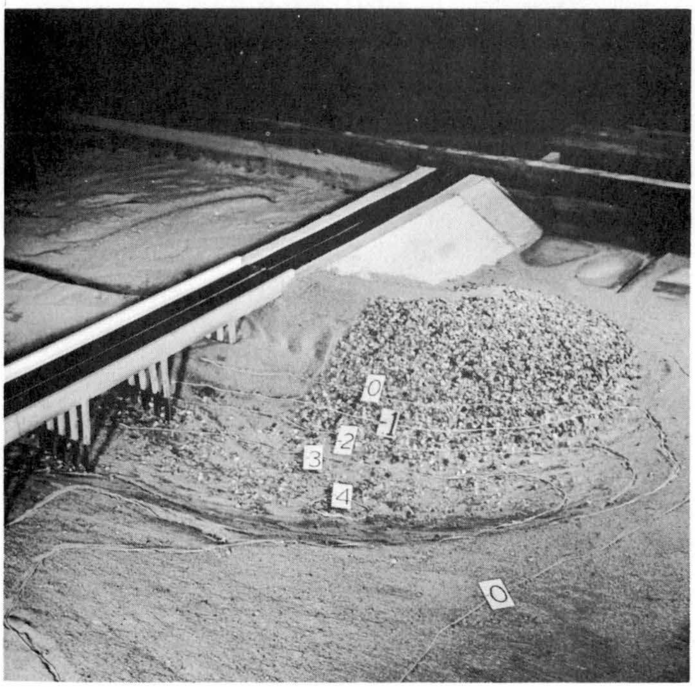


Fig. 40. 0.50 L_s on front face protected. 0.25 L_s on back face protected. 2 in. Toe riprap. Q_t = 5.0 c.f.s. Q_e = 2.03 c.f.s. Side slope = 2:1.

Skewed Embankments

The study with skewed roadways was limited to embankments projecting 45^0 upstream and 45^0 downstream with the direction of flow. Scour patterns at the abutments of embankments skewed with respect to the stream channel are not unlike scour patterns developed at normal crossings. The difference is, however, that the bridge extends into or away from the area of severe scour. If the scour hole forms at the end of the spur dike, and the bridge is skewed upstream into it, the abutment and bridge piers will project into the scour hole, while bridges skewed downstream will carry the piers away from the area of severe local scour. Tests conducted in the flume substantiated this reasoning as is shown in Figs. 41 and 42 with 2 and 3-foot spur dikes with the upstream skew and Figs. 43 and 44 with 2 and 3-foot dikes with the downstream skew. Flow quantities were the same for all four tests; 4.8 c.f.s. with no side flow and at 0.4 ft. depth. The effective width of opening was 8.0 ft. measured normal to the flume walls.

Studies were also made with side inflow to the flume. The results of these studies, as listed in Table 2, shows that for upstream skews, the greater the flow along the embankment, the deeper was the resulting scour hole. For the test conditions, the deepest point of scour was almost always at the bridge. Spur dikes did reduce scour, however, particularly when heavy embankment flows occurred. Scour at the bridge with no dike was 0.7 ft. while with a 3-foot spur dike, the scour was 0.35 feet.

If the embankment is skewed downstream, scour depths are not as great. In fact, as flow along the embankment increases, scour depth decreases because the greater momentum of the flow distributes itself farther away from the bridge abutment. Thus, for downstream skews, scour decreases with increasing length of spur dike, and also decreases with increasing flow along the embankment. The latter condition is the reverse of the upstream skew condition.

TABLE 2.

SKEWED EMBANKMENTS
Measured Data.

Run No.	L_s ft.	Q_e c.f.s.	Max. Scour ft.	d_s ft.	Condition
23	0	2.40	.45	.45	45^0 Upstream skew
24	0	2.65	.50	.50	" " "
25	0	2.90	.60	.60	" " "
26	0	3.15	.70	.70	" " "
27	1.5	2.40	.45	.45	" " "
34	1.5	2.65	.50	.50	" " "
35	1.5	2.90	.60	.60	" " "
36	1.5	3.15	.65	.55	" " "
28	2.0	2.40	.40	.40	" " "
29	2.0	2.65	.45	.45	" " "
30	2.0	2.90	.60	.60	" " "
31	2.0	3.15	.70	.55	" " "
33	3.0	2.40	.40	.40	" " "
32	3.0	2.65	.45	.35	" " "
40	0	2.40	0.40	0.40	45^0 Downstream skew
38	0	2.65	0.30	0.20	" " "
39	0	3.15	0.22	0.15	" " "
41	1.5	2.40	0.4	0.20	" " "
42	1.5	3.15	0.3	0.20	" " "
44	2.0	2.40	0.45	0.15	" " "
48	2.0	2.65	0.35	0.10	" " "
43	2.0	3.15	0.3	0.15	" " "
47	3.0	2.40	0.4	0.10	" " "
46	3.0	2.65	0.35	0.10	" " "
45	3.0	3.15	0.2	0.10	" " "

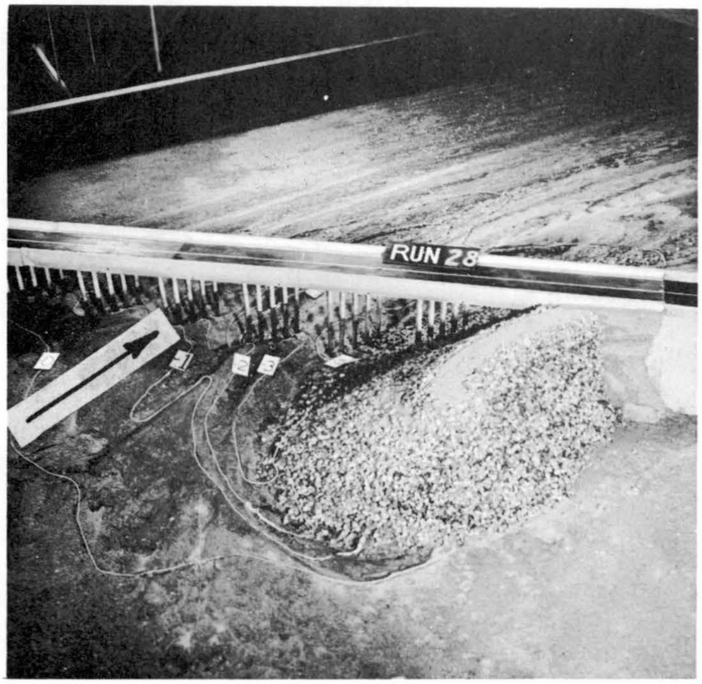


Fig. 41. Upstream skew. L_s = 2.0 ft.
Q_t = 4.8 c.f.s. Q_e = 2.40 c.f.s.

Fig. 42. Upstream skew. L_s = 3.0 ft.
Q_t = 4.8 c.f.s. Q_e = 2.40 c.f.s.

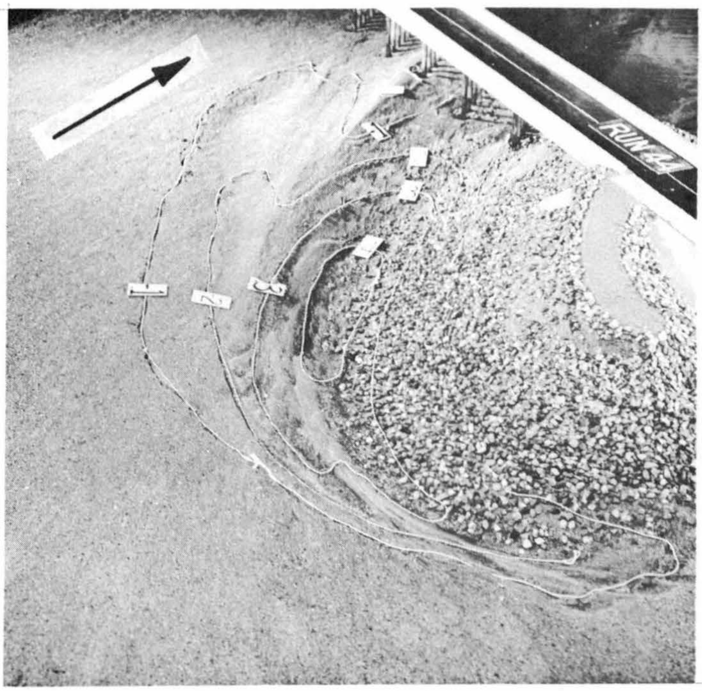


Fig. 43. Downstream skew. L_s = 2.0 ft.
Q_t = 4.8 c.f.s. Q_e = 2.40 c.f.s.

Fig. 44. Downstream skew. L_s = 3.0 ft.
Q_t = 4.8 c.f.s. Q_e = 2.40 c.f.s.

There was no attempt to develop criteria for design of spur dikes with skewed embankments because of the limited data. However, qualitatively the study has demonstrated that, in general, longer spur dikes will be required for embankments skewed upstream and shorter dikes can be utilized for embankments skewed downstream. The magnitude of the increase or decrease will be a function of the skew angle, and additional studies are needed to determine their values.

SUMMARY

This study of spur dikes has resulted in a tentative criteria for design derived from qualitative results. The criteria is set forth graphically in Fig. 23. Essentially, determining the length of a spur dike by use of this design curve is one of trial and error. To design a spur dike, the minimum length ratio L_s/L_e will be tried. With this ratio, the value of d_s/L_s will be determined. If the resulting estimate of scour is acceptable, the value of W_s/L_e is read from the curve. The width of spread, W_s, is subsequently calculated and Q_t* determined. Since Q_e is known, the ratio Q_e/Q_t* is computed. If this ratio is greater than or equal to the given value of Q_e/Q_t* on the curve, the spur dike length is satisfactory. If it is less, the spur dike is too short and a longer dike is required.

There are limitations to this design curve. First, the results are qualitative and were derived under the assumption of uniform approach flow. The curve does not apply to extremely long embankments. Another limitation is that an arbitrary time limit of 5 hours for each test was imposed for the study. Whether or not this time is representative of field conditions is open to discussion.

Despite these limitations, considerable progress is represented by this study. There are, however, many aspects of this total problem which needs further investigation:

(a) Determination of spur dike requirements for small bridges;

(b) Spur dike variations for skewed bridges;

(c) Possibility of employing other shapes of spur dikes in special cases.

(d) Determination of comparative effectiveness of permeable and nonpermeable dikes.

(e) Effect of sediment transport in the flood flow on spur dikes and scour.

(f) Analysis of flood hydrographs in its relation to spur dikes.