BULLETIN NO. 8

IOWA HIGHWAY RESEARCH BOARD

Scour at Bridge Crossings

by

Emmett M. Laursen

Iowa Institute of Hydraulic Research

State University of Iowa

Prepared by the

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LIST OF SYMBOLS

a — an exponent in the approximate form of the Laursen total-sediment-load relationship which depends on the shear-velocity/fall-velocity ratio.

b — a nominal width measured out from the bank (overbank constriction) or the abutment (encroachment) and equal to or slightly greater than 2.75 d_s (feet).

b_s — width of the scour hole measured in the direction of the bridge (feet).

B — width of stream between normal banks (feet); subscripts 1 and 2 refer to the uncontracted and contracted reaches, respectively, in a long contraction.

$\bar{c}$ — mean sediment concentration (percent by weight of mixture of sediment and water.)

d — mean diameter of the bed material (feet).

d_s — equilibrium depth of scour measured from the normal bed (feet).

F — Froude number, $V/\sqrt{gy}$.

g — acceleration of gravity (feet per second per second).

K — a coefficient in the approximate form of the Laursen total-sediment-load relationship which depends on the shear-velocity/fall-velocity ratio.

K_τ — a coefficient greater than unity for the effect on the depth of scour of a change in mode of sediment movement.

K_θ — a coefficient for the effect on the depth of scour of the angle of incidence between the approach fill and the direction of flow.

ℓ — effective length of an encroaching abutment (feet).

L — clear distance between the abutment and the end of the vegetal screen along the bank line (feet).

n — Manning roughness coefficient.

Q_t — total discharge of the stream (cubic feet per second).

Q_c — that portion of the total discharge flowing within the confines of the normal banks of the main channel of the stream (cubic feet per second).

Q_o — that portion of the total discharge flowing on the overbank area, or floodplain (cubic feet per second).

Q_b the discharge over a nominal width b (cubic feet per second).

Qe that portion of the overbank which remains on the floodplain if the abutment is set back from the normal bank (cubic feet per second).

Ql the discharge approaching an encroaching abutment (cubic feet per second).

Q_s rate of sediment transport (cubic feet per second).

r the ratio of the maximum depth of scour at an abutment to the depth of scour in a long contraction of width 2.75 d_s.

S slope of the energy gradient (feet per foot).

V_0 mean approach velocity of the stream (feet per second).

w mean fall velocity of the bed material (feet per second).

y depth of flow (feet); subscripts 1 and 2 refer to the average depth in the uncontracted and contracted reaches, respectively, of a long concontraction; subscripts 0 refers to the average depth over the nominal width b.

ϵ distance abutment is set back from the normal bank (feet).

ILLUSTRATIONS

SCOUR AT BRIDGE CROSSINGS

by

Emmett M. Laursen

THE GENERAL PROBLEM OF SCOUR AT BRIDGE CROSSINGS

Scope. Basically, scour is a consequence of an imbalance between the capacity of the flow to transport sediment out of an area and the rate of supply of sediment to that area. At a bridge crossing, the area of interest is the immediate vicinity of the bridge foundation—the piers and abutments—and the imbalance of capacity and supply can arise from a variety of causes, all of which should be considered in the prediction of the scour that may be expected around the piers and abutments. A useful distinction can be made by separating the various causes into two general categories: (1) those characteristic of the stream itself, and (2) those due to the modification of the flow by the bridge crossing.

In general, the scour in an unobstructed stream cannot be predicted, principally because the detailed flow pattern in the unobstructed stream cannot be predicted. Reasonable estimates can be made, however, on the basis of experience at similar reaches of similar streams, or, if time and the vagaries of stream flow allow, measurements at the particular site.

The scour which occurs because of the modification of the flow pattern by the bridge crossing can be further divided into two fundamentally distinct types of scour depending on whether or not sediment is supplied to the scour hole. A limiting equilibrium configuration of the bed is attained when a scour hole is enlarged to a size where the capacity to remove material from the scour hole is balanced by the rate at which sediment is supplied to the scour hole. During floods, a scour hole located in the main channel (i.e., around a main channel pier or an abutment adjacent to the main channel) will be supplied with sediment at a rate characteristic of the stream. Ignoring the complexities of material stratification that may exist below the stream bed, the material which is supplied will be essentially the same as the material which must be removed to form the scour hole.

If no sediment is supplied to the scour hole, the limiting, equilibrium configuration of the bed is not attained until the capacity to remove material from the scour hole is zero; i.e., when the shear of the boundary reaches the critical tractive force for the material which forms the boundary. Although fine material may be carried in suspension by the flow on the floodplain, the low velocities and vegetal cover will inhibit the rate of transport, especially of coarse materials. This is the type of scour to be expected at relief bridges on the floodplain. If the waterway opening spanned by a main channel bridge is larger than the width between the normal bank lines, the part of the bridge beyond the normal bank line should be considered a relief bridge. A scour hole on the floodplain around an abutment set back a considerable distance from the main channel will not be supplied with sediment of the size being scoured out.

This investigation, which has been carried out by the Iowa Institute of Hydraulic Research under the sponsorship of the Iowa State Highway Commission and the U. S. Bureau of Public Roads, has only considered scour where sediment is supplied to the scour hole by the main channel flow. In an earlier report (1) a method of predicting the scour at main channel piers was proposed. In a later section of this report a comparable method of predicting the scour at abutments adjacent to the main channel is presented.

Scour in an unobstructed stream. The channel of an alluvial stream is not fixed, but is in a constant state of flux. Changes may take place rapidly in a single flood, periodically during a water year, or slowly during a period of time comparable to the anticipated life of the bridge. The stream may be degrading over a considerable length—whether naturally, as an erosional agent of the geological cycle, or as the consequence of a dam some distance upstream or a straightening of the channel downstream. Natural degradation of any significance should be readily ascertainable in any settled region. Degradation caused by the works of man can also be predicted reasonably well—the difficulty being rather the anticipation of what men may do.

Particular streams and particular sites may have quite individualized characteristics. In a well-behaved meandering stream, the greatest depth will be at the outside of the bend. During high stages, the bends will scour and the crossings between bends will fill; during low stages, the crossings will scour tending to fill the next bend downstream. However, chutes can develop across the inside of a bend, and if the scour enlarges the chute sufficiently, the channel may be shifted to this new location. Quite commonly, the low-water flow occupies a channel which wanders within the confines of the main channel. At one particular site, which was examined as a possible location for field measurements of scour at a bridge pier, the low-water channel had a smaller radius of curvature than the main channel and at the bridge site was near the inside of the bend—the outside of the bend being a wide mudflat. From various items of evidence, it was readily apparent that during high stages the flow pattern in the main channel was as one would normally expect and that the mudflat would be entirely scoured out.

Braided streams, which are characteristically wide and shallow, will have one or more relatively deep channels which will shift erratically. In a meandering river a gradual, continuous displacement of the channel is to be expected, and over a long period of time the meander belt itself will shift.

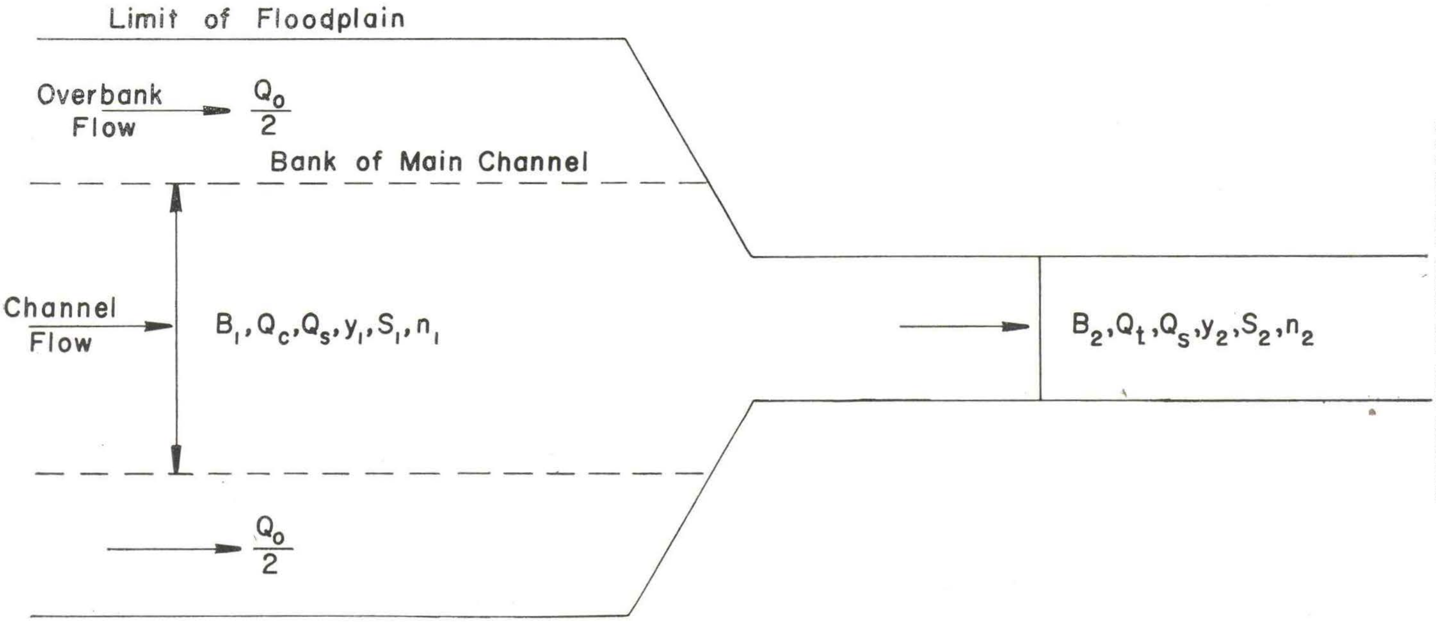


Figure 1. Definition sketch of a long contraction

Any contraction, whether of the main channel or of the overbank flow, will result in scour during high stages. With the assumption that the flow in the contracted and the uncontracted reaches is uniform, this case of scour can be solved analytically for the limiting, equilibrium depth of scour. It is this case of scour that is probably primarily responsible for the oft-quoted statement that a plains river will scour its bed as much as the water surface rises. That this too-commonly-held belief is unsound was demonstrated by Lane and Borland (2).

Scour in a long contraction. Some notion of the scour that can occur naturally in a river, solely because of the change of section from reach to reach, can be obtained by considering the river as a series of contractions and expansions. The contractions may occur in the main-channel, the overbank area, or in a combination of both. By assuming that the flow in the uncontracted and the contracted reaches is uniform, the ratio of the depth of flow in the two reaches can be obtained from the principle of continuity applied both to the water discharge and the sediment discharge as was first done by Straub. (3).

The necessary conditions are shown schematically in Fig. 1. The total flow Q_t in the contracted section must be equal to the sum of the flow in the main channel Q_c and the flow of the overbank Q_o. At the limiting condition, the rate of sediment transport Q_s in the two sections must be equal. This is assuming that the sediment transport on the overbank is zero—the worst condition. The flow in the main channel and in the contracted section can be described by the Manning equation as

$$\left.\begin{aligned} Q_c &= \frac{1.49}{n_1} B_1 \, y_1^{7/6} \sqrt{y_1 S_1} \\ Q_t &= \frac{1.49}{n_2} B_2 \, y_2^{7/6} \sqrt{y_2 S_2} \end{aligned}\right\} \qquad (1)$$

An approximate form of a total-sediment-load relationship recently proposed by the writer (4) can be used to evaluate the rate of transport.

$$Q_s \propto \bar{c}_1 \, Q_c = \bar{c}_2 \, Q_t \qquad (2)$$

where the mean sediment concentration (in percent by weight)

$$\bar{c} = \left(\frac{d}{y}\right)^{7/6} \left(\frac{V^2}{120 \, y^{1/3} \, d^{2/3}}\right) K\left(\frac{\sqrt{gyS}}{w}\right)^{a}$$

in which d and w are the diameter and fall velocity of the sediment and the exponent a and the coefficient K depend on the shear-velocity/fall velocity ratio. The variation of a is as follows:

$\sqrt{gyS}/w$	a
$< 1/2$	$1/4$
$= 1$	1
> 2	$9/4$

The shear velocity can be evaluated in terms of Q, B, y, and n by means of Manning's formula as

$$\sqrt{gyS} = \frac{Q\sqrt{g}\,n}{1.49\ B\ y^{7/6}}$$

Equation (2) written out in full is then

$$\left(\frac{d}{y_1}\right)^{7/6}\left(\frac{Q_c^{\,2}}{120\ B_1^{\,2}\ y_1^{\,7/3}d^{2/3}}\right)K\left(\frac{Q_c\sqrt{g}\,n_1}{1.49\ B_1\ y_1^{\,7/6}\ w}\right)^{a} Q_c$$

$$= \left(\frac{d}{y_2}\right)^{7/6}\left(\frac{Q_t^{\,2}}{120\ B_2^{\,2}\ y_2^{\,7/3}d^{2/3}}\right)K\left(\frac{Q_t\sqrt{g}\,n_2}{1.49\ B_2\ y_2^{\,7/3}d^{2/3}}\right)^{a} Q_t$$

Because they appear in the same way on both sides of the equation, the coefficients and the sediment characteristics will cancel out. Cross multiplication then results in

$$\left(\frac{y_2}{y_1}\right)^{\frac{7}{6}+\frac{7}{3}+\frac{7}{6}a} = \left(\frac{Q_t}{Q_c}\right)^{3+a}\left(\frac{B_1}{B_2}\right)^{2+a}\left(\frac{n_2}{n_1}\right)^{a}$$

or

$$\left(\frac{y_2}{y_1}\right) = \left(\frac{Q_t}{Q_c}\right)^{\frac{6}{7}}\left(\frac{B_1}{B_2}\right)^{\frac{6}{7}\,\frac{2+a}{3+a}}\left(\frac{n_2}{n_1}\right)^{\frac{6}{7}\,\frac{a}{3+a}} \qquad (3)$$

The ratio of the n values should not be too different from unity, and because the ratio is raised to a power at the most of 0.37, the roughness effect can be safely neglected. For a purely overbank contraction in which the flow on the floodplain returns to the main channel, the equation then reduces to

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \qquad (4)$$

for all values of the shear-velocity/fall-velocity ratio. This indicates that the depth ratio should be the same whether the mode of sediment movement is purely as bed load or principally as suspended load.

For flow confined within the main channel, however, the exponent of the contraction ratio does depend on the mode of movement. The general equation for this case reduces to

$$\left.\begin{aligned} \frac{y_2}{y_1} &= \left(\frac{B_1}{B_2}\right)^{0.50} & \frac{\sqrt{gy_1S}}{w} &< 1/2 \\ \frac{y_2}{y_1} &= \left(\frac{B_1}{B_2}\right)^{0.64} & \frac{\sqrt{gy_1S}}{w} &= 1 \\ \frac{y_2}{y_1} &= \left(\frac{B_1}{B_2}\right)^{0.69} & \frac{\sqrt{gy_1S}}{w} &> 2 \end{aligned}\right\} \quad (5)$$

If it is assumed that the material scoured out of the contraction is spread widely over the next wide reach of the river, so that the deposition can be taken as negligible, the depth ratio can be rewritten as

$$\frac{y_2}{y_1} = \frac{d_s}{y_1} + 1$$

on which basis Fig. 2 has been drawn. It is apparent in this figure that the average scour in a contraction can be a sizable fraction of the depth of flow and cannot be neglected in an evaluation of the scour at a bridge crossing. If there is deposition in the wide reaches, the scour will be less. This effect can be assessed by estimating the relative areas in which material will deposit or scour and by equating the volume of deposition to the volume of scour.

Scour around bridge piers and abutments. The contraction of the flow section by a bridge crossing is in many ways similar to the problem of the preceding section: the overbank flow is forced to return to the main channel, and the abutments may project into the main channel section, thereby reducing the flow section. The bridge-crossing constriction differs from the long contraction, however, in that the flow cannot be considered uniform. As a result of the non-uniformity the scour is not the same across the bridge opening. At the abutment the depth of scour will usually be much greater than for the equivalent long contraction, and in the center of the bridge waterway there may be no scour at all.

The bridge piers will further modify the flow pattern locally due to the formation of a consequent scour hole around each individual pier. As reported in Bulletin No. 4, the local scour around a pier is not measurably affected by the velocity of flow or the sediment size if the flow is subcritical ($F<1$) and the mode of sediment movement is as bed-load. The local depth of scour at a pier reduces then to a function of geometry alone—

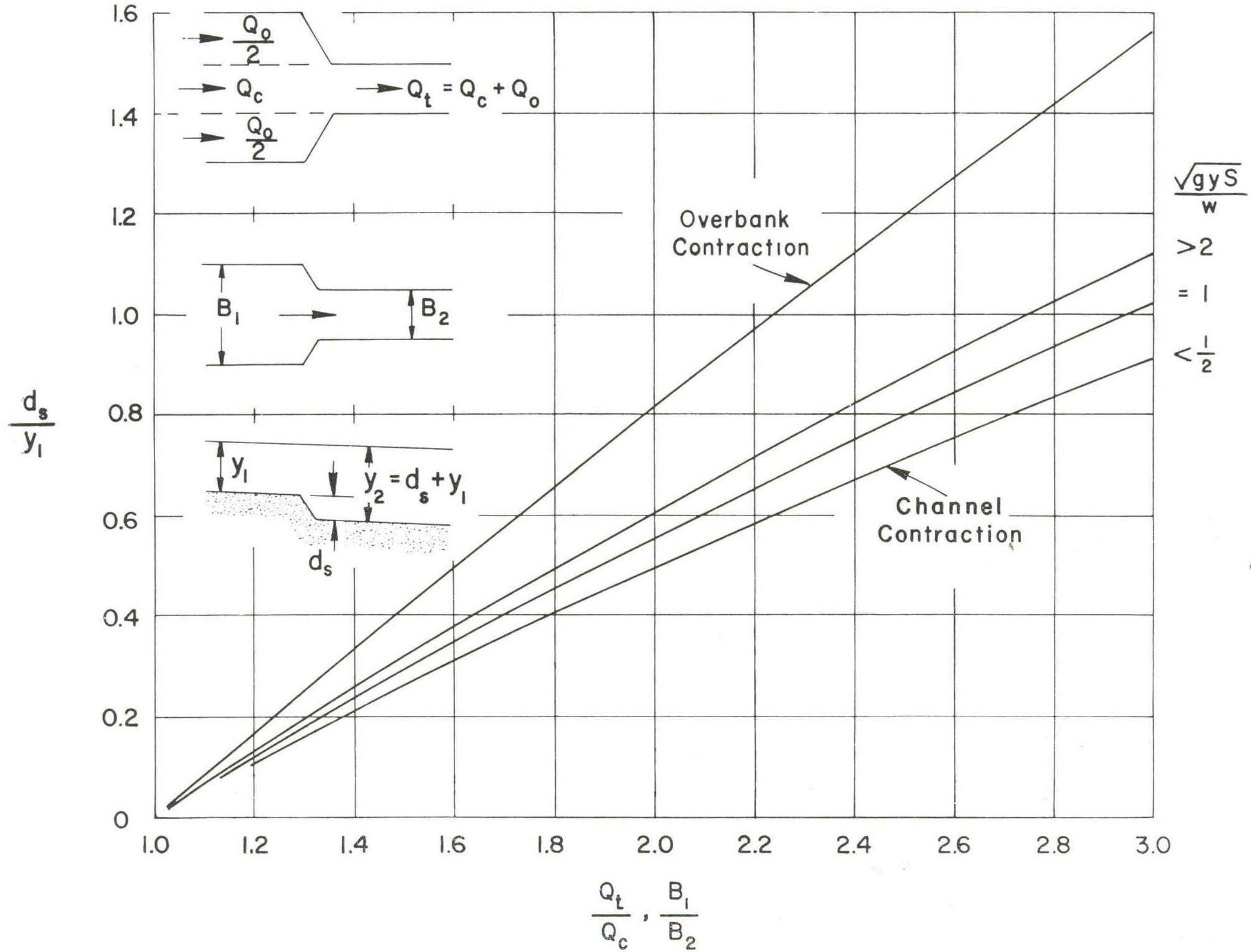
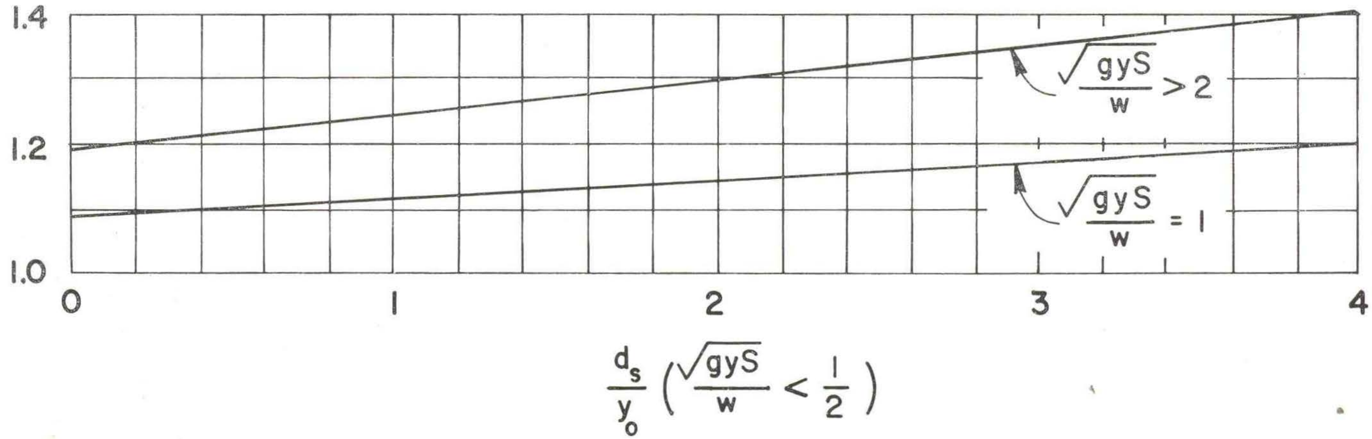


Figure 2. Depth of scour in a long contraction

depth of flow, angle of attack, and, of course, size and shape of pier. Based on the results of the experimental investigation, a method for predicting the local scour at a pier was evolved; this consisted of a basic design curve for rectangular piers at zero angle of attack, a family of curves for the effect of the angle of attack, and a table of coefficients for shape effect—the latter to be used only if the pier is aligned with the flow. In proposing this method of predicting scour at a pier, several points of caution were explicitly stated. The most important warning was that the design method could be recommended only for bed-load movement—hence, if the bed material moved primarily as suspended load, the design values should be used with caution.

Equations (5) can be used to indicate the effect of a change of mode of transport from bed load to suspended load. For the same contraction ratio, the depth of scour can be computed for the three different ranges of shear-velocity/fall-velocity ratio. Taking K_7 as the ratio of the depth of scour under suspended-load conditions to the depth of scour under bed-load conditions, Fig. 3 was prepared. It was readily apparent that the change in mode of movement has a sizable effect on the depth of scour. For the condition of the shear-velocity/fall-velocity ratio is greater than 2, the depth of scour may be more than 40% greater than calculated under the assumption of bed-load movement. To be noted, however, is the fact that if the shear-velocity/fall-velocity ratio is this large, the amount of bed material moving as suspended load will be at least twice that moving as bed-load; moreover, the bed material to be considered is that which is scoured out from around the pier, and should not include the fine material generally termed the wash load. Some experiments conducted with a very fine sediment (mean size = 0.04 mm) confirmed this effect qualitatively. The depth of scour was very difficult to measure; nevertheless, it was not more than 50% greater than for bed-load conditions, although the shear-velocity/fall-velocity ratio was approximately 20, the concentration in percent by weight was between 5% and 10% (i.e., between 50,000 and 100,000 ppm) and the ratio of suspended to bed-load was about 500 to 1.

The three Eqs. (5) could have been written as a single equation with an exponent which was a function of the shear-velocity/fall-velocity ratio, thereby showing explicitly that the depth of scour for a channel contraction was affected by the mode of movement. The fact that a single Eq. (4) was obtained, however, implies that there should be no change in the depth of scour for a purely overbank contraction with a change in the mode of transport from bed-load to suspended load. The depth of scour at an abutment can then be expected to be a matter of the geometry of the bridge crossing and the discharge ratio of the overbank flow and the channel flow. In the laboratory the detailed geometry of a typical bridge crossing cannot be successfully modeled. Indeed, it would not be especially desirable to



Note: The fall velocity, w, is that of the bed material being scoured out — not that of the suspended load.

Figure 3. Effect of shear-velocity/fall-velocity ratio on the depth of scour

attempt to do so—partly because of the extreme variability of those details, and partly because of the difficulty of assessing the effect of each individual one. Only the features that were considered the most important in determining the flow pattern around the abutment were modeled. These were the ratio of the overbank flow to the main channel flow, the width-depth ratio of the waterway opening, the set-back of the abutment, the presence of vegetation along the bank line, and the angle between the approach fill and the direction of flow.

The flow conditions. No matter what method might be used to predict the scour at a bridge pier or abutment, it is necessary that the flow conditions in the vicinity of the pier or abutment be known. In the case of the pier the characteristics of the flow which were found to be important were the depth of flow and the direction of flow. In order to obtain these two very elementary features of the flow, a study of flood probability is required as well as a prognostication of the channel positions, alignments, and cross-sections which could reasonably be expected within the life of the bridge. In the case of the abutment, the question of the rate of flow on the overbank becomes the most important and the most difficult problem that must be solved. It is quite obvious that the amount and character of the vegetal growth on the overbank can influence greatly the amount of flow on the floodplain.

Although this general question of the flow conditions at the bridge site is not a question of scour and has not been studied in this investigation, it cannot be too strongly stressed that the judicious evaluation of the flow conditions is vital to the proper application of the results of this study. Unless such an evaluation is made, the design conditions must be assumed to be the worst possible, with the result that the bridge foundations may be overdesigned.

INVESTIGATIVE EQUIPMENT AND TECHNIQUE

Experimental equipment and operation. The flume of 10-foot width used in the first qualitative studies of scour around bridge piers and abutments was remodeled as shown in Fig. 4 for this phase of the investigation. Seven feet were added to the length to improve the approach to the bridge constriction, giving a total flume length of 51 feet. A sand elevator was installed across half the width at the upper end and a pair of traps across the entire width at the lower end to permit continuous operation with a controlled rate of sediment transport. Two independent water-supply lines were run from the constant-level tank, one to control the simulated river flow and the other the overbank flow. The tailgate spanning the flume at the lower end was split to allow better control of the outflow, and tailgate gages modified from standard point gages were added to permit precise resetting of the gates.

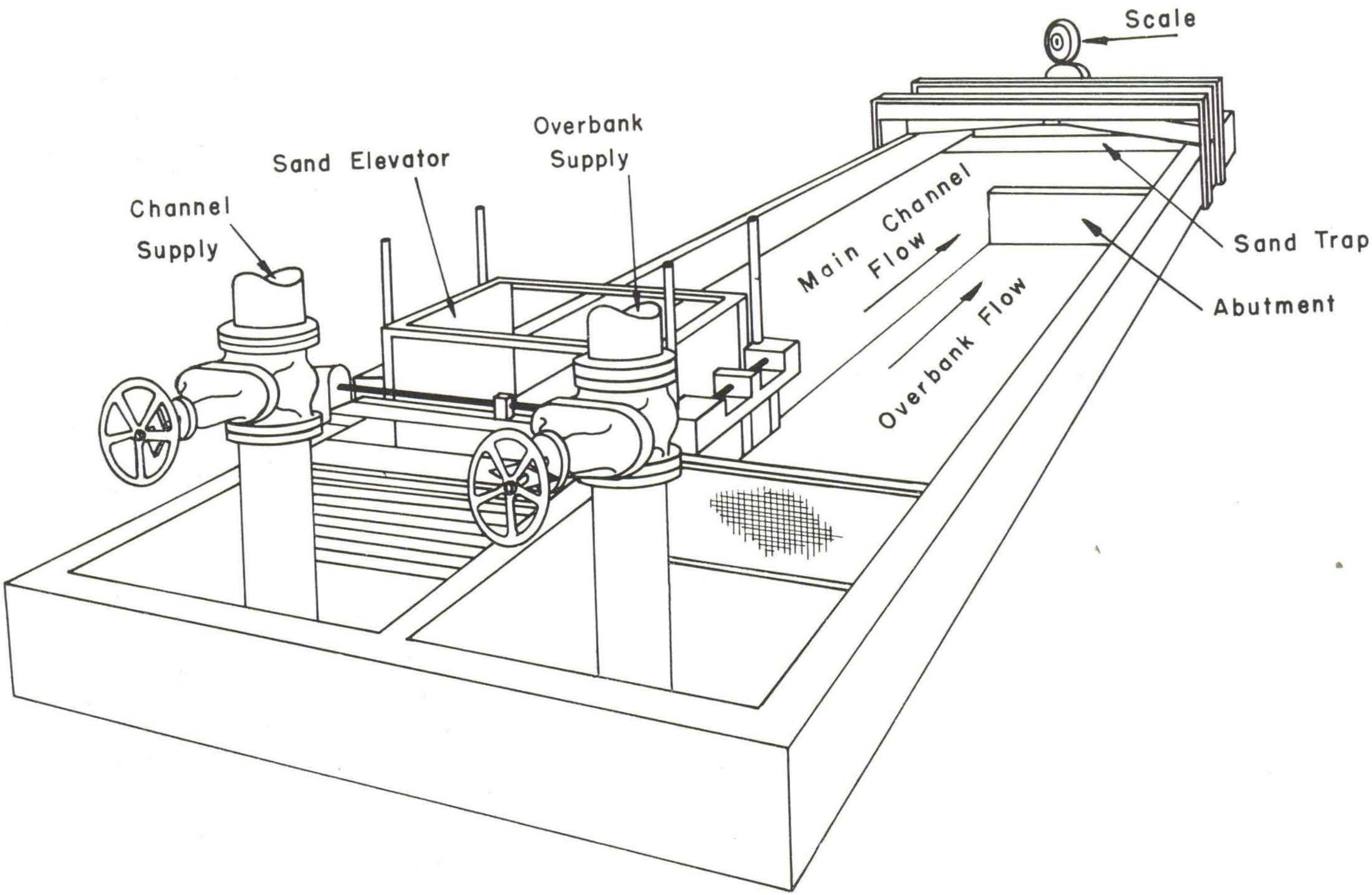


Figure 4. Flume for study of scour at bridge crossings

During preliminary tests, it was found advantageous to operate with a constant rate of sediment supply from the elevator. Even for overbank flow rates greater than the discharge in the river channel, there was no appreciable backwater, and the water surface at the constriction could be kept at a predetermined elevation with varying rates of flow on the overbank by manipulating the tailgates. The rates of sediment transport needed were established in runs in which a temporary brick wall separated the river channel from the overbank. In this manner the water surface and bed elevations were also established for certain velocities and depths of flow which were arbitrarily chosen on the basis of past experience.

The first run of any series was started with a level sand bed in the river channel. Both the depth of scour and rate of transport were checked for equilibrium before the run was considered complete. Equilibrium was considered established if the mean values of sediment transport and depth of scour remained unchanged for a period of four hours or, in some cases, more. In the first runs, both the scour meter described in Bulletin No. 4 and staff readings were used to measure scour depth. The comparatively large depths of scour, which were typical for the abutments, did not fluctuate as rapidly as was the case for the piers. Therefore, when the scour meter was needed for another project (an experimental M.S. thesis on scour at relief bridges) the staff readings were deemed sufficient.

Subsequent runs in a series were usually made with a variation in boundary or flow conditions that resulted in deeper and deeper scour holes. The bed configuration of the previous run was the initial condition of each new run of the series. Occasionally, the boundary condition would result in less scour than a previous run, and in this case the scour hole would fill. Because a scour hole tends to fill less rapidly than it will enlarge, this sequence was avoided whenever possible.

At the completion of a run, measurements were made of the water-surface and sand-bed elevations at selected profiles, and the scour-hole contours were outlined with black thread and recorded photographically.

In the first runs, the overbank area was paved with brick except in the vicinity of the abutment and the approach fill. The top of the bricks was slightly above the sand bed in the river channel. For large rates of flow on the overbank, this resulted in high approach velocities on the overbank; consequently, scour occurred all along the upstream face of the approach fill to depths greater than that at the upstream corner of the abutment. In order to reduce the velocity of approach on the overbank, the bricks were removed from the overbank area except for two rows next to the river channel which had to be left to contain the sand bed. As the depth of flow on the overbank was now greater than in the river channel, the velocity was less. Thus, a more realistic condition was imposed, and the deepest scour occurred at the upstream corner of the abutment. This experience, however, illustrates the difficulty of modeling a bridge crossing.

Simulation of bridge crossings. Included in the detailed geometry of a real bridge crossing are the characteristics of the river channel, of the floodplain, and of the bridge approaches and foundations. The river channel may be deep on one side and shallow on the other, it may be of constant depth, or it may be symmetrical but dish-shaped. The river banks may be high or low, steep and clearly delineated, or so flat that the distinction between the channel and the floodplain is arbitrary. The floodplain may be very wide or quite narrow, deep or shallow, open or covered with dense vegetation, almost flat or so rough that there are, in effect, secondary channels. The side slopes of the approach fill may be steep or gentle and the side ditches large or small. The approach may be straight or curved and may meet the river channel normally or at an angle. Finally, the position and geometry of the abutments and piers can vary greatly.

Even if all these details could be modeled to scale and retain a similarity of flow pattern in model and prototype, the difficulty of assessing the effect of the variation possible in all the different features would make the task impracticable. Fortunately, scour is controlled by the flow conditions in the vicinity of the pier or abutment. The simplified, and even distorted, model that was built for the investigation can best be considered as a schematic bridge crossing. Nevertheless, the results of the experiments indicate that the essential features of the crossing have been modeled successfully.

At the beginning of this phase of the experimental investigation, the one wall of the river-channel portion of the flume was considered to be a plane of symmetry. This interpretation severely limited the width-depth ratio of the waterway that could be simulated, since the width of the river channel portion of the flume was fixed at 5 feet and the depth could not be varied through a wide range. The smallest depth that could be used without introducing sizable errors of measurement was 0.25 foot. Because of the limitation of flow, the largest depth that could be used was 0.5 foot. Both represented rather small width-depth ratios of the waterway opening if the wall was considered to be a plane of symmetry. Later, however, it was realized that the wall could be interpreted as any streamline beyond the lateral extent of the scour hole, since such streamlines were not displaced from their original position in the unobstructed stream. The experimental results therefore simulate all values of the width-depth ratio large enough to prevent mutual interference of the scour holes from abutments on opposite banks. Interference is not likely to occur in the field except in very extreme cases.

Insofar as the flow conditions in the immediate vicinity of the abutment are concerned, the character of the floodplain is important only to the extent that it controls the overbank flow. The rates of flow in the river channel and the overbank, of course, could easily be controlled by valves at the two independent inlets to the experimental flume. The only other characteristic of the river that was simulated was the presence of vegetation at the bank line. This was done by adding perforated sheet metal, commercially available, which had an open area of about 33 percent.

The approach fill and abutment were represented by a square-ended, vertical wall 6 inches wide. The wall was usually placed normal to the flow, but in one series the angle with the flow was made 45, 90, 135, and 180 degrees respectively, the last being a wall parallel to the flow jutting upstream from the normal wall a distance of 24 inches. This condition could also be interpreted as representing a simple spur dike off the abutment. Several other simulated spur dike arrangements were also investigated.

Cutting off the wall resulted in a set-back, but whether the set-back should be considered in its ratio to a linear dimension, say the depth, or as a percentage of the overbank flow area is not clear. For small set-backs, the former seems a more reasonable interpretation; for large set-backs, the latter. However, for large set-backs the flow around the abutment might be expected to be relatively independent of the river flow and the problem to become a special case of the relief bridge. In this case, the scour is due to "clear water" and the findings of this phase of the total investigation are not applicable.

EXPERIMENTAL RESULTS

Flow and scour patterns. The flow pattern at a two-dimensional constriction is well known for its characteristic separation of the bounding streamlines and contraction of the jet just downstream from the constriction. As Kindsvater and Carter (5) have shown, the two-dimensional constriction typifies, at least qualitatively, the flow through a nonerodible, open-channel constriction. The primary difference between the two lies in the fact that the pressure variation in the area of non-uniform flow in the two-dimensional case becomes a variation in surface level in the open-channel case, because of the requirement that the pressure on the water surface be everywhere atmospheric.

A flow pattern similar to that of the two-dimensional constriction with separation and a contracting jet, as well as large variations in the water-surface elevations, could be observed fleetingly at the beginning of those runs in which there was no initial scour hole. As the scour hole quickly developed, this pattern completely disappeared and was replaced by one which was markedly three-dimensional.

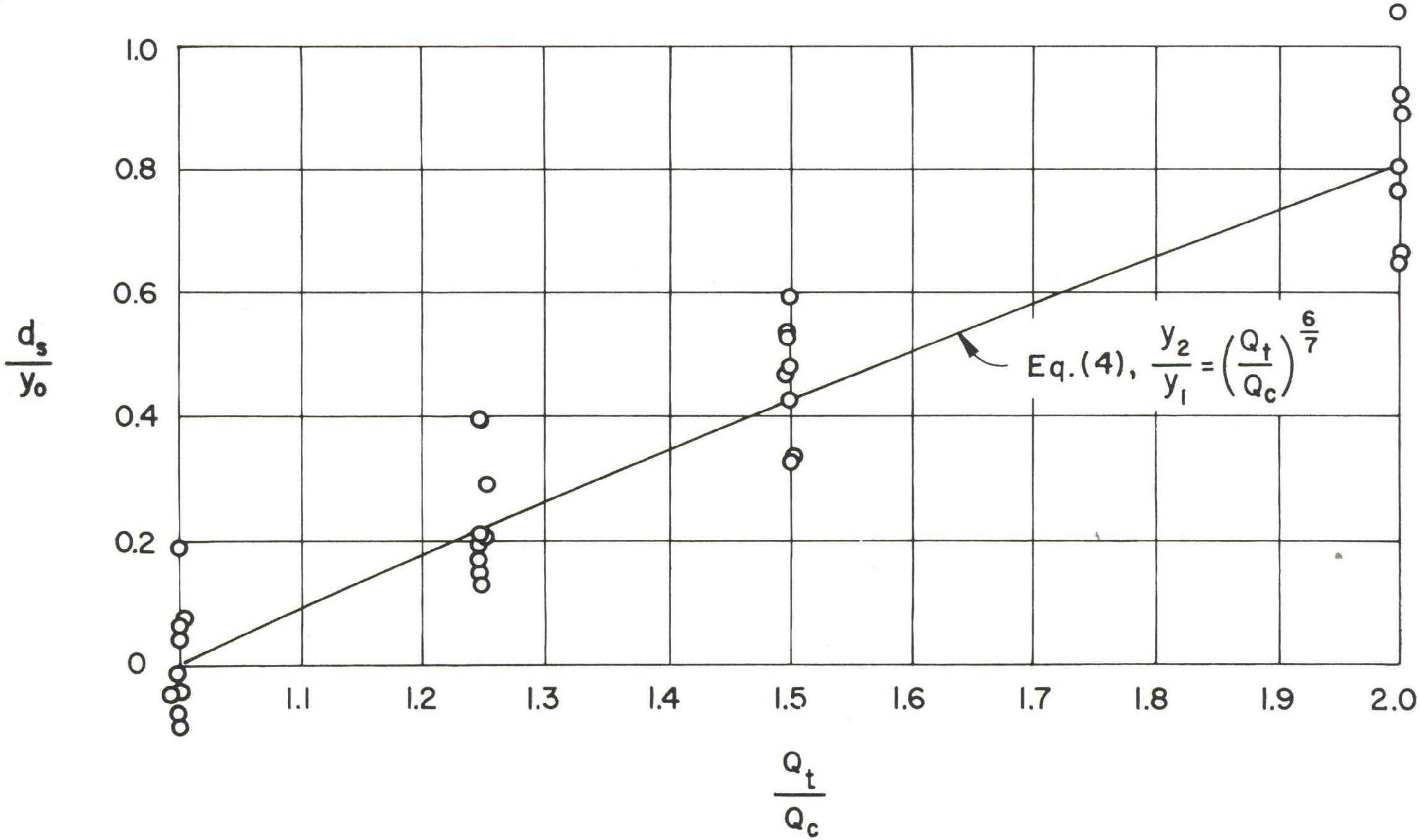


Figure 5. Depth of scour downstream from a bridge constriction

Once the scour hole developed at the abutment, even though it might not be as large as the final equilibrium scour hole, the overbank flow submerged and passed through the scour hole as a spiral roller. The main channel flow was almost completely unaffected by the return to the river channel of even large quantities of overbank flow. Separation did occur at the sharp upstream corner of the abutment, but the contraction of the channel flow was local and minor in its effect on the flow pattern. Some mixing occurred between the through flow on top and the spiral roller beneath, but the mixing was certainly not complete until the scour hole itself had spread completely over the channel downstream. There all the flow was concentrated into what was essentially the river channel, and the flume was not long enough to re-establish conditions comparable to those upstream from the constriction.

The scour downstream from the bridge crossing resulted in a dish-like cross-section. The average depth of scour in this section is well described by Eq. (4) as shown in Fig. 5. The scatter that is apparent is largely the consequence of the position of the dune at the time the measurement was made.

In Fig. 6, the scour holes from a series of runs with a normal crossing and no set-back are shown. Although deeper as the ratio of overbank flow to channel flow increases, the form of the scour hole does not change.

General relationship for scour at abutments. One of the most striking characteristics of the flow in the constricted section was that the flow in the center of the channel—i.e., away from the abutment and the scour hole around it—did not appear to be affected in the least by the constriction. In the previous study of scour around multiple cylinders, it had also been noticed that the scour was not dependent on the contraction ratio until adjacent scour holes overlapped one another. This limiting condition, where the degree of contraction becomes important, suggests a possible extension of the long-contraction analysis to the abutment problem.

Figure 7 shows a special case of the long contraction, which will be assumed to approximate the conditions at an abutment located at the bank of the low water channel. The flow on one overbank area, which must go around this abutment, is taken as Q_o, and the width of the river channel measured from the bank line as 2.75 d_s. Note that d_s is defined as the depth of scour at the abutment. (The width 2.75 d_s is an average value taken from the experiments of the width of the scour hole on a line extending the upstream face of the abutments.) The flow in the river channel farther than 2.75 d_s from the bank is completely disregarded as if there was a thin wall erected parallel to the bank. The average depth of flow over this width of the river channel is designated as y_0 and the discharge as Q_c. The depth of scour in the long contraction is assumed to be a fraction $1/r$ of the

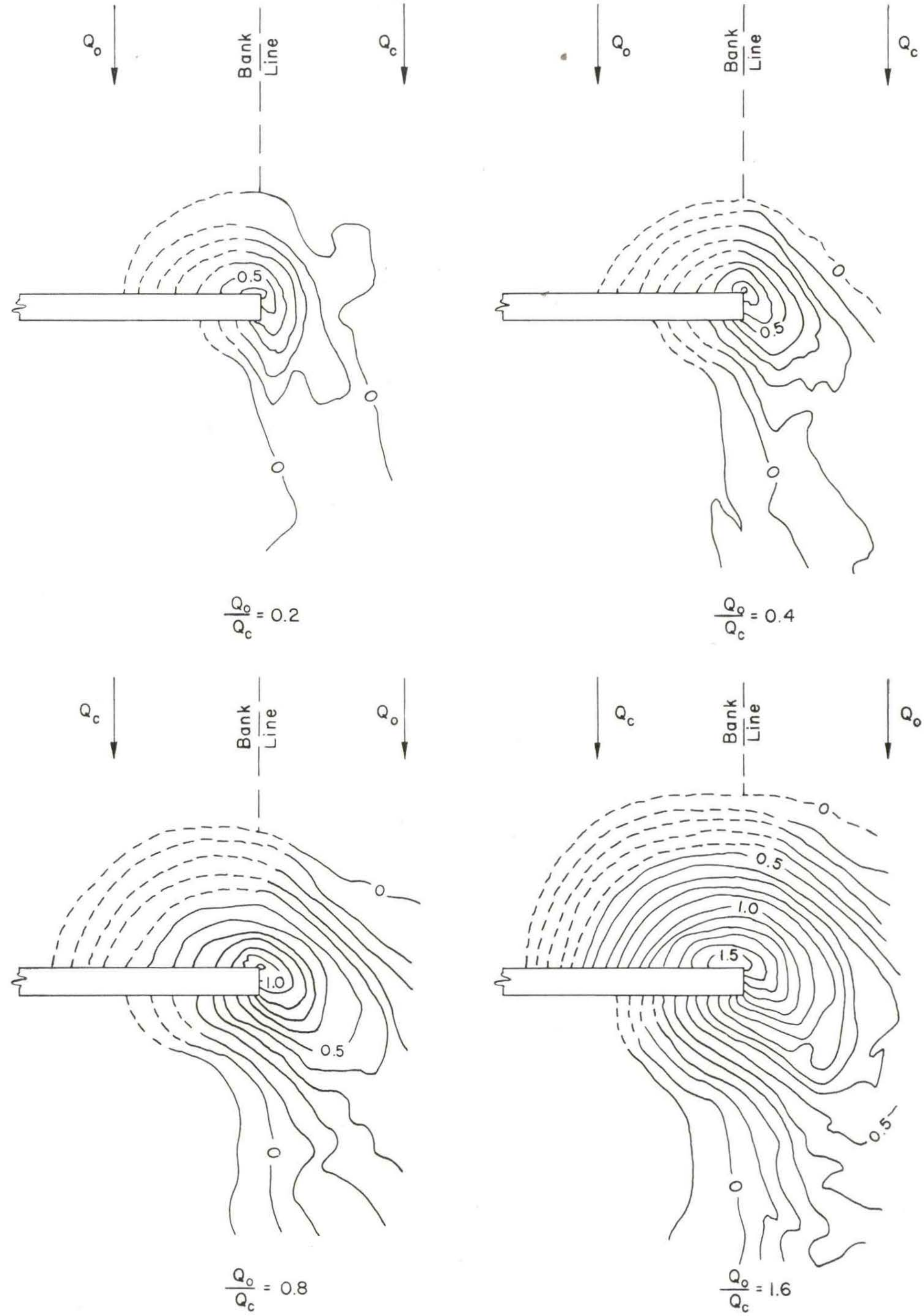


Figure 6. Typical scour patterns

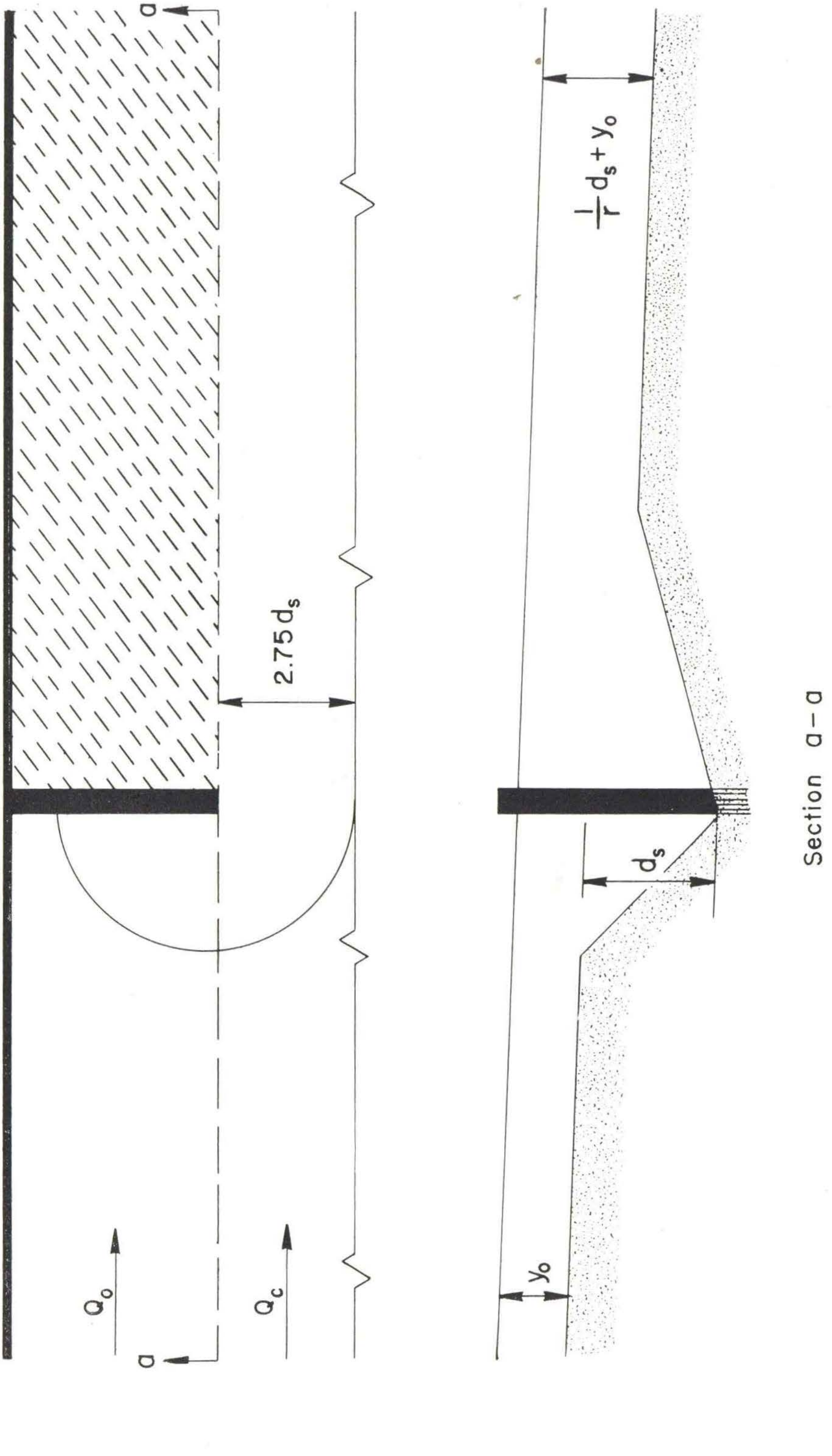


Figure 7. Definition sketch of an overbank bridge constriction

depth of scour at the abutment so the average depth of flow in the long contraction is $d_s/r + y_0$. Equation (4) rewritten for this special case would be

$$\frac{1}{r}\frac{d_s}{y_0} + 1 = \left(\frac{Q_c + Q_o}{Q_c}\right)^{6/7}$$

Because the depth of scour d_s is not known, the flow Q_c over the width 2.75 d_s cannot be evaluated. A trial-and-error solution is therefore necessary. If an arbitrary width b is chosen, the flow Q_b over this width can be evaluated. After successive trials b will be almost equal to 2.75 d_s and, therefore, the average velocity and the average depth of flow over the two widths can be assumed to be the same so that

$$\frac{Q_b}{b\, y_0} = \frac{Q_c}{2.75\, d_s\, y_0} \quad \text{or} \quad Q_c = \frac{2.75\, d_s}{b} Q_b$$

Substitution of this expression for Q_c in the special case of Eq. (4) above, after some algebraic manipulation, will result in

$$\frac{Q_o}{Q_c}\frac{b}{y_0} = 2.75\frac{d_s}{y_0}\left[\left(\frac{1}{r}\frac{d_s}{y_0} + 1\right)^{7/6} - 1\right] \qquad (6)$$

Equation (6) is plotted in Fig. 8 for various values of the parameter r, the ratio of the depth of scour at the abutment to the depth of scour in the fictitious long contraction, together with the data from the last—and best—set of runs. The dashed curves are for values of r of 2, 3, and 4. A value of r of 2 is probably the minimum to be expected with a triangular cross-section of scour at the abutment. The solid curve for a value of r of 4.1 passes through the points for high values of overbank contraction remarkably well. The discrepancy for low values of overbank contraction is due to the fact that Q_o has been taken as the discharge measured in the pipeline serving the overbank. The actual Q_o at the abutment is larger because of lateral cross flow from the channel to the overbank.

This cross flow could be observed by injecting dye near the bank in the approach to the bridge crossing. An approximate calculation based on the difference in elevation of the water surface in the channel and in the overbank area, and the cross-sectional area through which the cross flow could take place, moved the points for low overbank contraction over to the curve. The points have not been replotted, however, to emphasize the difficulty of estimating the overbank flow for this condition. In estimating the overbank flow in the field, the procedure would be based on a determination of the stage, slope, cross-sectional area, and an n value. Locally, however, the variation in resistance to flow and the resulting difference in

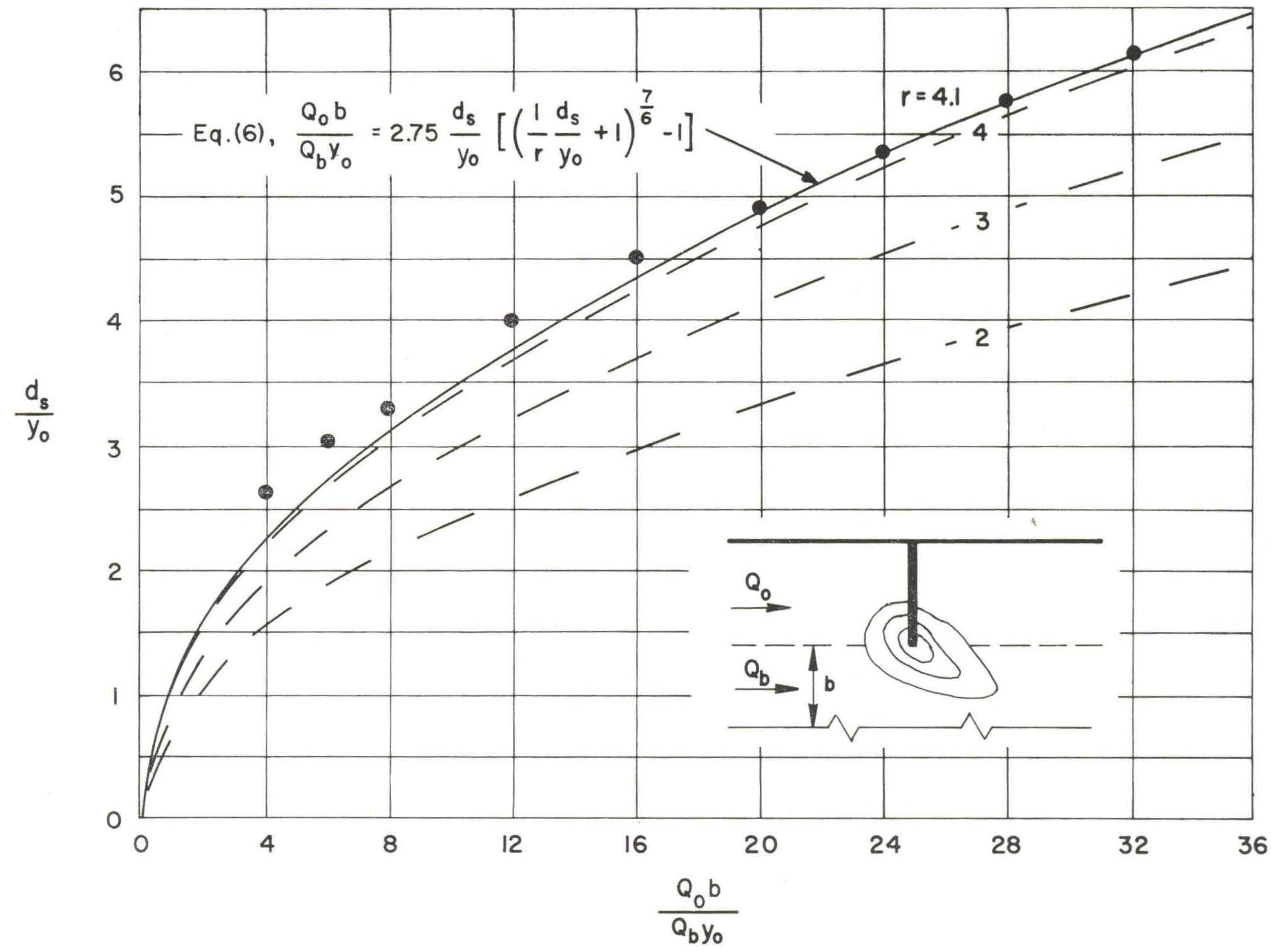


Figure 8. Depth of scour at an overbank bridge constriction

water-surface elevations will permit cross flow to and from the channel. For otherwise low rates of discharge on the floodplain, this can be an important consideration.

One limit in the applicability of the relationship shown in Fig. 8 is reached when there is interference between the scour holes extending out from the two opposite abutments. This is not likely to occur except in extreme examples of deep, narrow channels with a very large proportion of the total discharge on the floodplain. The minimum scour which can then be expected is given by Eq. (4). Interference can cause an increase in scour even before this limit is reached, however. If the two scour holes overlap, a rule-of-thumb correction can be made by increasing the estimated depth of scour to give a cross-sectional area of scour along the line of the upstream face of the abutments equal to the area of the two scour holes without interference. The slope of the scour hole can be taken as 2.75 horizontal to 1 vertical for most materials. If the natural angle of repose of the bed material is less than for normal sands a flatter slope would be indicated—approximately 2/3 of the natural angle of repose.

Another inherent limitation to the applicability of Eq. (6) is when Q_o Q_b, a condition which would result in large values of the parameter $Q_o b/Q_b y_o$. In the development of Eq. (3) and, therefore, also of Eq. (6), an approximate form of the writer's total-sediment-load relationship was used. The full form of the relationship for unigrannular material is

$$\bar{c} = \left(\frac{d}{y_0}\right)^{7/6}\left(\frac{\tau_0'}{\tau_c} - 1\right) f\left(\frac{\sqrt{\tau_0/\rho}}{w}\right)$$

where τ_0 is the total shear on the bed of the stream, τ_0' is that portion of the total shear associated with the sediment particles, and τ_c is the critical tractive force of the particles of size d and fall velocity w. The required function of the shear-velocity/fall-velocity ratio is given to a reasonable approximation by the exponential form $k(\sqrt{gyS}/w)^a$ with the values and limits as stated previously. Assuming $\tau_c = 4d$, the proposed evaluaton is τ_0'/τ_c is

$$\frac{\tau_0'}{\tau_c} = \frac{V^2}{120\, y^{1/3} d^{2/3}}$$

In a river in full flood, the condition of interest, τ_0' is considerably larger than τ_c so that the term –1 was dropped in the interest of a simple expression for the depth of scour. If Q_o is much larger than Q_b, however, this approximation is no longer justifiable. In the contracted reach (and in the scour hole) the mean concentration c will be very small since the sediment load Q_s is the same as in the uncontracted reach, but the discharge has increased from Q_b to $Q_o + Q_b$. If the term –1 is retained on both sides of Eq. (2), the resulting expression is extremely unwieldy; if, however, the -1 term is dropped for the uncontracted reach (the left hand side) where the concentration is large and τ_0' is considerably greater than τ_c, and retained in the contracted reach where the concentration is small and τ_0' is only slightly greater than τ_0, the expression can be reduced as before with the inclusion of one additional factor. Equation (6) then becomes

$$\frac{Q_o b}{Q_b y_0} = 2.75 \frac{d_s}{y_0} \left[\frac{\left(\frac{1}{r}\frac{d_s}{y_0} + 1\right)^{7/6}}{\left(1 - \frac{1}{t}\right)^{\frac{1}{3+a}}} - 1 \right] \qquad (6a)$$

where $t = \tau_0'/\tau_c = \dfrac{(Q_o + Q_b)^2}{120\, b^2 y^{7/3} d^{2/3}}$

Note that as t becomes very large Eq. (6a) approaches Eq. (6). For values of $r = 4.1$ and $a = 1/4$, Eq. (6a) is plotted in Fig. 9 for various values of the parameter t.

The case of a simple contraction of the stream where the approach fill and abutments actually encroach upon the channel can also be handled by an approximate analysis similar to the one leading to Eq. (6). In the first of Eqs. (5) if the length of the obstruction is designated as ℓ, the contracted width B_2 becomes $2.75\ d_s$, and the uncontracted width $\ell + 2.75\ d_s$. As in the analysis of the overbank case, d_s is the depth of scour at the abutment and d_s/r is the depth of scour in the ficticious long contraction. For shear-velocity/fall-velocity ratios less than ½—i.e., bed-load movement—the resulting equation is:

$$\frac{\ell}{y_0} = 2.75 \frac{d_s}{y_0} \left[\left(\frac{1}{r}\frac{d_s}{y_0} + 1\right)^{\frac{1}{0.59}} - 1 \right] \qquad (7)$$

For other values of the shear-velocity/fall-velocity ratio the only change is in the exponent. Equation (7) is plotted in Fig. 10 for values of r of 10 and 12, together with some experimental data. The open circles are data from the study of scour at multiple cylinders for which there was no interference effect. The open squares were obtained by R. C. Stiefel in his investigation of the relief-bridge problem, and are for a vertical, normal wall.

Effect of abutment positioning. At the inception of this phase of the general investigation of scour around bridge piers and abutments, it was felt that at least three aspects of the position of the abutment in relation

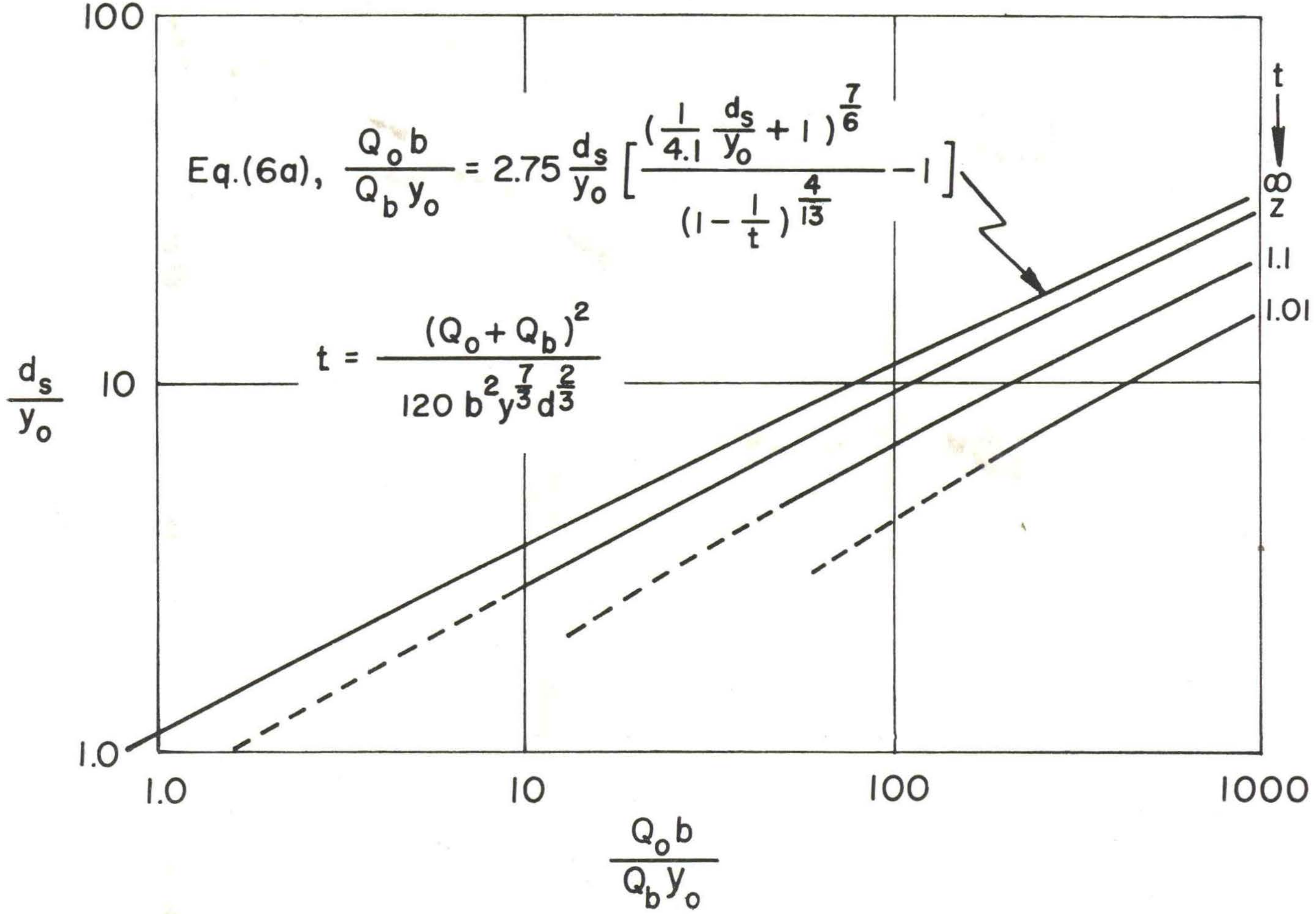


Figure 9. Depth of scour for large overbank flows

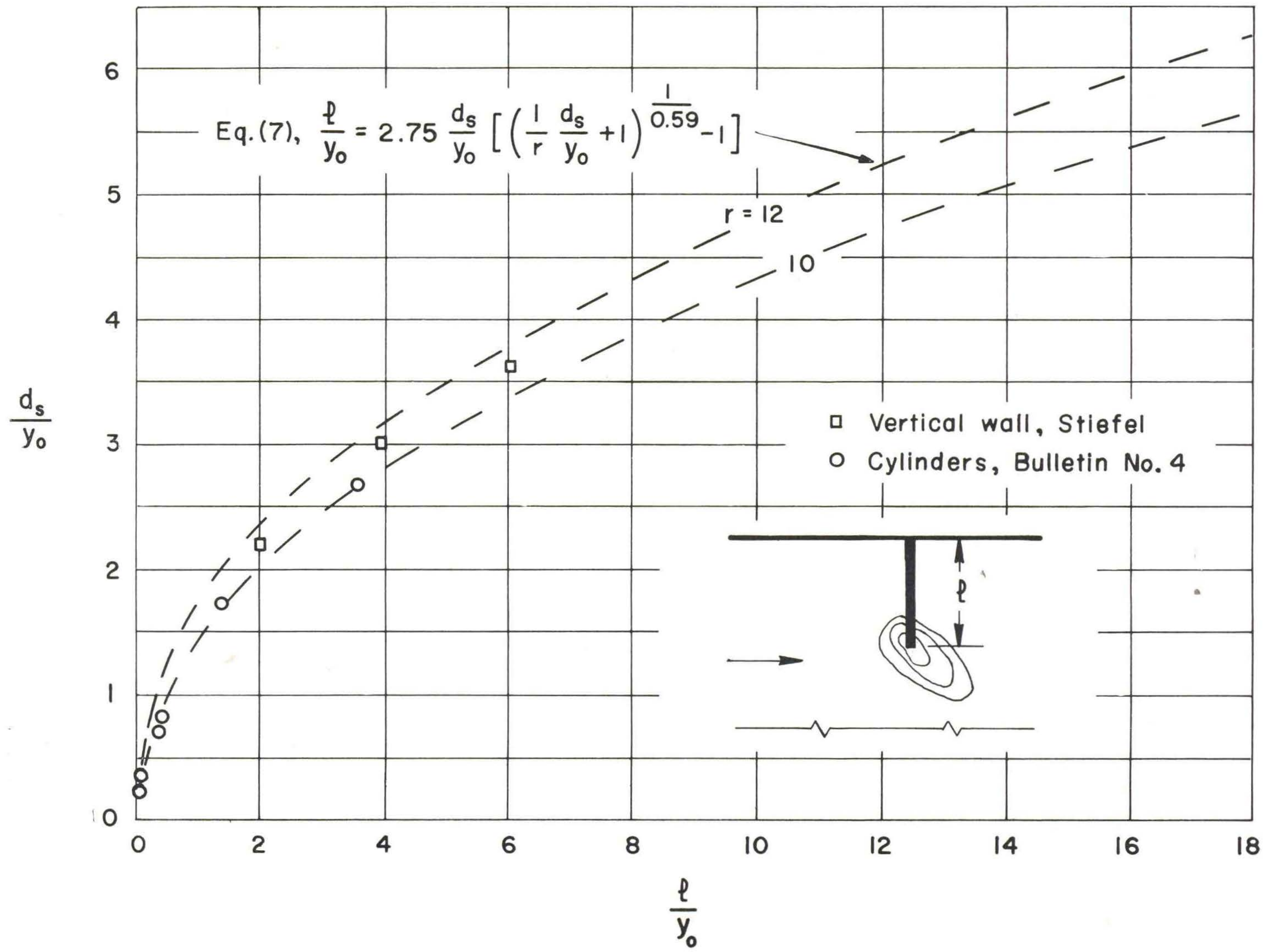


Figure 10. Depth of scour for an encroaching abutment

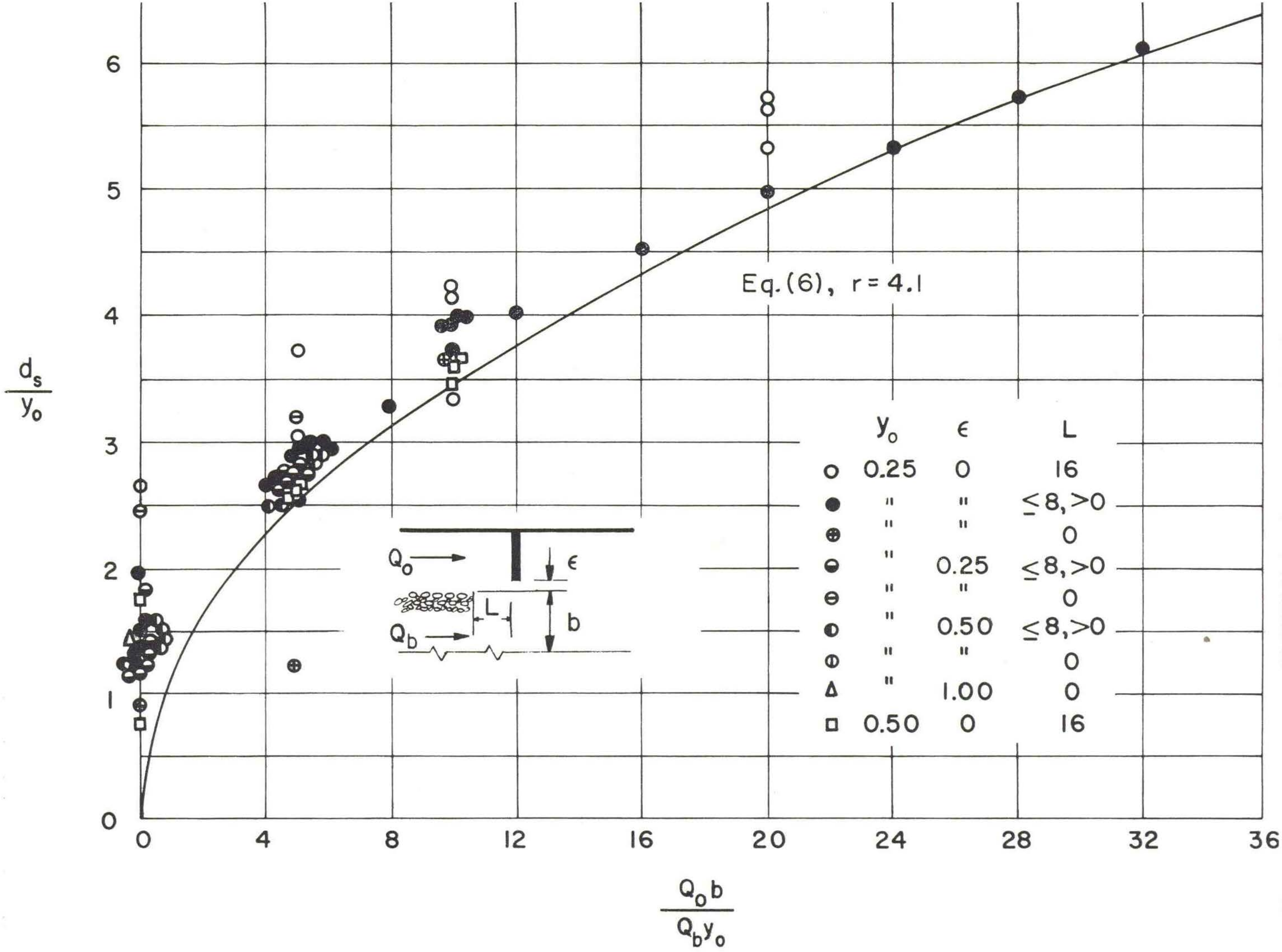


Figure 11. Effect of set-back and clearance on depth of scour

to its surroundings might have an important effect on the depth of scour at the abutment—the distance to the vegetal screen along the river bank, the set-back of the abutment from the bank, and the angle between the approach fill and the direction of the stream. These aspects of the general geometry of the bridge site all proved to be of secondary importance. All of the data obtained for various set-backs ϵ, and various clear distances between the abutment and the vegetal screen L are presented in Fig. 11, together with a curve representing Eq. (6) with r equal to 4.1.

Except for extreme conditions, the clear distance L did not have a measureable influence on the depth of scour. When the screen simulating the vegetation was entirely removed from the flume, the scour hole increased. The cause-and-effect relationship was not direct, however, but linked to the cross flow from channel to overbank. Because of the increase in the area through which cross flow from the channel to the overbank could take place, the rate of flow on the overbank increased with a resultant increase in the scour at the abutment. This effect, however, is more properly to be considered as a part of the problem of determing the overbank flow.

If the screen came down to the abutment so that there was no open area between the screen and the abutment, an alleviation of the scour at the abutment occurred. In this case the screen acted as a long, permeable dike along the river bank. The flow from the overbank to the channel was not concentrated at the abutment, but spread over a considerable length. As a consequence, the depth of scour was reduced. If, however, the screen was undercut, or if there was a small open area between the abutment and screen, the screen was ineffective.

The lack of effect of set-back was surprising, and, because the riverbank conditions could not be modeled, may be somewhat misleading. There are, however, two opposing tendencies which presumably counteract each other. Assuming for the sake of argument that the scour hole stays the same size and moves back away from the channel as the abutment is set back, there would be less sediment supplied to the scour hole. The scour hole would, therefore, tend to deepen and increase in size as a consequence of the decreased sediment supply. On the other hand, if the abutment is set back on the floodplain, some of the overbank discharge passes through the constriction and back to the floodplain as in a relief bridge. In fact, if the set-back is very large, the problem is that of a relief bridge and the presence of the river nearby is immaterial. Depending on the velocity and sediment size, the scour hole might then be smaller or larger than for a true river-channel abutment.

A simplified model to estimate the effect of set-back is shown in Fig. 12, and the analysis proceeds along the same lines as before; however, it is

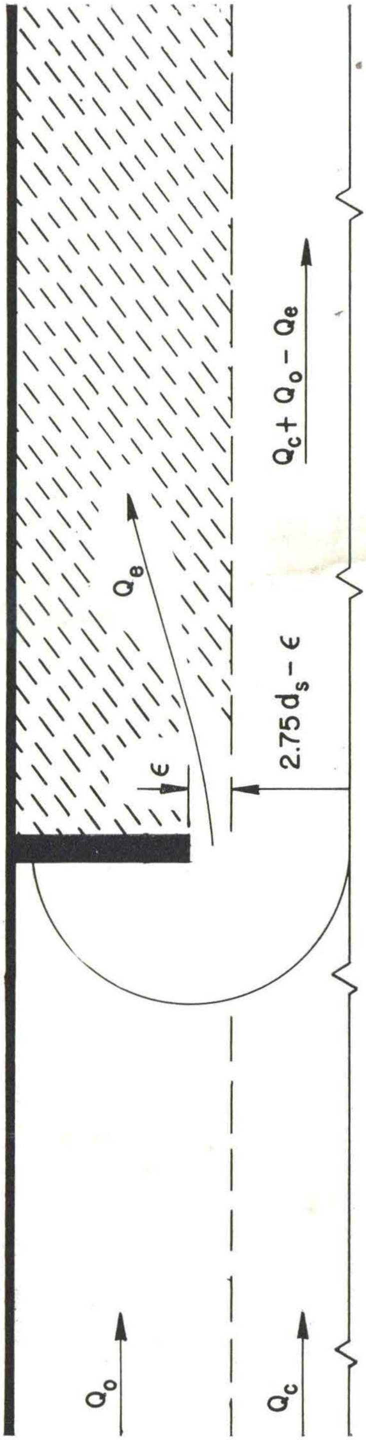


Figure 12. Definition sketch for the effect of a set-back

assumed that a portion Qe of the overbank flow does not contribute to the scour, and that this is equal to.

$$\frac{Q_b}{b}\epsilon\frac{d_s}{2y_0}.$$

The equation which results is:

$$\frac{Q_0}{Q_b}\frac{b}{y_0} = \left(2.75\frac{d_s}{y_0} - \frac{\epsilon}{y_0}\right)\left[\left(\frac{1}{r}\frac{d_s}{y_0} + 1\right)^{\frac{7}{6}} - \frac{2.75\frac{d_s}{y_0} - \frac{\epsilon}{y_0} - \frac{\epsilon d_s}{2y_0^2}}{2.75\frac{d_s}{y_0} - \frac{\epsilon}{y_0}}\right] \tag{8}$$

Equation (8) for r equal to 4.1 is plotted in Fig. 13 for several values of the set-back ratio ϵ/y_0. Although both the schematic analysis and the experimental measurements indicate that set-back is not effective in reducing the depth of scour at an abutment, this finding may be overly pessimistic in relation to conditions as they are encountered in the field. In the laboratory, and in the analysis, the depth of flow did not decrease when the abutment was set back from the bank. In the field, the depth of flow would decrease; presumably, therefore, so would the depth of scour. Moreover, bank materials usually contain fines and clays, and as a consequence they may be highly resistant to scour. A good turf and well-rooted brush may also inhibit scour—although whether or not the designer can rely on such protection is another question.

Evidence that the shape of scour holes did not materially change with the angle between the approach fill and the direction of the flow is shown in Fig. 14. The effect of angle of incidence θ on the depth of scour is given by the curve in Fig. 15. As might be expected, reentrant angles increase the depth of scour; however, the effect on the depth of scour is much less than it is on the coefficient of contraction for a comparable two-dimensional flow.

Alleviation of scour at abutments. Three different methods of reducing the scour at abutments appear expeditious. The first depends on increasing the erosion resistivity of the bottom of the scour hole with loose rock rip-rap or with solid or flexible matting. Although experiments on solid scour arrestors have not been carried out for the abutments, the conclusions based on D. E. Schneible's study (6) with piers should be equally applicable to abutments. As discussed in Bulletin No. 4, the necessary solid lateral extension of the foundation depends on its placement below the stream bed and the depth of the uninhibited scour hole. As a rule of thumb, the lateral extent of the scour arrestor should be such that the cross-sectional area normal to the stream is the same with and without the scour arrestor; i.e., if the scour is to be arrested at a depth y_a, the lateral dimension x_a must be such that $1.375\ d_s^2 = x_a\ y_a + 1.375\ y_a^2$.

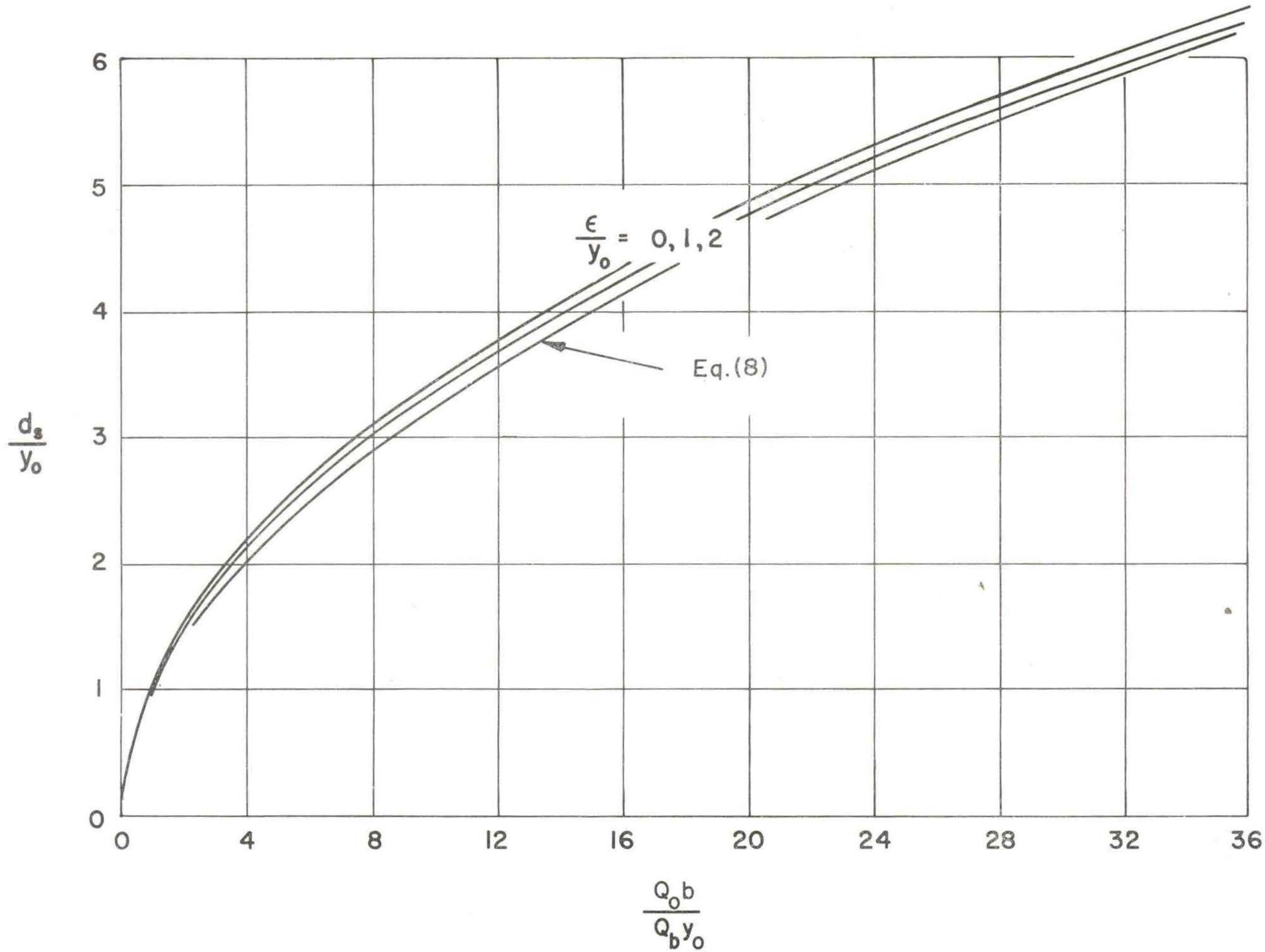


Figure 13. Approximate analysis for the effect of a set-back

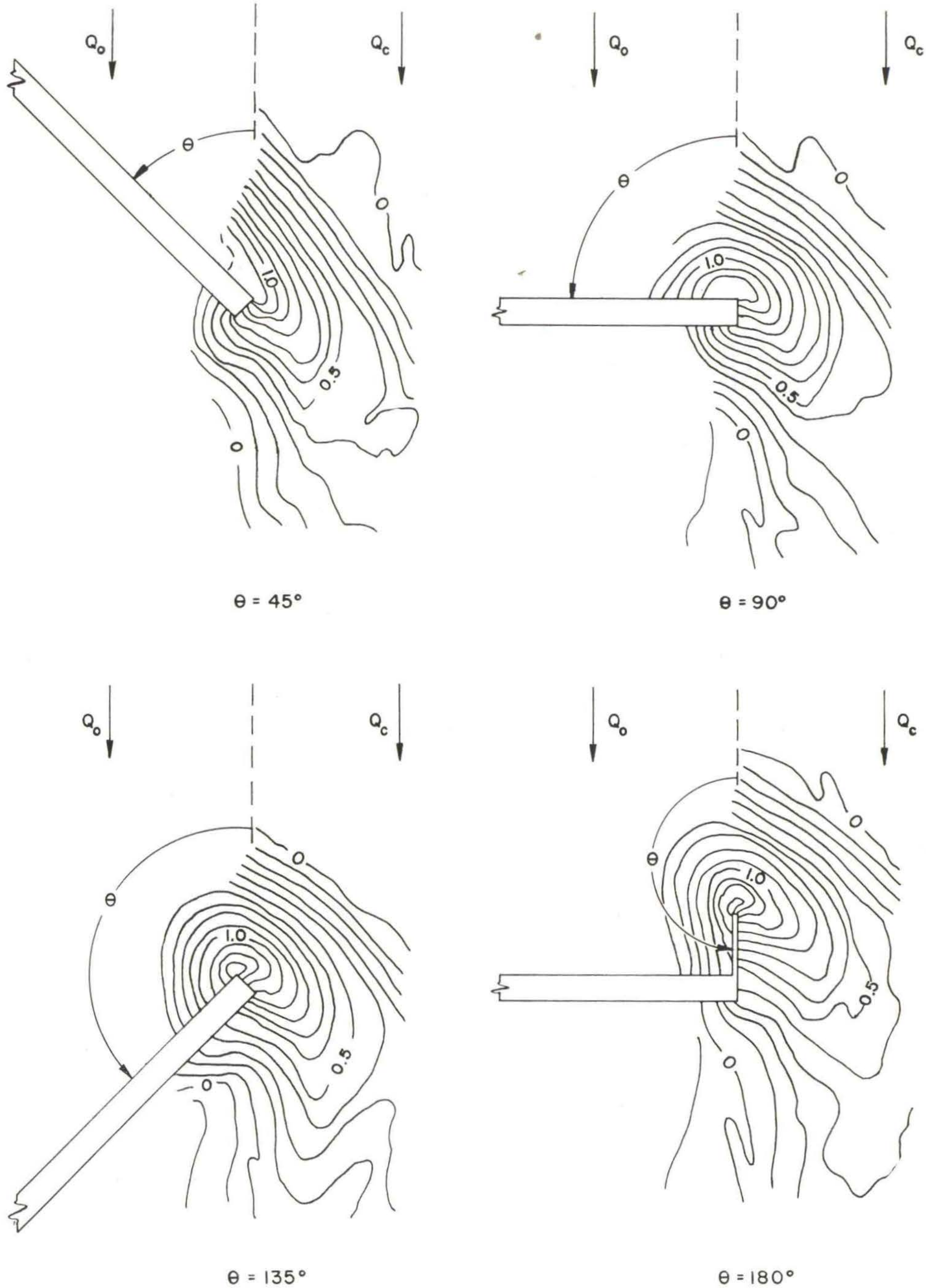


Figure 14. Patterns of scour for four angles of incidence

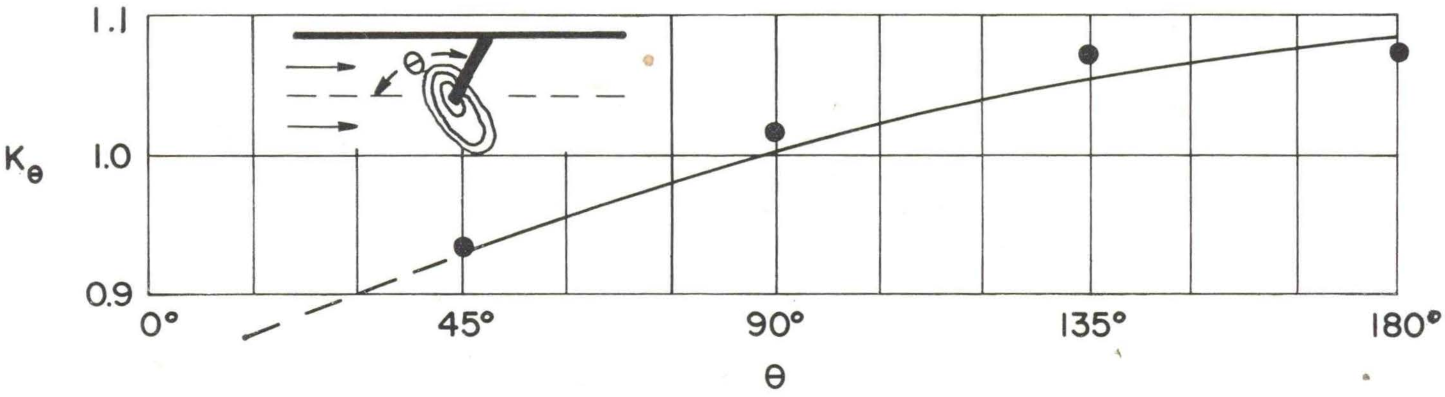


Figure 15. Effect of the angle of incidence on the depth of scour

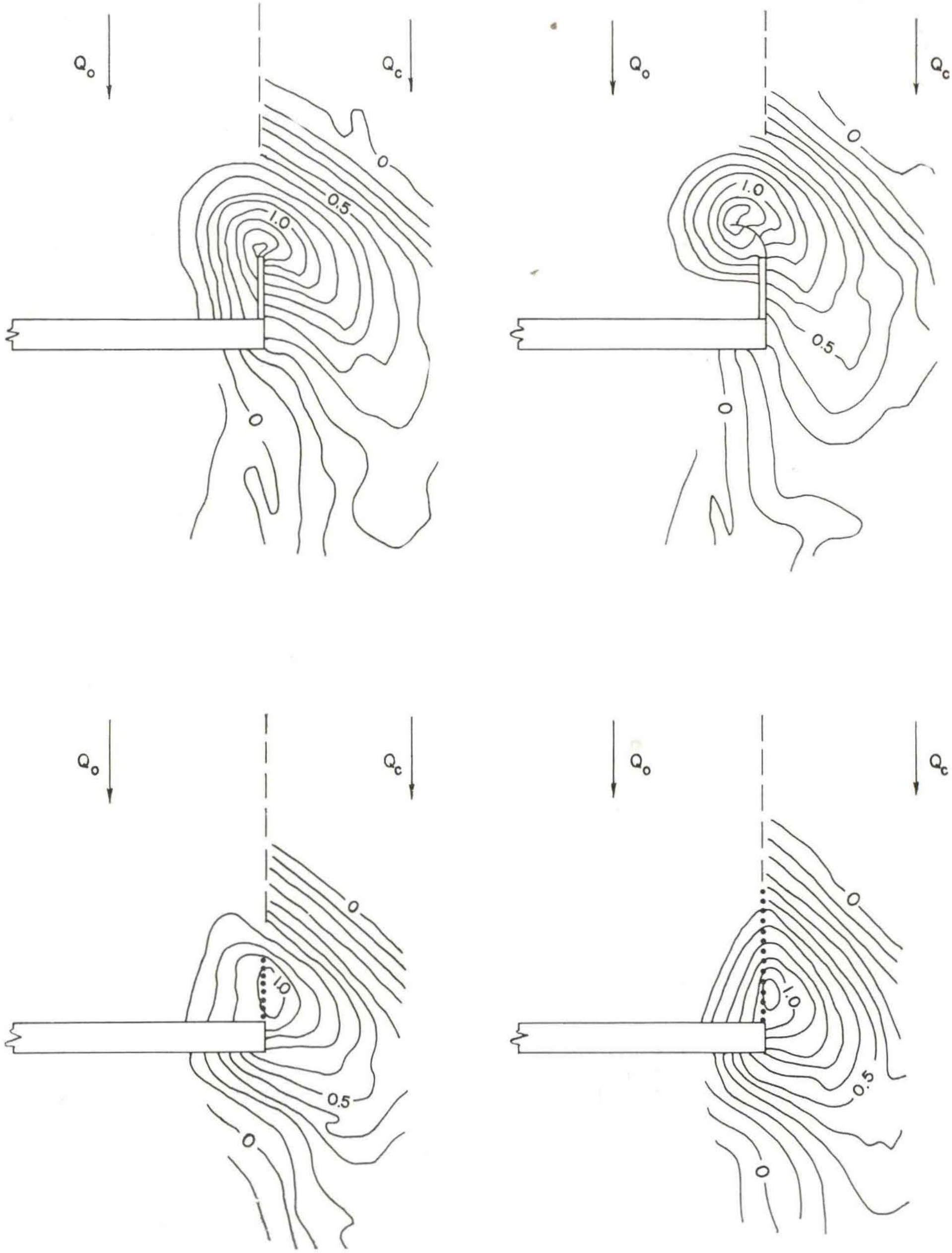


Figure 16. Patterns of scour with spur dikes

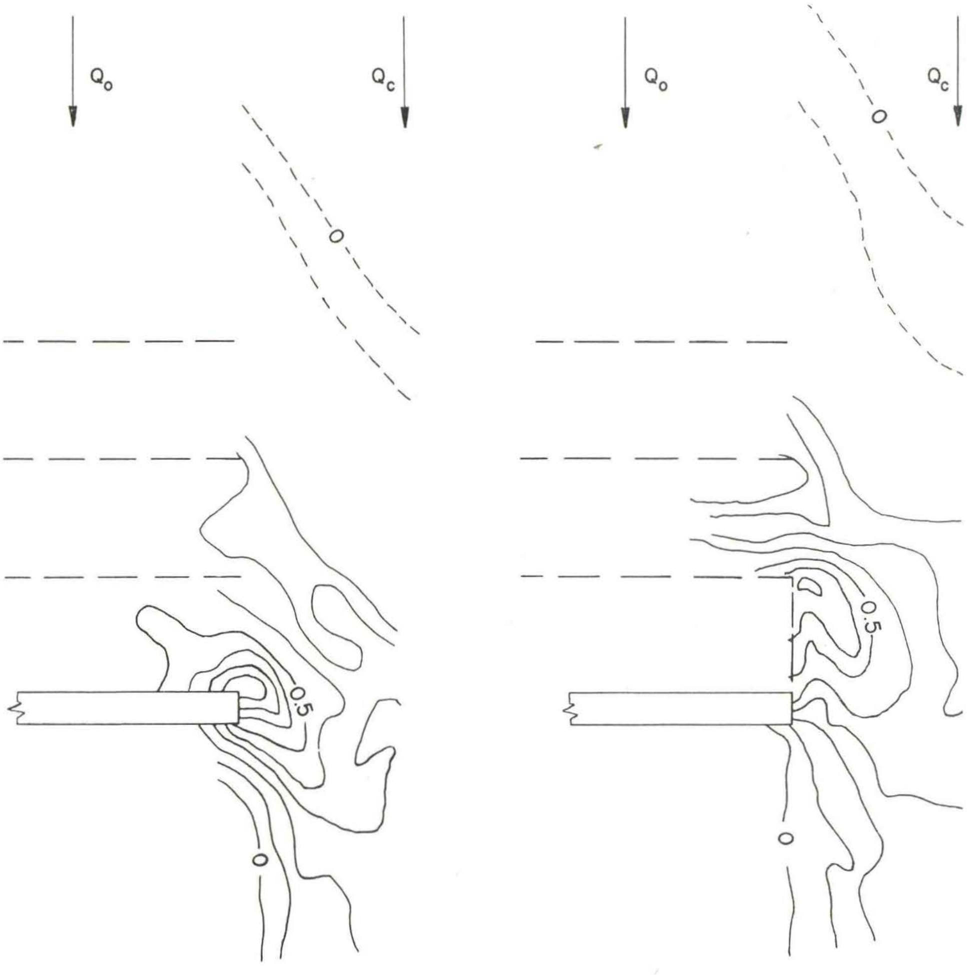


Figure 17. Patterns of scour with floodplain screening

The second method of alleviating the scour at the abutment consists of moving the scour hole upstream by means of a spur dike, solid or permeable, off the end of the approach fill and essentially parallel to the stream. The length of the necessary dike is dependent on the depth and shape of the scour hole and can be estimated from the typical shape of the scour holes in Figs. 6 and 16; the longer the dike, the farther the scour hole is shifted upstream and the less is the scour at the abutment. If a solid dike is three or more times the depth of scour, the depth of scour at the abutment itself should be nominal; i.e., approximately that to be expected in a comparable long contraction. The practicability of this method depends on the cost of the dike, which must be stable in itself, compared to the cost of deeper foundations for the abutment.

The third method depends on management of the floodplain upstream from the bridge so as to increase the resistance to flow on the overbank. If the effective n value of the overbank can be increased through plantings, such as multiflora roses and willows in rows normal to the flow, the flood waters will return to the main channel. Permeable dikes could also be used for this purpose. Because such barriers would not be impenetrable to the flow, the return to the channel would be over a considerable reach and the depth of scour would be minimized. In fact, if the barriers functioned properly, the maximum scour should not be much more than that in a long overbank contraction. In Fig. 17, the effect of two such screening systems is shown for a value of $Q_o b/Q_b y_o = 20$. The screening system on the left reduced the scour at the abutment to about 80% of what would otherwise have occurred. This reduction implies that 50% of the overbank flow was forced to return to the channel by the screens. The screening system at the right was even more effective with the maximum depth of scour reduced to 50% of what would have occurred without screens and the depth of scour at the abutment reduced to 25%. The minimum depth of scour that could have been expected is that of the long contraction and the depth of scour at the abutment is only about 60% greater than this absolute minimum.

If the depth of scour at the abutment is reduced by one means or another, it should be noted that the lateral extent of the scour hole in the area of interest is increased. This may add to the scour depth at the piers, and the economic justification must then give weight to the added cost of providing safe foundations for the piers.

APPLICATION TO DESIGN

Design relationships. Means are now at hand whereby the scour at bridge piers and abutments can be predicted with reasonable confidence for cases in which sediment is supplied to the scour hole—the relief bridge problem, and, therefore, abutments with large set-back are not included. Not all secondary effects have been investigated, but their influence should be small enough so that they can be safely ignored. The principal difficulty that remains is the forecasting of the flow conditions which will occur during the life of the bridge.

No one, of course, can foretell the future. Insofar as the discharge at the bridge is concerned, it is possible only to estimate the likelihood that a certain discharge will occur during a certain time period. (As outlined in Bulletin No. 4, one minus the probability of non-occurrence is the measure of likelihood desired.) The stage and the division of flow between river channel and overbank must also be predicted. It is because none of these predictions can be made with complete confidence that secondary effects upon the depth of scour lose their importance.

The added cost of constructing the piers and abutments for a discharge that represents a considerable factor of safety should be considered as an insurance premium against the possible loss of the bridge due to scour. Each bridge will be a case unto itself, of course, since the cost of the bridge, the cost of deeper foundations, the discharge-frequency relations, and the flow characteristics will be different for every site.

Figure 18 shows the basic design relationship for the scour at an abutment due to overbank contraction in which the approach fill is normal to the direction of flow. The variables which must be evaluated are Q_o the rate of flow on the floodplain, Q_b the channel discharge over a width b measured from the bank (b should be greater than 2.75 d_s), and the average depth y_0 over the width b. A trial-and-error solution may be necessary, since b must first be estimated and may require revision after d_s is calculated.

The vegetal screen along the bank line did not appear to have any effect on the depth of scour at the abutment except in the case of low rates of flow on the overbank aproaching the approach fill. The effect of the screen rests in its control over the cross flow from the river channel to the floodplain.

For large set-backs, i.e., greater than twice the depth of scour—the results of this study are not applicable. The river adjacent to the flow around the abutment is then incidental, and the conditions are those of a relief bridge in which the depth of scour is a function of the velocity of flow and the sediment size. For small set-backs—i.e., less than the depth of scour—the experimental evidence from this study indicates that the set-back has very little influence on the depth of scour. Although it seems likely that the decreased depth of flow might result in a decrease in the depth of scour, no such recommendation can be made at this time. After a successful conclusion of the relief-bridge investigation, the case of set-back should be restudied.

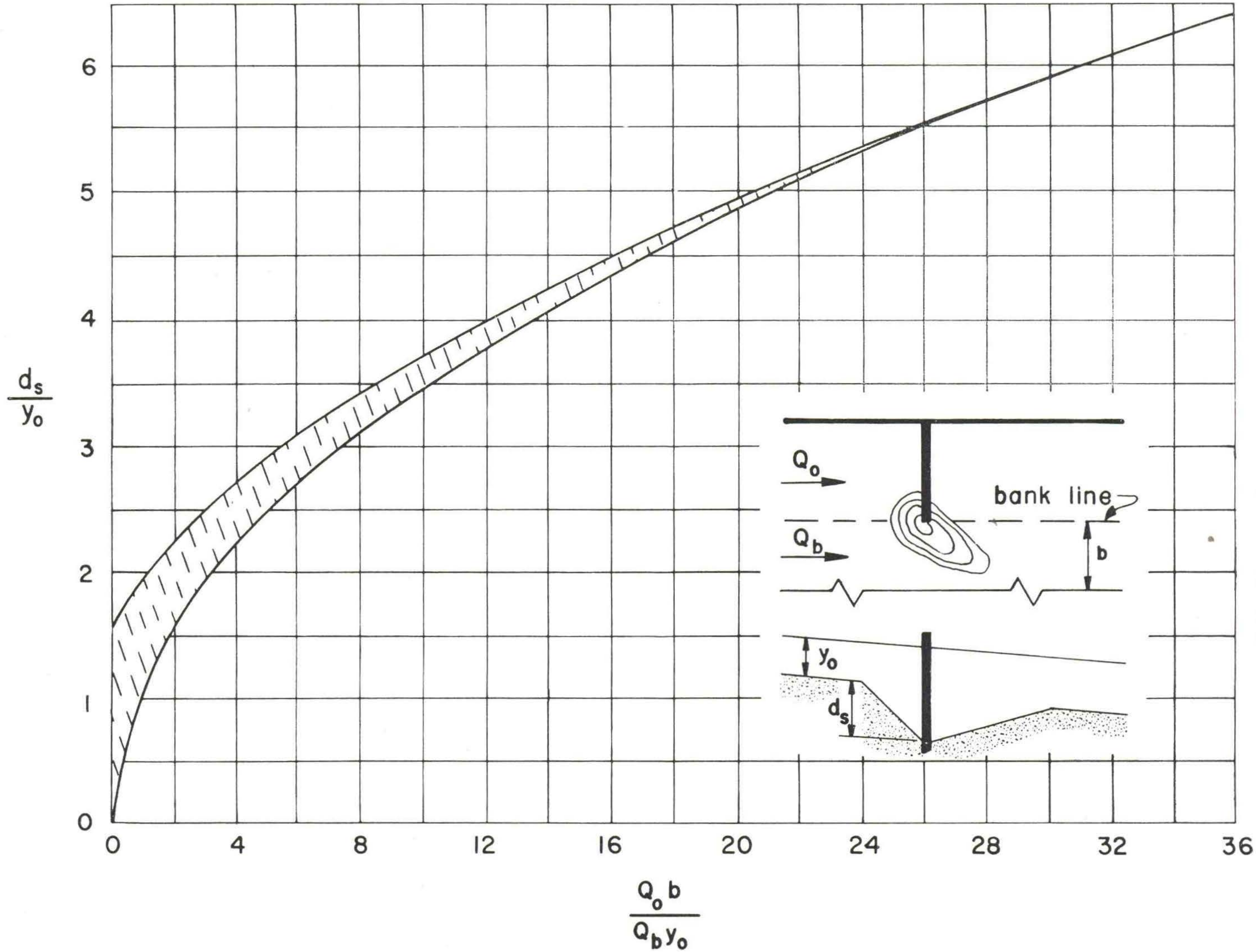


Figure 18. Basic design curve for an overbank bridge constriction

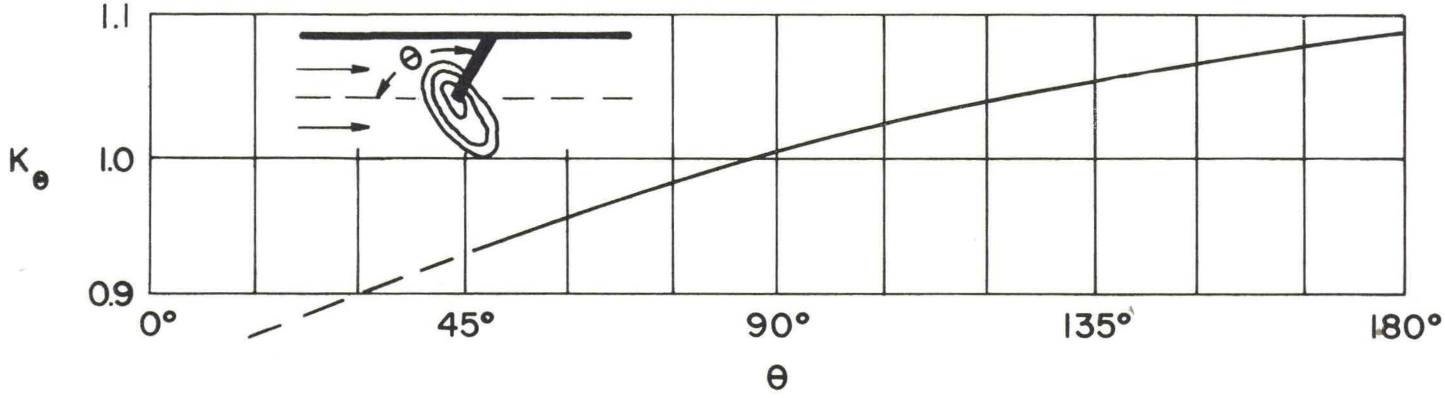


Figure 19. Design factors for the angle of incidence

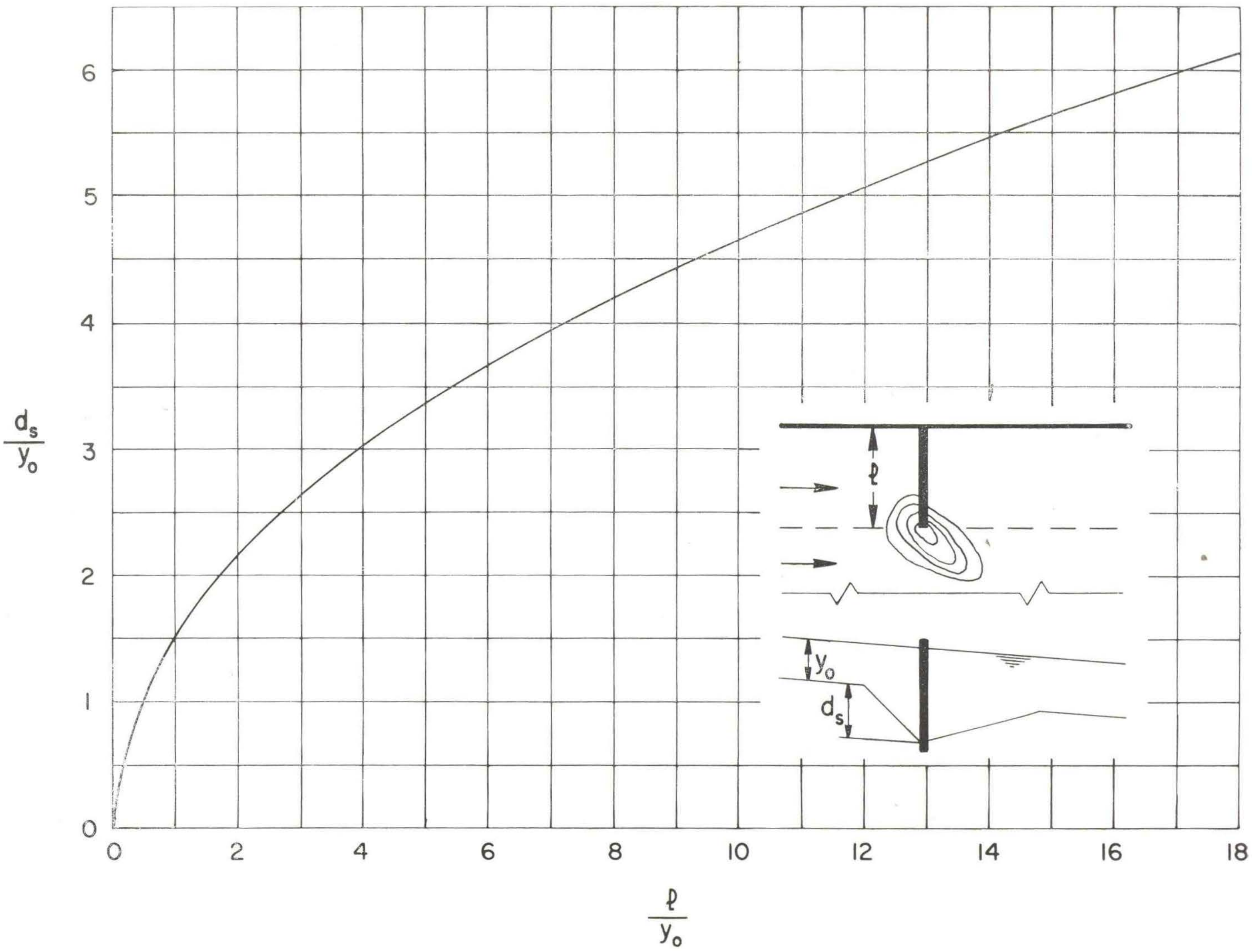


Figure 20. Basic design curve for encroaching abutments

If the approach fill is not normal to the direction of flow, Fig. 19 should be used to obtain a multiplying factor for the effect of the angle of incidence.

For the lower values of the parameter $Q_o b/Q_b y_o$ (in Fig. 18) a shaded zone rather than a single curve is shown. If the overbank flow is small and cross flow from the channel to the floodplain is not likely to occur in the reach immediately upstream from the constriction, the lower limit of this shaded zone can be used. If the amount of cross flow can be evaluated, the rate of flow on the overbank can be corrected and the lower limiting curve can be used. If cross flow is likely, but cannot be evaluated, a greater depth of scour than the lower limit is indicated. The upper limit of the shaded zone would correspond to conditions similar to those in the experimental investigation: an overbank area entirely clear and with minimal resistance to flow, and a bank line lightly screened.

Figure 20 shows the basic design relationship for the scour at an abutment which encroaches on the main river channel. The effective length of encroachment ℓ should be evaluated on the basis of the discharge Q which is obstructed, such that

$$\frac{Q_\ell}{\ell\, y_0} = \frac{Q_b}{b\ y_0}$$

wherein Q_b is the discharge and y_0 is the average depth over the width b. Again a trial-and-error solution is required, since b should be approximately equal to 2.75 d_s. If the approach fill encroaching upon the river channel is not normal to the direction of flow, the effect of the angle of incidence can again be taken from Fig. 19.

In the case of overbank contraction, the mode of movement—and hence the shear-velocity/fall-velocity ratio—should not have any influence on the depth of scour. In the case of channel encroachment, however, the mode of movement—whether bed load or suspended load—does affect the depth of scour. A multiplying factor can be obtained from Fig. 21 after the depth of scour has first been calculated assuming bed-load movement—i.e., that the shear-velocity/fall-velocity ratio is less than ½. Figure 21 can also be used to correct the depth of scour at piers as calculated by the procedures outlined in Bulletin No. 4 (Note that the fall velocity to be used is the median fall velocity of the material which will be scoured out; i.e., the presence of wash load in the stream flow should be ignored.)

A decrease in the depth of scour at the abutment can be achieved by shifting the principal area of scour away from the abutment. This can be done either by spur dikes jutting upstream from the abutment, or by screening to force the overbank flow back to the river channel upstream from the abutment.

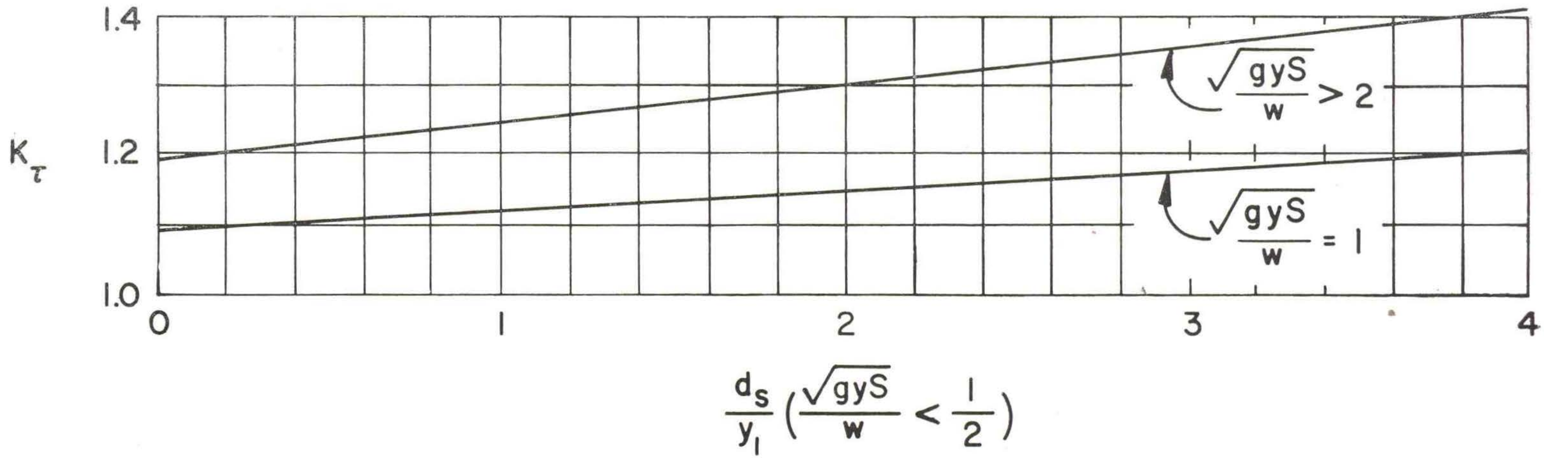


Figure 21. Design factors for changes in the mode of movement

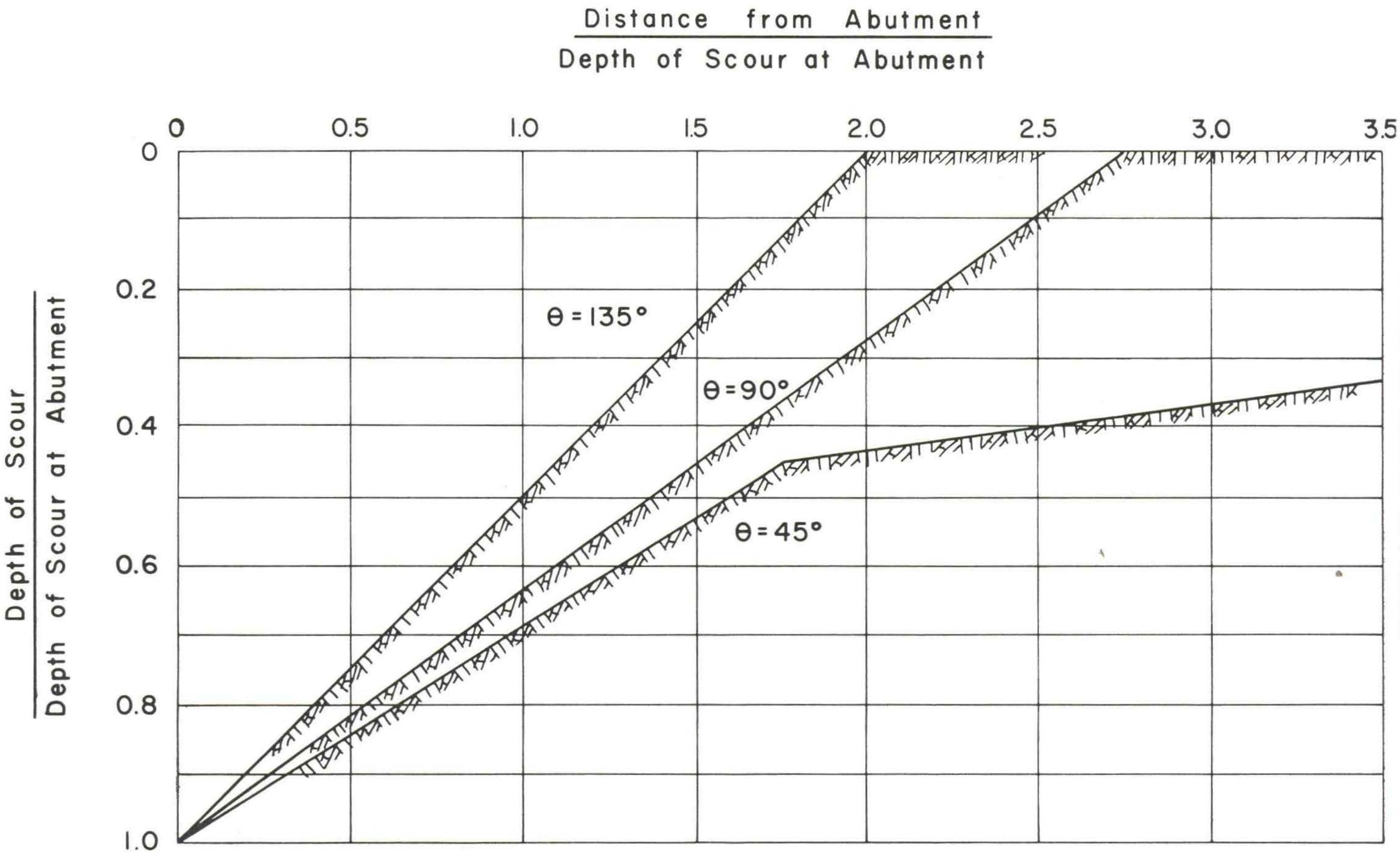


Figure 22. Lateral influence of scour holes

Any spur must, of course, be made sufficiently strong so that it will not fail during the critical time of a major flood. If the spur should fail, the scour at the abutment will merely be delayed, and it is not likely that the delay would be sufficient during a major flood to escape the high flows. A spur, however, need only withstand scour in itself, for there is no loading from the bridge superstructure. Only in individual cases can it be determined whether the cost of a spur is justified.

Screening, whether vegetal or artificial, requires a measure of control over the floodplain upstream from the bridge. This does not mean that the area must be entirely reserved; lines of screening approximately normal to the flow and connected back to the approach fill will suffice. The efficacy of the screening will depend on the extent to which it obstructs and turns the flow back to the river channel; its effect is to reduce the rate of overbank flow Q_o.

The lateral extent of the scour hole is important in relation to the depth of scour to be expected at nearby piers. For an unrestricted scour hole at an abutment, the lateral extent along the line of the upstream face of the bridge normal to the direction of flow can be taken as 2.75 times the depth of scour. If the bridge is at an angle to the direction of flow, the shape of the scour hole can be obtained from Fig. 14. The cross-sectional shapes of the scour holes for 45, 90, and 135° angles of incidence along the line of the bridge are shown in Fig. 22. For the 135° angle of incidence the bridge line follows the tail of the scour hole. An estimate of the limiting value can then be obtained from Eqs. (4) or (5).

If the scour hole is restricted in depth by riprap or by hardpan below the stream bed, the lateral extent will be greater and can be estimated by a trapezoidal cross-sectional area equal to the triangular cross-sectional area of the uninhibited scour hole. (As explained in Bulletin No. 4, riprap, to be effective should be placed well below the stream bed.) Shifting the scour hole by spur dikes or screening will also increase the lateral area of influence of the scour hole. The extent can be estimated from Figs. 16 and 17.

Examples of design procedure. As has been repeatedly stated in this bulletin and in Bulletin No. 4, a knowledge of the flow conditions at the bridge site is absolutely essential if the relationships provided by this investigation of the scour problem are to be used. However, although essential, prediction of the flow conditions is a problem apart from prediction of the scour depth. In the design examples following, therefore, the essential flow characteristics will be assumed as predetermined.

Example No. 1

The simple geometry of Fig. 23 represents schematically a normal bridge crossing in which the overbank flow is evenly divided between the

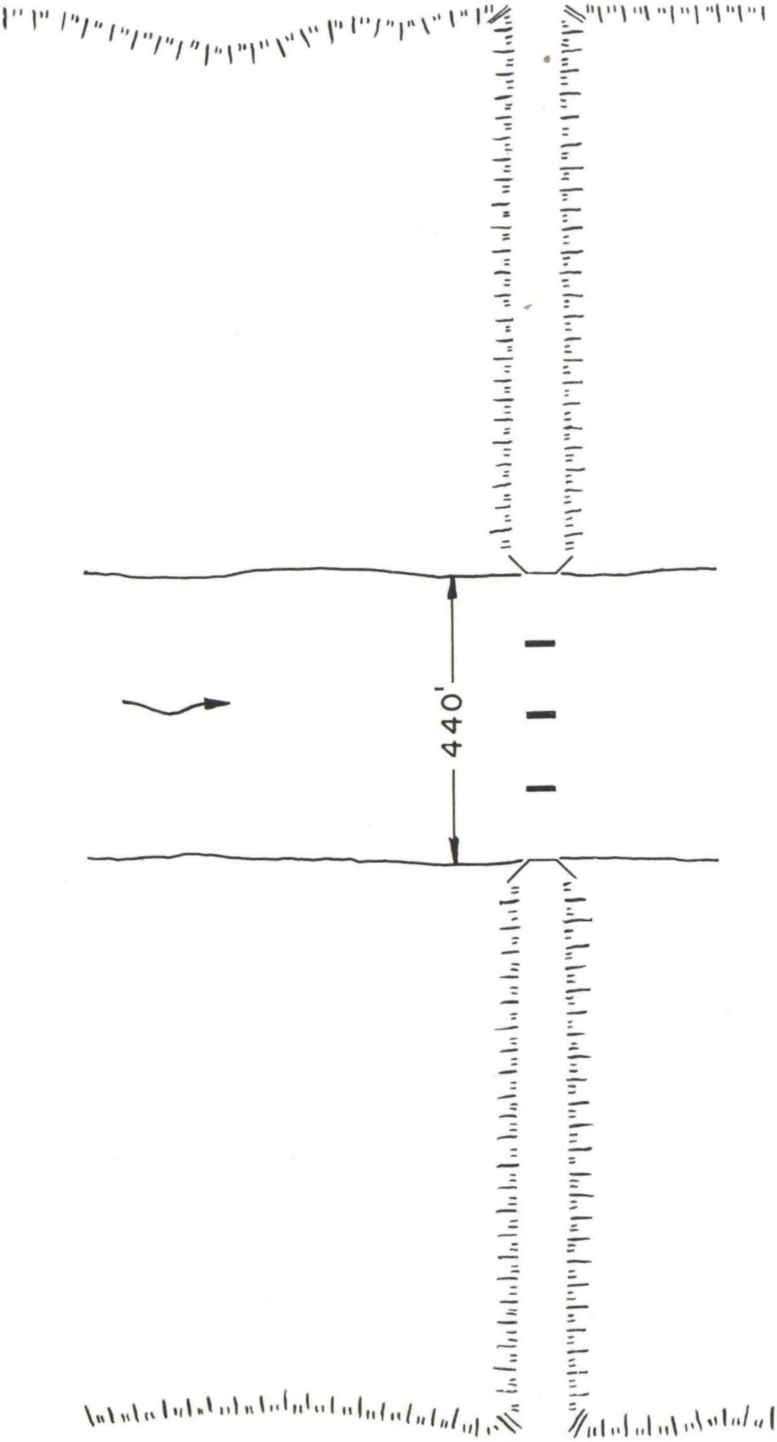


Figure 23. Design example No. 1

two sides of the valley, the resistance to flow along the floodplain is uniform so that there is no cross flow, and the flow within the normal channel is constant across the stream in both depth and mean velocity. The assumed characteristics of the flow are:

Flood Frequency (years)	Q_t (cfs)	Q_c (cfs)	Q_o* (cfs)	y_0 (feet)
10	28,500	28,500	0	12.2
25	36,700	33,400	3,300	13.4
50	44,800	35,500	9,300	13.9
100	56,300	37,600	18,700	14.4

* Q_o as tabulated is the total overbank flow and is evenly divided between the two sides.

Because in this simple case the flow is uniform across the section, it is not necessary to estimate d_s, b, and Q_b. In evaluating the parameter $Q_o b/Q_b y_0$, $Q_o = Q_o(\text{tab.})/2$, $Q_b = Q_c$, and $b = 440$. After evaluating the parameter for each flood being investigated by d_s/y_o is obtained from the lower limiting curve of Fig. 18. Finally, by multiplying by y_0 the value of the depth of scour is obtained.

Flood Frequency (years)	$\frac{Q_o b}{Q_b y_0}$	$\frac{d_s}{y_0}$	d_s (feet)	b_s* (feet)	Chance of occurrence in 50 years
10	0	0	0		
25	1.62	1.42	19.0	52.3	0.87
50	4.14	2.26	31.4	86.4	0.64
100	7.60	3.05	43.6	120.0	0.39

*The lateral extent of the scour hole for a 90° angle of incidence is equal to 2.75 d_s for sands with a normal angle of repose.

If the 440-foot width of the stream is bridged by four 110-foot spans, the scour hole for the 100-year flood would extend beyond the piers nearest the banks. To the scour due to the piers themselves (Bulletin No. 4) should be added a fraction of the scour at the abutment equal to 43.6 x 10/120 or 3.6 feet.

If the division of the flow between the two sides of the valley were not even, two solutions would be required (one for each abutment) and the scour depths would be different. The procedure, however, would be exactly as illustrated, taking into account the rate of flow on each overbank for each flood.

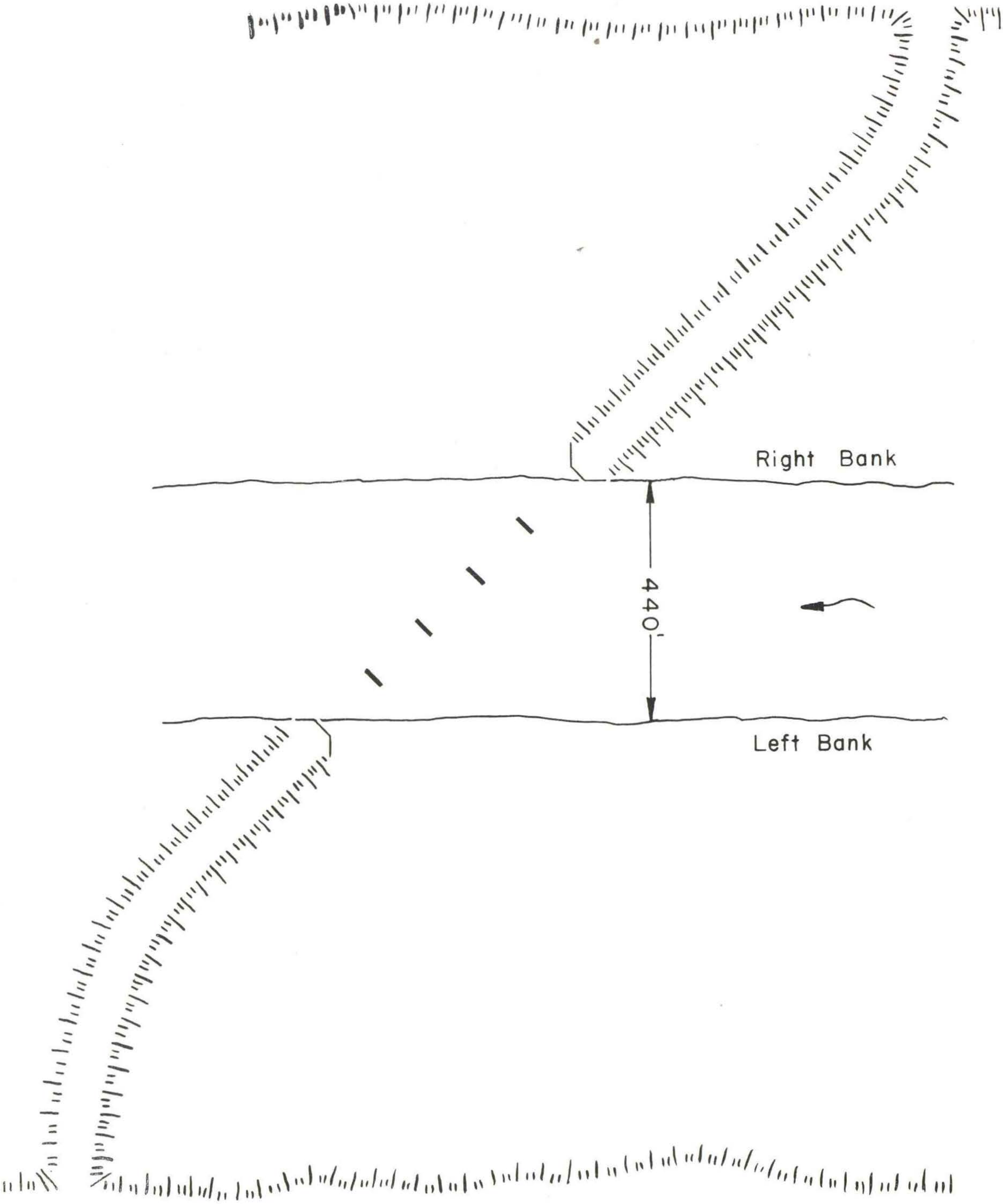


Figure 24. Design example No. 2

Example No. 2

As shown in Fig. 24 the valley and stream of the first example are crossed at an angle of 45°. The angle of incidence at the right bank (looking downstream) is thus 45° and at the left bank 135°. From Fig. 19, the multiplying factors K_Θ are 0.93 and 1.05 for the right and left abutments, respectively. These coefficients, applied to the depth of scour obtained in the first example, give the depth of scour at the two abutments. The span length, 124 feet in this case, divided by the depth of scour locates the piers with reference to the lateral extent of the scour hole in Fig. 22.

Flood Frequency (years)	d_s (right bank) (feet)	d_s (left bank) (feet)	span/d_s (right bank)	span/d_s (left bank)
25	17.7	20.0	6.7	5.9
50	29.2	33.0	4.2	3.7
100	40.5	45.8	3.0	2.7

If the piers are numbered in order from the right bank, the depth of general scour at Piers No. 1 and 4 can be obtained from Fig. 22 as a fraction of the scour at the adjacent abutment. An estimate of the scour at Pier No. 2 can be obtained from Eq. (4) or Fig. 2.

Flood Frequency (years)	d_s* Pier No. 1 (feet)	d_s* Pier No. 4 (feet)	d_s* Pier No. 2 (feet)
25	2.8	0	1.1
50	9.3	0	3.1
100	14.6	0	6.0

* To the scour at the piers tabulated here must be added the local scour due to the pier itself—see Bulletin No. 4.

Example No. 3

To illustrate the trial-and-error solution that usually will be required, the stream of uniform depth and velocity of the first example is modified to have the shape and distribution of flow shown in Fig. 25. The flood flows and the division of flow between channel and overbank are assumed to be the same as in the first example.

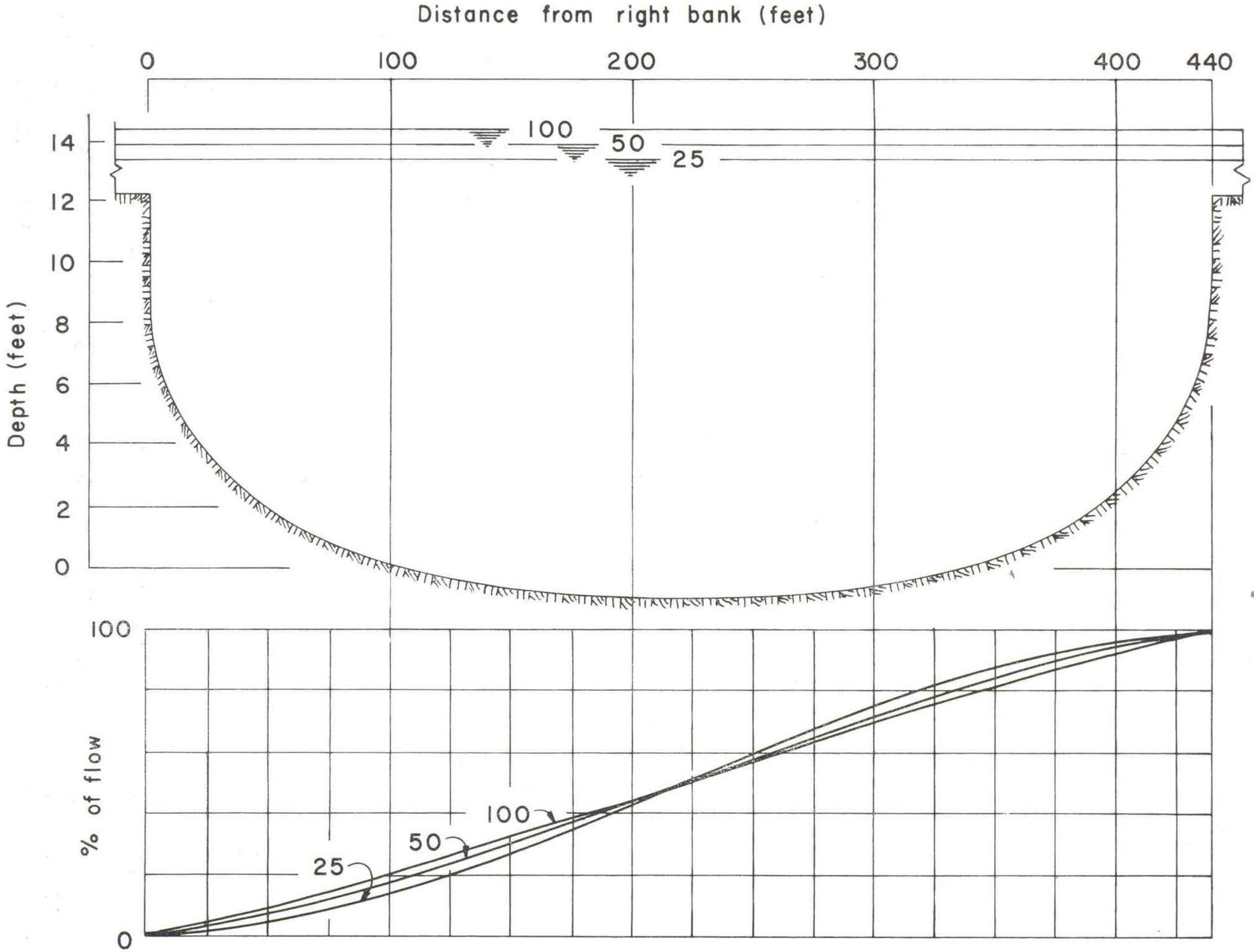


Figure 25. Design example No. 3

Flood Frequency (years)	b (feet)	Q_b (cfs)	y_0 (feet)	$\frac{Q_o b}{Q_b y_0}$	$\frac{d_s}{y_0}$	d_s (feet)	b_s (feet)
First trial							
25	80	3,340	10.9	3.62	2.12	23.1	63.5
50	100	6,240	12.0	6.21	2.75	33.0	90.6
100	140	11,300	13.5	8.59	3.22	43.5	119.5
Second trial							
25	60	2,000	10.2	4.86	2.45	25.0	68.9
50	90	5,330	11.7	6.71	2.86	33.5	92.3
100	110	8,270	12.8	9.71	3.42	43.7	120.1

A third trial could be made using b values of 70, 95, and 120, but in practice it would not be justified in view of the approximations, assumptions, and estimates that must be made in arriving at the values used to describe the flow characteristics. Note that the depths of scour obtained in the two trials are not very different.

Example No. 4

The site geometry shown in Fig. 26 again modifies the simple case of the first example, this time to show the effect of general contraction in the reach and the problem of zero overbank flow. Assuming that the flow conditions approaching the upstream bridge are the same as in the first example, the conditions at the downstream bridge are quite different. There will be general scour at the downstream bridge due to the contraction of the flow above, there will nominally be no overbank flow, and because of the general scour the depth of flow will be increased. Equation (4) or Fig. 2 can be used to evaluate the general scour.

Flood Frequency (years)	$\frac{Q_t}{Q_c}$	$\frac{d_s{*}}{y_0}$	$d_s{*}$ (feet)	y_0(new) (feet)
25	1.10	0.085	1.1	14.5
50	1.26	0.220	3.1	17.0
100	1.50	0.415	6.0	20.4

* General scour across the section caused by the upstream contraction.

Although the upstream contraction has presumably forced all the overbank flow into the main river channel, in the 1500 feet between the bridges there would be a cross flow, because the difference in water-surface slope in the channel and the overbank areas would create a slope in the lateral direction. Assuming that there is only light vegetation along the bank, the upper limiting curve of Fig. 18 should be used, and for zero

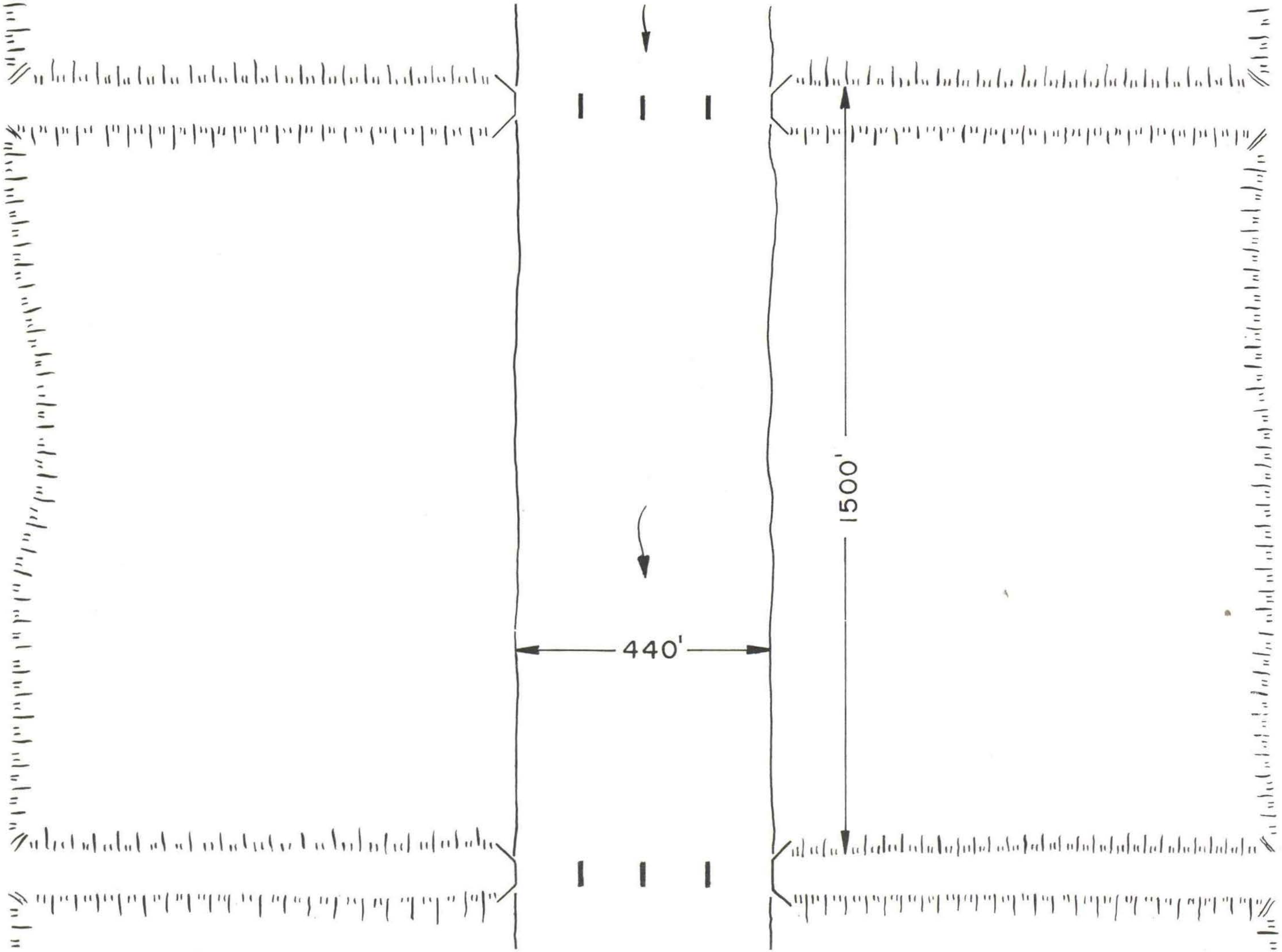


Figure 26. Design example No. 4

overbank flow $d_s/y_0 = 1.6$. The depth of scour thus obtained should be added to the depth of general scour to yield the total depth of scour.

Flood Frequency (years)	d_s (abutment) (feet)	d_s (total) (feet)
25	23.2	24.3
50	27.2	30.3
100	32.7	38.7

The depth of scour as computed here is almost the same as in Example No. 1, and is probably greater than should be anticipated. Unless the cross flow can be evaluated, however, a conservative design such as this is inadvisable.

Example No. 5

In Fig. 27, the case of channel encroachment in a wide shallow stream, which is leveed some distance back from the low-water channel, is shown. The slope of the stream is 3 feet per mile, and the bed material has a diameter of 0.2 mm ($w = 6$ cm/sec). For simplicity the fractions approaching the abutments on each side are considered equal; for unequal division the procedure would be the same, although two solutions would be required.

Flood Frequency (years)	Q_t (cfs)	Q_c (cfs)	$(Q_t - Q_c)/2$ (cfs)	y_0(max) (feet)
25	18,500	17,500	500	7.0
50	26,500	24,000	1,250	8.0
100	35,800	30,800	2,500	9.0

For the first trial, the entire flow between one abutment and the centerline will be considered, so that $Q_l = (Q_t - Q_c)/2$, $Q_b = Q_c/2$, $b = 500$ feet, and y_0 is the average depth of flow between the abutments. The effective length of the contracting approach fill can then be evaluated ($l/y_0 = Q_l b / Q_b y_0$) and, from Fig. 20, the depth of scour obtained.

Flood Frequency (years)	y_0(av) (feet)	l/y_0	d_s/y_0	d_s (feet)	$2.75 d_s$ (feet)
25	5.0	5.7	3.6	18.0	49.5
50	6.0	8.7	4.3	25.8	71.0
100	7.0	11.6	4.9	34.3	94.5

For the second trial, b values of 60, 80, and 100 will be assumed and the flow between the abutments distributed according to the 5/3 power of the average depth.

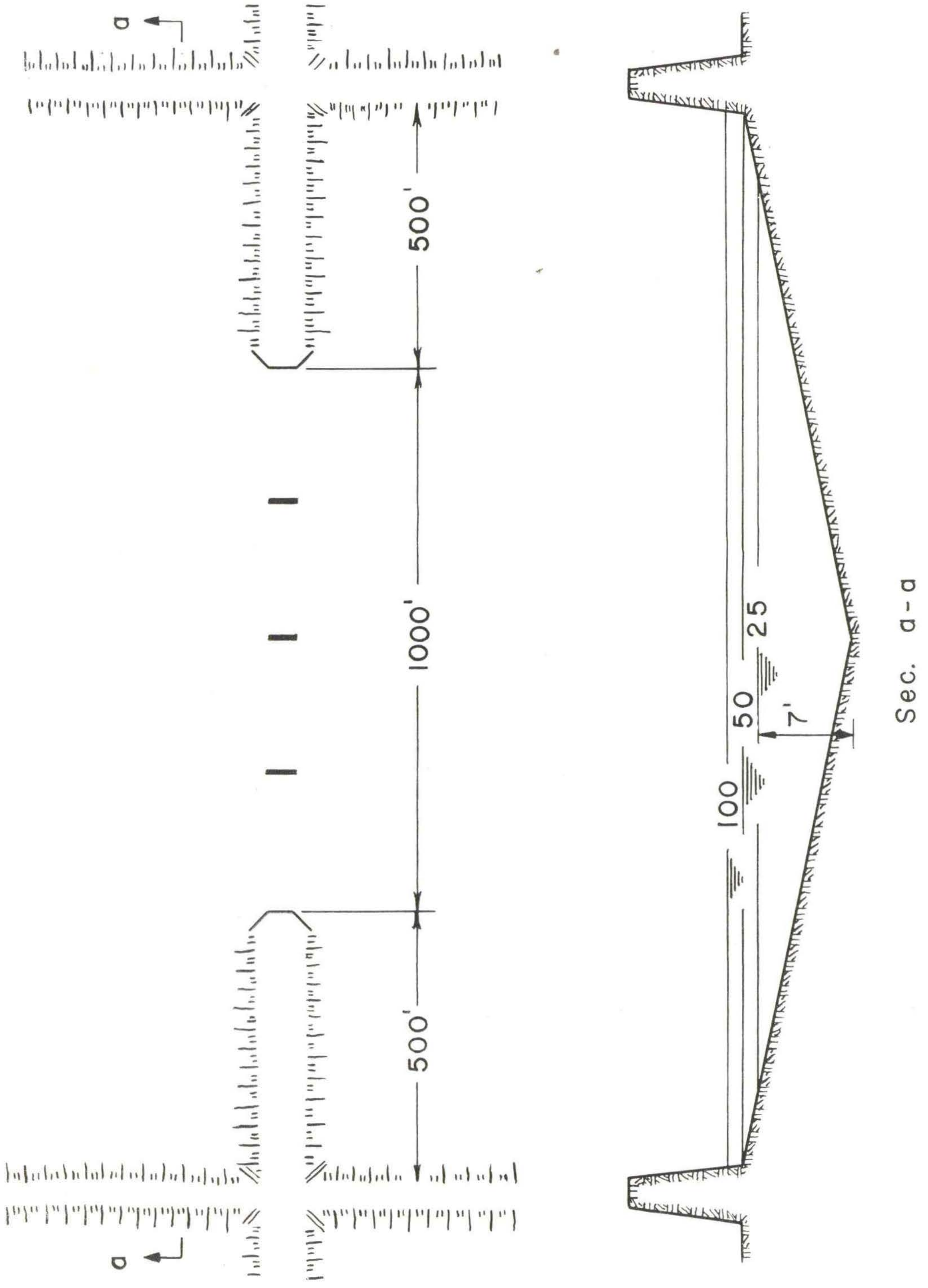


Figure 27. Design example No. 5

Flood Frequency (years)	b (feet)	y_0(av) (feet)	Q_b (cfs)	ℓ/y_0	d_s/y_0	d_s (feet)	$2.75d_s$ (feet)
25	60	3.24	510	18.2	6.2	20.0	55.0
50	80	4.32	1,110	20.9	6.7	29.0	80.0
100	100	5.40	2,000	23.2	7.2	39.0	107.0

For this case the possible effect of the shear-velocity/fall-velocity ratio should be checked. For the assumptions that the bed material which will be scoured has a mean size of 0.2 mm and a fall velocity of 6 cm per sec, and that the slope of the stream is 3 feet per mile; K_T can be obtained from Fig. 21 and the depth of scour corrected.

Flood Frequency (years)	$\sqrt{gy_0S}$ (fps)	$\frac{\sqrt{gy_0S}}{w}$	K_T	d_s (corrected) (feet)
25	0.30	1.5	1.30	26.0
50	0.33	1.7	1.35	39.0
100	0.36	1.8	1.40	55.0

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