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by

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INTRODUCTION

Construction work on the Old London Bridge began in the year 1176 and was completed in 1209. With its narrow stone arches, the bridge severely constricted the waterway, taking up more than 5/6th of the original width. The result was to create a 5 foot head between the opposite sides of the structure. The water roared through the openings like a mill race. (Figure 1)

For the high velocities which existed the Old London Bridge was very well built. Cutwaters of stone and timber were constructed around each pier. One of their purposes was to prevent detritus and ice from damaging the piers. Another purpose was to guide the water smoothly through the constriction. The cutwaters worked so effectively that they prevented erosion of the piers for more than five centuries. In 1757 as part of a plan to improve navigation the two small spans were converted to one large span. The concentrated flow of water through the new arch caused heavy scour of the piers and soon began undermining the entire bridge.

Engineers for centuries have been aware of the problem of erosion of bridge piers and abutments. In ancient days they lacked the technical knowledge and equipment to construct large span bridges, and almost all older bridges were built with narrow spans which constricted the waterway and increased velocities between bridge abutments and caused

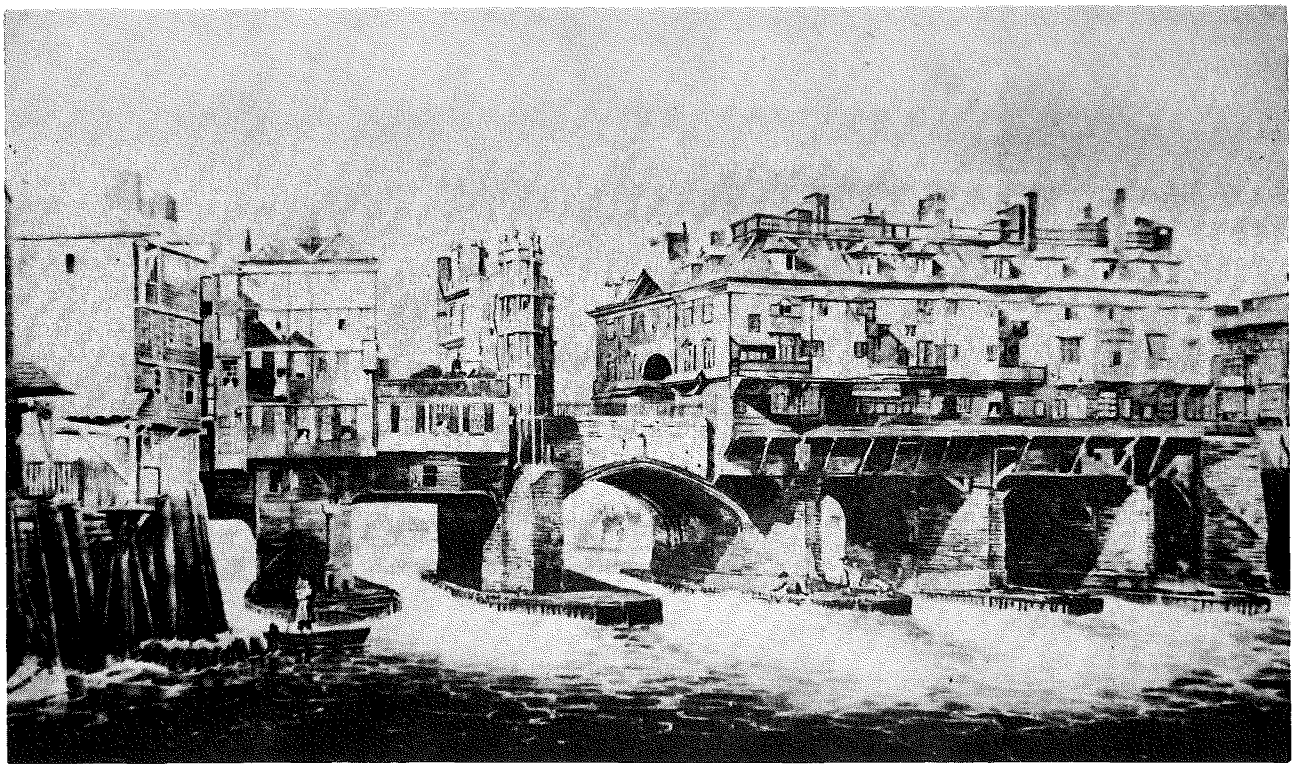

Fig. 1 Old London Bridge 1750, from a watercolour drawing signed J. Varley, in the British Museum.

erosion. In almost all cases the solution for erosion was the same. The engineer added cutwaters around the piers.

The Old London Bridge was pre-Renaissance in design and construction and a product of Anglo-Saxon culture. Oriental culture of the same period conceived and built the wooden cantilever bridge of the type at Srinagar, India. (Figure 2) Most of these bridges were built on a soft foundation which was easily scoured. To reduce erosion the cutwater was built around each pier. By chance or intelligent design another safety factor was included, the piers were built of stacked logs which formed an open network. During periods of peak flow the river flowed through the open piers thus reducing the effect of constriction.

In modern times many long span bridges were built which either did not constrict a waterway or else had a negligible effect on it. Erosion for these bridges is not a problem.

However, there are many highway bridges built over narrow rivers which flood severely and cause damage to abutments. In the State of Connecticut alone the damage in August and October 1955 amounted to $30,000,000. Fifty percent of the damage was on state bridges and roads, and fifty percent on town and city bridges and roads. In Northern California during the floods of December 1964 and January 1965 three large railroad bridges and twenty five major highway bridges were lost causing damage of about $48,000,000.

BRIDGE FAILURE DUE TO SCOUR

The large percentage of the bridge damage could be attributed to failure of foundations caused by scour. In the past, the bridge

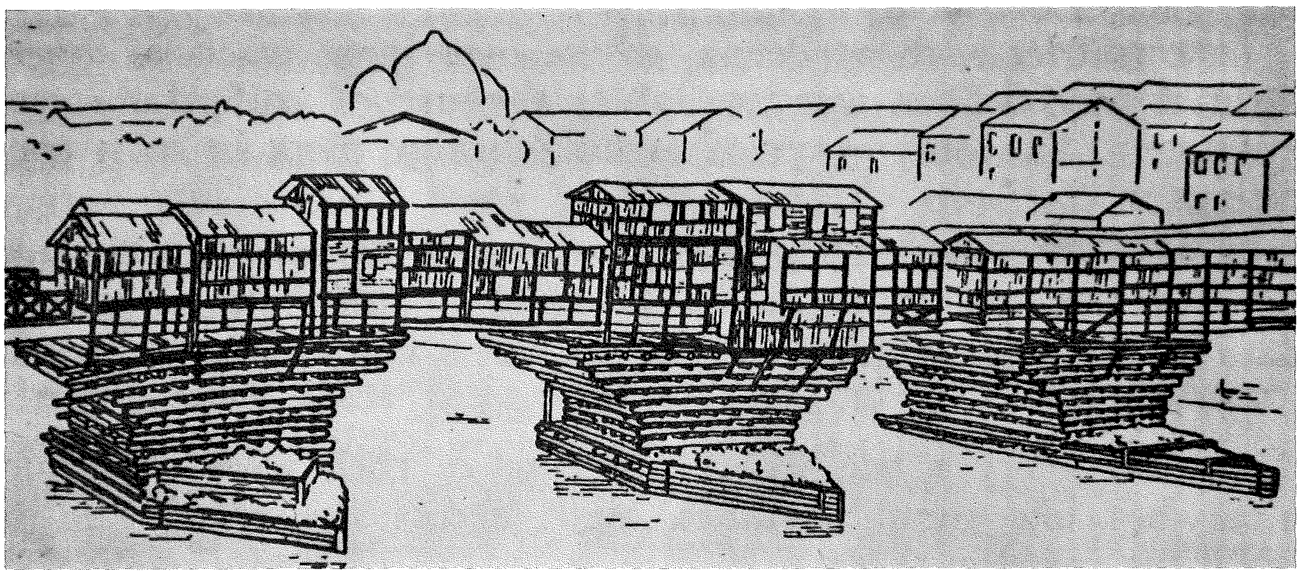

Fig. 2 A primitive cantilever bridge over the river Jhelum at Srinagar.

superstructures were designed meticulously from the structural point of view, while the hydraulic design of piers and abutments was based on the "rule of thumb". Undermining of structures because of scour is still very much of a problem, especially with the present trend towards high approach embankments to bridges, with consequent deep flood plane flow. The main difficulty with the existing structures which were built 20-25 years ago, is the fact that these bridges in many cases, contracted the flow excessively.

Constriction of the flow caused by approach embankments or piers results in a rise in the water level immediately upstream of the constriction, and an increase in the stream velocity in the constricted area. Under normal flow conditions, the higher velocities may not be sufficient to produce significant disturbance of the material on the stream-bed but, in times of flood, the greatly increased velocities may produce severe scouring action, particularly at the bases of abutments and piers, which causes partial or complete collapse of the bridge structure. In the case of a short span bridge, the scour effect increases with the extent of constriction, so that the shorter (and generally the more economical) the bridge structure at a particular site subject to flood scour, the greater is the danger of flood damage due to scour.

Scour is also noticeable at the piers and abutments of bridges which are badly located, and especially at points of severe stream curvature, resulting in deep scouring at the outside of the bend. Answers to all of the problems in highway hydraulics are not at hand. In perhaps no other field of hydraulic construction is the designer so handicapped by lack of knowledge of the basic factors involved. However, some progress has been made during the past several years in studying the problem of scour around bridge piers and abutments.

DESCRIPTION OF SCOUR PHENOMENA

"The following description of scour phenomena is extracted from an article by E. J. Sanden and C. R. Neill printed in the periodical "Public Works in Canada" in September 1963, and subsequently reprinted under the title of "Measuring Scour Around Bridge Foundations in Floods."

"Most books on bridge and foundation design give solemn warnings on the dangers of scour, but restrict themselves to generalized explanations and poorly supported rules of thumb for estimating foundation depths. Research by hydraulic engineers in various countries over the past 25 years has produced a good deal of qualitative information but very little of it is readily available to bridge designers. The picture that emerges is roughly as follows:

During the high flood stages, the bed of an alluvial river is in a highly active condition. In sand rivers, the bed is ruffled into a complex system of ripples, dunes and waves, with sand blowing over the shifting dunes and being whirled into suspension by gusts and eddies of the current. In gravel rivers, the larger material rolls along rapidly and the finer material jumps or goes into suspension. Contrary to old beliefs still prevalent there is no appreciable general lowering of a river bed during a flood, and no appreciable depth if bed is in motion at any one time, but there is usually a substantial redistribution of material in the form of scour and fill. As a result, longitudinal and cross profiles may be quite different at low-water and flood stages, much in the same way as desert and snowfield topography changes in windstorms. Thus, even without the interknowledge of the basic factors involved. However, some progress has been made during the past several years in studying the problem of scour around bridge piers and abutments.

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ference caused by a bridge, certain parts of a river channel, particularly at bends, are subject to scour at high stages and refilling at low stages.

A bridge may cause further scour in two ways: First, unless it spans the entire flood plain, it generally causes some constriction of high flood flows; and second, the pier and abutment foundation offer obstruction to the current. Research indicates that scour due to constriction tends to occur across the width of the channel and that it is closely related to the degree of constriction, while scour due to obstruction is localized and is closely related to the width of the obstacle normal to the direction of the current.

The formation of local scour holes around obstacles such as cylinders and bridge piers, that is, the obstruction type of scour, has had considerable attention in hydraulic laboratories, and reports of several model studies may be found. The phenomenon may be observed in any situation where a turbulent fluid flows past an obstacle projecting above a loose granular bed."

PREVIOUS STUDIES

(i) Local Scour at Bridge Contractions - Whereas the general scour in a long channel constriction results in a more or less uniform lowering of the bed, local scour refers to the development of holes of limited extent, produced in regions of high local velocity. These generally are in the vicinity of sharp changes in bed or wall boundary alignment where the flow separates from the boundary so as to produce eddies and zones of high turbulence.

In the case of short span bridges, the extent of channel contraction appreciably effects the mean velocity and the local high velocities in the constricted section, and hence the local scour. With bridges of long span, the local velocity and scour effects appear to be less dependent on the overall flow geometry and are treated as purely local phenomena.

An early laboratory study of the problem of scour around abutments was reported in 1894 by Engels in Germany, although reference was made by him to previous work carried out in France by Durand-Claye in 1873. The Engels study was confined to narrow limits, however, and no attempt was made at generalization or prediction of scour patterns.

Investigation in this field seems to have lapsed for many years, and it was not until 1949 that an analytical approach was attempted by the U. S. Department of Agriculture[1]. Also, Posey[2] studied briefly the scour around a pier in the Rocky Mountains Hydraulic Laboratory. This was followed by an investigation by the U. S. Geological Survey in 1953[3].

After the disastrous floods in Iowa in the early fifties, the State University of Iowa began investigations into scour around bridge piers and abutments. This work was described by Laursen and Toch[4] in 1956, and further work was reported by Laursen[5] in 1958 and 1960. Some of their conclusions are mentioned below.

Various empirical formulae have been proposed for the depth of local scour in the case of long span bridges. Some of these express the

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[1] "Flow Through Diverging Open Channel Transitions", U. S. Department of Agriculture.

[2] "Why Bridges Fail in Floods" by C. J. Posey, Civil Engineering, February, 1949.

[3] "Computation of Peak Discharge at Contractions" by C. E. Kindsvater, R. W. Carter, H. J. Tracy, U. S. Geological Circular No. 283, 1953

[4] "Scour Around Bridge Piers and Abutments" by E. M. Laursen and A. Toch, Iowa Highway Research Board Bulletin No. 4, 1956.

[5] "Scour at Bridge Crossings" by E. M. Laursen, Iowa Highway Research Board Bulletin No. 8, 1958

depth of scour hole below the water surface (D_s) as a multiple of Lacey's regime depth (D_L) in the contracted section. For example, the Khosla and Inglis formulae are of the type

$$D_s = k\ D_L = k \times 0.47\ (Q/f)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

where Q is the total flow rate, f is Lacey's silt factor, and k is a factor varying from 20 to 4, depending upon the local geometrical form. Blench and Ahmed similarly relate the depth of maximum scour to a mean flow rate and, to some extent, to the bed material. Laursen[5] on the other hand, maintains that, with bed load movement continuing during the scouring process, the maximum local scour is independent of the sediment size and flow velocity, and depends only on the normal flow and the length of the obstruction. He concluded that the maximum depth of scour hole below the stream bed may be four times the depth of the general scour in the case of an embankment extending to the edge of the main channel, with neighboring scour holes overlapping; and as much as twelve times, when the main channel is constricted, with no overlap of adjacent scour holes.

Following the August 1955 floods in Connecticut, the State Highway Department[6] made careful measurements of maximum and average high water, mode of failure, debris, and channel characteristics. A formula was developed relating the average depth of scour to the difference between the sediment load in the approach flow and the transport capacity under the bridge

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[6] "Report on Investigation of Scour at Bridges caused by Floods of 1955 by L. K. Moulton, C. Belcher, B. E. Butler, Highway Research Abstracts, Vol. 27, No. 8, pp. 14-31, September 1957.

The laboratory study by Liu, Chang, and Skinner[7], (1961) indicates that the effect of flow velocity on scour may be appreciable, and suggests that Laursen's conclusions to the contrary, holds only for Froude Numbers of less than 0.5 in the unconstricted channel. It concludes that, if the bed load is appreciable, the constriction ratio has no appreciable effect on the depth of scour; but that if there is no bed load, the limiting scour is a function of the constriction ratio. This laboratory investigation yielded experimental curves relating equilibrium and maximum local scour to the flow geometry and flow rate (ratio of length of embankment to normal depth, and the normal Froude Number, respectively). The authors point out, however, that their results should be used only with caution by designers until prototype verification is obtained.

Neill[8,9] discussed the physical nature and cause of river-bed scour phenomena and outlined some of findings of experimental research. The author also described a few case histories and gave qualitative recommendations for dealing with scour in bridge design and maintenance.

Herbich and Brennan[10] found through analytical study and field observations that systematic relationships exist among the significant,

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7 "The Effect of Bridge Constriction on Scour and Backwater" by Liu, Chang, Skinner, Publication of C.E. Section, Colorado State Univ., Report No. CER60HEL22, Feb. 1961.

8 "River-Bed Scour. A Review for Bridge Engineers" by C. R. Neill, Research Council of Alberta Contribution No. 281. Canadian Good Roads Assoc., Ottawa, Canada, December 1964.

9 "Measurements of Bridge Scour and Bed Changes in a Flooding Sand-Bed River" Proc. Inst. of Civil Engrs., Vol. 30, pp. 415-436, February 1965.

10 "Prediction of Scour at Bridges" by H. P. Herbich and L. M. Brennan, A Report of an Investigation. Dept. of Civil Engineering, University of Windsor, Windsor, Ontario, Canada, December 1965

relevant, dimensionless parameters affecting scour. The relationships which were presented in graphical or equation form permitted prediction of depth of scour at several bridges with fairly good accuracy.

(ii) The Use of Spur Dikes - Spur dikes have been used in a number of cases in the United States to "streamline the flow" through a bridge opening in an attempt to eliminate separation and the accompanying scour. (Figure 3) In some cases they are permeable, such as loose rockfill timber cribs, rockfill embankments, and open timber pilings; others, consisting of earth embankments or solid timber sheeting, are impermeable.

The first study on the effect of spur dikes on the flow pattern in this country was sponsored by the Georgia State Highway Department. The model spur dikes were made to simulate dikes constructed of timber cribs. It was reported by Carter[11] in 1955, that for spill-through type abutments, a dike of length equal to 0.08B (where B = width of opening) at a distance of 0.08B from the beginning of abutment curvature, and at angle of 0° to the flow, proved to be the most efficient. No other details were given in the paper.

Some studies were conducted in Sweden in 1957 by Hartzell and Karemyr[12] where dikes were used to align the flow and secure a uniform velocity between the abutments. It appeared that a dike some distance away from the abutment end, and at 10° angle with the direction of flow, gave best results. However, the tests were inconclusive.

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[11] Carter, R. W., "Highway Hydraulics" Proc. of the Fourth Annual Georgia Highway Conference, February, 1955.

[12] Hartzell, G., and Karemyr, I.,"Anordningar För Minskning av Risken for Erosion Utanfor Vagbankar (Methods Used for Reduction of Scour at Abutments), Chalmers Tekniska Hogskola, Sweden, 1957

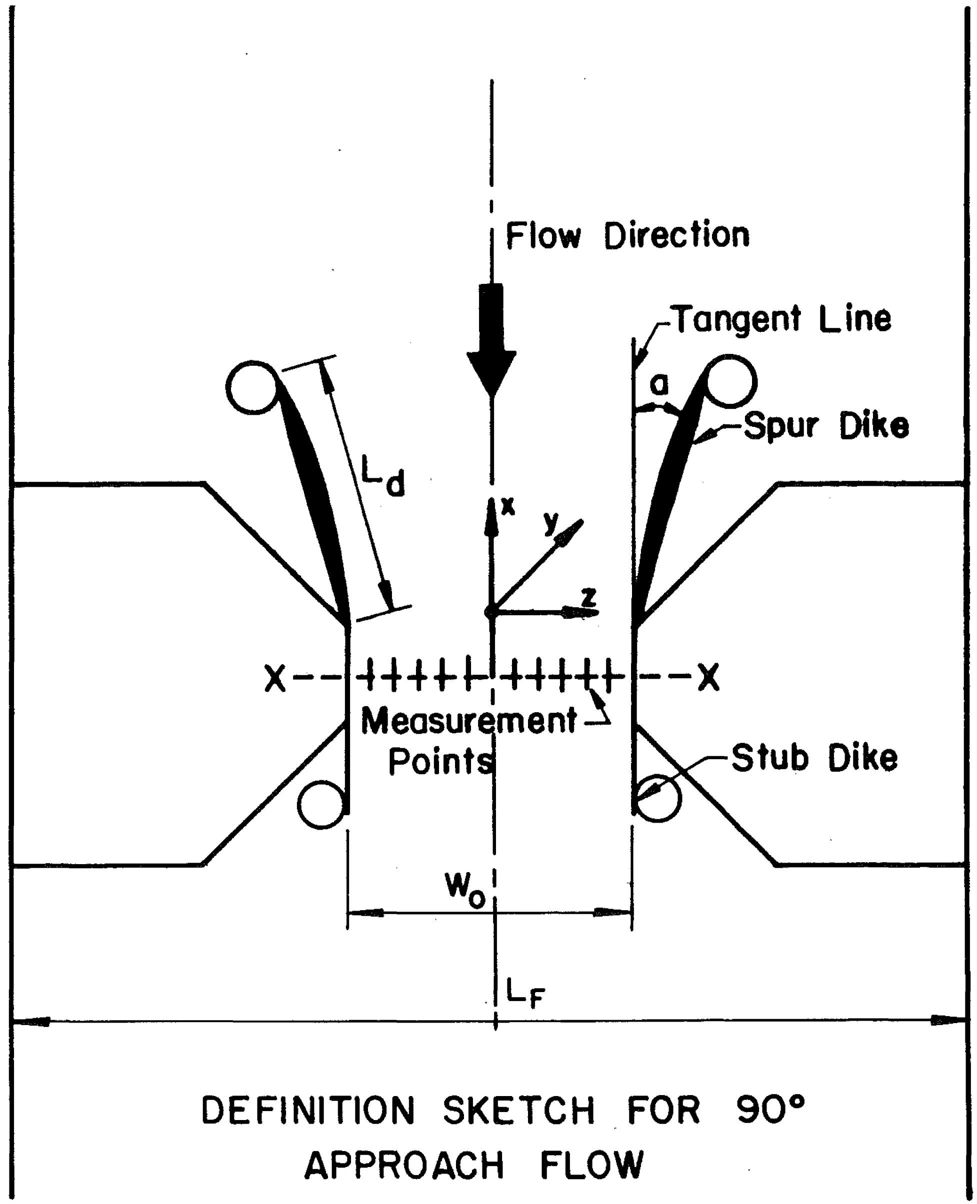

Fig. 3 Definition sketch for 90-degree approach flow.

In another Swedish model study[13] of possible erosion at a proposed bridge site, it was found that short guide banks extending upstream from the ends of the abutments resulted in appreciable reduction of local scour.

Karaki[14,15] conducted studies on the effect of spur dikes in a movable-bed model while Herbich[16] investigated efficiency of spur dikes both in fixed and movable bed models.

The conclusions drawn by Karaki were that spur dikes are effective in reducing local scour; that their effectiveness depends upon the geometry of the roadway embankments, the flow on the flood plain, and the size of bridge opening, and that the dike should be curved with its toe alignment tangential to the end of the abutment (that is, parallel to the flow in the constriction). With a sloping bank spur dike, this results in the centerline of the dike intersecting the embankment some distance from the end of the embankment.

Herbich reported that the movable-bed studies confirmed the predictions based on the fixed-bed investigation that curved spur dikes,

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13 Reinius, E., "Modellundersökning av Erosion I Ett Brolage (Model Studies of Erosion at a Bridge Site) Institute of Hydraulics, Bulletin No. 7, Chalmers Tekniska Hogskola, Sweden, 1956.

14 Karaki, S. S., "Hydraulic Model Study of Spur Dikes for Highway Bridge Openings", Civil Engineering Sec. Report CER 59 SSK 36, Colorado State University, September 1959.

15 Karaki, S. S., "Laboratory Study of Spur Dikes for Highway Bridge Protection", Paper presented at 39th Meeting of the Highway Research Board, January 1960.

16 Herbich, J. B., "The Effect of Spur Dikes on Flood Flows Through Bridge Constrictions", Paper presented at the National Convention of the American Society of Civil Engineers, Boston, Massachusetts, October, 1960.

in providing a smooth transition for the flow, were extremely effective in reducing scour at the abutments. At some points along the abutments deposition occurred where, without dikes, scour would have developed.

FIXED-BED STUDIES

Objectives - The study was conducted to determine the shape, length and size of dikes necessary to prevent excessive scour for generalized field conditions.

The principal reason for commencing the studies in a fixed-bed model, was the fact that the problem of scour between bridge abutments is a very complicated one, involving a great number of variables. In addition, there is a question of "scale effect" between the movable-bed model and the prototype. Employment of the fixed-bed model reduced the number of variables considerably, facilitated the study of velocity distribution, and presented a much clearer picture of the effect of spur dikes on the flow between bridge abutments.

Variables involved - A great number of variables is involved in this problem:

(a) Geometrical

f_D = shape of dike

f_A = shape of abutment

θ = angle of abutment skew

α = angle of dike

L_F = waterway width

w_o = width of abutment opening

L_d = length of dike

x,y,z = coordinate axes

y_s = depth of scour

(b) Dynamical

V = velocity

f_v = velocity distribution

g = gravitational acceleration

τ_o = boundary shear stress

V_f = fall velocity of a particle

Q = discharge

(c) Fluid and Sediment properties

ρ = fluid density

μ = fluid viscosity

ρ_s = particle density

d_s = mean particle size

To simplify the research some of the variables were eliminated and some of the factors were kept constant. The sediment factors were eliminated by confining the first part of the investigation to fixed-bed models. The second part of the study was conducted in the movable--bed models.

One of the constant factors was the abutment shape, made similar to the type used by the Pennsylvania Department of Highways. The wing-walls are at 45 degree angle to the abutment face, and the sides slope at 45 degree angle. The spur dikes were made straight during initial studies. Two angles of abutment skew were used.

From these simplifications emerged several important ratios, each of which is significant for the particular geometry chosen:

$$\frac{w_o}{L_F} = \frac{\text{Width of Abutment Opening}}{\text{Width of Flume}}$$

$$\frac{L_d}{w_o} = \frac{\text{Dike Length}}{\text{Width between Abutments}}$$

$$\frac{z}{w_o} = \frac{\text{Width}}{\text{Width between Abutments}}$$

$$\frac{x}{w_o} = \frac{\text{Length}}{\text{Width between Abutments}}$$

$$\frac{V}{V_a} = \frac{\text{Velocity}}{\text{Approach Velocity}}$$

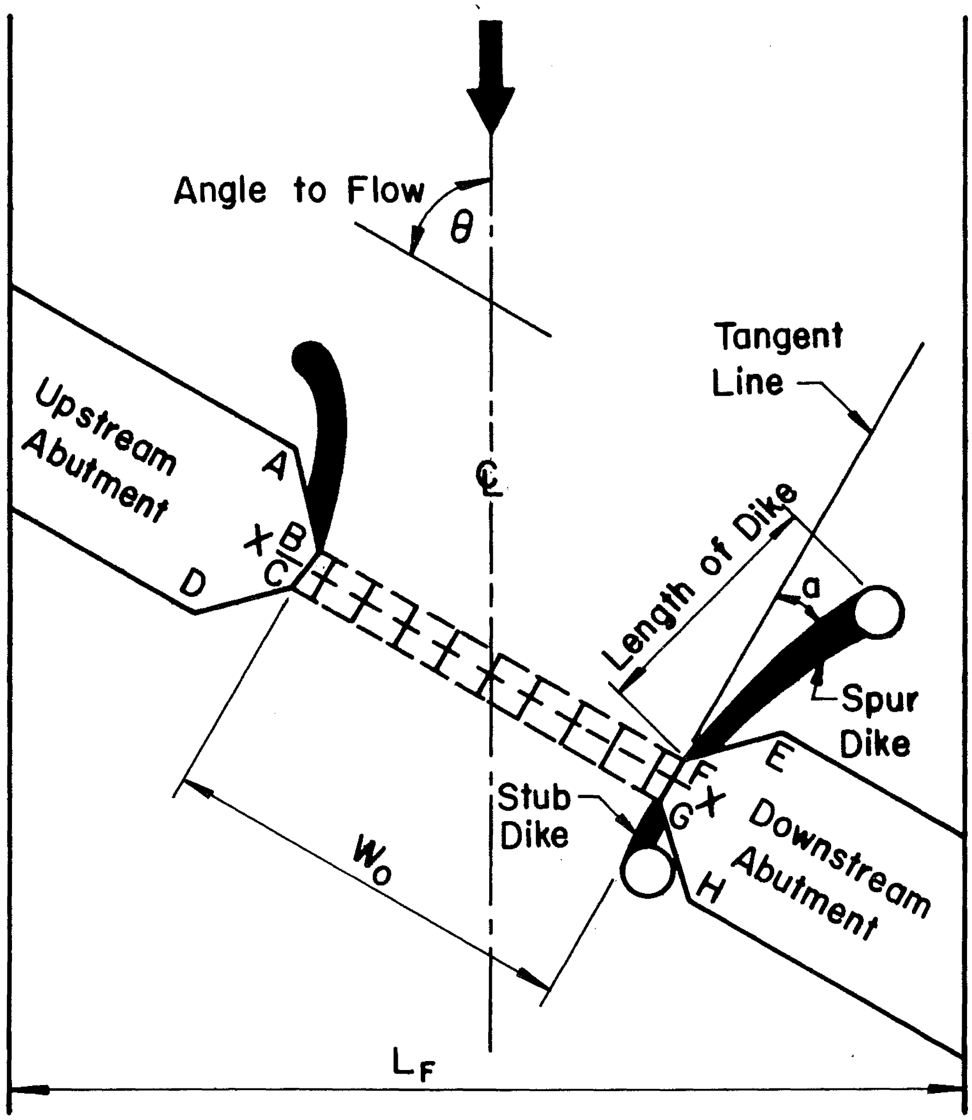
$$\frac{V}{V_o} = \frac{\text{Velocity at a Point with Dikes}}{\text{Velocity at a Point Without Dikes}}$$

$$\frac{\mathbb{F}}{\mathbb{F}_o} = \frac{\text{Froude Number with Dikes}}{\text{Froude Number without Dikes}}$$

Separation of flow at the abutments results in further contraction of the flow, and hence higher velocities through the constriction. The investigation had as its objective the evaluation of spur dikes in producing a more uniform velocity distribution and a lower mean velocity with consequently less liability of scour, through the constriction.

The geometrical factors (Fig. 3 and 4) varied were:

1. The percentage opening, $w_o/L_F \times 100$, a measure of the extent of constriction, w_o being the distance measured along the line of the



DEFINITION SKETCH FOR SKEWED ABUTMENT

Fig. 4 Definition sketch for skewed abutment.

embankment between abutments, and L_F the normal width of channel. The opening percentages ranged from 25 to 50, approximately.

2. The angle of approach, θ , two values tested being 90 degrees (normal crossing) and 60 degrees (the angle between the line of the embankments and the channel centerline).

3. The length of dike, L_d. Three values: 1.5; 2.25; and 3 feet were used, giving ratios L_d/w_o ranging from 0.3 to 1.5.

4. Dike angles (α) measured from the normal to the line of the embankments. Three values: 0^o; 15^o; and 30^o were used.

The discharge was maintained constant for most of the tests, and the dikes were in general straight, although several were conducted with curves dikes. Depths and velocity distributions were determined generally throughout the channel, and in particular, along the centerline of the constriction

RESULTS AND COMMENTS

(a) Velocity Distribution - In the case of the 90-degree approach (Fig. 3), the spur dikes produced a marked improvement in the uniformity of the velocity across the constriction. The length of dike appeared to be unimportant in reduction of velocities (provided that the length was not over a certain minimum length), but the contraction ratio w_o/L_F is important (Fig. 5). In Fig. 5 the change in velocity along the centerline of abutments due to addition of spur dikes is plotted against z/w_o. The average reduction in velocities to about nine/tenths of

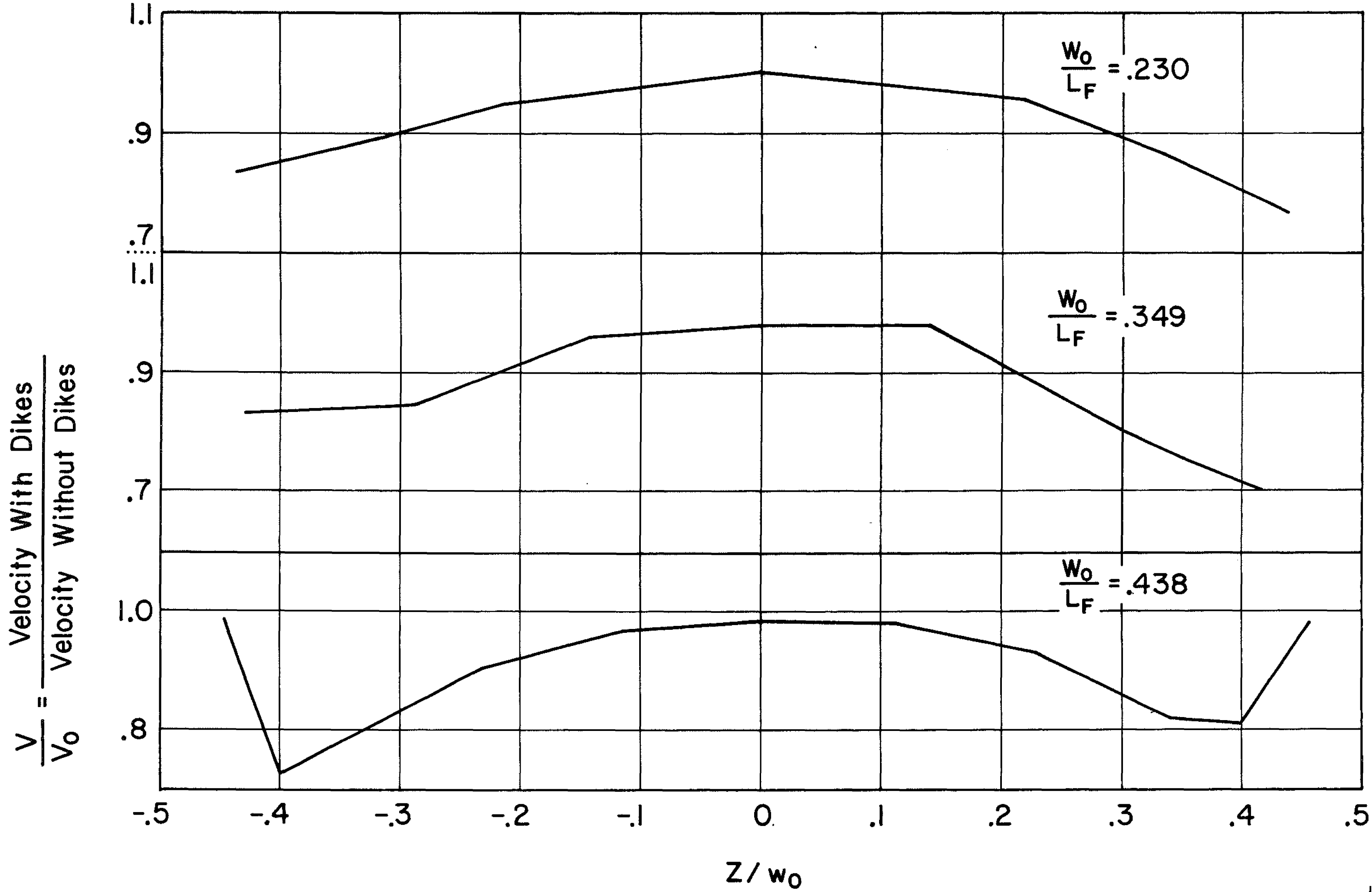


Fig. 5 Typical patterns of velocity reduction at centerline between abutments by using spur dikes. 90-degree approach flow.

the original is evident for each of the contracting ratios (w_o/L_F). However, the patterns of reductions are a function of the contracting ratios and it should be noted that the maximum reduction occurs near the abutment, where it is important to prevent high velocities. Thus it may be stated that the average reduction is not as significant as the pattern of reduction.

With the abutments skewed at 60 degrees to the flow, the addition of dikes decreased the velocities along the left-hand abutment to as low as sixty percent of the original. On the right side the velocities increased for the twenty three percent contraction but decreased for the other contractions (Fig. 6). That the greatest contraction should produce the worst condition may be explained by the fact that the fluid flow is deflected toward the right abutment by the dike.

(b) Use of Continuity Equation - For a rectangular channel the continuity equation may be written as Q = V by where Q = discharge, V = mean velocity over the section, b = width and y = depth. The equation may be written in a logarithmic form as $\log_e Q = \log_e V + \log_e b + \log_e y$ which when differentiated takes the form of

$$\frac{dQ}{Q} = \frac{dV}{V} + \frac{db}{b} + \frac{dy}{y} \qquad (2)$$

For constant discharge $\frac{dQ}{Q} = 0$ and $\frac{dV}{V} + \frac{dy}{y} = -\frac{db}{b}$ (3)

The effectiveness of channel conveyance increases by introduction of dikes upstream of the abutments. If the effective width of the channel without spur dikes is called b_o and the width with spur dikes is called b, then the difference between the two widths $\Delta b = b - b_o = n\, b_o - b_o$, where n is the measure of the conveyance effectiveness of the channel.

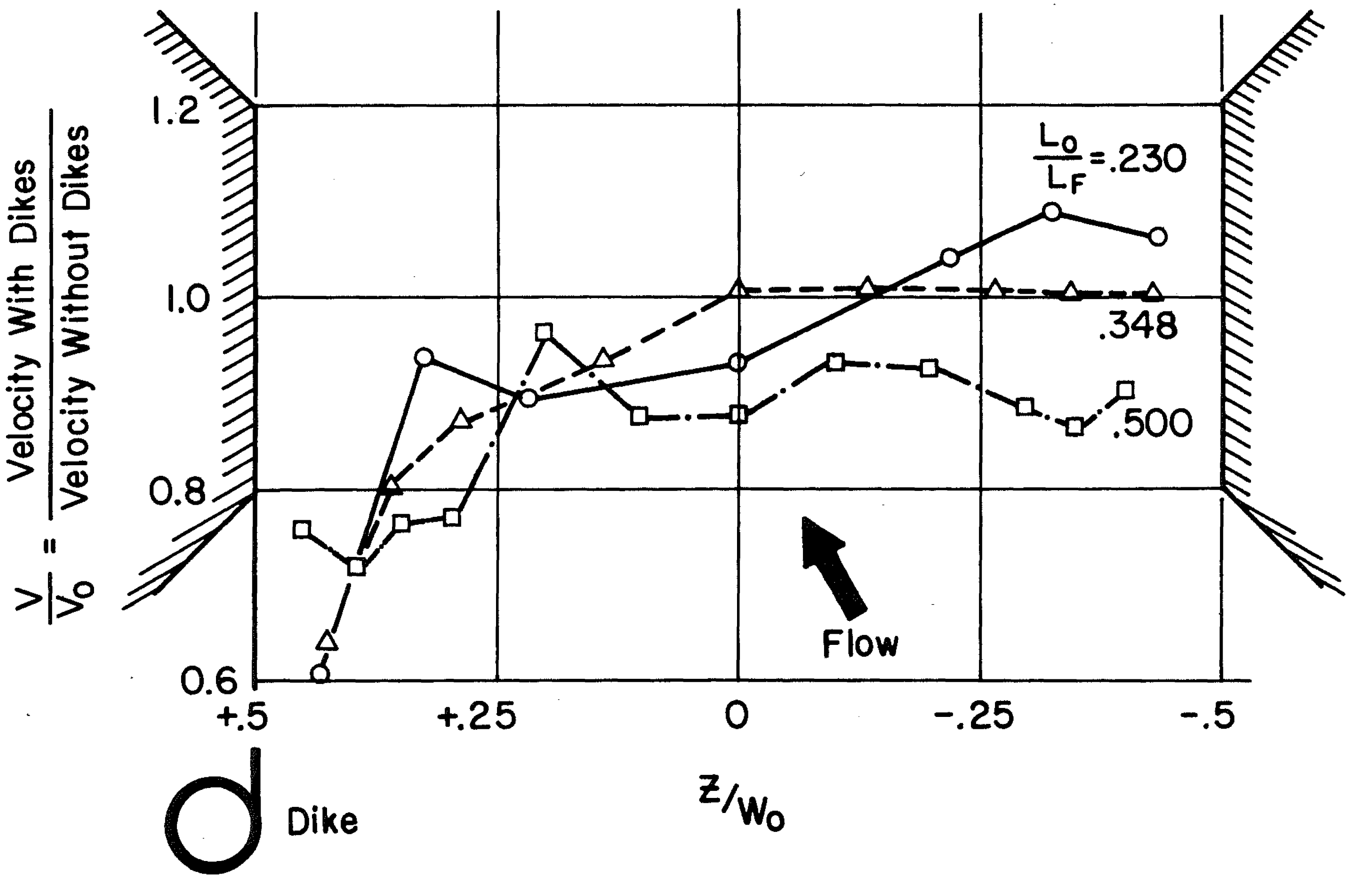


Fig. 6 Velocity reduction produced by spur dikes. 60-degree approach flow.

Writing Equation (3) in terms of differences

$$\frac{\Delta V}{V_o} + \frac{\Delta y}{y_o} = - (\frac{n-1}{b_o}) b_o \qquad (4)$$

or

$$n = 1 - \frac{\Delta V}{V_o} - \frac{\Delta V}{y_o} \qquad (5)$$

In most cases n was found to be greater than one when dikes were employed which indicates an increase in the conveyance efficiency of the channel.

(c) Effect of Dike Angle on Change of Froude Number - Introduction of spur dikes causes changes in depth and velocity between abutments. In general, well-designed dikes will cause an increase in depth and consequently decrease in velocity.

If Froude Number without dikes is defined as $FF_o = \frac{V_o}{\sqrt{gy_o}}$ and the Froude Number with dikes $FF = \frac{V}{\sqrt{gy}}$ then it may be stated that the Froude Number will decrease when dikes are constructed to guide the flow pattern abutments.

Figures 7, 8 and 9 show the effect of dike angle (α), length of dike (L_d) and opening width (w_o) on the change in Froude Numbers (with and without the dike). These figures, which are for FF equal to 0.520, 0.845 and 0.968 respectively, indicate that the effect of dike angle varies to a large extent with the Froude Number.

(d) The Discharge Equation - Physical analysis of flow at a constriction by the energy approach is complicated by the occurence of vertical as well as lateral contraction and by the variable extent of drowning of the jet. The momentum approach is restricted by lack of

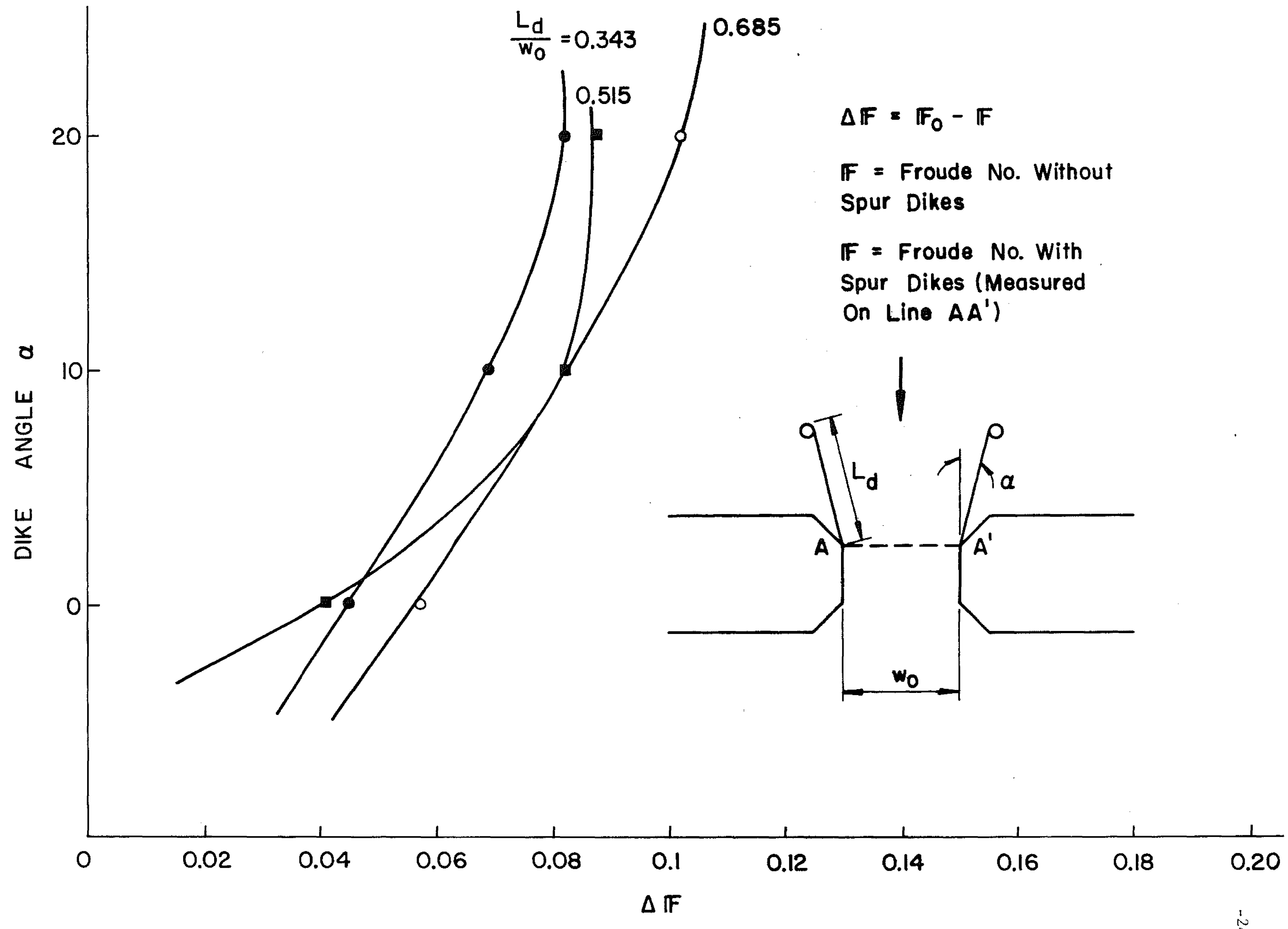


Fig. 7 Effect of spur dikes on flow pattern. Froude Number = 0.52.

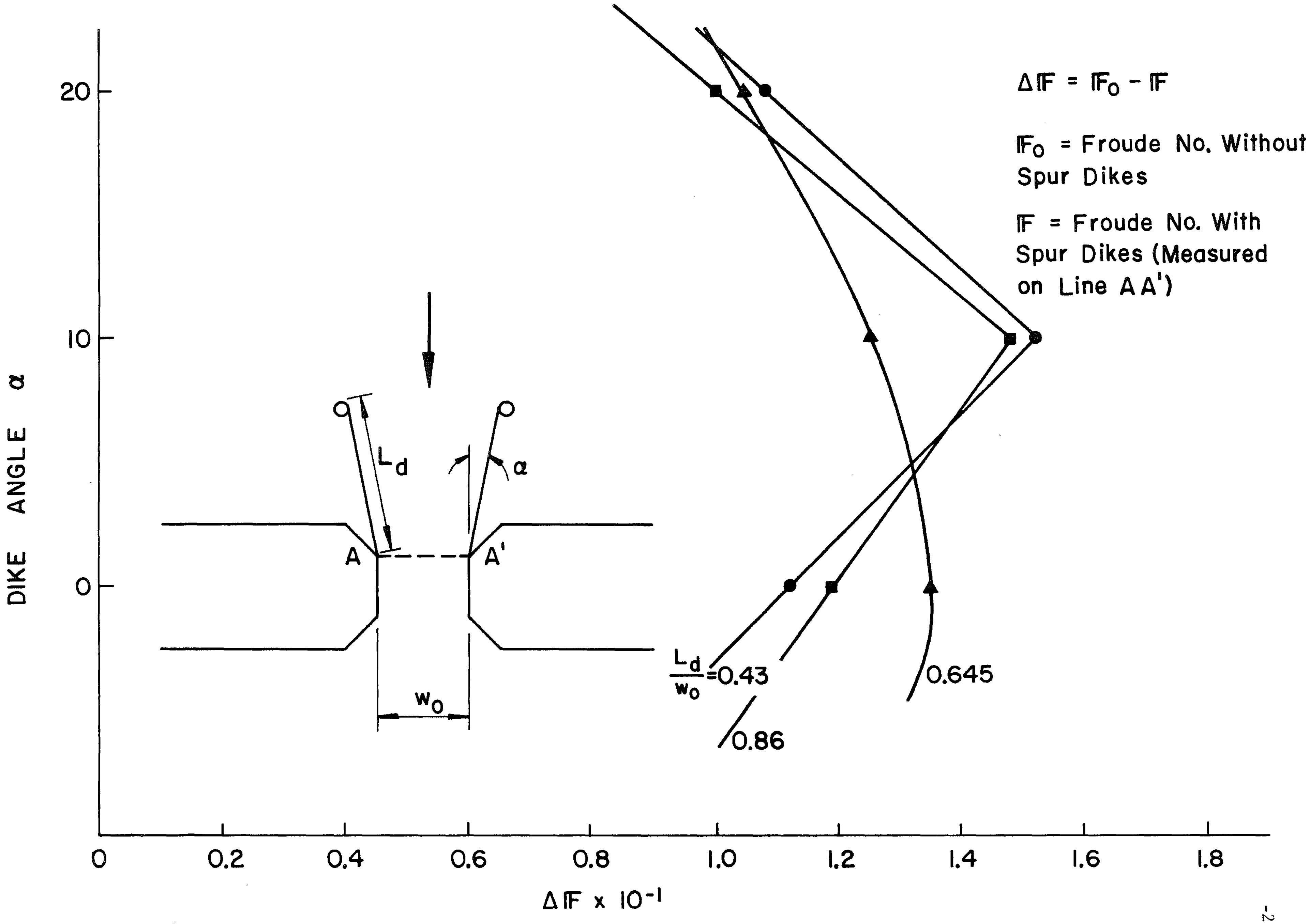


Fig. 8 Effect of spur dikes on flow pattern. Froude Number = 0.845.

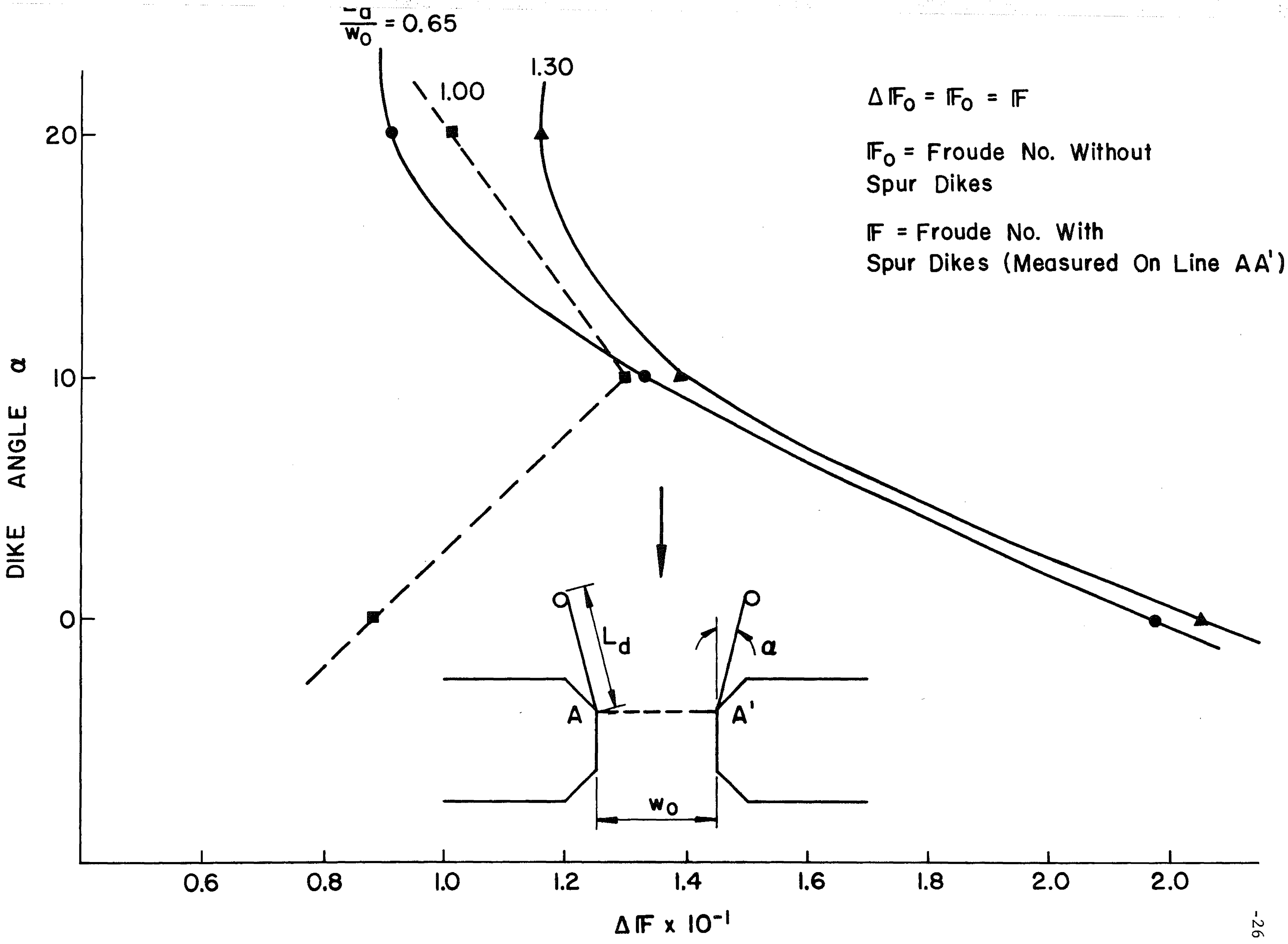


Fig. 9 Effect of spur dikes on flow pattern. Froude Number = 0.968.

sufficiently reliable knowledge of boundary forces in the region downstream from the constriction. Vallentine[17] suggested that an expression of the form

$$Q = C\, w_o\, y_w^{3/2} \tag{6}$$

may be used where the discharge coefficient is

$$C = f\left(FF_o\, ,\ \frac{w_o}{L_F}\, ,\ \frac{y_o}{L_F}\right) \tag{7}$$

where y_w = average water depth with spur dikes. This coefficient embodies the effect of lateral and vertical contraction of the jet and also the effect of dikes.

Figure 10 indicates the effect of Froude Number on the coefficient C. It appears that the effect of dike angle is negligible and that within the narrow range of Froude Number (between 0.42 and 0.88) the coefficient C is directly proportional to the Froude Number.

(e) Dike Coefficient - The dike coefficient may be defined as

$$K_{FF} = \frac{FF}{FF_o}\ \frac{L_d}{w_o} \tag{8}$$

The coefficient, which is dimensionless, measures the effectiveness of spur dikes and embodies the effects of length of dike and width of opening between abutments. Figure 11 indicates the effect of L_d/w_o ratio on coefficient K_{FF}. This diagram is of practical value because from it one

[17] Vallentine, H. R., "Flow in Rectangular Channels with Lateral Constriction Plates", La Houille Blanche, No. 1, pp. 75-84, January - February 1958.

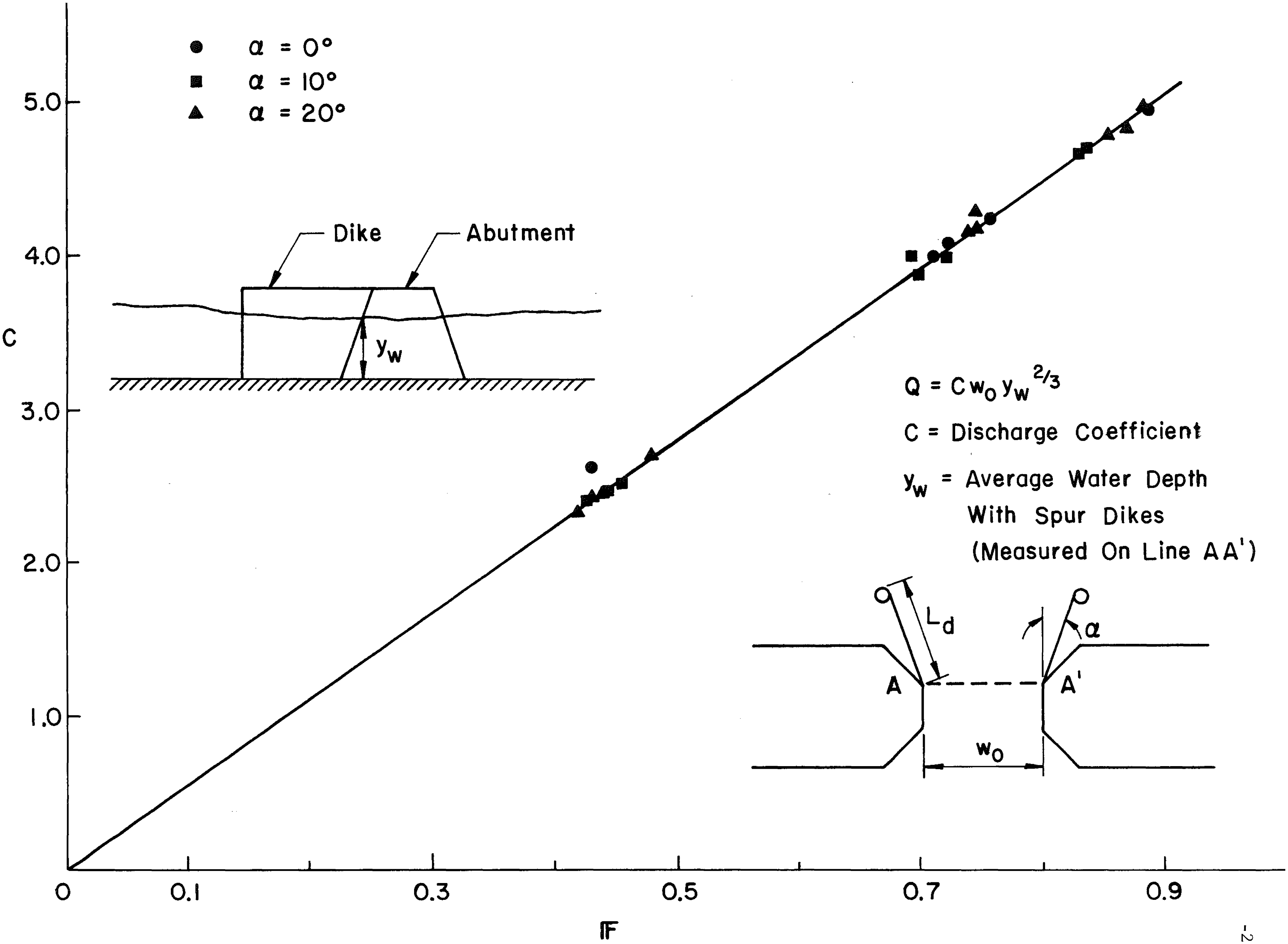


Fig. 10 Discharge coefficient as a function of Froude Number.

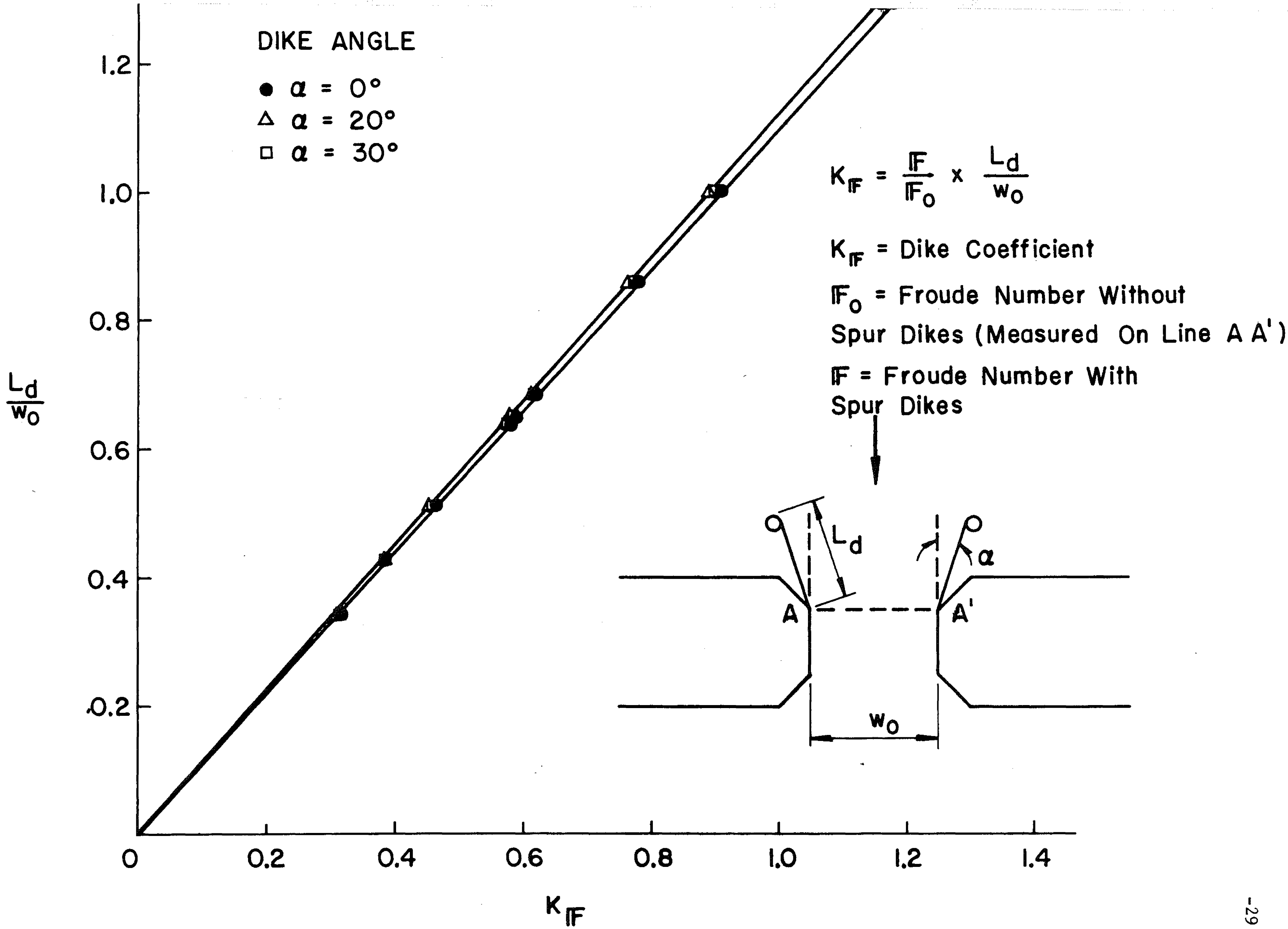


Fig. 11 Dike coefficient as a function of dike length ratio.

one can predict the Froude Number between the abutments equipped with spur dikes, if the flow Froude Number without spur dikes, the length of dike, and the width between abutments are known. The dike angle has only a small effect on the other variables as indicated in Fig. 11.

MOVABLE-BED STUDIES

The greater part of scour investigation was conducted with clear water and the scour produced in this case may be referred to as "clear water scour".[18] A few tests were conducted with sediment-transporting flow which will be called the "sediment supply scour". There is a difference between two types of scour and it should be emphasized that conclusions reached from experiments on either type cannot be simply transposed to apply to the other. In limited tests it was found that the scour with clear water was greater than the scour with sediment transporting flow.

(i) Results and Comments - 90-Degree Approach - The movable-bed studies confirmed the predictions based on the fixed-bed investigation that curved spur dikes, in providing a smooth transition for the flow, were extremely effective in reducing scour effects at the abutments. At some points along the abutments deposition occurred where, without dikes, scour would have been developed.

(a) Studies of spirally-shaped spur dikes indicated that such dikes will protect the abutment from damage due to scour. Not only

[18] Laursen, E. M., "Some Aspects of the Problem of Scour at Bridge Crossings", Federal Inter-Agency Conference, Jackson, Mississippi, February, 1963.

did the dikes significantly reduce maximum scour depths, but they moved the points of deep scour away from the abutments.

(b) The assumptions made in the fixed-bed investigation that uniformity of flow and reduction of eddies produced less scour were verified by the movable-bed model study.

(c) In the scour studies of dikes-abutments it was found that the mean depth varied as the two-thirds power of the discharge. The same proportionality was reported by Leopold and Wolman[19] for scour between bridges.

Clean water scour results for a 90 degree bridge crossing are presented in Figures 12 and 13. Deep local scour at the abutments which is very evident when dikes are not used (Fig. 12) is not present when dikes are employed (Fig. 13), and the deepest scour occurs at the center part of the channel. The contraction ratio is sufficiently high so that the scour patterns generated from each abutment overlap.

(ii) 60-Degree approach

(a) The condition at a bridge site with skewed abutments is much more severe than with right-angles abutments, and the scour occurs at comparatively low discharges.

Typical clear water scour results for 60-degree crossing are presented in Figures 14 and 15. Heavy scour which occurred particularly at the downstream abutment (Fig. 14) is completely eliminated by the use of dikes. (Fig. 15).

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[19] Leopold, L. B. and Wolman, M. G., Professional Paper 252, U. S. Geological Survey, 1960.

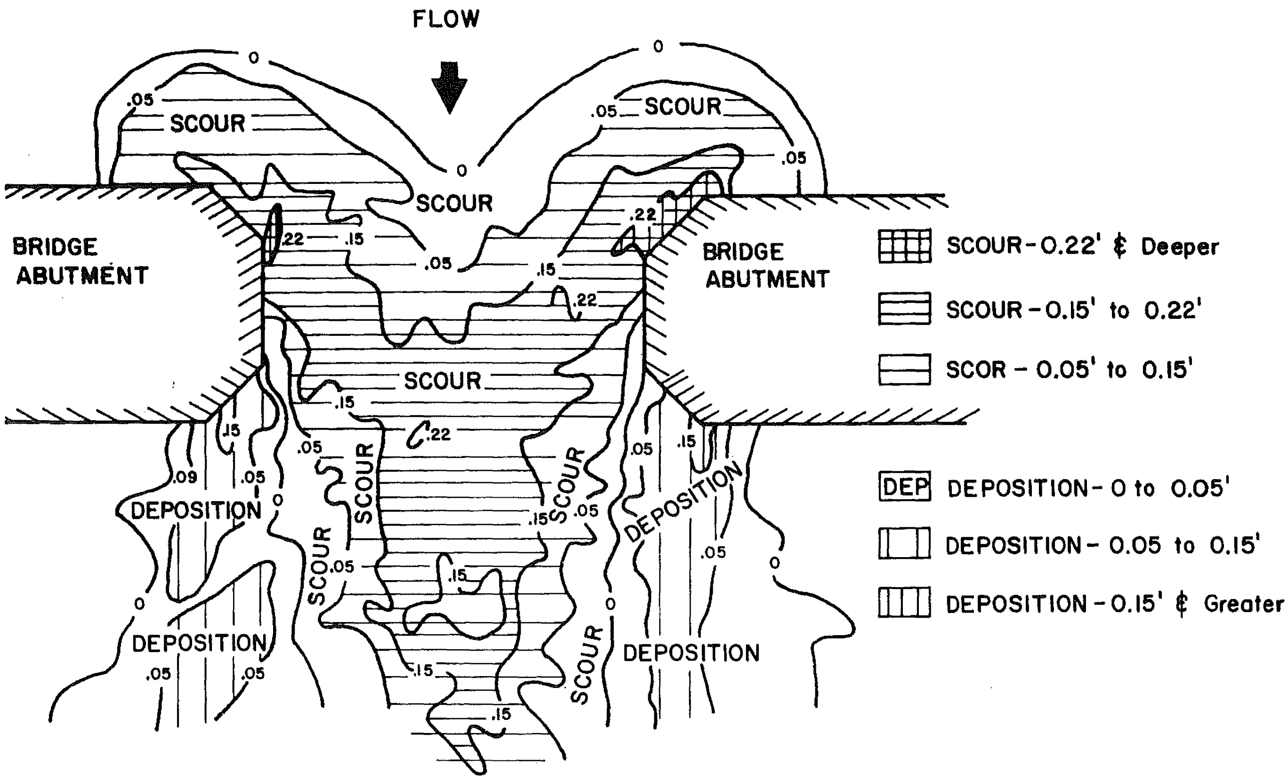

Fig. 12 Scour pattern between abutments without dikes, width between abutments 41.5 inches, $\frac{Wo}{L}$ = 34.6%.

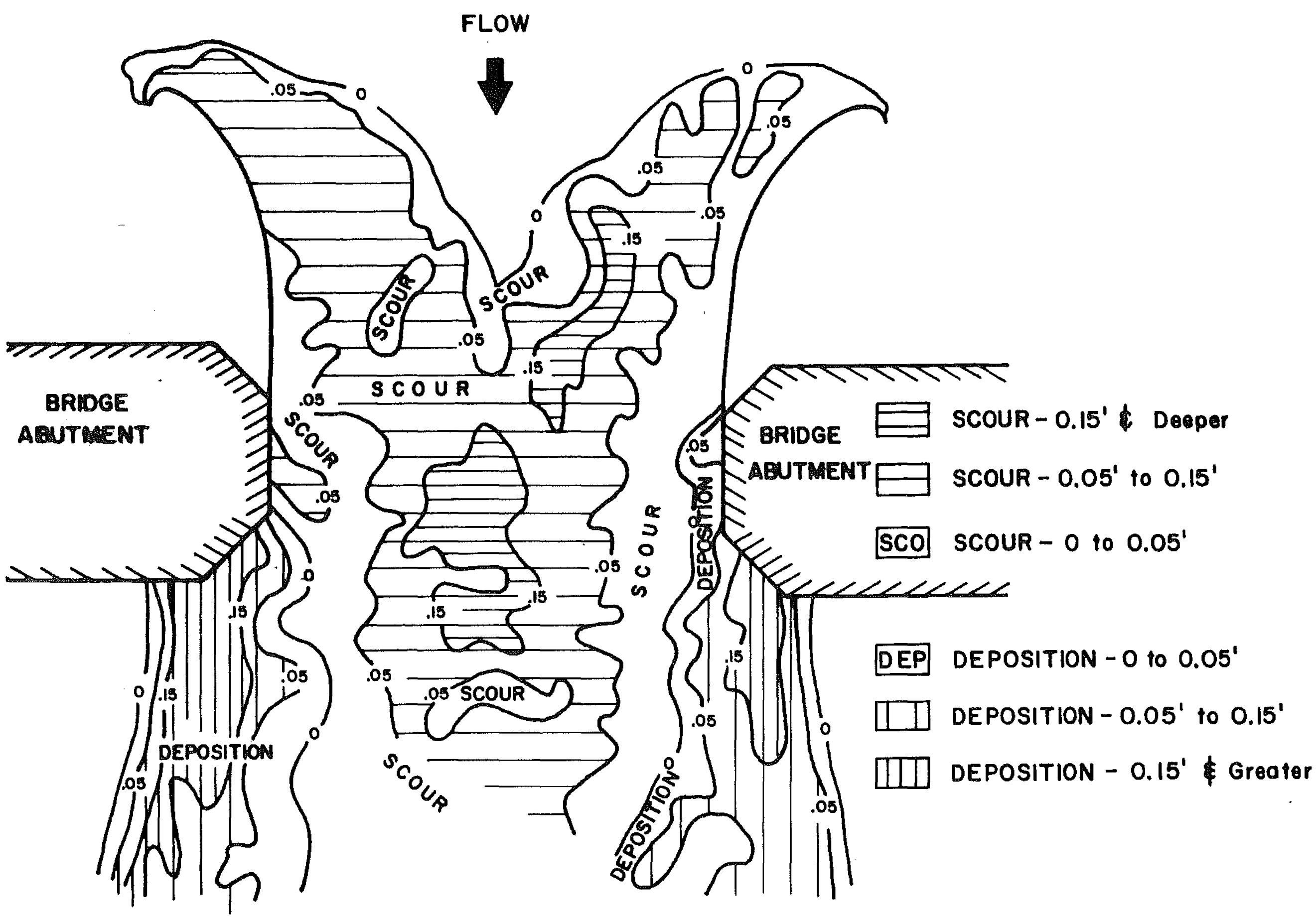

Fig. 13 Scour pattern between abutments with dikes, width between abutments 41.5 inches, $\frac{W_o}{L} = 34.6\%$.

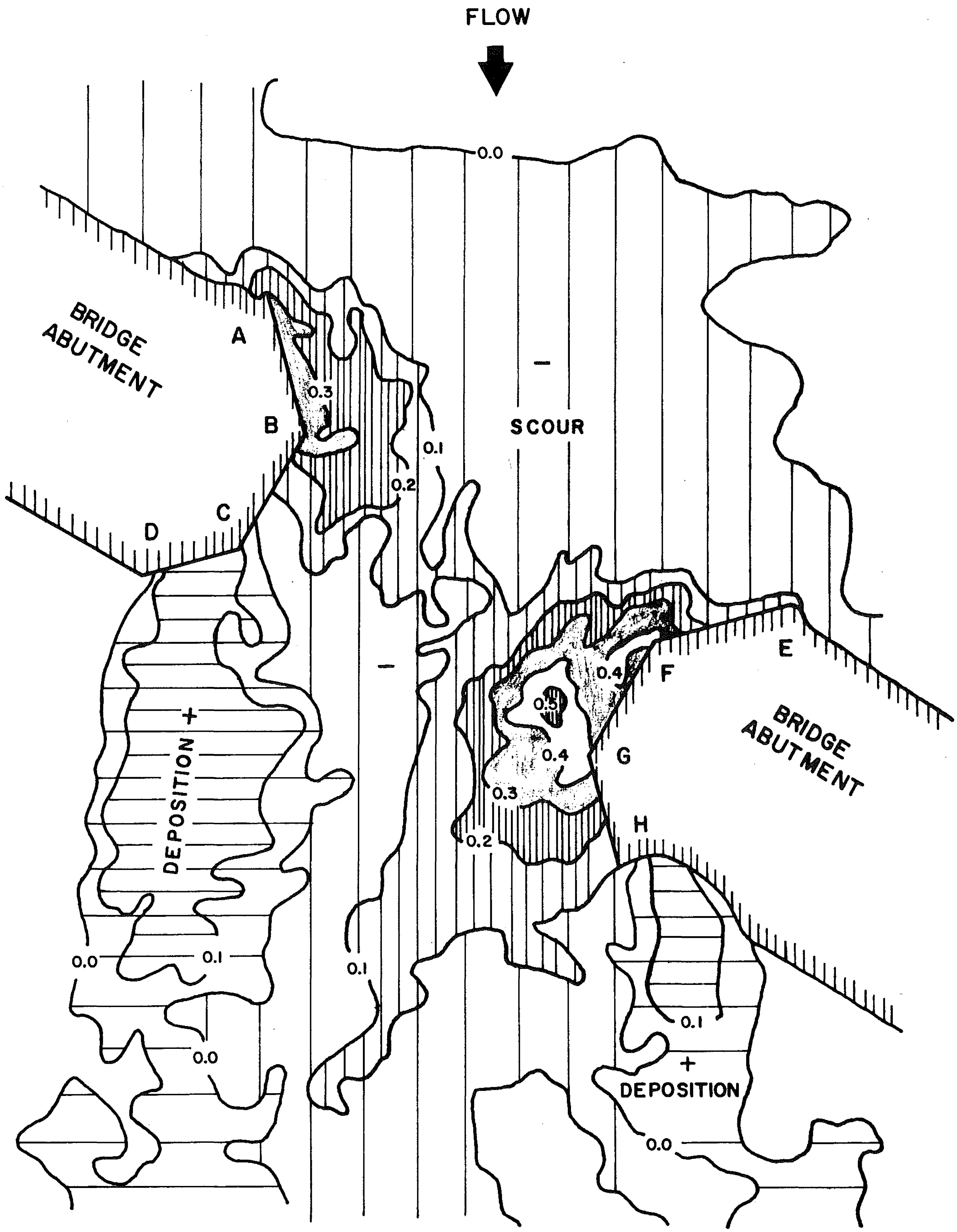

Fig. 14 Scour pattern between abutments. 60-degree skewed crossing. Width between abutments 41.5 inches, $\frac{W_o}{L}$ = 34.6%.

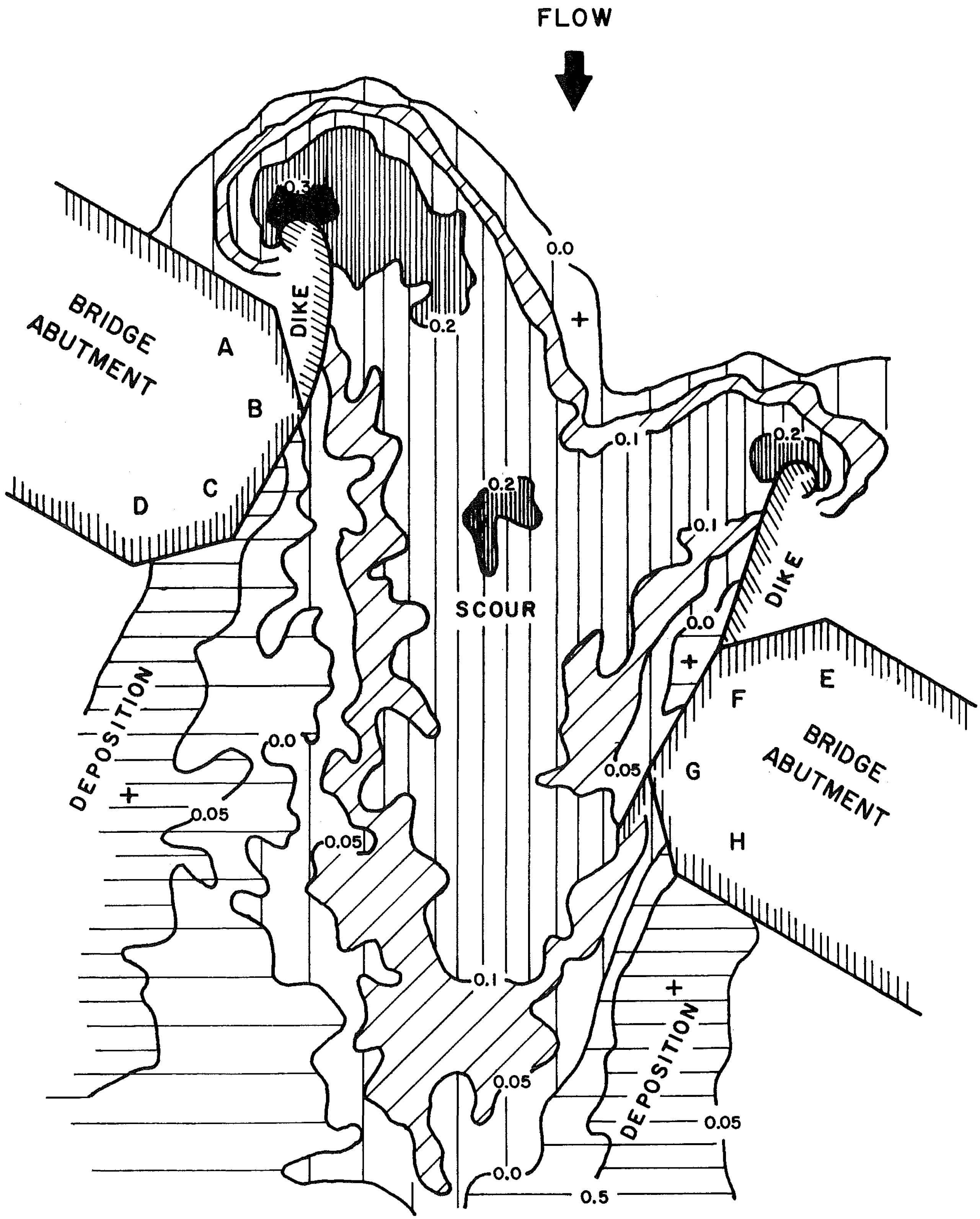

Fig. 15 Scour pattern between abutments with dikes. 60-degree skewed crossing. Width between abutments 41.5 inches, $\frac{W_o}{L} = 34.6\%$.

PRELIMINARY DESIGN RECOMMENDATIONS

90-Degree Approach

(a) A curved dike should be used as it reduces eddying at the head of dike, eliminates eddying at the junction of dike and abutment and provides uniform velocities between abutments.

(b) Experimental studies indicate that a spiral shape fulfills these requirements. The dike should join the abutment tangentially (Fig.3).

(c) The length of dike itself is not important, provided that it is over a minimum length. The length required to develop a certain shape will usually be greater than the minimum length desired.

(d) The dike shape should be determined for maximum flow to be expected. This will provide a satisfactory flow for lower discharges.

(e) Shape and length of dike depends upon discharge. In case of high discharge, the shape of the dike should change very gradually. This would cause the dike to be longer than for the case of lower discharge where the transition need not be so gradual.

(f) It should be borne in mind that highest velocities would occur along the dikes in the transition zone and measures should be taken to protect the dike embankment with rip-rap or rock fill. (Fig. 16)

60-Degree Approach

(a) Comments discussed under (a), (d), (e) and (f) apply equally to the 60-degree approach.

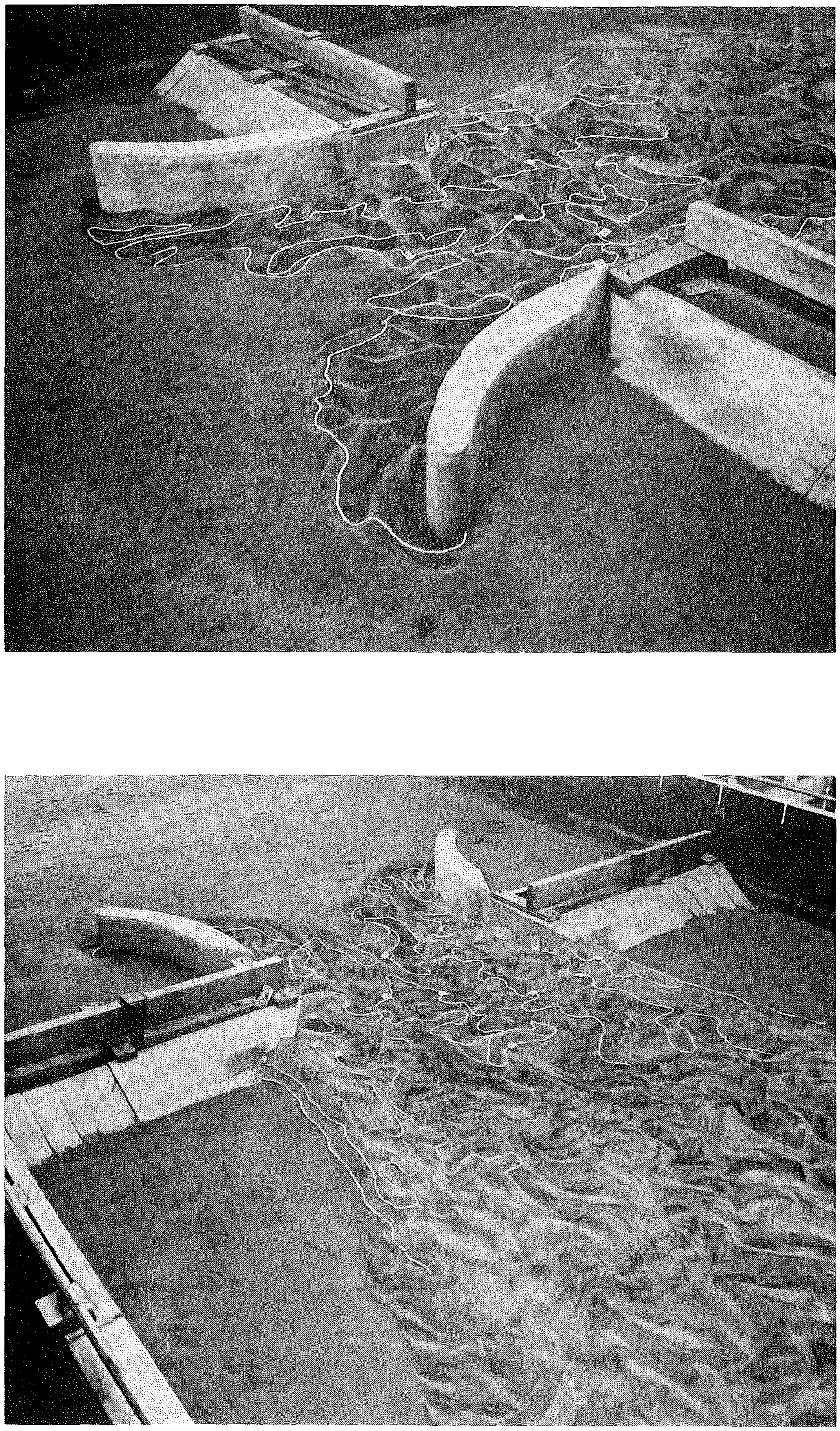

Fig. 16 Views of scour patterns

(b) Dikes at both abutments are necessary. The most effective shape for the upstream dike is elliptical with axis ratio 2-1/2 to 1 (upstream of point B, Fig. 4), and that for the downstream dike is the straight at 5-degree inclination toward the center of the opening (upstream of point F). A stub dike, curved in shape is necessary at the downstream corner (point G) of the downstream abutment.

(c) For the upstream abutment, although a shorter dike is quite sufficient to eliminate scour in front of the abutment, scour at the end of the shorter dike would reach the abutment from behind -- consequently a longer elliptical dike is required there.

Suggestions for Further Research - 1. Determination of optimum curvature and length of curved dike for

(a) $\frac{\text{abutment opening}}{\text{width of stream}}$ ratio

(b) rate of flow.

2. Effect of $\frac{\text{abutment opening}}{\text{width of stream}}$ ratio on scour pattern. It will be noted that the constriction ratios studied were such that the scour pattern overlapped. The effectiveness of dikes for smaller constriction ratios should be evaluated.

3. The difference between "clear water scour" and "sediment supply scour" should be evaluated.

4. The difference between constant discharge and the discharge varying according to a hydrograph on scour depth should be thoroughly studied.

5. Determination of "scale effect", if any, for various sediment sizes should be evaluated.

6. Field performance of spur dikes at bridges should be observed and evaluated. Some observations were recently reported by Schneible[20].

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APPENDIX - NOTATION

The following symbols have been adopted for use in this paper:

b	= width (with spur dikes)	L
b_o	= width (without spur dikes)	L
B	= width of opening	L
C	= coefficient of discharge	$L^{\frac{1}{2}}/T$
d_s	= mean particle size	L
D_s	= depth of scour hole below water surface	L
f	= Lacey's silt factor	-
f_A	= shape of abutment	-
f_D	= shape of dike	-

[20] Schneible, D. E., "Field Observations on Performance of Spur Dikes at Bridges". Paper presented at the ASCE Transportation Conference, Philadelphia, Pennsylvania, October 1966.

f_V	= velocity distribution	-
$\mathbb{F}$	= Froude Number with dikes = $\frac{V}{\sqrt{gy}}$	-
$\mathbb{F}_o$	= Froude number without dikes = $\frac{V_o}{\sqrt{gy_o}}$	-
g	= acceleration due to gravity	L/T^2
k	= a factor	-
$K_{\mathbb{F}}$	= dike coefficient	-
$\log_e$	= logarithm to the base e	-
L_d	= length of dike	L
L_F	= width of waterway	L
n	= conveyance effectiveness of channel	-
Q	= flow rate	L^3/T
V	= velocity (with spur dikes)	L/T
V_a	= approach velocity	L/T
V_f	= fall velocity of particle	L/T
V_o	= velocity (without spur dikes)	L/T
w_o	= width between abutments	L
x	= coordinate axis	L
y	= coordinate axis; depth	L
y_s	= depth of scour	L
y_w	= average water depth (with dikes)	L
z	= coordinate axis	L
α	= angle of dike ~~depth (with dikes)~~	-
Δ	= difference	-
θ	= angle of abutment skew	-
μ	= fluid viscosity	$\frac{M}{LT}$
ρ	= fluid density	$\frac{M}{L^3}$
ρ_s	= particle density	$\frac{M}{L^3}$
τ_o	= boundary shear stress	$\frac{M}{LT^2}$

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