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Scour Around Bridge Piers And Abutments

by

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fied. In any area where, as a result of the modified pattern, the capacity for transport out of the area is greater than the rate at which material is supplied to the area, scour will occur. Conversely, where the transport capacity is less than the rate of supply, deposition will occur. The resultant changes in the stream bed will further modify the flow pattern—and the capacity pattern—until equilibrium between capacity and supply is again achieved at every point on the stream bed. An analytic solution would have to combine a prediction of the flow pattern and a description of the local transport capacity of the flow. Although an approximation of the flow pattern might be attempted, a comparable solution for the capacity is not yet possible.

The experimental approach has been tried in the past with limited success, usually because the goal was restricted to a particular installation or to some special phase of the general problem. The earliest report on a laboratory study which has come to the attention of the writers is that of Engels at Dresden, Germany, in 1894. In the paper describing that study reference is made to an earlier one in France in 1873 by Durand-Claye. Neither these early experiments nor subsequent studies by various investigators in various countries have been sufficiently general to obtain the desired result—a means of predicting scour in the field.

The experimental approach was chosen as the investigative method for this study, but with due recognition of the importance of field measurements and with the realization that the results must be interpreted so as to be compatible with the present-day theories of fluid mechanics and sediment transportation. This approach was chosen because, on the one hand, the factors affecting the scour phenomenon can be controlled in the laboratory to an extent that is not possible in the field, and, on the other hand, the model technique can be used to circumvent the present inadequate understanding of the phenomenon of the movement of sediment by flowing water.

PROGRAM OF INVESTIGATION

In order to obtain optimum results from the laboratory study, the program was arranged at the outset to include a related set of variables in each of several phases into which the whole problem was divided. The phases thus selected were:

1. Geometry of piers and abutments
2. Hydraulics of the stream
3. Characteristics of the sediment
4. Geometry of channel shape and alignment

Planning for the necessary field program was deferred pending the initial laboratory results and the development of a satisfactory means of obtaining field measurements.

SCOUR AROUND BRIDGE PIERS AND ABUTMENTS

by

Emmett M. Laursen and Arthur Toch

INTRODUCTION

Man's never-ending search for better materials and construction methods and for techniques of analysis and design has overcome most of the early difficulties of bridge building. Scour of the stream bed, however, has remained a major cause of bridge failures ever since man learned to place piers and abutments in the stream in order to cross wide rivers. The bridge builder's concern with scour in the days of the masonry arch is evidenced by the treatises of that time. The massive piers and short spans typical of old arch bridges resulted in extreme contractions of the flow section and, consequently, severe scour. Moreover, the timber-crib foundations placed at or near the original stream bed were particularly vulnerable to undermining. Modern steel and concrete bridges can be built with long spans and relatively small piers. Pile and caisson foundations can be sunk far beneath the stream bed. Yet every year additions are made to the list of bridges that have failed because of scour of the stream bed around the piers and abutments. In fact, the considerable bridge losses in the State of Iowa in the year 1947 were in large measure responsible for the determination of the Iowa State Highway Commission to sponsor an intensive study of the problem with the goal of evolving means for predicting probable scour depths.

Considering the overall complexity of field conditions, it is not surprising that no generally accepted principles (not even rules of thumb) for the prediction of scour around bridge piers and abutments have evolved from field experience alone. The flow of individual streams exhibits a manifold variation, and great disparity exists among different rivers. The alignment, cross section, discharge, and slope of a stream must all be correlated with the scour phenomenon, and this in turn must be correlated with the characteristics of the bed material ranging from clays and fine silts to gravels and boulders. Finally, the effect of the shape of the obstruction itself—the pier or abutment—must be assessed. Since several of these factors are likely to vary with time to some degree, and since the scour phenomenon as well is inherently unsteady, sorting out the influence of each of the various factors is virtually impossible from field evidence alone.

An analytical approach is equally difficult. If an obstruction, such as a pier, is placed in a stream, the flow pattern in the vicinity of that obstruction will be modified. Because the capacity for the transport of sediment is a function of the flow, the transport-capacity pattern will also be modi-

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The original program was modified as the study progressed. The second and third phases were combined in the laboratory and are discussed together in this report. Certain elements of the fourth phase pertaining to the relationship between the channel and the piers and abutments were included in the first phase, while those pertaining to the bridge-crossing geometry were deferred for study in a separate investigation. A field scour meter was installed during the later part of the program when laboratory experience had been gained in the design and use of electrical scour meters. Together with the laboratory measurements, the field data thus obtained have permitted an evaluation of the scale effect.

A final report is being submitted at this time because the concept of the last phase has changed gradually during this investigation and because a means for predicting the local scour at a pier has been evolved. The continuation of the study will be more concerned with the overall bridge-site geometry and has as its goal a means of predicting the general scour at the contracted opening.

INVESTIGATIVE EQUIPMENT AND TECHNIQUE

Studies of relative depth of scour. For the qualitative study of the effect of pier and abutment geometry, two flumes (Fig. 1) 35 feet long

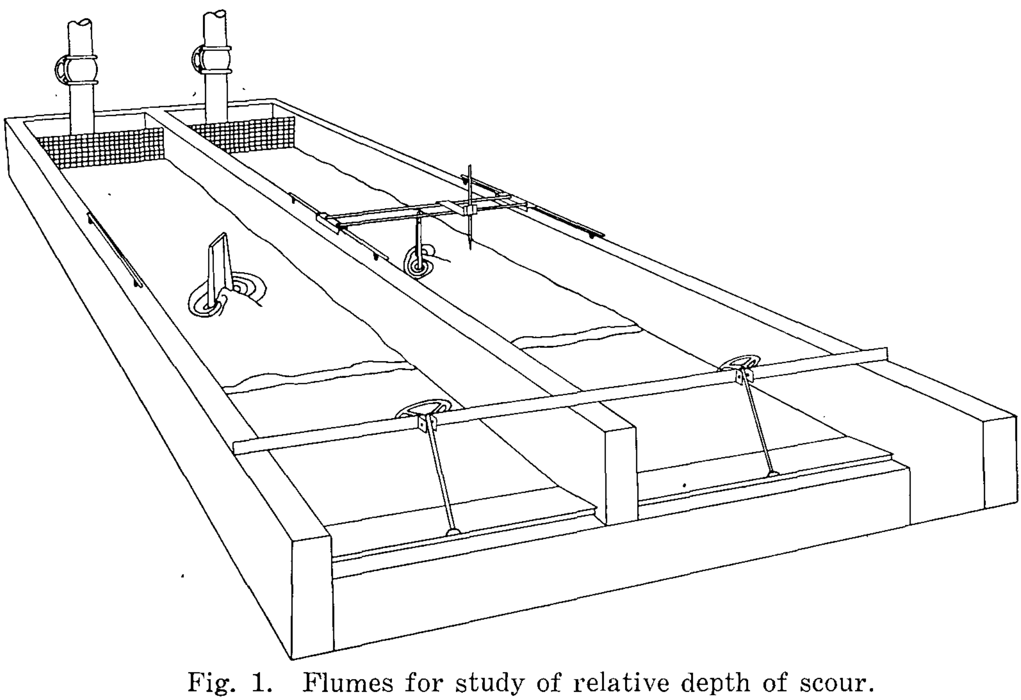


Fig. 1. Flumes for study of relative depth of scour.

and 5 feet wide were constructed in the laboratory annex of the Iowa Institute of Hydraulic Research. Because a wider flume was also considered necessary, a common wall was used, removal of which converted the two flumes into a single one of double width. Flow was supplied to each flume by the general pumping system of the building and measured by means of an air-water manometer connected to an orifice in the 10-inch supply line. Simple tailgates controlled the depth of flow. Both flumes were provided with level rails and a gage carriage. A portion of the floor near the middle of each flume was depressed sufficiently to permit the development of the scour holes around the models.

Because the form of bridge piers has evolved largely according to structural and architectural criteria, countless different shapes exist. The pier shapes chosen for the study, as shown in Fig. 2, are typical forms used in Iowa but do not model any specific design. The rounded or conical forms represent the type of piers used in conjunction with multiple-span truss or girder bridges, and the square forms represent the type used

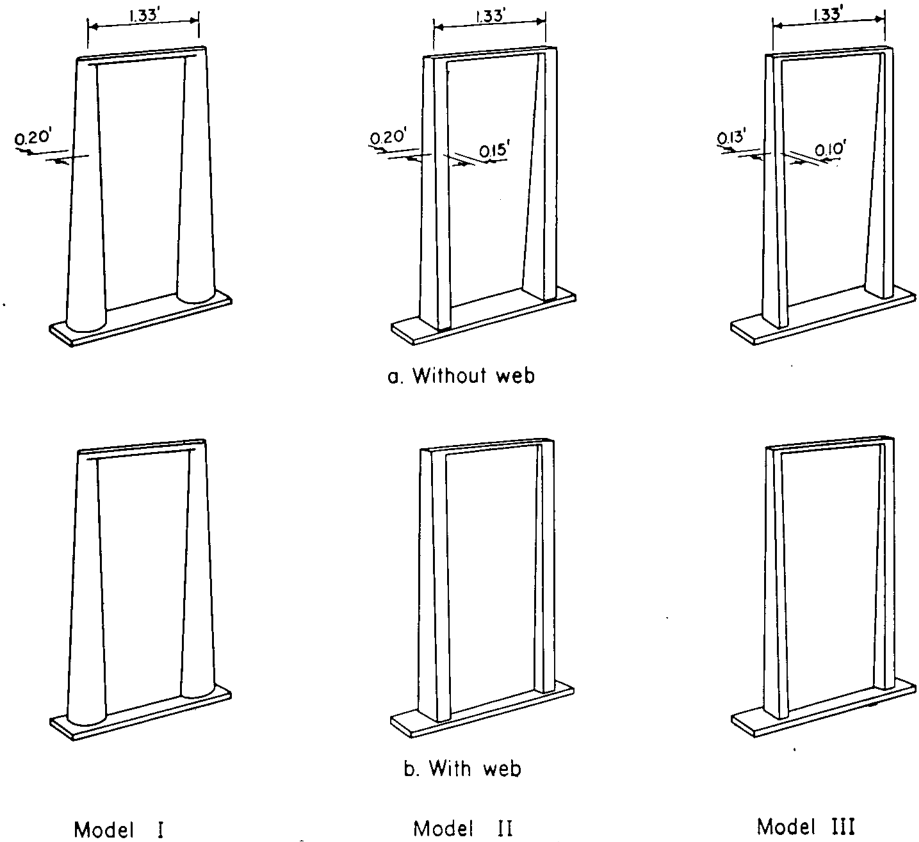


Fig. 2. Models of typical Iowa piers.

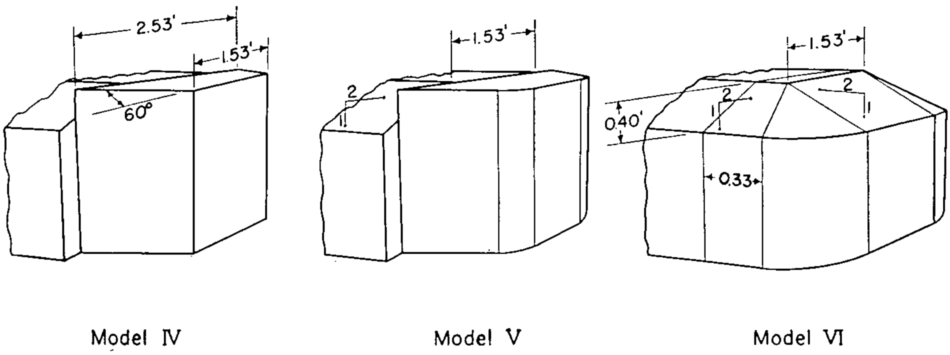


Fig. 3. Models of typical Iowa abutments.

with rigid-frame reinforced-concrete bridges. It is believed that the pier shapes used in this study typify almost all the modern piers being built in Iowa.

In a similar manner the many possible abutment forms are simulated by those shown in Fig. 3. The sloping form is representative of the modern stub abutment, in which the bridge seat is supported on piles and the approach fill is contained within sheet piling. The vertical form is a typical gravity wall with the bridge seat contained in the abutment proper. Since high velocities might be expected near the sharp corners of this type of abutment, a model in which these corners were rounded was also tested.

The most important geometrical relationships of pier and abutment to the stream were investigated in these tests, although they might well be considered a part of the fourth phase. These relationships include, in the case of piers, the angle of approach α of the stream, and, in the case of the abutments, the relative length of the approach fill, or the contraction ratio β.

To overcome the difficulty of observing the development of the scour hole, horizontal layers of colored sand 0.01 foot thick were placed at intervals of 0.1 foot below the original sand surface. These layers permitted the observation of the scour pattern as it progressed and also served to delineate the contours of the hole of greatest extent. Photographs taken from a fixed relative position formed the final record.

The sorting effect that can occur in the scour phenomenon was eliminated by the use of a closely graded silica sand having the mean size of 0.58 millimeter and the size distribution shown in Fig. 9. The colored sand was produced by dying the sand with an aniline dye (Sudan III) in a 1-percent benzene solution. All other physical properties of the colored layers were identical to those of the remainder of the sand bed.

Prior to the establishment of a standard operating procedure, a series of runs was made with different model sizes, depths of flow, and velocities. On the basis of this series, the size of the models shown in Figs. 2 and 3 was chosen and the following procedure for the tests on the piers adopted. With the tailgate in its highest position and the flume filled with water, the flow was fixed at 1.875 cubic feet per second—a condition which did not cause movement of the sand. The tailgate was then lowered until a depth of flow of 0.3 foot was reached, with an approach velocity of 1.25 feet per second. The times of starting, of initial movement, and of water-surface establishment were recorded. As the scour hole increased in depth, the time of appearance of each red layer was also noted. At the end of three hours the tailgate was raised until the sand movement ceased. The supply valve was then closed and the water drained from the flume. Any slight excess of sand obscuring the red contours was removed and a standard photograph taken. For the abutment tests the same procedure was adopted but the single wide flume was used with a proportional increase in the discharge.

The study of pier-shape effect was continued by D. E. Schneible of the Bureau of Public Roads as an M.S. thesis project. In contrast to the previous part of the investigation, which was concerned with the comparative scour tendency of modern Iowa designs, this study had as its purpose the gathering of data in a search for pier forms or devices which

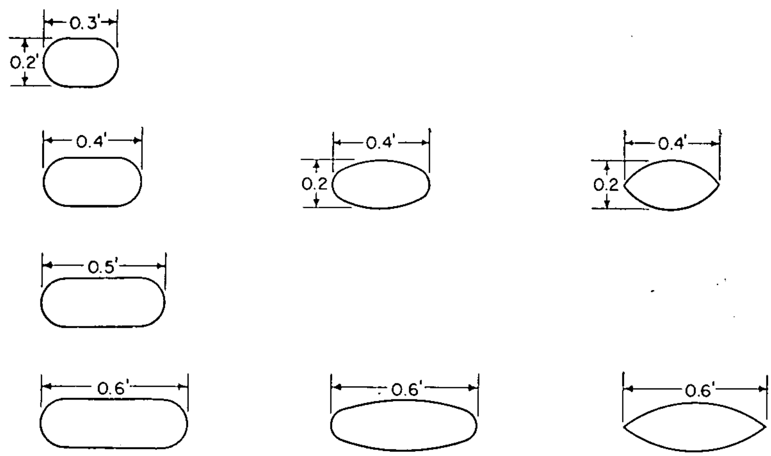


Fig. 4. Basic geometrical pier shapes.

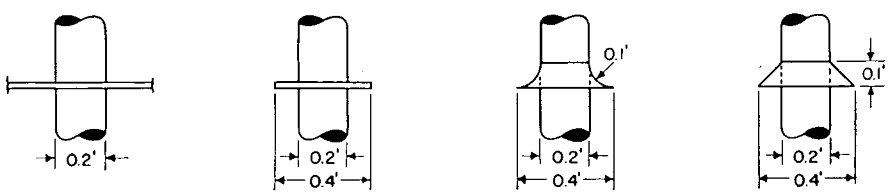


Fig. 5. Scour arrestors—ring type.

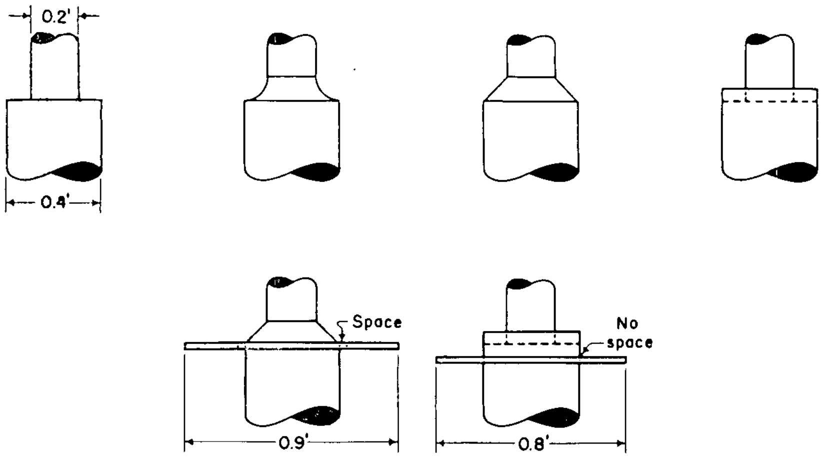
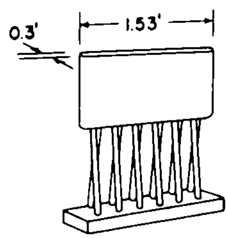


Fig. 6. Scour arrestors—caisson type.

would be characterized by relatively small scour depths. The geometric forms tested, Figs. 4-6, therefore, do not necessarily have any prototype equivalents. Except for the length of run, which was reduced to 60 minutes, the laboratory procedure for this study was, in general, the same as described previously.

Following these tests on basic geometrical forms a pier design proposed for use in Texas, in which battered piles extended up from the river bed to about low-water elevation, was studied. The model shown in Fig. 7 was tested with the pile tops at bed level and 0.1 and 0.2 foot above bed level. At each of these elevations the pier was tested at four angles of attack: 0°, 15°, 30° and 45°. The procedure was the same as that of Schneible's tests.

Debris which collects on the nose of a pier has the effect of changing the geometry of the pier. In order to assess qualitatively its effect on the scour hole, debris was permitted to collect on a Model IIb pier aligned with the stream. The debris used consisted of twigs and small



Note: Pier body set at three levels

Fig. 7. Model of Texas pile pier.

tree branches broken to a size roughly compatible with the model scale. For some runs these sticks were allowed to float freely against the pier; in others bunches of sticks were tied together by cloth strips to simulate different densities of accumulation. The test procedure was the same as described before except that the length of the run was set at 90 minutes to insure the development of the maximum scour hole in all runs. Photographs were taken during each run and after the water had been drained from the flume. The debris was then removed and black thread was placed at 0.1-foot contour intervals to delineate the scour hole, whereupon additional photographs were taken.

Studies of equilibrium depth of scour. For the study of the effects of the stream and sediment characteristics, the equipment described above was not sufficient. Because the transport capacity of the flow is dependent upon its depth and velocity, it was necessary to construct a flume wherein the rate of sediment movement could be measured and sediment introduced at an equivalent rate at the upstream end. The flume that was built was similar to those used in the qualitative study and had the same dimensions. A view of this transport flume is shown in Fig. 8. The measurement of the sediment load was accomplished at the downstream end by means of a scale connected to a sand trap. The sand was trapped continuously and the accumulated weight could be read on the scale at any convenient time interval. Sand was introduced at the upstream end by an elevator. The relationship between a unit scale reading and a revolution of the elevator drive shaft depended only on the lever arms at the scale, the gear ratios of the elevator drive, the area of the elevator, and the density of the sand in place. Since all of these were constant, the amount of sand added could be made equal to the amount of sand trapped in any time interval.

Besides the sand used for the study of the geometry of pier and abutment, three other sizes of uniform sand and one mixed sand were tested. The characteristics of all the sands are shown in Fig. 9.

A method of placing the sand in the elevator was developed which gave a constant density throughout the elevator. A hopper with a long slot in the bottom was filled with sand and the slot was then opened to let the sand drop about one foot through the air before reaching the surface of the water which filled the elevator. The hopper could be moved laterally across the elevator so that the sand deposit accumulated uniformly over the entire area of the elevator. Since the voids ratio of the uniform sands was the same, the density of these four sands in place was equal. A slightly greater density was obtained for the mixed sand, as would be expected. The trap was found to be efficient in catching the sediment except at very high rates of transport, when small amounts of material could be observed being carried over the tailgate.

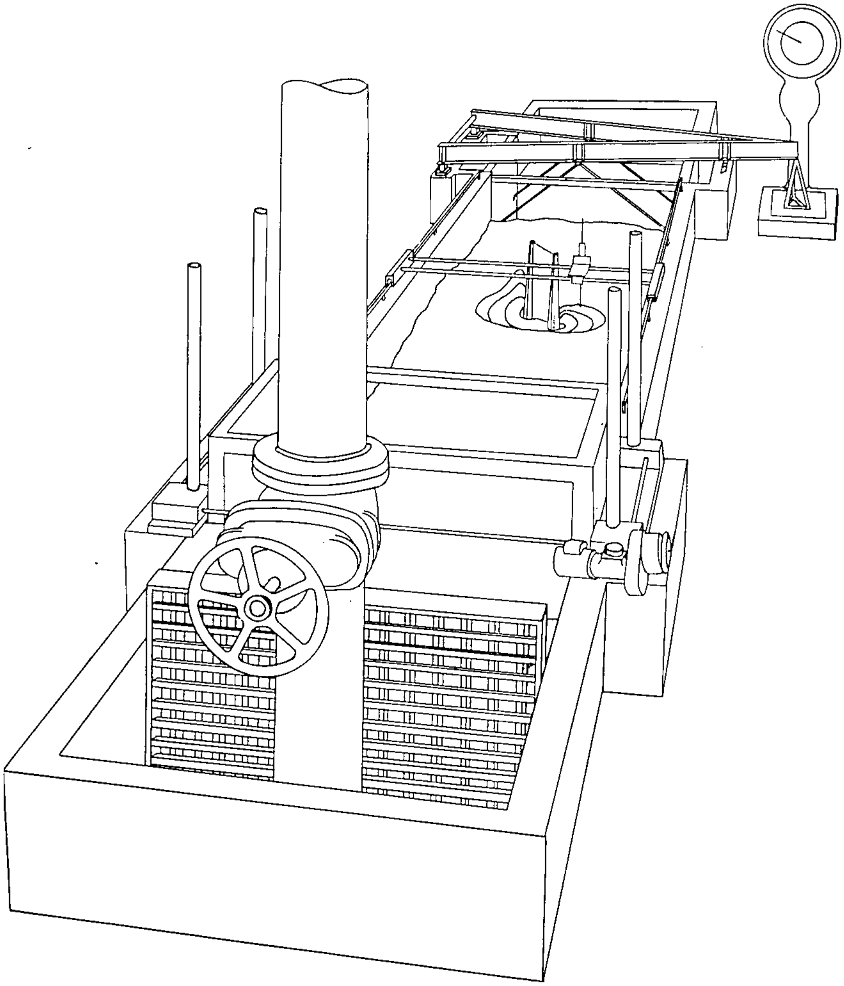


Fig. 8. Flume for study of equilibrium depth of scour.

The rate of flow of water was measured by means of an orifice meter and controlled by a valve in a 12-inch line from the laboratory water-supply system. A tailgate was used to control the water-surface elevation. Level rails on the walls of the flume permitted movement of a carriage for measurements with a point gage and facilitated leveling the bed for each run.

A Model IIb pier made of plastic and set at an angle of 30° to the flow was used throughout the study of the effect of velocity, depth of flow, and sediment size. The depth of scour at the vertical face of the upstream shaft was measured during each run by means of a device utilizing the difference in electrical resistance of the water and the sand-water mixture.

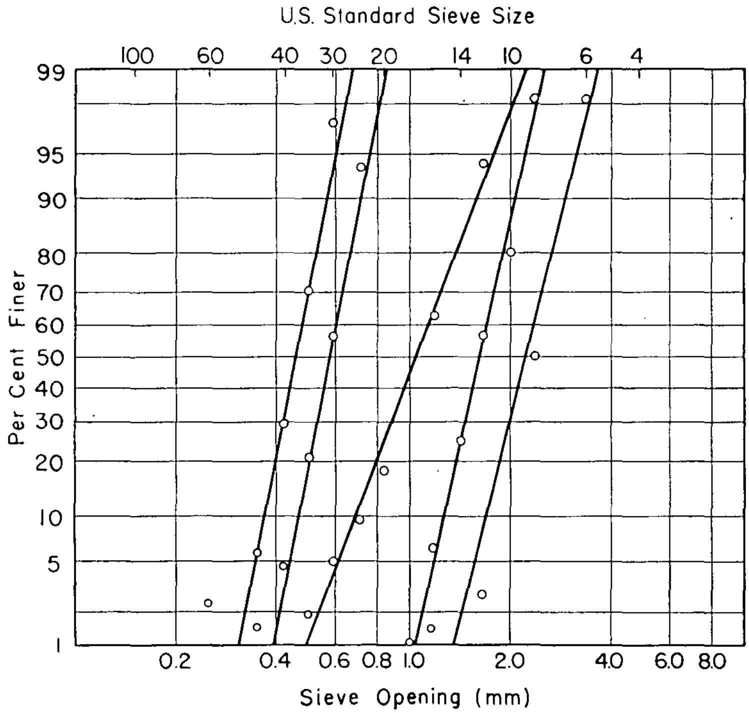


Fig. 9. Sand characteristics.

Electrodes were recessed in the front face of the pier. For the two fine sands and the mixed sand an electrode spacing of 0.02 foot was used, but for the two coarse sands a spacing of 0.04 foot was found necessary.

During the course of the investigation the electrical scour meter was modified and improved several times. The principles of the design and construction of the various scour meters are presented in the appendix to this report.

The procedure for starting a run was the same as that employed in the studies of relative scour depth. Additionally, a reading was taken on the trap scale at the time the proper depth of flow was established and at five-minute intervals thereafter. To keep the volume of sand in the flume constant, the elevator was raised a corresponding amount for each increment of scale reading. With the first scour meter, readings were taken every thirty seconds by means of a manual switching device and plotted to yield a graphic record of the scour depth as it varied with time. An automatic recorder was incorporated in the later versions of the scour meter, so that the manual operation and plotting were unnecessary. At the end of each run with the 0.58-mm sand and of many of the later runs, the contours of the scour hole were determined and plotted.

The above procedure was adopted after a number of trial runs had been made. It was observed during the trial runs that the elevation of the sand bed near the pier was fortunately maintained at a sensibly constant elevation when the amount of sand in the flume was kept constant. Profiles of the sand bed taken during and after the run were used to determine any small correction necessitated by a building-up of the bed near the pier. Leveling of the bed for the next run always gave opportunity for checking the total amount of material in the flume and its distribution.

As can be seen from the typical mass curve shown in Fig. 10, the rate of transport gradually attained a constant mean value, with minor short-period fluctuations due to dune formations. Equilibrium of transport is indicated on such a mass diagram when a straight line can be drawn as a mean curve of the plotted data. The slope of the straight line then indicates the mean rate of transport. In the experiments with the 0.58-mm sand, continuous operation of the elevator and intermittent operation at 5-minute intervals were found to give the same rate of transport and scour conditions. Intermittent operation proved to be more expedient and was used for all further experiments.

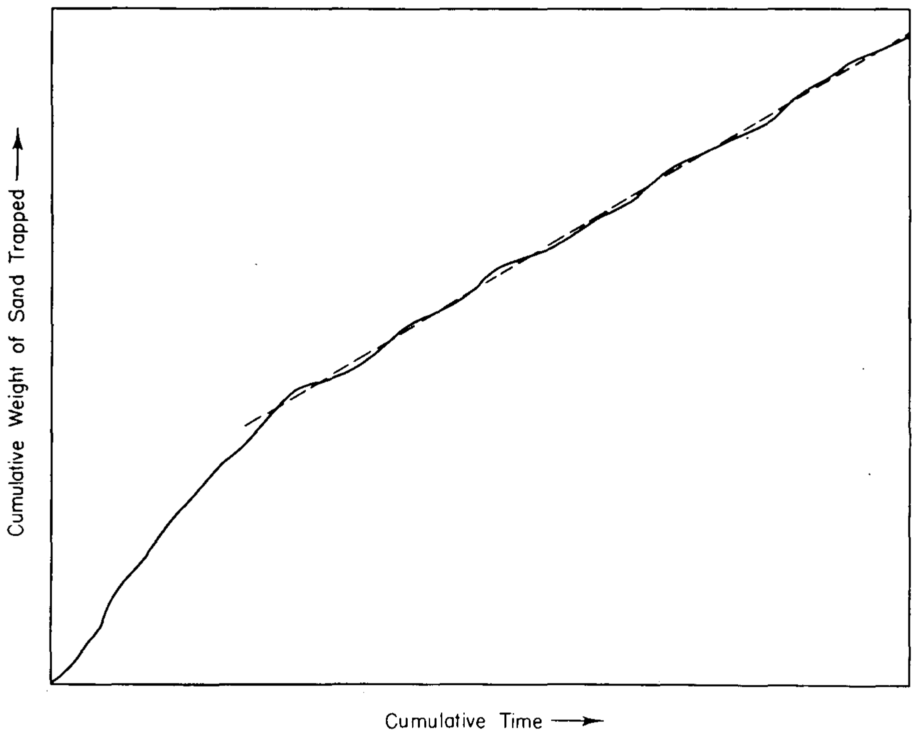


Fig. 10. Mass curve of sediment transport.

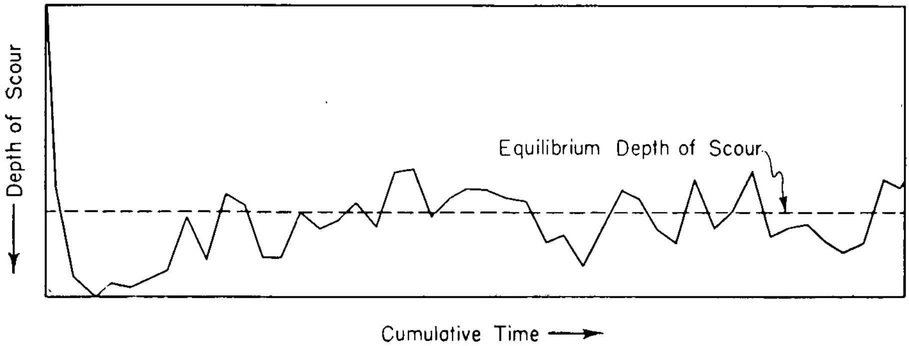


Fig. 11. Typical scour-depth/time relationship.

The scour depth approached an equilibrium condition in the manner shown in Fig. 11, a typical scour-time graph. To give a workable plot the scour depths are averaged over ten-minute intervals; changes in the average depth of scour could be noted in this shorter graph more readily than in the original record. For each run the initial rate of scour was very high. As the scour hole increased in size, the rate of scour decreased, until a maximum depth was obtained at the time the first large dune reached the hole. Partial refilling of the scour hole usually occurred slowly thereafter as the equilibrium transport condition was established in the flume. Determination of the final mean equilibrium depth took considerable time because of the fluctuations in scour depth with the passage of dunes. Operating time for the test runs varied from 8 to 50 hours.

A total of forty successful runs were made in the study of the effect of velocity and depth of flow and of sediment size. The velocities used ranged from 1.00 to 2.50 feet per second, the depths of flow from 0.2 to 0.9 foot, and the mean sediment size from 0.46 to 2.3 millimeters. The resulting rate of sediment transport, probably the most significant single indicator of the range of the tests, varied from 0.0041 to 0.272 pound per second, dry weight.

The transport flume was also ultilized for a study of the effect of contraction on the depth of scour around circular piers. Right circular cylinders 0.1, 0.2, 0.8, and 2.0 feet in diameter were used. One or more of a given size were placed in a line normal to the flow to vary the contraction ratio to a maximum of 50 percent. Depths of flow of 0.3 and 0.9 foot, a velocity of 1.75 feet per second, and a sand size of 0.58 millimeter were used in this series of tests. Electrode sets with a spacing of 0.015 foot were recessed in the front face of the center pier of the combinations. Otherwise the experimental procedure remained as described previously.

Field measurements. To obtain prototype data a bridge site was selected for field measurements of scour. The basis of selection was a set of specific requirements which would simplify modeling the bridge pier as well as permit analysis of the prototype data. Basically, these requirements were that there be a sand bed of considerable depth at the pier, that the angle of approach between the oncoming flow and the pier be the same at all stages, and that the crossing geometry be such as to produce local scour at the pier rather than overall degradation because of excessive contraction.

After careful consideration of many sites in Iowa, a crossing was finally chosen on the Skunk River about 6 miles below Ames. A picture of the bridge at low water is shown in Fig. 12. The river was artifically straightened some 40 years ago and has a long straight approach to the bridge as well as a straight course for a considerable distance downstream. The single pier of the bridge is located in the center of the 135-feet-wide flow section and is aligned with the river. Levees on both sides of the stream will presumably contain any flows up to a depth of about 12 feet at the site. The sandy bed, relatively free of cementing substance, and the absence of abutments jutting into the stream make this as desirable a site as is likely to be found.

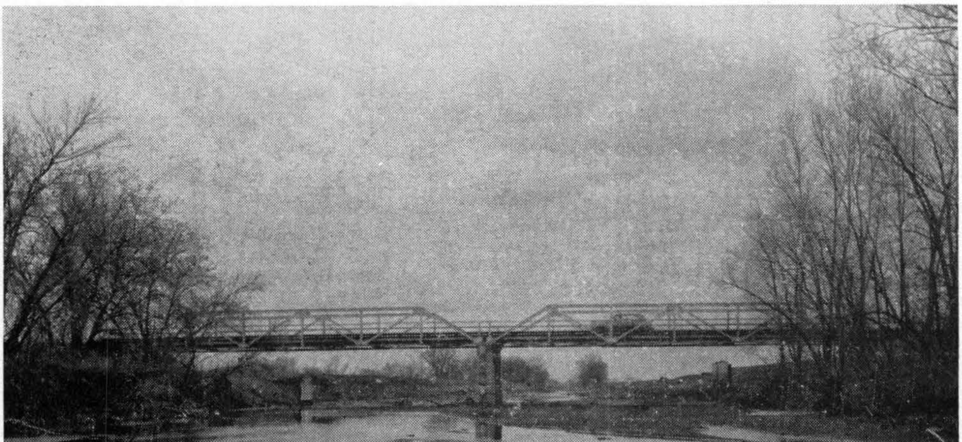


Fig. 12. Site of prototype measurements—Skunk River near Ames.

A concrete pile, 1 foot square, into one face of which 29 stainless-steel electrodes had been cast, was driven into the bed of the stream just upstream from the pier. The spacing of the electrodes was ½ foot on center. Figure 13 shows the pier details and the relation of the pile to the pier. The wires from the electrodes were carried in a pipe inside of the pile to a point near its top and from there in a conduit to the recording apparatus at the top of the pier. To obtain a record of the depth of flow a stage-indicating device, consisting of a rheostat driven by a float inside an 18-inch pipe mounted on the downstream end of the pier, was included in the scour-meter installation. This field adaptation of the scour meter developed for the laboratory is described in the Appendix.

For the model component required in the establishment of a model-prototype relationship, two scale models, Fig. 14, of the Skunk River bridge pier were constructed and tested in the transport flume. The scale ratios used were 1:12 and 1:24. The size of the larger of the two models was limited by the depth of flow in the river and in the model, and that of the smaller by the precision of the measurement of the elevation of the sand surface.

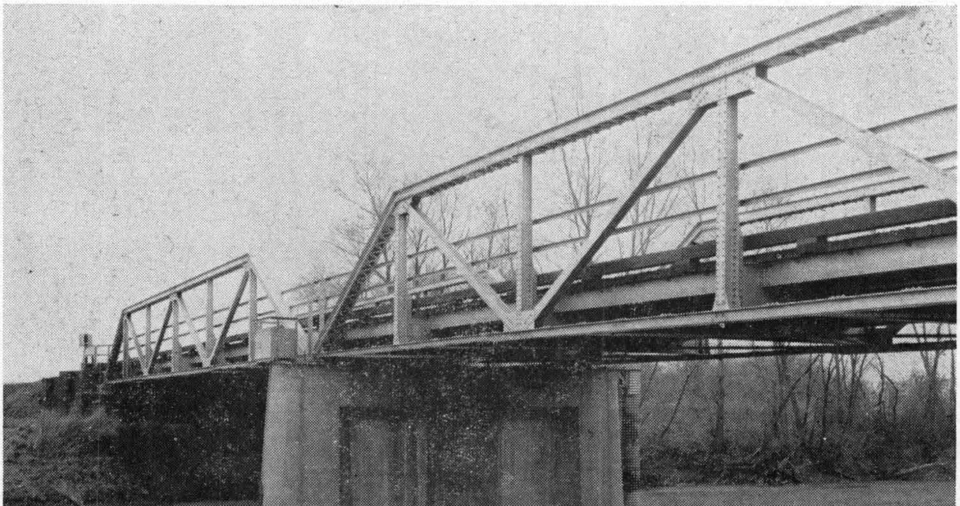


Fig. 13. Prototype pier after installation of measuring equipment.

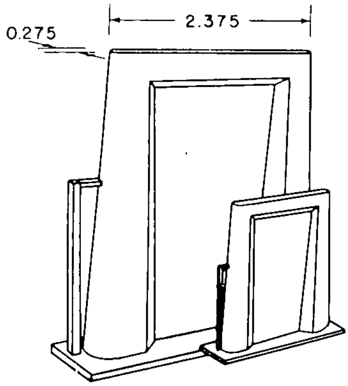


Fig. 14. Models of Skunk River pier.

The experimental procedure for these tests was the same as previously described for the investigation of the effects of velocity and depth of flow. For all runs the velocity was held constant at 1.75 feet per second and the standard 0.58-millimeter sand was used. The depth of flow, of course, was varied from run to run. The electrodes for determining the scour depth were mounted in scaled-down piles in front of the pier just as in the prototype. The electrode spacing used was 0.015 foot for both models.

EFFECT OF PIER AND ABUTMENT GEOMETRY

Representative Iowa pier and abutment designs. The standard photographs taken at the end of each 3-hour run were used as the bases of Figs. 15 to 20, the scour patterns around the piers, and Figs. 21 to 23, the scour patterns around the abutments. The shape and relative depth of the scour holes are indicated by the contour lines. Although these values cannot be interpreted in terms of actual depth of scour to be expected in the field, they nevertheless embody a comparative measure of scour tendency as a function of pier and abutment geometry.

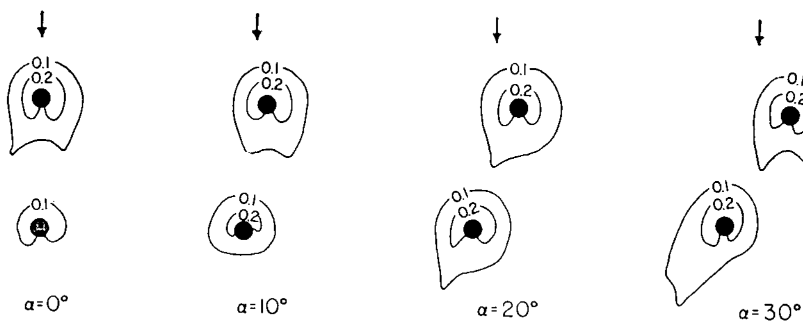


Fig. 15. Scour patterns around Model Ia.

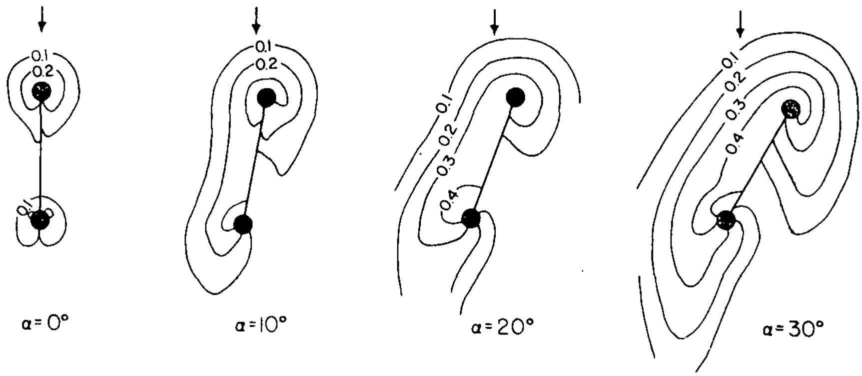


Fig. 16. Scour patterns around Model Ib.

Several general characteristics are common to all the scour patterns around the piers. The upstream portion of the hole has the approximate form of an inverted cone, sometimes distorted from the circular, with side slopes equal to the angle of repose of the sand. The zone of greatest depth is generally displaced from the face of the pier. Deposition which occurs in the low-velocity area behind the pier divides the downstream portion of the scour hole into two separate tails.

In the absence of a web, a separate scour hole is formed at each shaft of the pier. At small angles of approach (0 to 10 degrees) the downstream shaft is shielded by the one upstream, with an accordingly shallower scour hole. As the angle increases, however, not only is this protection lost, but the downstream shaft becomes subject to currents of higher velocity produced by the shaft upstream; this results in a shift of the region of deepest scour to the downstream hole. At large enough angles, of course, the interference effect practically disappears.

When the pier as a whole is parallel to the flow, exactly the same pattern results whether or not the web is present. At increasing angles, however, basically different patterns obtain. The web then plays an increasingly important role, changing the position of the maximum depth of scour and increasing the depth twofold at an angle of 30° and two-and-one-half-fold at 45°. Little if any local variation can be ascribed to the shape of the shafts at large angles; both the flow and the scour pattern then approximate those which would occur around a simple flat plate.

The most pronounced difference between the effects of round and rectangular shafts is observable at small angles, when local shape is important. As a consequence of greater disturbance of the flow resulting from the sharp edges of the rectangular shaft, the scour depth is increased some 15 percent beyond that for a cylinder of the same breadth. A comparison of the structurally equivalent round and rectangular piers (i.e., the round and the small rectangular) is difficult to make because

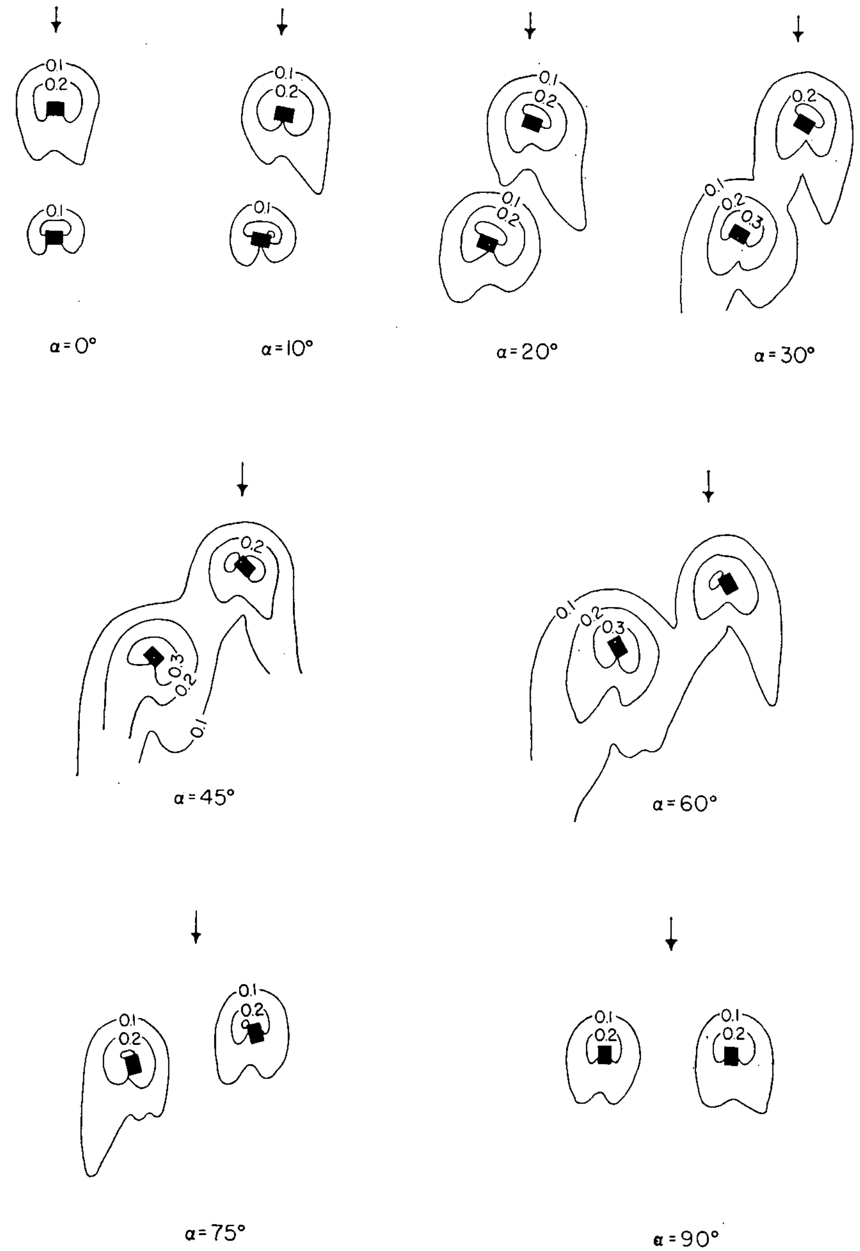


Fig. 17. Scour patterns around Model IIa.

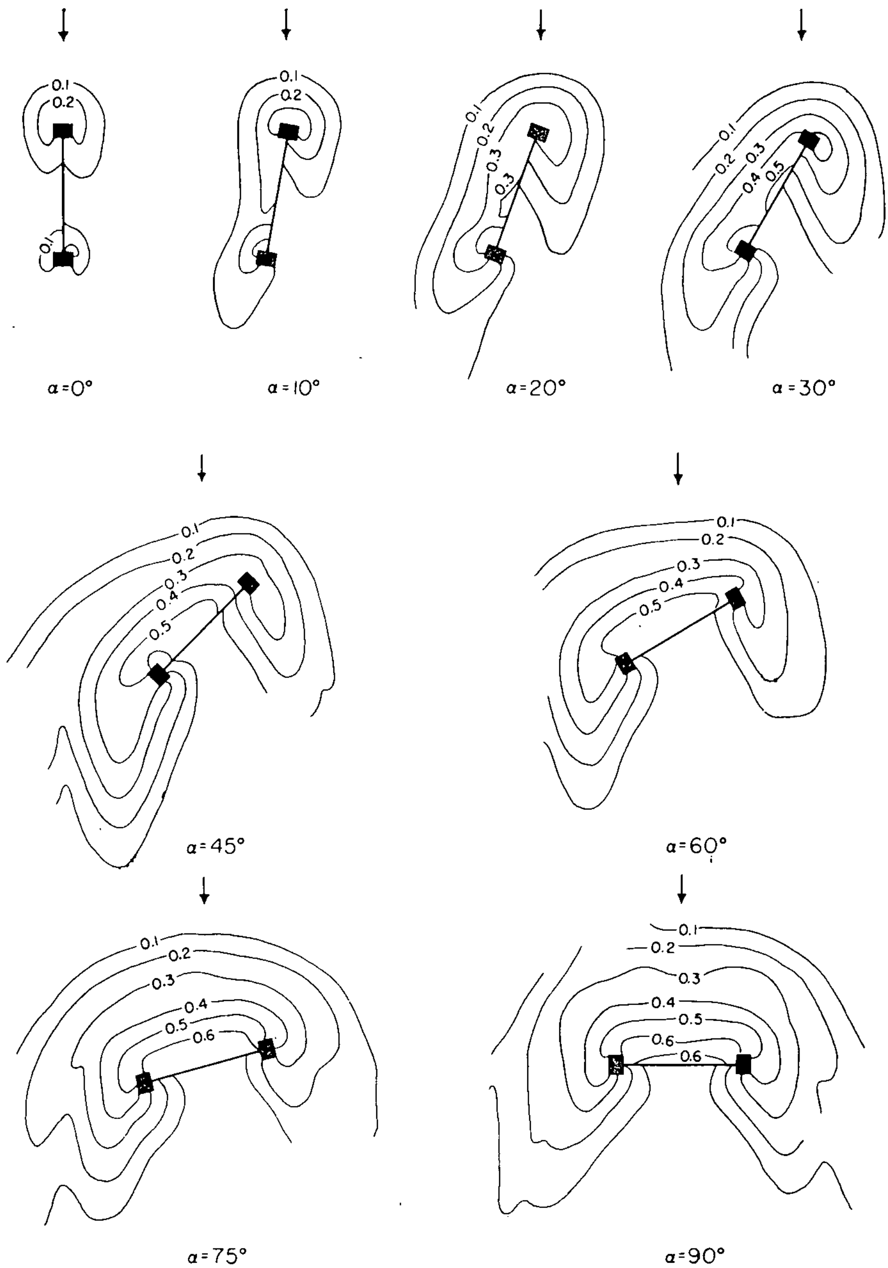


Fig. 18. Scour patterns around Model IIb.

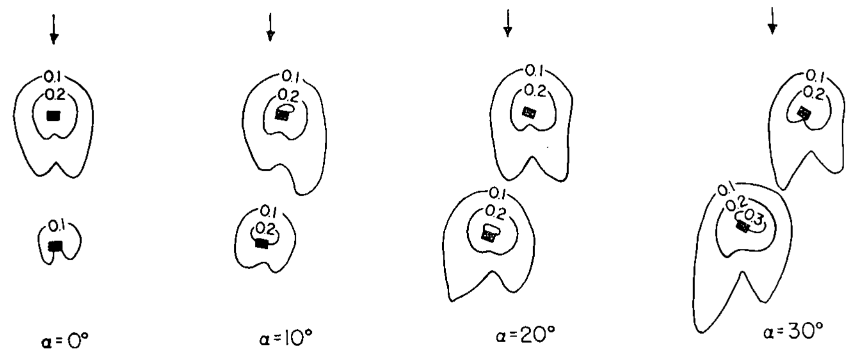


Fig. 19. Scour patterns around Model IIIa.

of the change in width-depth ratio. It is believed, however, that the equivalent rectangular pier will produce a slightly greater scour depth.

Since a partial web extending down to low water is often used, a test was run with the web extending one-half the depth of flow below the surface. A scour depth intermediate between those for a full web and no web was obtained. With greater depth of web it is likely that the increased velocity of flow under the web would result in scour as deep as that for the full web. By replacing the full web of the model with a screen, the condition of a lattice web was simulated. Again the scour was less than that of the fully webbed model. While this construction might prevent large debris from lodging between the shafts, it would eventually become clogged with small debris, thus defeating its special purpose.

Another set of tests was made to indicate the effect of a footing which became exposed during the scour process. When the footing was set slightly above the bottom of the scour hole which otherwise would have been formed, the scour action was inhibited. A footing set too high was undermined, but in no instance was the scour depth greater than that which would have occurred without a footing present.

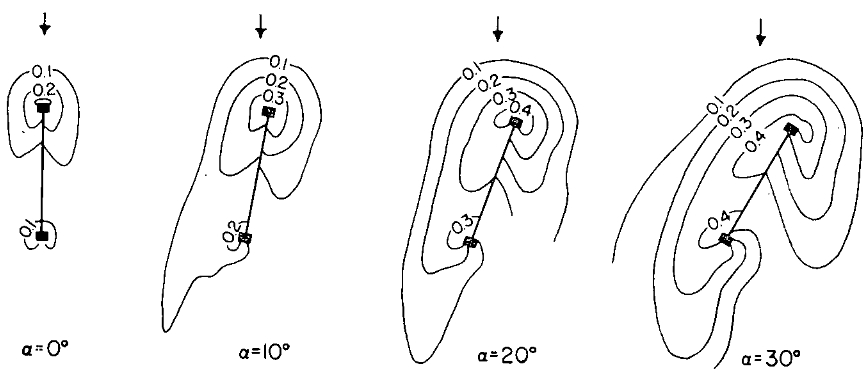


Fig. 20. Scour patterns around Model IIIb.

For the abutment forms, the scour-hole formation was similar to that for the piers, the only essential difference being that the flow passed—and hence the scour occurred—on one side only. In these tests the velocity of approach was held constant for all runs with different abutment forms and different degrees of contraction. For each form the maximum depth of scour increased with increasing contraction. A lesser, but still noticeable, effect could be ascribed to the geometry of the abutment. The gravity abutment with a sharp edge between breast wall and wing walls produced the greatest depth of scour. The moderate amount of rounding given the corners in the other gravity-abutment model resulted in a reduction in scour depth of about 15 percent. The form of the stub abutment resulted in a depth of scour between those of the two gravity walls. Noteworthy is the fact that the scour hole was centered on the upstream corner of the abutment for all forms and conditions tested.

When piers and abutments are considered in combination, a multitude of geometrical arrangements are possible. For exploratory purposes, two tests were made with a combination of stub abutment and fully webbed pier. Very little change can be noted in the scour adjacent to the abutment, but the scour pattern around the pier is similar to that of the pier alone at an angle of 45° to the current, Fig. 24. It would seem then that the pier exerts little influence on the flow around the abutment, but that the abutment, in effect, swings the current against the pier. The importance of further investigation of similar combinations is obvious. Such a study, however, forms more logically a part of the investigation of general bridge-crossing geometry.

Basic geometrical forms. Two approaches to the problem of decreasing the depth of scour are immediately apparent. The first consists of shaping the pier of desired size to obtain minimum disturbance of the flow. By thus decreasing the capacity for transport within the scour hole, the depth of scour will be minimized. The second approach lies in placing an inerodible surface around the pier to increase the resistance to the scouring action. Both methods were studied by Schneible and are reported in detail in his thesis.

Of the various pier shapes investigated by Schneible, only those indicating a promising line for further research are summarized in Table I. The tests on inclined piles, icebreaker nose forms, and flared conical bases have not been included. The results for "streamlined" shapes in Table I are especially interesting because they show (1) that streamlining does reduce the depth of scour and (2) that this effect is lost if the pier is not aligned with the flow. The latter result is most apparent if the length-width ratio of the pier is large.

The possibility of arresting the scouring action by means of an inerodible surface placed below the bed level was first studied with flat

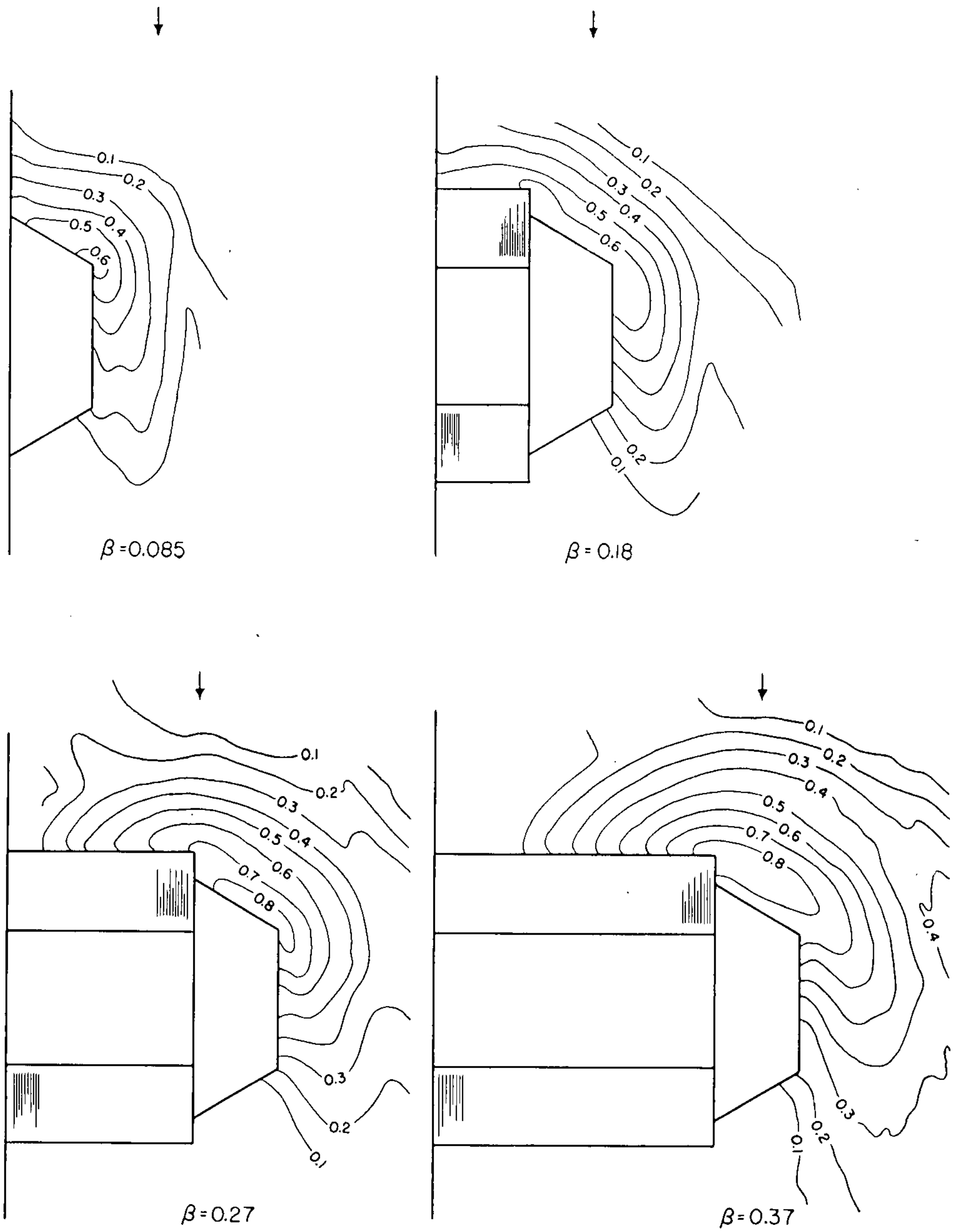


Fig. 21. Scour patterns around Model IV.

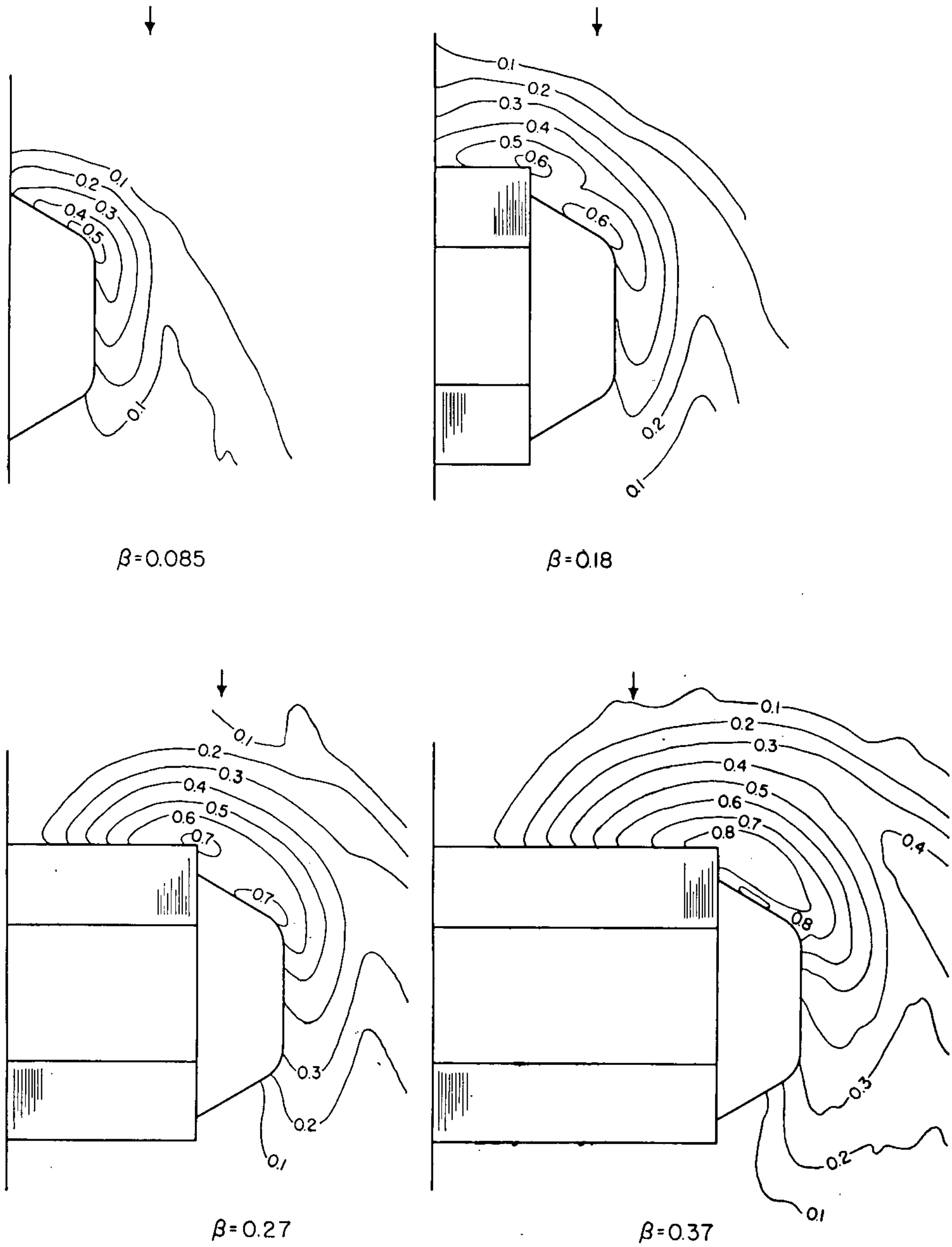


Fig. 22. Scour patterns around Model V.

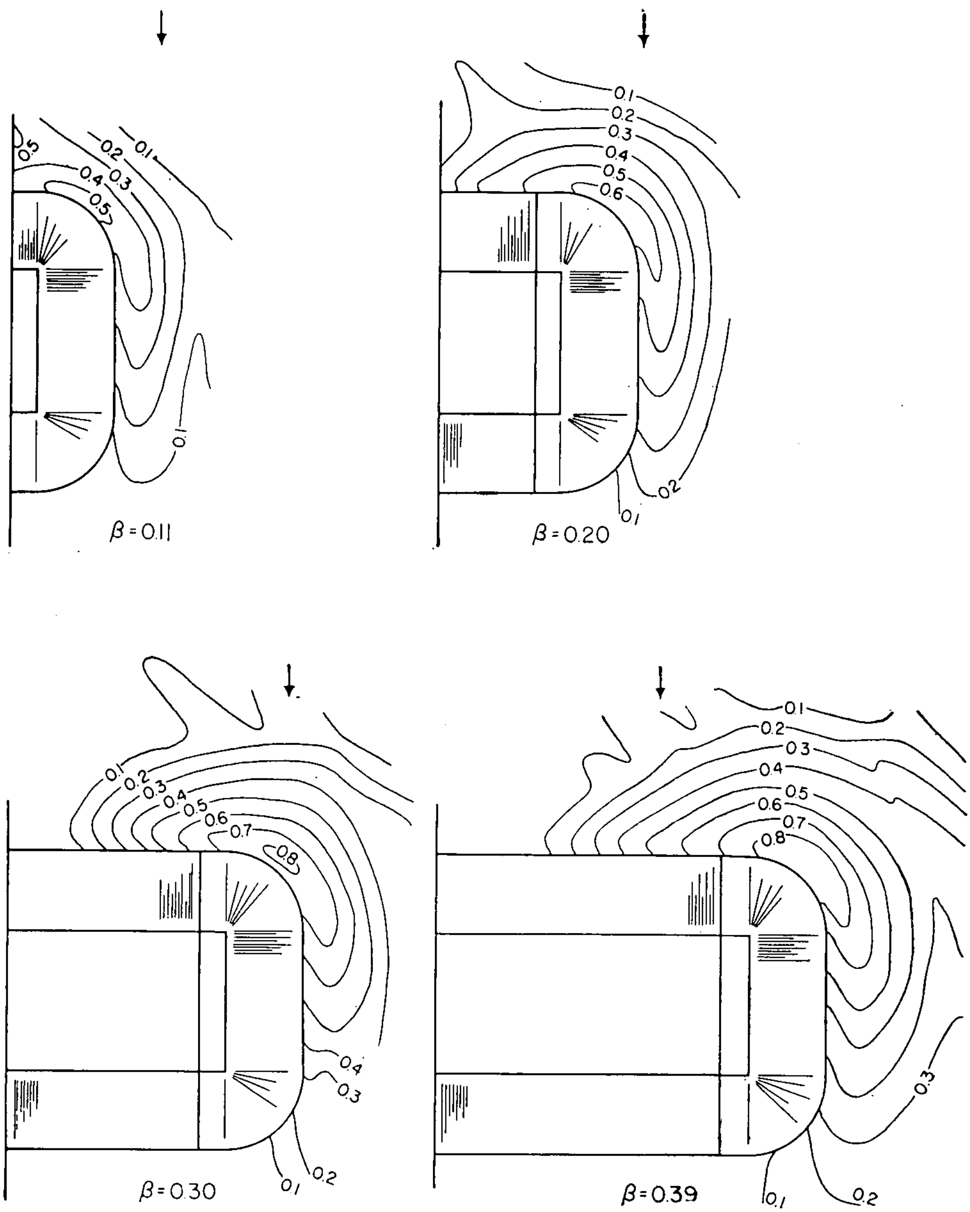


Fig. 23. Scour patterns around Model VI.

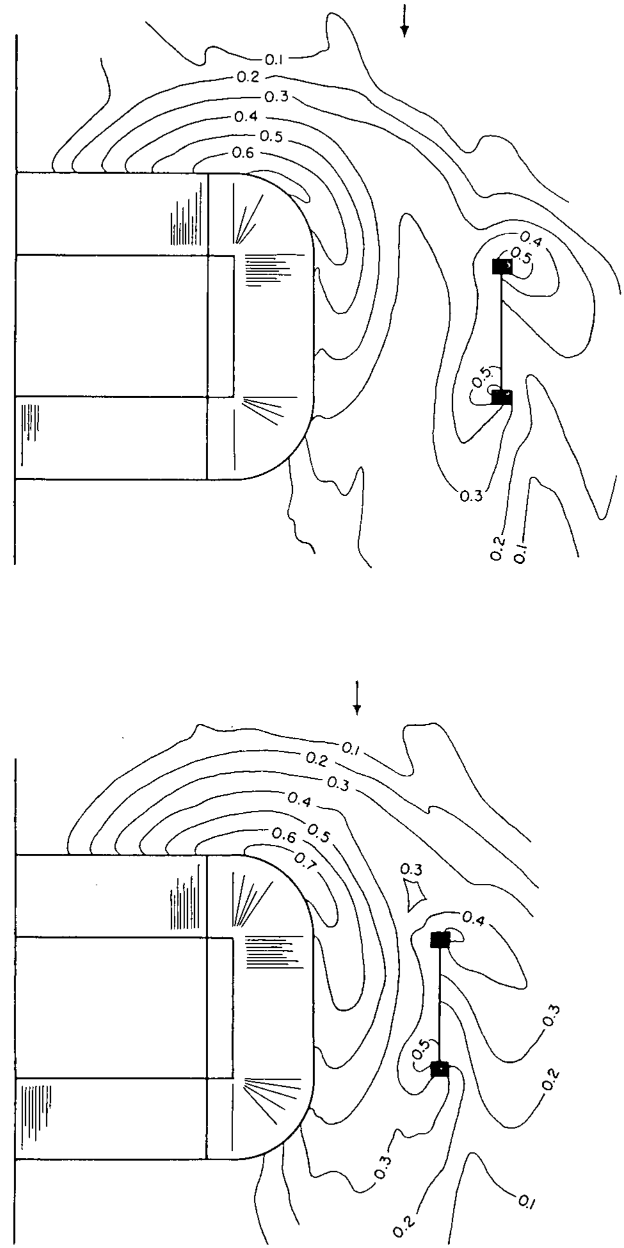


Fig. 24. Scour patterns around pier and abutment.

TABLE I

Scour depths around basic geometrical shapes

Angle of attack (degrees)	Length-width ratio	Relative scour depth[1] Model shape		
		Oblong	Elliptic	Lenticular
0	1:1	1.00		
	3:2	1.00		
	2:1	1.00	0.91	0.91
	3:1	1.00	0.83	0.76
10	3:1	1.02	0.98	0.98
20	3:1	1.13	1.06	1.02
30	3:2	1.13		
	2:1	1.17	1.13	1.13
	5:2	1.24		
	3:1	1.24	1.24	1.24

[1] Depth relative to depth of scour at single round shaft (0.2-foot diameter).

plates having a lateral extent greater than that of the scour hole formed. Clearly evident in the summary of results in Table II is the fact that the lateral extent of plate needed to prevent undermining is large if the plate is only slightly below the normal bed elevation. As would be expected, the required size of the plate decreased as the plate position was lowered.

A conical or concave collar, with the base of the collar resting on the horizontal plate, increased the lateral extent of the scour. A disk placed above the plate at the level of the sand bed reduced the extent of scour. The collars positioned so that the base was at the bed level, either upright or inverted, had about the same effect as the disk. The reduction is indicated in Table II.

Combinations of disks and of a disk and a collar were also tested and are summarized in Table II. Considerable reduction in the depth of scour resulted for some combinations, but equally noteworthy is the fact that even when these shapes were undermined the scour was never greater than for the pier by itself.

An enlarged base simulating a caisson construction with various transitions between pier shaft and caisson was also investigated. As shown in Table III the conical and convex transitions were not as effective as the abrupt transition. In fact, a lip extending up from the caisson was the best form tested. A combination of the caisson and a disk was also tried. In these tests it was found that the joint between the disk and the caisson must be tight. The effectiveness of the disk was almost completely destroyed if the joint was open.

TABLE II

Scour around ring arrestors

Flat plate set below stream bed

Relative elevation of flat plate[1]	Relative exposed area of flat plate[2]
0.18	21
0.37	14
0.56	6
0.74	5
1.00	1

Disks set at or below stream bed

Relative elevation[1]	Relative diameter[3]	Relative scour depth[4]
0.18	2.0	0.85
0.37	2.0	0.70
0.56	2.0	0.59
0.37	2.0	0.59
0.18	1.5	0.78
0	2.5	0.52

Collars above flat plate (Flat plate 0.1 foot below stream bed)

Type of transition	Relative area exposed[2]
Concave	17
Conical	24
Concave inverted	10
Conical inverted	13

[1] Elevation relative to depth of scour at single round shaft (0.2-foot diameter).
[2] Area relative to area of single round shaft.
[3] Diameter of disk relative to diameter of shaft.
[4] Depth relative to depth of scour at single round shaft.

TABLE III

Scour depths around caisson arrestors

Caisson

Type of transition	Relative elevation of caisson[1]	Relative scour depth[2]
Flat	0	1.00
Concave	0	1.04
Conical	0	1.07
Lipped	0	0.85
Flat	0.37	0.59
Concave	0.37	0.70
Conical	0.37	0.89
Lipped	0.37	0.48

Caissons with disk[3]

Type of transition	Relative elevation of caisson[1]	Relative elevation of disk[1]	Relative scour depth[2]
Conical	0	0	0.89
Conical	0	0.18	1.07
Conical	0	0.37	1.26
Flat	0	0.37	0.52

[1] Elevation relative to depth of scour at single round shaft (0.2-foot diameter).
[2] Depth relative to depth of scour at single round shaft.
[3] Disk not tight around caisson with conical transition.

Texas pile pier. The wide latitude in geometrical variation available to the designer is well illustrated by the pile pier tested. It is readily apparent that for this type of pier not only can the length-width ratio be varied, but also the spacing of the piles and the elevation of the bottom of the pier body relative to the bed level. Since this design is not in general use, a specific pier proposed for use in Texas was modeled as typical and was tested at different angles and relative elevations.

Under and around the pile pier a dish-shaped scour hole was formed as shown in Fig. 25. In addition a small scour hole was formed around each individual pile. The maximum depth of scour for various angles and relative elevations is summarized in Table IV. The effect of angle of at-

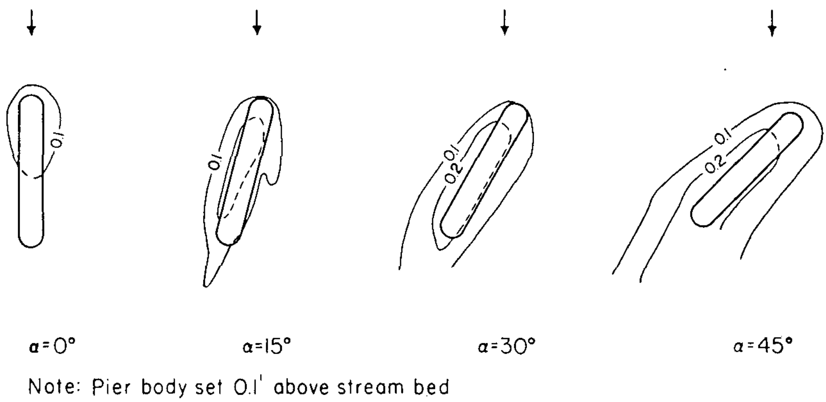


Fig. 25. Scour patterns around Texas pile pier.

tack can be seen to agree with all the previous results; the greater the angle, the deeper the scour. Even at 45°, however, with the pier body set at bed level the scour produced was less than for a solid pier of equal size projecting into the bed. For higher settings of the pier body the scour was even less.

Debris forms. A comparison of the effect of various shapes of debris cones collected at the front of the pier can be made on the basis of the photographs in Figs. 26-29. As a result of the added disturbance to the flow, the presence of the debris resulted in scour holes which were both deeper and larger in extent than the scour hole that formed around the clean, unencumbered pier. An exception occurred in the case illustrated by Fig. 29, because the debris was partially buried in the bed. In this case the debris acted also as a scour arrestor. However, for debris to attain such a position relative to the scour hole implies deeper scour in the past.

The greatest depths of scour, although not the greatest lateral extent, were obtained in the last two runs of the series, for which masonite boards,

TABLE IV

Scour depths around Texas pile pier

Elevation of pier body above bed[1] (feet)	Angle of attack (degrees)	Maximum scour depth[2] (feet)
0	0	0.22
	15	0.29
	30	0.36
	45	0.40
0.1	0	0.20
	15	0.26
	30	0.30
	45	0.33
0.2	0	0.18
	15	0.24
	30	0.30
	45	0.28

[1] Depth of flow 0.3 foot.
[2] Maximum occurs at an individual pile.

0.4 and 0.8 foot wide, respectively, were attached to the front of the pier, which was otherwise 0.2 foot wide. These two conditions simulate a very dense debris mass which has been silted to such an extent that it is virtually impervious.

Conclusions. No conclusions can be reached regarding absolute values of scour depths on the basis of these qualitative studies. Relative scour tendencies, however, can be assessed. The combined results of all the experiments support very definitely the viewpoint that the depth of scour is closely related to the degree of disturbance of the flow. Together with the length-width ratio, the angle between pier and flow is the most important geometrical characteristic of a pier. Only for piers truly aligned with the flow can streamlining be fully effective.

A solid extension of the pier in the form of a disk or enlargement of the base can reduce the depth of scour. The lateral extent of the arrestor, however, must be approximately that of the unarrested scour hole. Otherwise, the depth of scour may be as great as without the arrestor. The design, as well as the evaluation, of arrestors depends, therefore, on a means of predicting the absolute size of the scour which can be expected.

Debris, in effect, enlarges the pier and thus results in increased scour depths and areas. The difficulty in evaluating even qualitatively the effect of debris is that the permeability and the position are as important as the overall size of the debris mass.

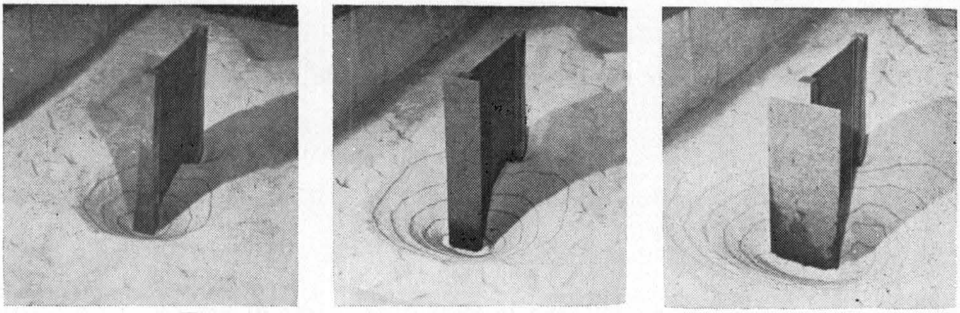


Fig. 26. Scour patterns due to solid debris.

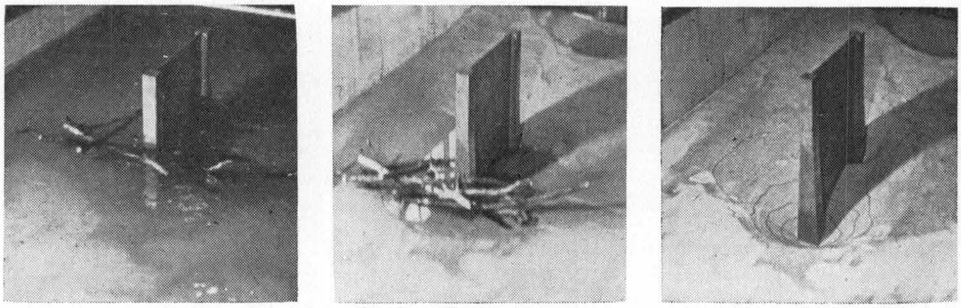


Fig. 27. Scour pattern due to floating debris.

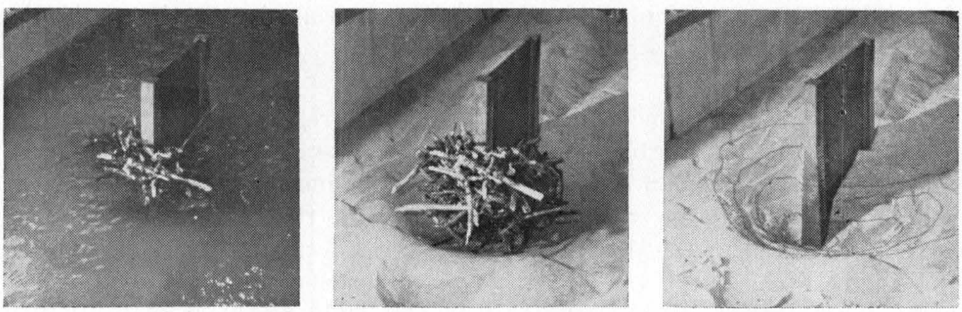


Fig. 28. Scour pattern due to submerged debris.

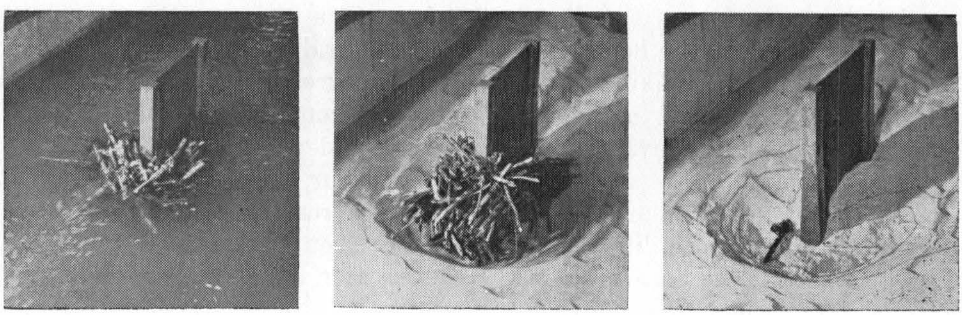


Fig. 29. Scour pattern due to buried debris.

The open structure of a design such as the pile pier can also be effective in reducing the depth of scour. The lower the pier body is placed, however, the less effective is the openwork of the piles. In addition, debris collecting against the piles may quite completely block the open area.

In the case of abutments the important finding is the location of the maximum scour at the upstream corner. Especially for the sheet-piling ring of stub abutments this point can easily be strengthened. Even for gravity abutments a deeper foundation at this point can be designed and built without much difficulty. The important geometrical characteristic of abutments was found to be the contraction ratio. The shape of the abutment proper was of less importance, although again a streamlined shape resulted in a reduced scour depth. Whereas there can be no question of the importance of the contraction in the case of abutment scour, it should be noted that a contraction in the field is generally much more complex than the contraction in the laboratory flume.

EFFECT OF VELOCITY, DEPTH OF FLOW, AND SEDIMENT SIZE

Development of the scour hole. Because the active process of scour is a result of the imbalance of supply and capacity, the development of the scour hole must be considered in terms of the general transport conditions. A description of the scour action will be of help in understanding the phenomenon. The initial condition of the bed in the laboratory experiments was a level sand surface. During the establishment of flow, some scour occurred around the pier, the spiral roller which formed at the base of the pier being the active agent of erosion. At the beginning of a run the velocity of the roller was such that the capacity for transport out of the scour hole was much greater than the supply to the scour hole from the flume. As the scour hole enlarged, the velocity of the spiral roller decreased, reducing the capacity for transport out of the hole. In addition, as the hole became progressively deeper, the volume which had to be removed for any given change in depth also increased. Thus, the rate of scour, as measured by the change in depth of the scour hole, decreased gradually from an originally high value.

That the rate of scour approached zero does not mean that the transport out of the scour hole also approached zero, but only that the transport out of the scour hole was almost equalled by the supply to the scour hole. The transport capacity in the flume, whether partially or fully developed, determined the supply and thereby assumed an important role in the later stages of development of the scour hole.

As stated before, the initial condition in these experiments was that of a level bed. After establishment of the flow, ripples appeared on the bed, gradually growing to fully formed dunes. The bed slope was largely molded by a large foreset dune which moved down from the elevator.

Until this dune reached the scour hole, the transport rate in the upstream portion of the flume was considerably less than normal. If the time required for the foreset dune to move down to the scour hole was relatively large, an oversize scour hole could develop, because the rate of supply to the scour hole was less than normal. The scour hole began filling when the dune formation and foreset dune had established the normal bed conditions. Complete equilibrium of transport and scour was then gradually attained, with both transport rate and scour depth fluctuating about their temporal mean values.

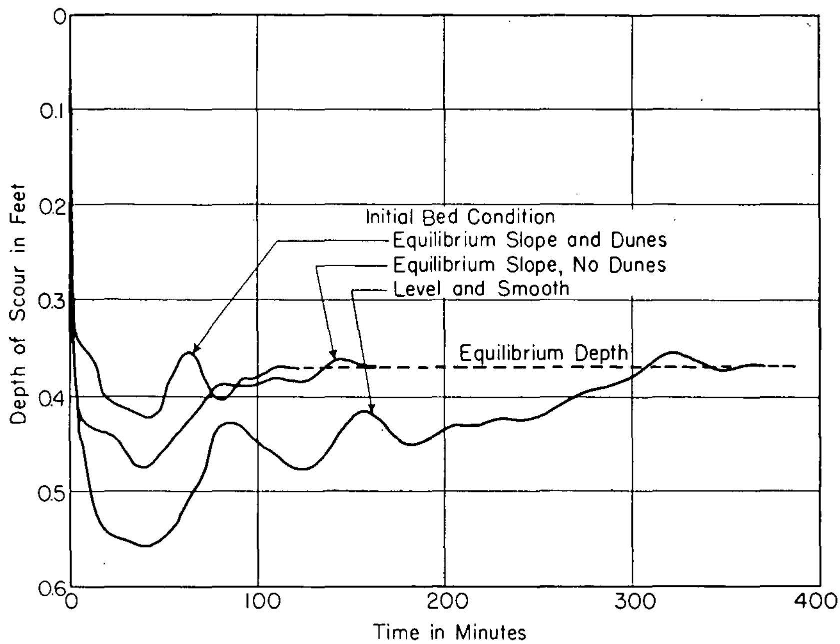


Fig. 30. Excess depth of scour.

That the excess depth of scour, which oftentimes occurred before equilibrium conditions were established, was a result of a need for both a required slope and a required bed configuration is shown in Fig. 30. The three runs represented in this figure were made consecutively under identical conditions except for the initial slope and form of the bed. The first run was started with the usual level bed. After the conclusion of the run the scour hole was filled and the bed smoothed to the existing equilibrium slope. A test was again run to completion, and the scour hole was then filled for the third run. The existing slope and bed configuration from the second run were the initial conditions for the third run. Under these changed conditions, the excess depth for the second run was less than for the first, and during the third run little more than the usual fluctuations of the equilibrium depth could be noted.

Equilibrium depth of scour. As used in this report, the term "equilibrium" does not have any implication of conditions being static. Equilibrium refers to the condition of balance wherein the amount of material removed from the scour hole is equalled by the amount of material supplied by the normal transport of the approaching flow. Because the sediment supply varied with the dune movement and the capacity in the scour hole varied with the fluctuations of the spiral roller, fluctuations occurred in

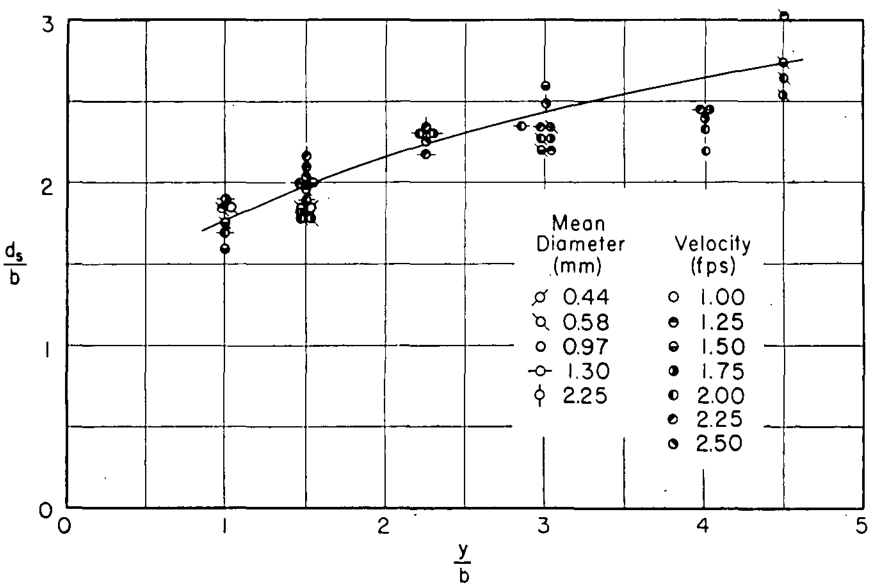


Fig. 31. Results of equilibrium scour study.

the depth of scour. Equilibrium conditions, therefore, were determined as a temporal average over time periods of 3 or more hours, depending on the irregularities of the scour trace.

The results of all the runs concerned with the effect of velocity, depth of flow, and sediment size are summarized in Fig. 31. In this plot no discernible influence can be ascribed to either the velocity of flow or the sediment size. The scatter of the experimental points is of the same order of magnitude as the experimental error and is not systematic. The depth of flow, however, does have an influence on the depth of scour, the latter increasing therewith.

The lack of effect of the velocity of flow and the sediment size, although contrary to customary belief, can be rationalized on the basis of the necessary balance between the transport capacity of the roller and the transport capacity of the approaching flow. Given an equilibrium scour condition with a certain velocity and depth of flow and a certain sediment size, if the mean velocity is increased, the angular velocity of the roller could be expected to increase proportionally. The velocity at the grain level both in the scour hole and in the upstream reach will also increase in the same proportion. The absolute rate of sediment movement would therefore increase, but the balance between capacity in the scour hole and supply to the scour hole required for equilibrium would still be maintained. Similarly, if the sediment size could be changed, the absolute rate of movement would change, but the balance between capacity and supply would still be maintained.

The effect of the depth of flow can be rationalized from a similar argument. The angular velocity of the roller is assumed to be little affected by an increase in the depth of flow as long as the mean velocity remains constant. The velocity at the grain level in the upstream reach would be decreased whereas the velocity at the grain level in the scour hole would then remain relatively unchanged. Because the rate of sediment movement

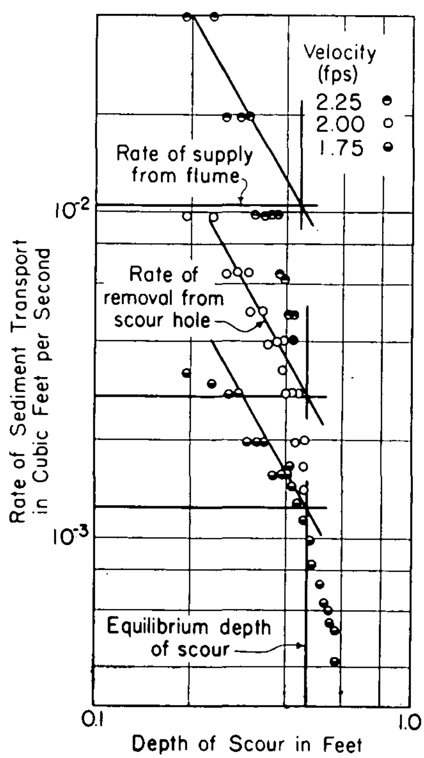


Fig. 32. Confirmation of logical concept of scour.

in the upstream reach would be decreased, the amount of sediment supply to the scour hole would be reduced. The excess capacity in the scour hole would, therefore, enlarge the scour hole until the capacity of the roller was reduced sufficiently to reestablish a balance between capacity and supply.

Several qualifications should be emphasized which restrict the generality of these findings. In all experiments the mode of movement of the sediment, both in the flume proper and in the scour hole, was as bed load. The flow conditions were such that the sediment movement was general throughout the flume. All runs were made with the flow in the subcritical ($F = V/\sqrt{gy} < 1$) range. Any application of these results must be confined, therefore, to flow conditions conforming to these restrictions.

This logical concept of scour is confirmed by the experiments summarized in Fig. 32. To obtain the points plotted there, the movable sand was confined to a small area around the pier, thus almost eliminating the transport into the scour hole. The variation of transport capacity in the scour hole was determined from the change in volume of the hole and plotted as a function of the depth of scour. The flume-transport values and the equilibrium depths are those obtained previously from standard test runs. Only the middle portion of the curves for rate of scour was used, because the experimental data were influenced at small depths by the unsteadiness resulting from flow establishment and at large depths by the small, but noticeable, rate of movement into the hole from the limited unpaved area of the bed. If the rate-of-scour curve through the middle portion of each set of data is extrapolated, it intersects the equilibrium depth of scour at a value corresponding to the normal rate of supply to the scour hole.

Conclusions. The results of this phase of the investigation eliminated the dynamic aspects of the scour process from consideration in determining the equilibrium depth of scour. At least as a first approximation the equilibrium scour depth, with certain qualifications as to the flow conditions, appears to be a function only of the geometry, i.e., the relative depth of flow, the shape of the pier, and the angle of attack.

Two important corollary conclusions based on this finding are immediately apparent. Insofar as the equilibrium depth of scour in a model is concerned, the velocity of flow, the sediment size, and the rate of sediment transport do not need to be scaled. Exactly the same depth of scour should result in the model, no matter what velocity or sediment is used, as long as there is general bed-load movement and the Froude number is everywhere less than unity. To insure similar flow patterns in the model and the prototype the usual criterion that the Reynolds number of the model be large enough for fully developed turbulence must be observed. In addition, the Froude number does not need to be the same in model and prototype as long as the flow in both is somewhat below a value of unity, preferably no greater than 0.8.

The other corollary must be qualified. To the extent that velocity of flow and sediment size (and, therefore, rate of transport and intensity of boundary shear) do not influence the equilibrium depth of scour, the scour depth of model and prototype should be related simply by the geometric scale ratio between model and prototype. Because secondary effects of velocity and sediment size which could not be detected in the limited range of the laboratory data may become important at larger scale, the validity of this conclusion can only be tested by model-prototype conformity studies. It is possible that excess scour depths such as occurred in the laboratory may also occur in the field. For this condition of scour the dynamic aspects of the flow are assuredly important, as are the duration and rate of rise of the flood flows.

EFFECT OF FLOW CONTRACTION

Multiple cylinders. The simplest geometric arrangement of a number of piers in a stream, right circular cylinders evenly spaced across the flow, was used to assess the effect of a contraction of the flow section on the depth of scour. In general, one would expect such a contraction to result in an increased depth of scour because of the increase in velocity in the section. Certainly, for a long contraction the bed will be scoured to a lower elevation than in the uncontracted flow. The latter is, in effect, a limiting case for the scour at the piers. A solution for scour in a long contraction was presented by L. G. Straub in 1939 at the First Hydraulics Conference at Iowa City. For the condition of general movement his solution reduces to

$$\frac{d_s}{y} = \frac{1}{(1 - \beta)^{9/14}} - 1$$

in which $(1 - \beta)$ is the ratio of the contracted to the uncontracted width.

The results of the present experimental study are plotted in Fig. 33, together with Straub's relationship for the long contraction. It will be noted that not until the depth of scour at the pier approaches the depth of scour which would occur in a long contraction does the contraction of the flow section appear to have any effect on the scour around the piers. If the spacing between piers is much larger than the width of the piers, the depth of scour is determined by the depth of flow relative to the width of the pier. As the spacing between piers becomes sufficiently small, the depth of scour is determined by the contraction ratio. Straub's solution seems to give a fair approximation to this limiting value. Between the two limits the depth of scour is a function of both the relative depth and

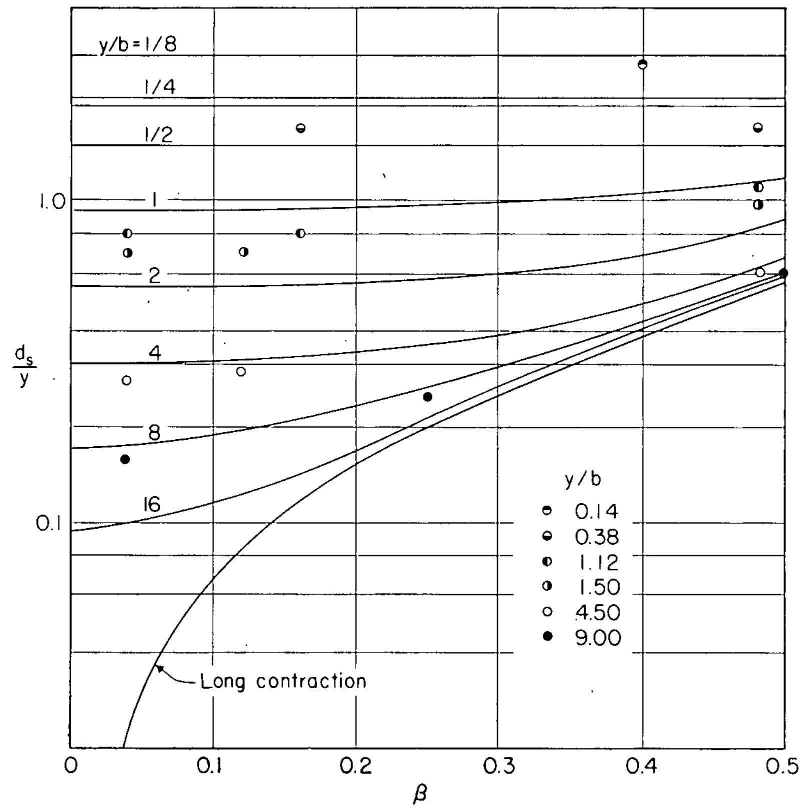


Fig. 33. Depth of scour around multiple cylinders.

the contraction. The value at which the contraction exerts a substantial effect depends on the relative depth, being less as the relative depth becomes greater.

Conclusions. In modern bridge design the effect of pier contraction will seldom be important. Even for a slender pier in a deep stream the contraction would have to be of the order of 10% for the depth of scour to be affected appreciably. It should be noted, however, that a distinction must be made here between the contraction due to the piers and the general contraction due to approach fills and abutments.

The results of this phase of the investigation also indicate that an entire river crossing need not be modeled in investigating the local scour at a pier. A true model of a bridge pier in a stream would necessitate either a very wide flume or a very small pier. Since the contraction in the model can be distorted without introducing serious errors, the distance between piers need not determine the flume width.

Finally, the fact that pier diameters of 0.1 to 2 feet resulted in a consistent pattern of scour depths lends credence to the proposition that scour depth can be considered as a function of the geometry alone.

MODEL-PROTOTYPE CONFORMITY

Field measurements. Two floods occurring during May and June, 1954, and one during August of the same year produced measurable depths of scour at the pier on the Skunk River at which the scour meter had been installed. The records from the scour meter were transcribed and the data are presented in Figs. 34, 35, and 36 as depth of scour versus depth of flow. On the curve for each of the floods, arrows indicate the temporal continuity of the stage-scour relationship.

Differences between the measurements on the rising and falling stages of the floods raise the question as to whether or not it was the equilibrium depth of scour that was measured. It is readily apparent that the scour at any given depth of flow would be the same during rising and falling stages if the equilibrium depth obtained at all times. It is also evident that, if the scour on the rising stage were less than the equilibrium value, the depth of the hole would continue to increase after the peak of the flood, resulting in a counterclockwise loop on the graph. Were an excess depth of scour to obtain during the rising stage, a clockwise loop would form if at the time of the peak the scour hole were being filled, but a counterclockwise loop would form if the hole were still being enlarged.

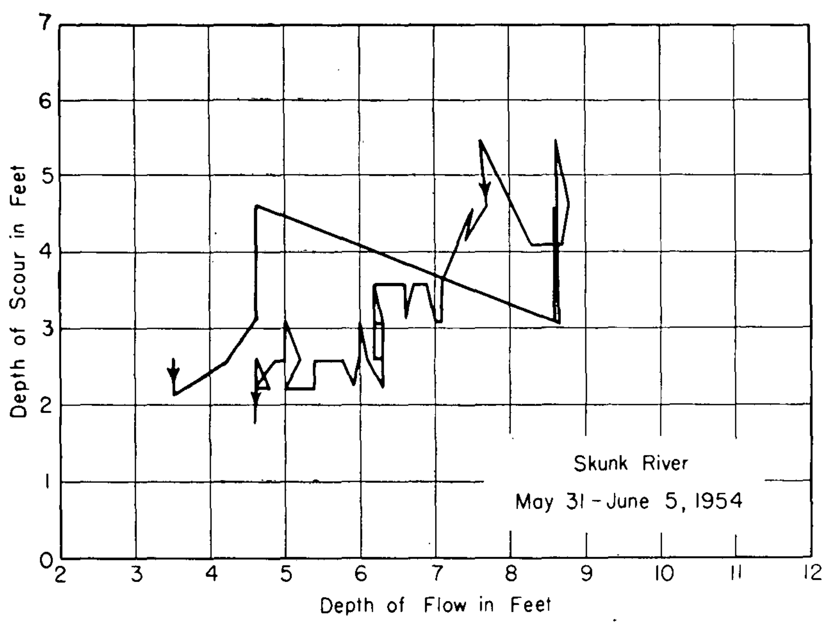


Fig. 34. Field measurements—May-June 1954.

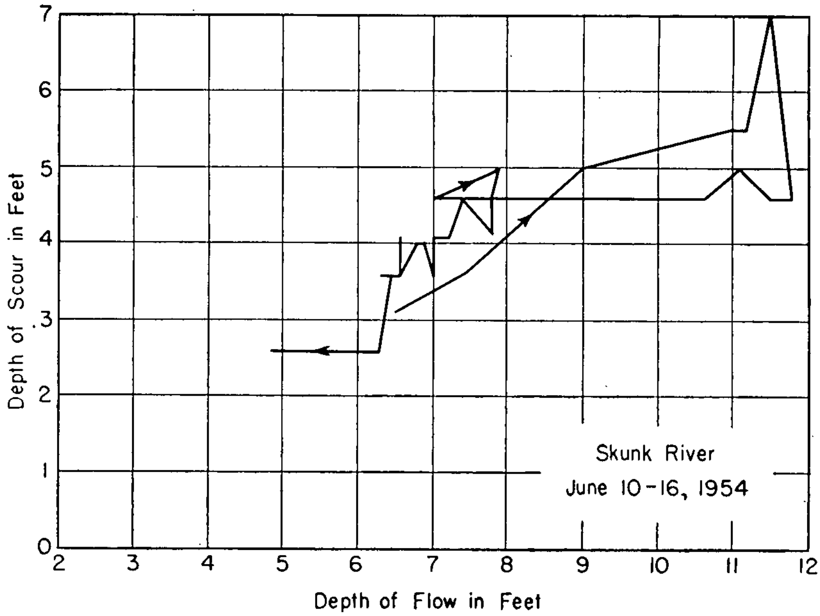


Fig. 35. Field measurements—June 1954.

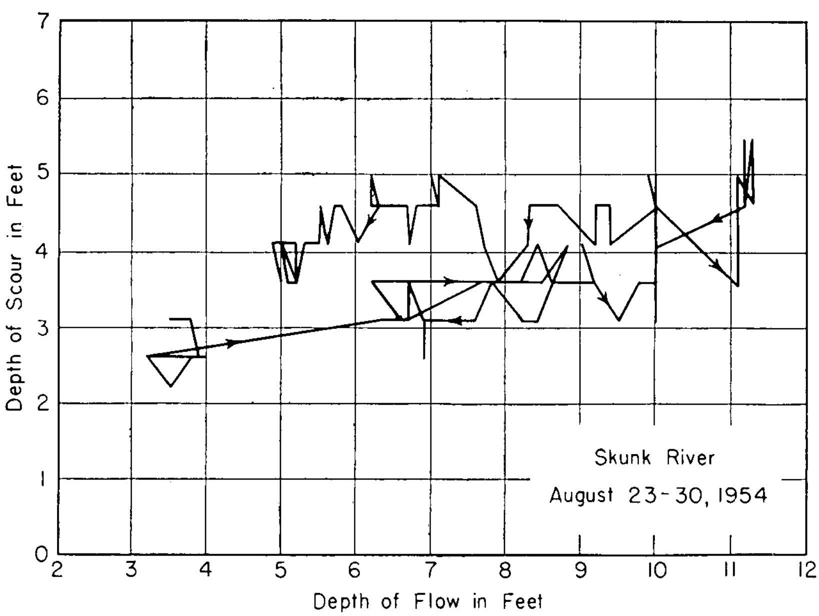


Fig. 36. Field measurements—August 1954.

Thus, in interpreting field data, superposition of the rising and falling traces would indicate that equilibrium depths obtained at all times; a clockwise loop would indicate excess depth of scour; and a counterclockwise loop would indicate that the scour had probably not reached the equilibrium depth, although excess depth in this case is a possibility. In the figures the two smaller floods can be seen to have counterclockwise loops near the peak and the largest flood to have a clockwise loop. Near the peak of the largest flood, however, the stage indicator reached its limit and the depth of flow had to be estimated by extrapolation. The evidence indicating excess depth of scour is, therefore, not reliable. Moreover, the difference between rising and falling traces is of a magnitude comparable to the precision of the scour depth measurement. It would appear on the basis of these measurements that the depth of scour measured in the field was approximately equal to the equilibrium value.

Models of field installation. Figure 37 contains, besides the prototype scour curves, the experimental data obtained from runs with 1:12 and 1:24 models of the prototype pier, through which a mean curve could be drawn. Scour depths in the models were considered to have geometric similarity to the prototype and were multiplied by the appropriate scale ratio. A mean curve through the model results would lie slightly above the curves obtained from the prototype. This could indicate simply that the depth of scour measured in the field was less than the equilibrium depth that

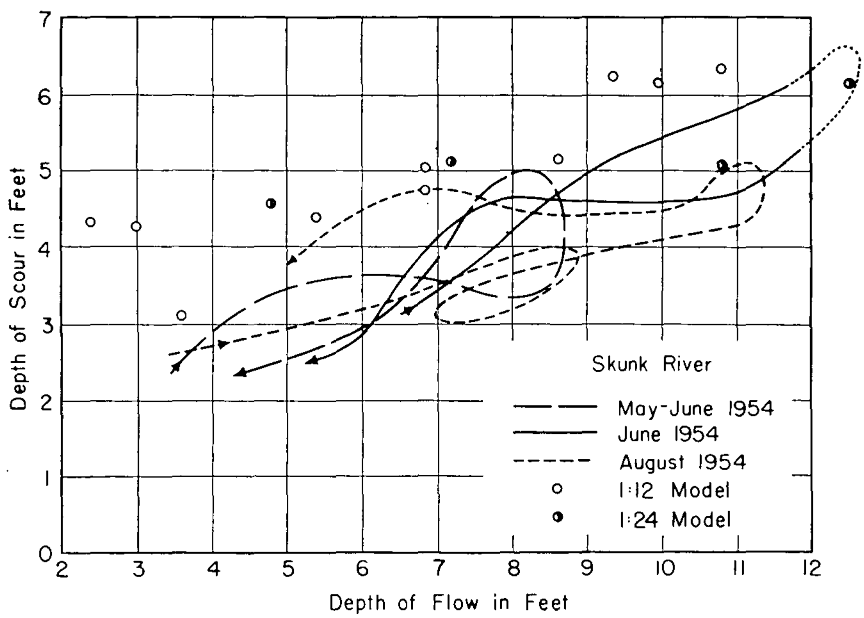


Fig. 37. Model-prototype conformity.

would have resulted if the time periods of the flood had been greater. It could also mean, however, that the relationship between model and prototype scour depths is more complex than the assumed geometric scale ratio.

Conclusions. Although more model-prototype conformity studies are needed, this first example represents a major step in the solution of the bridge-scour problem. The field adaptation of the scour meter provides the means for securing two kinds of information. Measurements at sites of simple geometry will permit more model-prototype comparisons which are needed for the laboratory results to be used with confidence. Measurements at sites of extremely complex geometry can build a fund of knowledge for conditions outside the range of any design procedure that may be evolved.

On the basis of all the accumulated evidence, both laboratory and field, it appears that the depth of scour can be regarded as a function of the geometry alone and that the scour depth can be treated like any other length in the comparison of model and prototype. Furthermore, it appears likely that the equilibrium depth of scour is the maximum depth which will occur, except possibly in the case of flash floods with an extremely short time to peak.

APPLICATION TO DESIGN AND MAINTENANCE

General qualifications. The final objective of this study is to find means whereby the probable depth of scour around piers and abutments can be predicted by the bridge designer. An understanding of the process of scour and the effect of each of the many contributing factors is, of course, necessary, if the method of prediction is to be used with confidence. The preceding pages contain descriptions of the scour process and of the effect of many of the contributing factors. A concept of scour is presented which explains, at least qualitatively, the effect of these factors. Although more measurements are needed, in the laboratory and especially in the field, before the prediction of scour can be made with complete confidence, sufficient evidence has been accumulated so that design criteria can be proposed. These criteria should be used with caution and with due regard for the qualifications stated.

The scour referred to is the local scour around a pier or abutment and is measured from the general level of the bed at the bridge crossing. If the bridge proper spans a contraction, whether natural or man-made, of the flooded stream, general scour may occur in addition to the local scour. (This general scour is the subject of the continuing investigation.) Moreover, general degradation may occur due to stream straightening or an upstream dam. All lowering of the stream bed should be considered additive in determining the requirements for a bridge foundation.

Local scour at piers. All of the available data were adjusted to scour conditions at a zero angle of attack for a rectangular pier and plotted non-dimensionally as equilibrium depth of scour against depth of flow with the width of pier at stream bed used as the repeating variable. The design curve presented in Fig. 38 was drawn somewhat conservatively and represents the predicted local depth of scour for a rectangular pier, the sides of which are aligned with the current, as a function of the depth of flow and the width of the pier at stream bed.

In the laboratory investigation it was found that the angle between the axis of the pier and the direction of the current is more important than any other geometric detail of solid or webbed piers. If the pier is skewed to the current, the scour depth predicted from Fig. 38 must be multiplied by a factor greater than unity. Values of this factor $K_{\alpha L}$ are presented in Fig. 39 as a function of the angle of attack and the length-width ratio of the pier at stream bed. In estimating the angle of attack it should

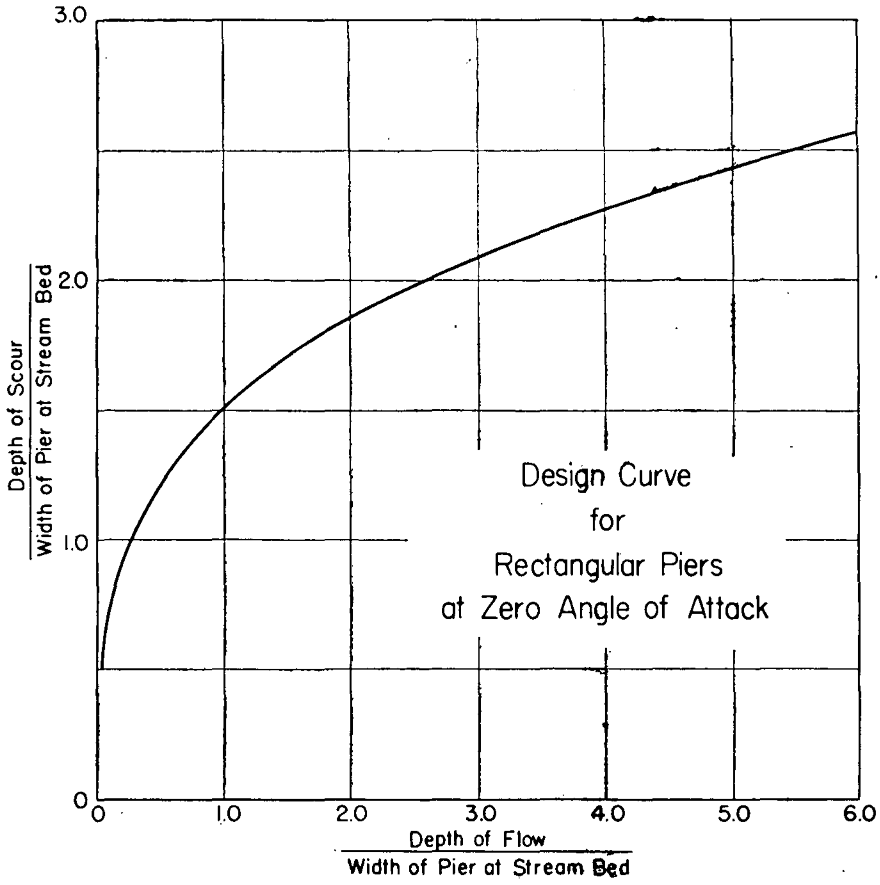


Fig. 38. Basic design curve for depth of scour.

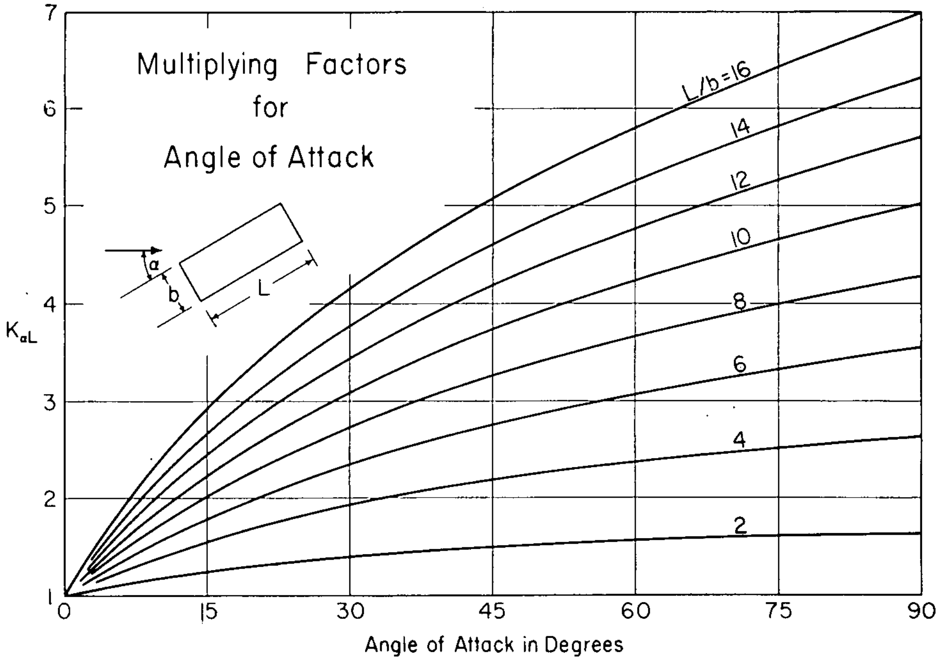


Fig. 39. Design factors for piers not aligned with flow.

TABLE V

Shape coefficients K_s for nose forms

(To be used *only* for piers aligned with flow)

Nose form	Length-width ratio	K_s
Rectangular		1.00
Semicircular		0.90
Elliptic	2:1	0.80
	3:1	0.75
Lenticular	2:1	0.80
	3:1	0.70

be realized that the direction of the flood flow is likely to be influenced by the valley direction if there is overbank flow. Furthermore, the presence of abutments and road fills will affect the direction of flow, especially at the piers closest to the bank.

As the angle of attack changes from zero, the point of greatest scour shifts from the front of the upstream end of the pier to the side exposed to the oncoming flow. On piers consisting of two shafts and a web the maximum scour will then occur at the downstream shaft. In this connection it is well to recall that for double-shaft piers without web the depth of scour may be greater at the downstream shaft than at the upstream shaft at some angles of attack, but that the maximum depth will never be as great as that for the comparable webbed pier.

Only if the pier is aligned with the flow at all stages can the depth of scour be decreased appreciably by streamlining the pier. Table V presents multiplying factors K_s for a number of shapes that have been tested for relative scour. These factors should be applied to the depth of scour found from Fig. 38, but must *not* be used if the multiplying factors of Fig. 39 are employed.

Several words of caution should be noted by anyone using these suggested design values. All tests have been confined to the subcritical range ($F = V/\sqrt{gy} < 1$); the design criteria should not be applied to supercritical flow conditions, because the flow pattern will be entirely different. Another prerequisite for the use of these design criteria is the existence of general movement of the bed material as bed load. It is unlikely that an alluvial river in flood stage would not exhibit general movement of the bed in the main channel. There may not be general movement on the flood plain, however, so the procedure is not applicable to the design of relief bridges. Moreover, it is possible that the bed material may move primarily as a suspended load. Therefore, for fine bed materials the design values should be used with caution, and for very fine bed materials the design procedure is not recommended at this time.

Under certain circumstances scour depths in excess of the equilibrium depth of scour can occur. Because this is a phenomenon associated with unsteady flow conditions, it is recommended that the predicted depth of scour be increased by at least fifty percent if the river is subject to flash floods.

Debris is likely to accumulate on any bridge pier during a flood. If the debris is removed periodically as a routine measure, it will not tend to become a dense, enlarged nose on the pier. A permeable mass of brush and trees, as accumulated during a single flood, will result in a scour depth somewhat greater than that at a clean pier, but still less than a solid enlargement of the pier. If practicable, removal of debris during major floods would be a prudent measure.

Scour conditions at abutments. Tentative design criteria, such as have been presented for piers, cannot at this time be proposed for abutments. In the course of the investigation it became clear that scour at an abutment is related to the geometry of the whole bridge crossing and not simply to the geometry of the abutment itself. Although not even tentative values for the depth of scour can be suggested, the laboratory tests on abutments definitely indicated that the upstream corner of the abutment is subject to the deepest scour. Field observations of flood damage, reported by D. E. Schneible of the Bureau of Public Roads in the August 1954 issue of *Better Roads*, confirm this finding. Added protection at the upstream corners, in the form of deeper sheet piling for the stub abutment or deeper footings for the gravity abutment, would obviously increase the safety of the structures. Furthermore, because scour is the result of an imbalance between capacity and supply, a reduction of the distortion of the flow pattern adjacent to the abutment should result in less scour. In the general layout of the bridge crossing, therefore, an effort should be made to keep to a minimum conditions which would cause an increase of velocity in the vicinity of the abutments.

Riprap protection. Riprap is commonly considered as a method of reducing scour. Although this phase of the general problem has not been studied explicitly, the understanding of the scour process which has been gained does permit a few qualitative recommendations. Riprap should be placed well beneath the stream bed—it should not be dumped around the pier or abutment on top of the stream bed. The greatest protection will be obtained if the riprap material is placed just above the level that is considered dangerous to the structure. The capacity and competence of the roller in the scour hole thus allowed to form are the least possible and the riprap is, therefore, most likely to remain in place. The size of the riprap material should be considerably larger than any material in the stream bed—preferably larger than any material that can be moved by the flow between the piers. A mixed material with minimum voids will give the greatest protection to the underlying material, but the fine fraction of the mixture should not be so small as to be readily removed by the action of the flow.

Examples of design procedure. The successful use of the recommended design procedure to foretell the local depth of scour at a bridge pier depends on an ability to predict the flow pattern of the stream and on a knowledge of the probability of occurrence of flood stages. For selected stages of the stream the pattern of flow must be visualized in order to estimate the angle of attack between the flow and the piers. A change in the flow pattern with stage, moreover, may indicate a shifting in the stream-bed configuration which is characteristic of the stream itself and must be considered in determining the reference level for the local depth of scour at the pier. The river stages which should be studied are those characterized by conditions conducive to large depths of scour and by a probability of occurrence high enough to constitute a real danger.

A stage-frequency relationship will not usually be available but can be synthesized from discharge-frequency data such as those presented in Bulletin No. 1 of the Iowa Highway Research Board, *Iowa Floods, Magnitude and Frequency,* and from a computed stage-discharge relationship. The stage-discharge relationship for a reach having channel control can be obtained from the Manning equation, after area and slope values have been determined and a value of the Manning n has been estimated. Otherwise, backwater curves may have to be computed from a downstream control section.

The choice of the design flood is, of course, a matter of economics, as the increased cost of a bridge made safe for a larger flood must be balanced against the decreased chance of occurrence of the flood during the anticipated useful life of the structure. The importance of such economic studies makes a short review of flood probabilities pertinent at this point.

A flood having a chance of occurrence p in any one year will have in n years a probability P(x) of occurring exactly x times such that

$$P(x) = \frac{n!}{x!\,(n-x)!} p^x (1-p)^{n-x}$$

The probability of occurring one or more times in n years is then the same as $1 - P(0)$ or

$$P(\text{at least once}) = 1 - (1-p)^n$$

This probability, of at least one occurrence of the design flood, is of greatest interest in connection with the problem of bridge-pier design, because a single flood of the presupposed magnitude will presumably destroy the bridge or at least severely damage it.

The chance of 25-, 50-, and 100-year floods occurring at least once in periods of 25, 50, and 100 years is summarized in the accompanying table.

	Chance of Occurrence		
Frequency (yrs)	25	50	100
Period (yrs)			
25	0.64	0.40	0.22
50	0.87	0.64	0.39
100	0.98	0.87	0.63

The tabular values are based on the assumption that floods are a random phenomenon and that the chance of occurrence in any one year of a flood of 25-year frequency or recurrence interval is 0.04.

The following examples have been chosen to illustrate typical problems that might be encountered in the field. To obtain clarity and emphasis on method, the examples are presented as simply and schematically as possible without sacrificing reality of conditions. In the numerical calculations, the results of which are summarized in a table for each design example, the lengths and widths of the piers involved were chosen to be reasonably commensurate with each geometry. The depth-of-scour values have been rounded off to the nearest foot, except in the first example, in which small differences require values to the nearest 0.5 foot.

Example No. 1.

The leveed stream of Fig. 40 represents the simplest bridge-site geometry that can be encountered. Let it be assumed that this original simplicity is to be retained, so far as possible, by bridging the stream at right angles to its course, burying the abutments in the levees, and placing a single round-nosed pier having a width of 4 feet in the center of the stream. The depth of flow y is equal to the stage, since the stage has been referred to the stream bed and the stream is not contracted.

From Fig. 38 the relative depth of scour d_s/b for a rectangular pier aligned with the flow can be obtained for various relative depths y/b.

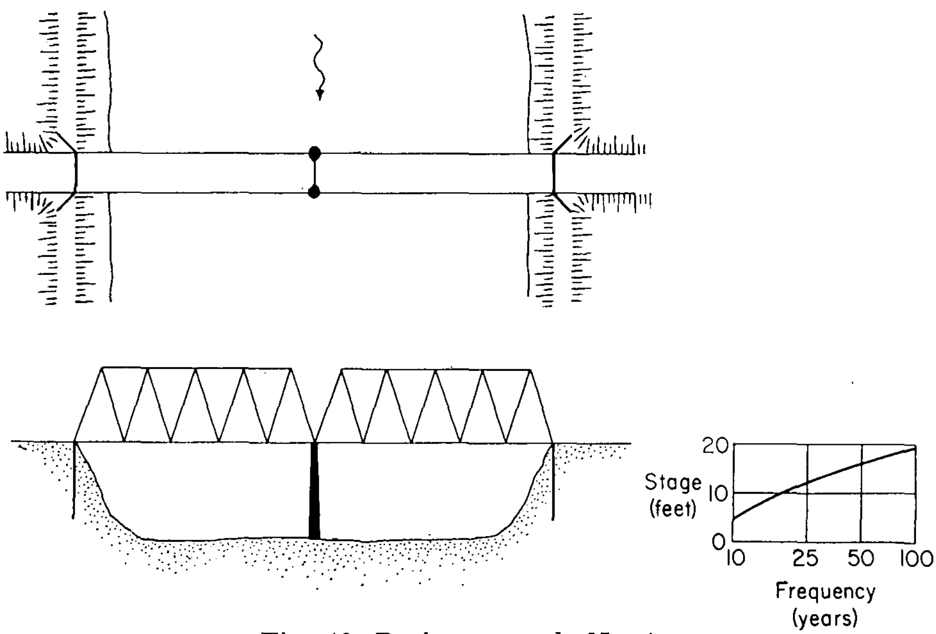


Fig. 40. Design example No. 1.

Since the pier is aligned with the flow, there is no angle-of-attack correction. The circular nose form, however, permits the use of a shape correction factor K_s of 0.9 from Table V. The depth of scour is then $d_s = K_s b\, d_s/b = 0.9 \times 4\, d_s/b$ and is shown in the last line of the accompanying table.

Frequency (yrs)	25	50	100
y (ft)	12.0	15.8	19.4
y/b	3.00	3.95	4.85
d_s/b	2.10	2.25	2.40
d_s (ft)	7.5	8.0	8.5

The local depth of scour as computed in this case should be the depth of scour to be expected. In other cases, however, there may be general scour or general degradation of the stream bed which must be added to the local depth computed.

Example No. 2.

Modification of the first example in two particulars is sufficient to illustrate further applications of the design procedures. Assume that the

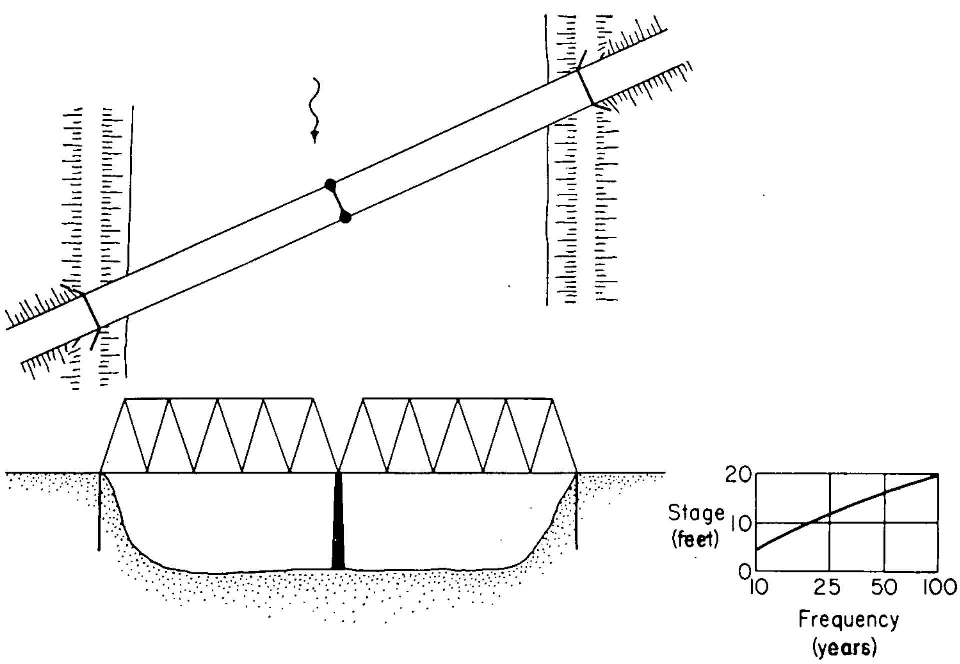


Fig. 41. Design example No. 2.

leveed stream of the first problem is now (Fig. 41) bridged at an angle so that an angle of attack other than zero exists between the flow and the pier. Moreover, two types of piers are to be compared: a standard dumbbell with a full web, and a hammerhead in which the bridge seat is cantilevered out from the pier shaft. Each of the piers is assumed to be 4 feet wide; the dumbbell pier to be 36 feet long, the hammerhead pier to be 12 feet long.

	Dumbbell Pier			Hammerhead Pier		
Frequency (yrs)	25	50	100	25	15.8	100
y (ft)	12.0	15.8	19.4	12.0	3.95	19.4
y/b	3.00	3.95	4.85	3.00	2.25	4.85
d_s/b	2.10	2.25	2.40	2.10	24	2.40
α (deg)	24	24	24	24	3	24
L/b	9	9	9	3	14	3
$K_{\alpha L}$	2.65	2.65	2.65	1.60	50	1.60
d_s (ft)	22	24	25	13	1.60	15

The first part of the computations which are summarized above are exactly the same as in the previous example. In this case, however, the relative scour depth d_s/b for a rectangular pier at zero angle of attack is multiplied by an angle factor $K_{\alpha L}$ which depends on the angle of attack and the length-width ratio. This correction factor, which is always greater than unity, is obtained by entering Fig. 39 with the appropriate values of the angle of attack α and the length-width ratio L/b. The depth of scour is then $d_s = K_{\alpha L}\, b\, d_s/b$. Note that a correction for pier shape is now not permissible.

With the predicted values of the depth of scour, an economic comparison of the two piers in this example can be made. A comparison can also be made with the aligned pier of the first example by considering the cost of skewing the superstructure of the bridge on the pier.

Example No. 3.

The case of a stream that varies in flow pattern with stage but is still confined within leveed banks is shown in Fig. 42. Although there is very little, if any, contraction of the stream, a lowering of the stream bed at Pier No. 1 should be anticipated, as the chute carries a larger percentage of the flow at higher stages. Therefore, the depth of flow is equal to the difference between the stage and the stream-bed elevation. The amount of lowering of the bed has been estimated as shown in the computation summary, as has the angle of attack at various stages. The piers are considered to have a 5×40-foot cross section.

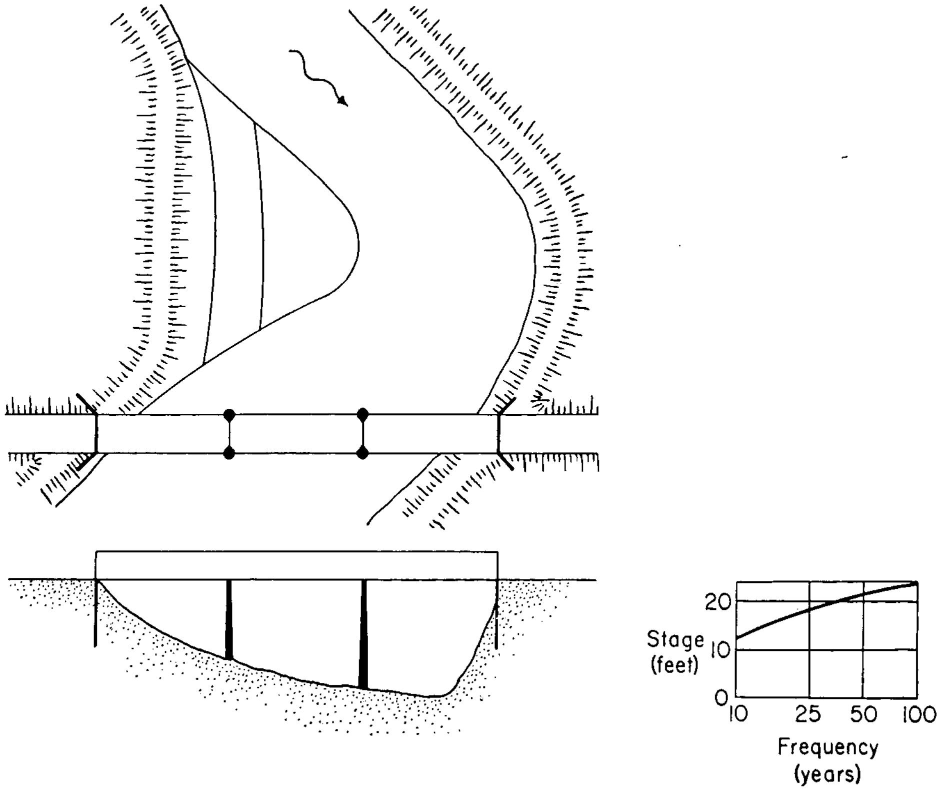


Fig. 42. Design example No. 3.

	Pier No. 1			Pier No. 2		
Frequency (yrs)	25	50	100	25	50	100
Stage (ft)	18	21	24	18	21	24
Stream-bed elevation (ft)	8	4	2	2	2	2
y (ft)	10	17	22	16	19	22
y/b	2.00	3.40	4.40	3.20	3.80	4.40
d_s/b	1.85	2.20	2.35	2.15	2.25	2.35
α (deg)	57	33	16	42	42	42
$K_{\alpha L}$	3.60	2.85	2.10	3.20	3.20	3.20
d_s (ft)	34	32	25	35	— 36	38
Scour-hole elevation (ft)	— 26	— 28	— 23	— 33	36	— 36

The values of d_s/b for the rectangular pier at zero angle of attack and of $K_{\alpha L}$ for the various angle-of-attack corrections were obtained from Figs. 38 and 39, respectively. The last entry, the elevation of the bottom of the scour hole, shows that the scour at one pier may be markedly different from that at another on the same bridge and that, if the angle of attack decreases with increasing stage, the maximum stage does not necessarily result in the maximum depth of scour.

Example No. 4.

In this example the relatively simple geometry of the valley shown in Fig. 43 includes the feature of a flood plain. Up to a stage of 20 feet the stream will remain within its channel, but above this stage it will go over its banks and flood its valley. Up to bank-full stage, therefore, the piers will remain aligned with the stream, but at all higher stages the flow pattern will be changed, because the approach fills will interfere with the normal flow of water on the flood plain and angles of attack other than zero will occur between flow and piers. Depending on the percentage of flow on the flood plain and the proximity of the pier to the abutment, the first pier may be subjected to flow at a large angle of attack. The flow pattern will be changed less in the vicinity of the other piers, but they also will be subjected to some change in angle of attack.

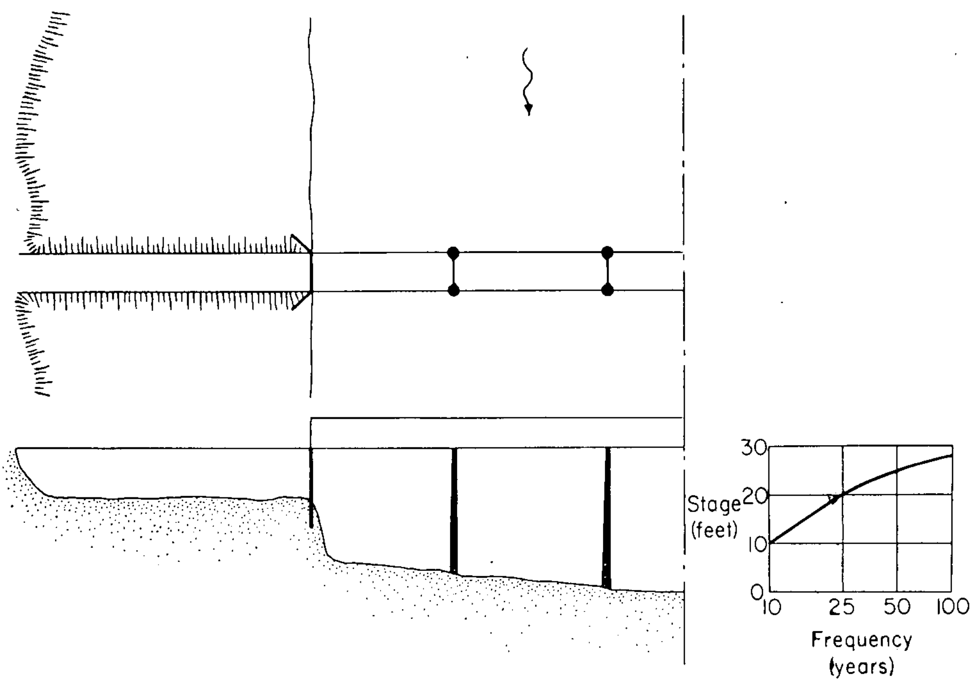


Fig. 43. Design example No. 4.

	Pier No. 1			Pier No. 2		
Frequency (yrs)	25	50	100	25	50	100
Stage (ft)	20	25	28	20	25	28
Stream-bed elevation (ft)	4	4	4	1	1	1
y (ft)	16	21	24	19	24	27
y/b	2.67	3.50	4.00	3.17	4.00	4.50
d_sb	2.05	2.20	2.30	2.15	2.30	2.35
α (deg)	0	15	30	0	1	5
$K_{\alpha L}$	1.00	1.80	2.35	1.00	1.05	1.30
d_s (ft)	12	24	32	13	14	18

The pier widths and lengths are assumed as 6 and 36 feet, respectively. The angle of attack for different stages has been estimated in the computation summary above, permitting the evaluation of the angle correction factor $K_{\alpha L}$. The depth of flow has been taken as the difference between the stage and the stream-bed elevation. It should be noted that the last line in the summary is the local depth of scour d_s, to which must be added an estimated depth of general scour due to the contraction. This general scour will be centered on the abutment for a simple geometry such as this, but would probably cause some lowering of the stream bed, at least at Pier No. 1.

Example No. 5.

The geometry of the bridge crossing in this example (Fig. 44) probably approaches the usual field condition more closely than any previous one. If there is considerable vegetation on the flood plain, the problem of predicting flow patterns could be extremely difficult—especially since the character of the vegetal cover may change completely in 25 or 50 years. Nevertheless, estimates of the angle of attack must and can be made for various arbitrary conditions. Once the stage and flow-pattern estimates are made, the procedure is exactly the same as for the previous examples.

If the changes in angle of attack are assumed as shown in the table and the pier size is again 6 feet by 36 feet, the computed values for each pier will be as indicated.

In this example again the last line is the local scour at the pier; any general scour or degradation must be added to this value to obtain the total lowering of the stream bed around the pier. At Pier No. 1 it may be noted that the scour decreases with stage because the decrease in angle more than compensates for the increase in depth. For other geometries it is quite possible that the angle of attack would increase with stage and the depth of scour would then increase markedly.

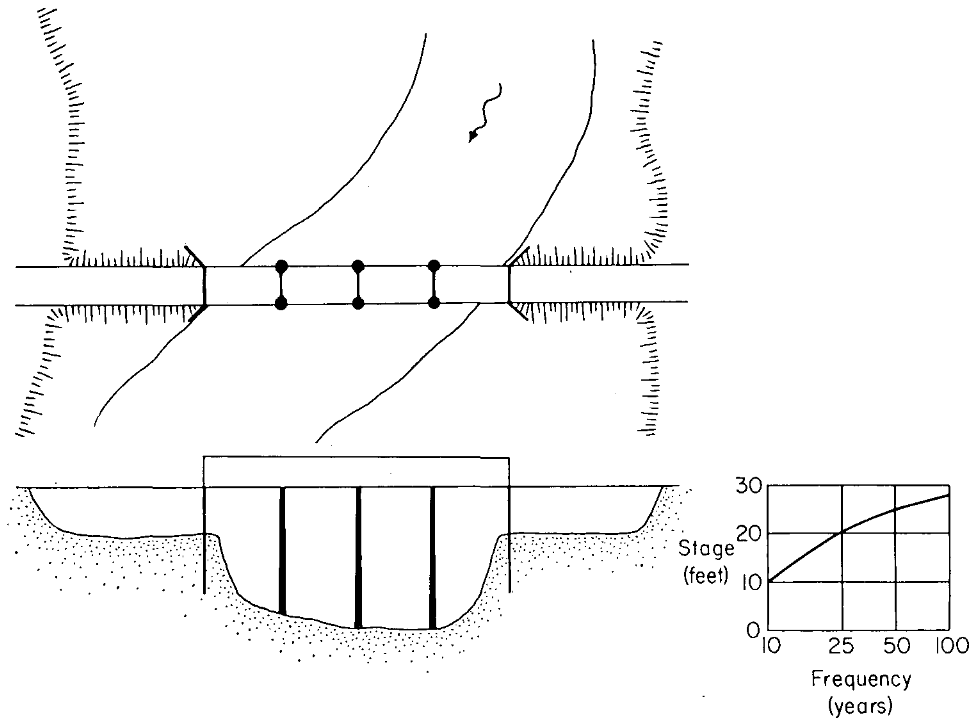


Fig. 44. Design example No. 5.

	Pier No. 1			Pier No. 2			Pier No. 3		
Frequency (yrs)	25	50	100	25	50	100	25	50	100
Stage (ft)	20	25	28	20	25	28	20	25	28
Stream-bed elevation (ft)	4	4	4	1	1	1	0	0	0
y (ft)	16	21	24	19	24	27	20	25	28
y/b	2.67	3.50	4.00	3.17	4.00	4.50	3.33	4.17	4.67
d_s/b	2.05	2.20	2.30	2.15	2.30	2.35	2.18	2.32	2.37
α (deg)	50	30	10	50	40	25	50	50	50
$K_{\alpha L}$	2.85	2.35	1.55	2.85	2.65	2.20	2.85	2.85	2.85
d_s (ft)	35	31	21	37	37	31	37	40	40

Example No. 6.

Figure 45 shows a pile-trestle bridge spanning a small tributary in the valley of a major stream and its location relative to the main crossing. The design procedure used in the previous examples could be used here as long as only the small tributary flows under the trestle bridge. Once the main stream goes out of its banks, however, the trestle will act as a relief bridge for the whole valley and the flow of the tributary will be lost in the general overbank flow.

One of the basic premises of the analysis of bridge-pier scour presented in this report is that the sediment supplied to the scour hole has the same characteristics as the material being scoured from the river bed. Sediment transportation, especially of coarser material, will normally be inhibited on the flood plain because of the vegetal cover, and only fine material may be supplied to the scour hole at a relief bridge. Scour at such a bridge, therefore, should be classed as a case of clear-water scour—basically the same, except for the difference in geometry, as the scour below the outlet of a dam. The limiting condition for this type of scour is a boundary shear equal to the critical tractive force for the beginning of movement, which obviously involves both the sediment size and the velocity of flow. The general concepts of bridge-pier scour as described herein are, therefore, not applicable to the conditions of this example.

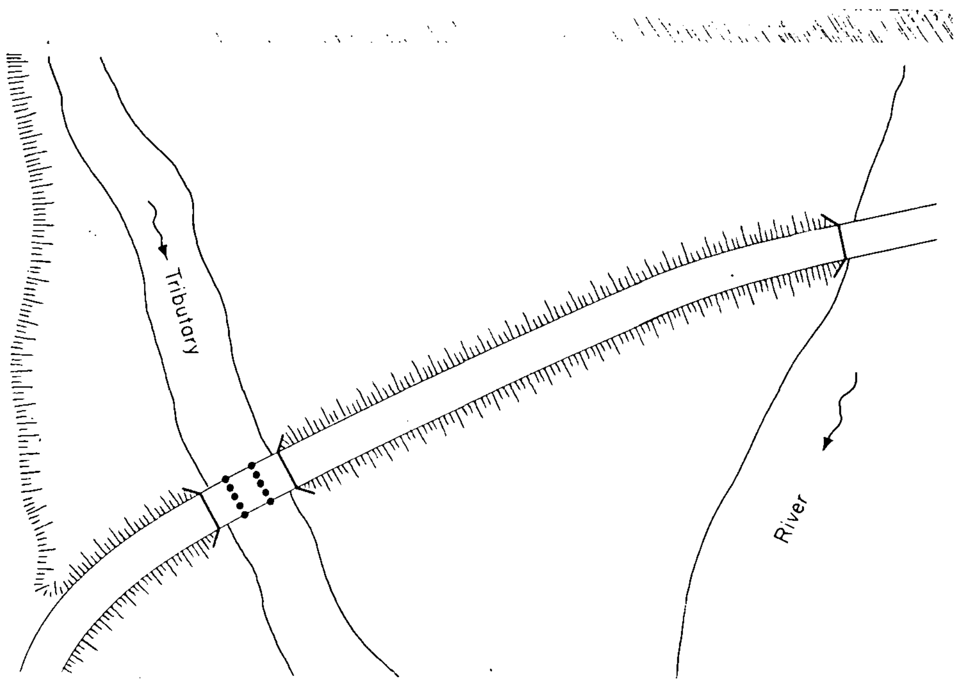


Fig. 45. Design example No. 6.

Both the local scour around the piers and the general scour at the contracted opening will be greater than those for a comparable river-crossing geometry by amounts that are still unpredictable.

One method of protecting this type of bridge is mentioned in the report: the placing of riprap over the area of the whole opening. Such riprap should be well below the normal bed of the tributary to permit maximum utilization of its protective qualities and should be of a size large enough to remain in place in zones of even the greatest local velocities.

APPENDIX

by Philip G. Hubbard

DETAILS OF SCOUR-METER CONSTRUCTION AND OPERATION

In designing the electrical instruments to make the measurements described in this report, primary emphasis was placed upon adaptability to various operating conditions. For example, scour depths vary from less than a foot in a model to perhaps dozens of feet at a large pier in a deep river, and the electrical conductivities of the water and bed material will vary considerably for different conditions. Of several methods attempted during the course of this investigation, the electrode-impedance method has proved to be the most reliable and adaptable.

To apply the electrode-impedance method, a series of electrodes are supported in a vertical line extending from the normal bed level down to the maximum expected depth of scour. The spacing of the electrodes, and hence the total number required, will depend on the desired accuracy. Alternating current at any convenient power frequency is passed from each of these electrodes to a nearby common ground. The impedance to the flow of current through the bed is much greater than the impedance to its flow through water, so that the difference can be used to determine which electrodes are covered by the bed material.

Alternating current is used in making the measurement, because the impedance at any measuring point changes rapidly with time if a unidirectional current is used. This phenomenon is often referred to as polarization, and is caused by the changing ion concentration near the electrodes when a current flows. When alternating current is used, the net flow of ions is zero, and a constant impedance results. The standard power frequency of 60 cycles per second is high enough to give the desired results for this application, and has been used because of its ready availability. Considerably higher frequencies will work equally well, but undesirable effects due to capacitance would become evident if frequencies above 1000 cycles per second were used.

In the earlier laboratory tests, the electrodes were switched manually into a circuit which indicated impedance by the deflection of an ordinary electric meter, and the operator recorded the location of the first electrode showing a sharp change in impedance. After the reliability of the method had been established with this initial equipment, automatically recording methods were developed. Two different techniques have been used to distinguish between the two impedance levels which serve as the basis of operation. Both techniques utilize a paper chart which is pulled past marking elements so that a continuous record is obtained.

For the laboratory measurements, a circuit has been used (Fig. 46) in which an electrode is placed in one arm of a bridge which is continuously supplied with alternating current. The other arms of the bridge are set so that it will be in balance when the impedance of any electrode is roughly midway between the values corresponding to complete coverage by bed material or complete exposure to the water. The phase of the output voltage from the bridge therefore has two distinct values, 180° apart, corresponding to the two electrode conditions. Though the magnitude of the output voltage may vary slightly from one electrode to the next due to

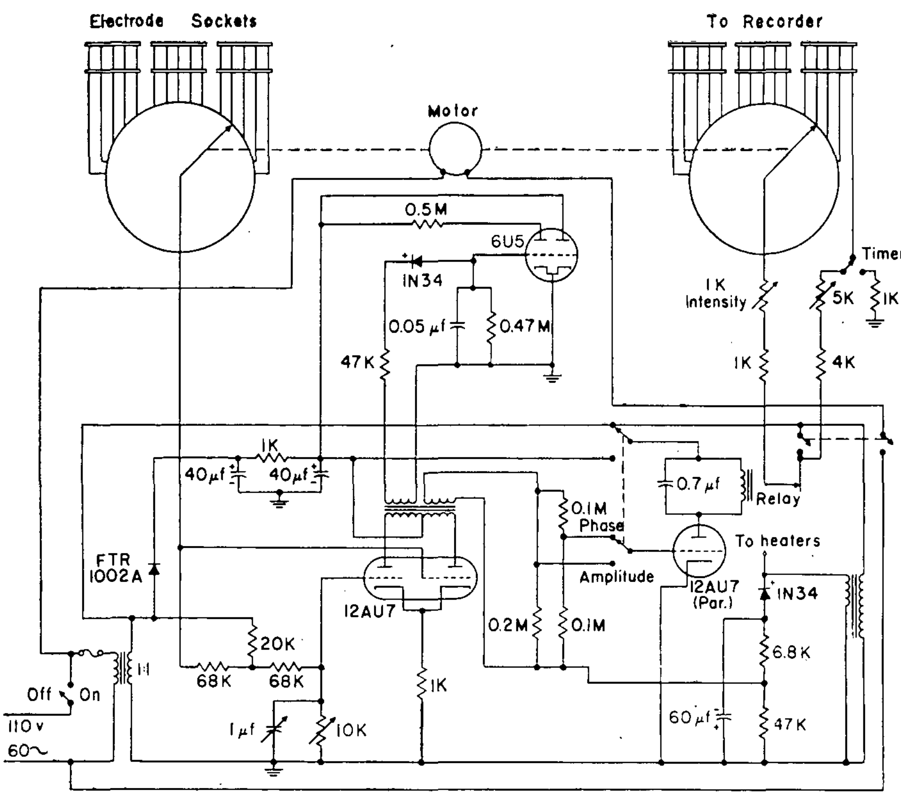


Fig. 46. Circuit diagram for laboratory scour meter.

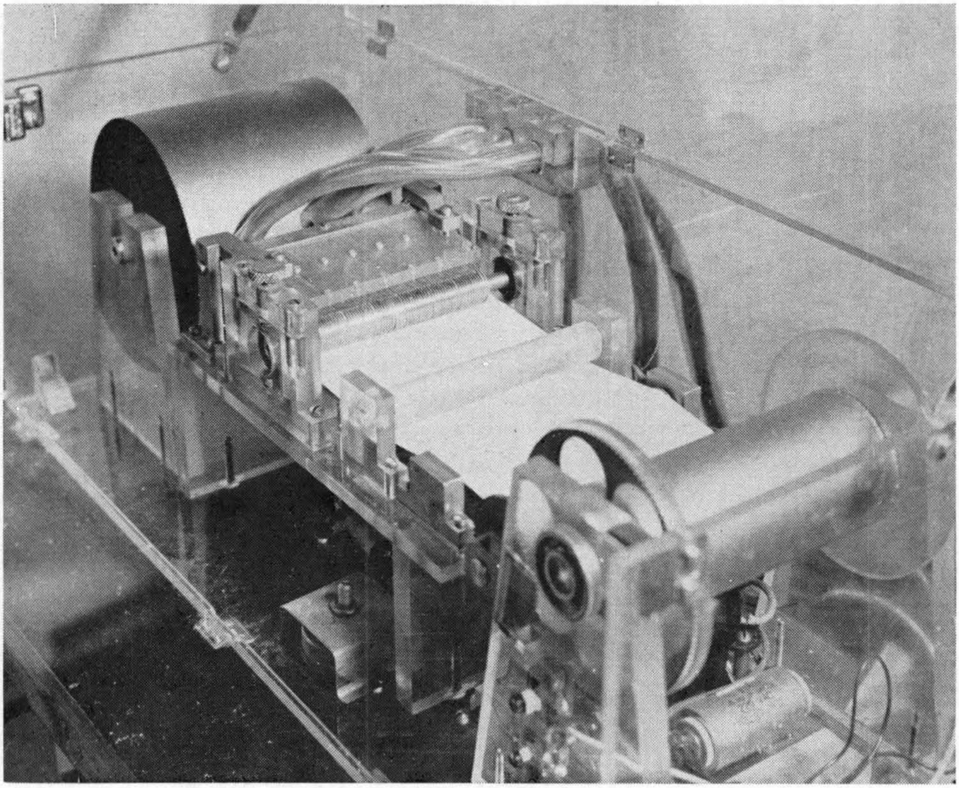


Fig. 47. Recording component of laboratory meter.

small differences, the phase will have two stable values which are a reliable indication of whether the scour has reached a particular electrode.

The recording medium for the laboratory instrument is Teledeltos, a paper which turns from a silvery color to black at the point where an electric current is passed into it from a fine wire or other conductor with small contact area. By using the aforementioned phase difference to control the passage of current, the marks on the paper can be made to correspond to the condition of an electrode exposed by scour. The circuit used to perform this function consists of an amplifier and a thyratron tube whose conduction is controlled by the amplified bridge output voltage. The plate current of this tube is passed through the paper by means of wire styli (in earlier work) or brass rollers (see Fig. 47).

When this method was first applied, a separate indicating circuit was used for each electrode, and difficulty was experienced in maintaining the circuits in essentially identical condition over a satisfactory length of time; that is, the marks on the paper were the result of comparing not only the electrode impedance, but also the bridge values, the amplifier characteristics, and the stylus performance. By changing the form of the instrument to that of Fig. 46, in which each electrode has a corresponding stylus but the remainder of the circuit is common to all, this difficulty was largely eliminated. A motor-driven switch connects the electrodes in turn to the bridge, and another switch on the same shaft connects the thyratron output to the corresponding stylus or roller. This modified method of operation has the disadvantage of giving an intermittent rather than a continuous record of the conditions at each electrode, but if the switch speed is high enough that the time required for one cycle is less than the time required for the scour to progress one electrode space, then this disadvantage is a minor one.

Another source of erratic operation was the fine wire styli which were used to conduct current to the paper. Because of their flexibility, it was difficult to maintain the proper spacing between the writing tips, and unequal wear of the styli (due primarily to arcing) complicated this problem. In addition, bits of insulating material collected under some of them so that the operation became erratic unless they were frequently cleaned with fine sandpaper. Substitution of the brass rollers eliminated these defects, and operation over long periods without servicing is now routine. Electrical connection to the rollers is made by means of small brushes, and the rollers are insulated from each other by the steel-cored Lucite shaft.

For the field measurements, a modified method of applying the electrode-impedance principle was utilized, because a lower switch speed was permissible, operation over much longer intervals without servicing was necessary, elimination of vacuum tubes was desirable, and a self-contained power supply was needed to eliminate the danger of power failure during floods. As may be seen from Fig. 48, the basic idea of a single measuring circuit for all electrodes is retained and carried one step farther by having a single marker for the chart. The circuit has been reduced to bare funda-

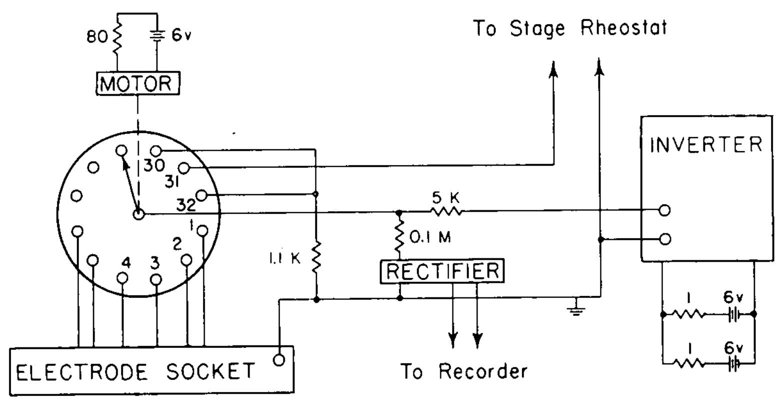


Fig. 48. Circuit diagram of field scour meter.

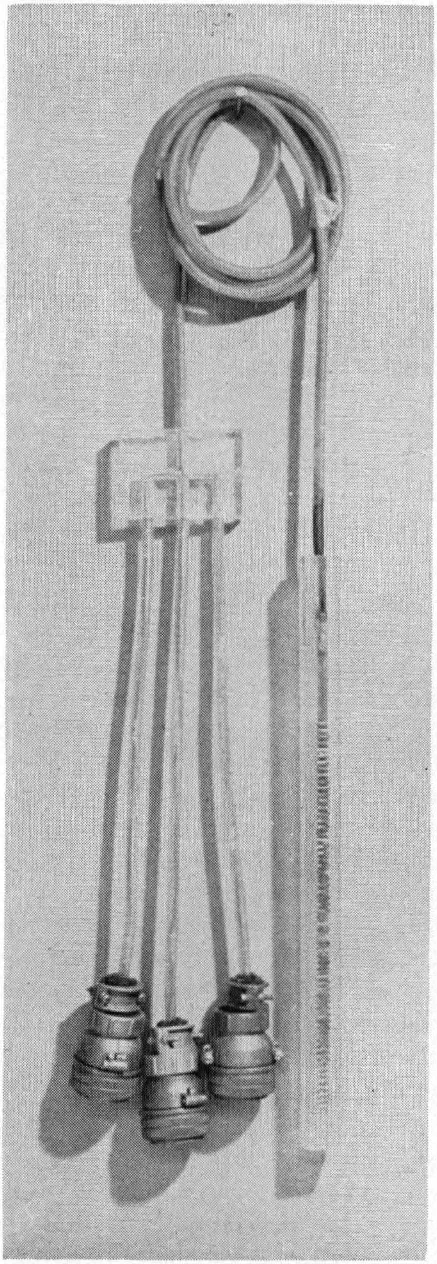


Fig. 49. Sensing element of laboratory meter.

mentals, and is simply a recording impedance meter to which the electrodes are connected in sequence. Because the electrodes for field use are much larger, their impedance is much lower and they can absorb sufficient power to make direct recording (without an amplifier) possible.

Ordinary 6-volt automobile storage batteries supply all of the electrical power necessary to operate the field meter, and the 60-cycle current necessary to obtain satisfactory performance is supplied by an inverter of the type used to operate small appliances from an automobile's cigarette-lighter supply. Current from this inverter is passed through a fixed resistor and into the electrodes through a slowly rotating switch (driven by a small 6-volt motor). As it moves from one contact point to the next, this switch momentarily shorts them, and the resulting low impedance causes the output indication to decrease. In this way, a clear dividing line is established between successive electrodes regardless of their impedance. A rectifier and a recording direct-current milliammeter connected across the common terminal of the switch complete the instrument. The position of the river bed at any time can be ascertained by simply counting the number of electrodes showing high impedance.

Because the river stage is an important factor in the scour process, it is desirable to include an indication of stage on the same chart with the scour data. This indication has been provided for in the field meter by using a standard float, cord, and pulley attached to a precision rheostat. An extra terminal on the electrode switch is simply connected to this rheostat so that its resistance is recorded on every revolution of the switch. Two other contacts on this switch are connected to a fixed resistor, so that a standard is always available which can be compared with other values to indicate variations in overall sensitivity of the measuring circuit due to any cause. This standard deflection on the chart also serves as an index for locating the first electrode on the pier.

In selecting materials for the electrode assembly, two factors should be considered: the exposed surface should resist corrosion and mechanical damage, and the insulation should stand up well under conditions of continual immersion. For the laboratory, platinum foil over a brass support served quite well as electrode material, and the individual electrodes with attached leads are cast into a convenient strip (see Fig. 49) by imbedding them in a special casting plastic; this strip can then be placed in a pier or other boundary, or used alone where its presence does not change the scour pattern. For the field, a similar casting process was used, but stainless-steel electrodes were cast in concrete, using a regular pile form.