velocity of flow and the sediment size as well as the geometry of the opening and its environs. The limiting condition for this type of scour is a boundary shear that is equal to the critical for the material of which the boundary is composed. Since there is no supply of sediment coming into the scour hole, at the limit there can be no sediment going out of the scour hole. This limit is finite, but is reached asymptotically in infinite time. It is very difficult in the laboratory to scale all factors controlling clear water scour and a combination of experiment and analysis is indicated.

For an abutment which is set back from the normal bank less than the depth of scour, the relationships proposed herein should be applicable. An abutment set back over three times the depth of scour should be considered a relief bridge. Following a successful investigation of the relief bridge problem, it would seem in order to study the effect of set backs between these limits.

One could, of course, go on studying the effect of various geometries of piers, abutments and overall crossing. However, the results of this investigation indicate that the effects of these details are minor, and of less importance than the error that can be expected in the evaluation of the flow conditions at the site. Further studies of geometry should probably be deferred until field measurements indicate they are of sufficient importance.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the contributions of the many others who were associated with him in this investigation of scour at bridge crossings. On the one hand there was the continued support and encouragement of Dr. Hunter Rouse, Director, Iowa Institute of Hydraulic Research; Mr. Mark Morris, Director of Highway Research, Iowa State Highway Commission; and Mr. Carl Izzard, Chief, Hydraulic Research Branch, Bureau of Public Roads; and on the other, the aid of the many research assistants who labored so long and so well, and especially Mr. Arthur Toch, Research Engineer, I.I.H.R. who, as a colleague for years, contributed in many ways, and Dr. Phillip G. Hubbard, Research Engineer, I.I.H.R., who was responsible for the development of the scour meter.

DISCUSSION

T. BLENCH,[9] F. ASCE.—The writer draws attention to the collection of data of scour around bridge piers and other obstacles made by C. C. Inglis, from 1924 to 1942.[10] The data of severe scour around piers numbered 17 and were for floods from 30,000 cfs to 2,600,000 cfs. They appeared, with descriptions,

[9] Prof. of Civ. Engrg. and Cons. Engr.

[10] "The Behaviour and Control of Rivers and Canals," (with the aid of models), by Sir C. C. Inglis, Govt. of India, Central Water Power Irrigation and Navigations Research Sta., Ponna, 1949.

as Table 8-1 in the reference and were plotted in its Fig. 8-1 to show the relation of scour depth (measured from water surface to bottom of scour) to the Lacey regime depth for the river at the flood discharge; the line in Fig. 8-1 was an excellent fit and indicated that, all other things being equal, scour depth varies as the cube root of discharge exactly as the regime theory based on controlled canals shows; the ratio to regime depth averaged almost exactly 2.0. The writer has replotted[11] the Inglis data along with model data of Inglis, of the present author (some time ago), and of workers at the University of Alberta. The coordinates were chosen in terms of estimated discharge intensity for the sake of uniformity and future applications; allowing for the discharge intensities of the rivers being averages over the whole river whereas for models they are discharge intensities of attack, the river and model data were in good accord with each other and with the regime theory prediction of variation as the two thirds power of discharge intensity—all other factors being equal. What to do when other factors vary has been explained.[11]

Inglis has described,[10] also, experiments on model bridge piers, similar to the recent Iowa ones, has plotted the results, and has given a formula to fit the graph. The writer has reduced this formula to a rather simpler form:

$$d_s/d_r = 1.8\left(b/d_r\right)^{\frac{1}{4}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where d_s is scour depth from water surface, d_r is regime depth of the river, and b is the breadth of the pier projected at right angles to the direction of attacking flow.

It is interesting to note that the mean depth of flow between piers (as distinct from the local scour round them) is shown by Leopold[12] to follow the regime law of variation as the two thirds power of discharge intensity in the bridge over the Powder River at Arvada, Wyoming.

The writer feels that Eqs. 7 and 8 should not have been derived from considerations of total load since it is bed load that is mainly effective in determining depth and that the formula is rough; furthermore, he notes that y seems to be unrelated to regime depth so is open to objection as a standard for comparison. Considering the evidence of regime theory, plus the Inglis and Leopold observations on rivers, the writer would replace Eqs. 7 and 8 by their equivalents from

$$\text{regime depth} \propto \sqrt[3]{q^2/F_b} \qquad \ldots\ldots\ldots\ldots\ldots (13)$$

and then apply geometric coefficients to various obstacles to obtain scour;[11] q is discharge intensity and F_b is the appropriate bed factor.

JOSEPH N. BRADLEY,[13] M. ASCE.—The model results on scour at bridge abutments and piers have been treated in a logical and commendable manner. The consistency with which the model results plot could be misleading how-

[11] "Regime Behaviour of Canals and Rivers," by T. Blench, Butterworths Scientific Publications, London and Toronto, 1957.

[12] "The Hydraulic Geometry of Steam Channels and Some Physiographic Implications," U.S.G.S., Prof. Paper 252, 1953, Fig. 4.A.

[13] Hydr. Engr., Internatl. Engrg. Co., Inc., Dacca, East Pakistan.

ever, by making the prediction of scour appear as a simple routine procedure. Since Mr. Laursen has said little with regard to the limitations of his results, a few remarks on this phase of the subject may be appropriate.

Figs. 5 and 6 represent the results of model studies on abutment scour made under essentially ideal conditions, for example, with bed of granular material free to move, a rectangular flow cross section, and a uniform velocity distribution. One can compare the author's results with those of a completely independent set of experiments made with different size and gradation of bed material, a constant depth of flow and a uniform velocity distribution, performed at Colorado State University,[8] and find that the two are in close agreement. This indicates that the model results are consistent and easy to duplicate.

Where streams meet similar specifications in nature as those of the models, there should be a reasonable correspondence between model and prototype. There are streams in India and Pakistan which do approach these so-called ideal conditions; the stream beds present an unlimited depth of alluvial material, the river channels are extremely wide with more or less constant width-to-depth ratio, and the velocities are low due to an unusually flat gradient (the average is 2 ft in 4 miles in East Pakistan). Under such conditions the velocity distribution cannot vary greatly across the stream. Field measurements of scour at bridge abutments, spurs, and guide banks for the rivers of India and Pakistan are on record[14] and these show surprisingly good correlation with the model results.[15]

Limited experience with scour on rivers in the United States show less favorable comparisons. The reason is obvious; the gradients are steeper, the cross sections are irregular, the velocities are higher, and the velocity distributions are far from uniform. Under these conditions the greatest scour does not necessarily occur at the abutments but is more likely to be found in the portion of the channel where the depth of flow and velocity are greatest. Records of the United States Geological Survey of bridge sites in Mississippi, where the beds are generally of alluvial material, show this to be true. There is also evidence from past floods in various sections of the United States that the settlement of piers in the deeper portion of the channel is more common than abutment failures due to scour. Yet if Figs. 3 and 6 are consulted, in order, it is found that prediction from the model gives scour depth up to three times the pier width for the center of the channel, while scour up to six times the depth of flow is supposedly possible at abutments. The latter can result in fantastic figures, which are true in the case of the model with rectangular cross section, where all scour is concentrated at the abutments; but such predictions are unreasonable when applied to irregular cross sections in the field.

This discussion was not written to confuse the issue or discredit the model results (which are valid for the conditions tested). Rather it is to point out some of the remaining unknowns and (1) encourage investigators to make a

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[14] "The Behavior and Control of Rivers and Canals," by Sir Claude Inglis, Research Publication 13, Part II, Central Water Power Irrigation and Navigation Report, Poona Research Sta., 1949.

[15] "Field Verification of Model Tests on Flow Through Highway Bridges and Culverts," by Carl F. Izzard and Joseph N. Bradley, Proceedings, 7th Hydr. Conf., Iowa, 1958, p. 225.

concerted effort to take soundings of streams at constrictions both before and during floods for the purpose of better understanding the field problem and making better utilization of the model results; and (2) to warn engineers to not use the model information blindly but to treat each river crossing as an individual problem, using the model results as a guide rather than a definite solution. Returning to item (1), further model studies will not produce the answer desired; field measurements are the only alternative. A further comment on item (2) is that model results applied to abutment scour in the United States will certainly fall on the side of safety. The extra cost of unduly deep footings may, therefore, be sufficient to finance a comprehensive field study in a very short time.

D. V. JOGLEKAR.[16]--It is agreed that each river and each reach must be studied to understand its individual, almost personalized, characteristics and that scour at bridge piers is closely related to (a) the river conditions upstream and downstream (such as indicated by flood hydrographs from year to year); (b) the meandering tendency of the river that, in turn, depends on the detritus load carried by the river during floods, (c) whether the river is flowing in its alluvial plain or has its sides and bed resistant to scour, (d) whether the river is flashy or has sustained floods, and (e) the constriction of the river section caused by the bridge. The reach of the river also has an important bearing on scour at bridge piers, for example, bridges on hill torrents, bridges on alluvial plains with mild slopes, and bridges on the tidal reaches of the river. In a braided river near a hilly region the river changes are so rapid that it is very difficult to estimate the waterway required for bridges across various streams of the braided river. An illuminating case of violent movement of Manas River[17] in Assam can be cited. In 1909, three bridges were provided on the three tributaries of this river:

1. on the Manas River which had 10 spans of 100 ft,
2. on the Bholookadoba Branch - 2 spans of 75 ft,
3. on the Beki Branch - 4 spans of 12 ft.

Due to changes in river courses, the bridge had to be rebuilt several times on the Bholookadoba Branch between the period 1909 to 1945:

1. 2 of 75 ft,
2. 3 of 75 ft,
3. 1 of 250 ft and 2 of 150 ft.

Changes in the Beki Branch were more violent and larger waterways had to be provided during this period as follows:

1. 4 x 12 ft,
2. 4 x 20 ft,
3. 9 x 20 ft,
4. 1 x 50 ft, 1 x 30 ft, 6 x 19 ft,
5. 1 x 50 ft, 2 x 40 ft, 6 x 19 ft,
6. 2 x 40 ft, 3 x 100 ft,
7. 7 x 150 ft.

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[16] Adviser, Central Bd. of Irrig. and Power, Poona, India.
[17] "River Training," Railway Board Technical Paper No. 334.

The river is still uncontrollable in spite of various training measures.

Experiments[18,19] were carried out at the Central Water and Power Research Station, Poona, India, in 1938 and 1939 for finding scour round a single pier placed in the center of a parallel sided flume, embedded in sand of mean diameter 0.29 mm. The following relation was worked out:

$$\frac{d_s}{b} = 1.70 \left(q_c^{\frac{2}{3}} \big/ b\right)^{0.78} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

in which b is the width of the pier, q_c is the discharge per foot in the center of the flume upstream of the pier, and d_s is the maximum depth of scour at the pier below water level. As it was difficult to correlate this with the depth of scour at prototype piers, due to q (intensity of discharge per foot) depending upon the curvature of the river upstream - which varies from river to river, it was considered desirable to study cases of actual scour in prototypes and work out a general, empirical relationship. Besides, it has to be remembered that the angle of repose of bed material in the model and the prototype is the same, hence, the extent of scour in plan in the vertically distorted model is found always relatively greater than in the prototype. This in effect reduces the discharge intensity at the pier due to greater dispersion of flow and hence the depths of scour obtained in the model would be relatively less. Data[19] were, therefore, collected for scour round bridge piers of various bridges constructed in India and a general relationship[19,20] was worked out as follows:[21]

$$d_s = (2)\ 0.473 (Q/f)^{\frac{1}{3}} = 2\ D(\text{Lacey}) \quad \ldots\ldots\ldots\ldots \quad (15)$$

in which Q is the maximum flood discharge in cu sec; d_s is the maximum depth of scour below highest flood level; f is = $1.76 \sqrt{m}$, and m is the mean grade of bed material in millimeters. In Eq. 15, a representative f value has to be used. From bore data, values of f for each strata is to be worked out to ascertain that the anticipated depth is not based on the f value which is higher than that appropriate at that depth. Recent advances in foundation engineering have made it possible to take the pier foundation sufficiently below the maximum probable depth of scour to provide adequate grip length. Where this cannot be done, high level stone protection, though costly in the long run, has to be employed. Another advantage of deep foundation is that, because of increased side friction on the pier sides embedded in sand, the load bearing capacity of the pier increases considerably as compared to that of a pier with shallow foundation. Generally this additional load bearing capacity is not taken into consideration in the design but is kept as a margin of safety. The previous railway practice was to work out the depth of bridge foundations according to the observations

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[18] Technical Annual Report, Central Water and Power Research Station, Poona, India, 1938-39.

[19] "Behaviour and control of rivers and canals with the aid of models - part II," Research Publication No. 13, Chapter VIII, Central Water and Power Research Sta., Poona, India.

[20] Technical Annual Report, Central Water and Power Research Station, Poona, India.

[21] "Stable Channels in Alluvium," by G. Lacey, Journal of the Institution of Civil Engineers, Paper No. 4736, 1929.

made by Spring[22] and Gales.[23] In the case of shallow foundation, protection has to be provided by stone pitching and if this is at high level, a lot of the stone (stone used generally weighs 80 to 120 lb and is called one man stone) is washed away downstream due to turbulence and has to be replenished, even during floods, to ensure the safety of the bridge. Due to this turbulence very deep scour occurs downstream of the piers as in the case of Hardinge Bridge[24] on the river Ganges, depth of scour being of the order of 200 ft.

The current railway practice is to provide a grip length for the pier equal to half the depth of scoured bed below H.H.F.L. so that the total length of piers below H.H.F.L. is 3D(Lacey).

Experiments[25,26] were carried out at the Central Water and Power Research Station, Poona, India, for testing the design of training works, waterway, and length of piers of a railway bridge at Mokameh on the river Ganges near Patna (Bihar State). The pier foundation of this bridge has been taken to a depth of about 200 ft below H.H.F.L. which is equal to 3D(Lacey). As the foundations are deep enough, protection by way of stone pitching round piers is not provided.

Various cases of bridges, for which rivers had to be trained to ensure the safety of piers, have been experimented on. These experiments are described in the Technical Annual Reports of the Central Water and Power Research Station, Poona, India, for the year 1937-38, 1938-39, 1939-40, 1940-41, 1944 to 47, 1949 and 1952 to 1958.

In the case of flash flood type rivers, fixing the waterway is much more difficult. In such cases the flood rises and falls so rapidly that the river has no time to scour its bed with the result that the afflux (difference in water surface upstream and downstream of bridge) increases enormously; the bridge is likely to fail by outflanking. A railway bridge on Luni River in Rajasthan State[27] failed in this manner.

In the case of inerodible bed material, it is difficult to estimate the maximum depth of scour. Hydraulic model experiments are unable to reproduce this scour due to obvious limitations. Recourse has, therefore, to be taken to field data. In some cases, the maximum flood level is underestimated and the waterway provided is insufficient. If the bed is inerodible afflux increases beyond the safe limit, with the result that standing wave conditions prevail downstream of the bridge, which lead to undermining of bridge piers. The railway bridge on Yeshwantpur River[28] in Andhra Pradesh failed and collapsed for similar reasons.

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22 "River Training and Control of the Guide Bank System," by F. J. E. Spring, Railway Board's Technical Publication No. 153.

23 "Principles of River Training for Railway Bridges and their Application to the Case of the Hardinge Bridge over the Lower Ganges at Sara," by R. Gales, Journal of the Institution of Civil Engineers, December, 1938, Paper No. 5167.

24 Investigations carried out by means of models at the Khadakwasla Hydrodynamics Research Station, near Poona in connection with the protection of the Hardinge Bridge which spans the river Ganges near Paksey, by C. C. Inglis and D. V. Joglekar, East Bengal Railway, Public Works Department, Bombay, India, 1936, Technical Paper No. 55.

25 Technical Annual Report, Central Water and Power Research Sta., Poona, India, 1950.

26 "Manual on River behaviour, control, and training," Ch. VI, Pub. No. 60, Central Board of Irrigation and Power.

27 Technical Annual Report, Central Water and Power Research Station, Poona, India, 1954.

28 Technical Annual Report, Central Water and Power Research Station, Poona, India, 1955.

The writer agrees with the author that afflux caused by a bridge depends so much on the erodibility of the bed material. Thus in the case of the railway bridge on the River Ganges[25] at Mokameh, the afflux caused by the bridge was only a couple of inches for a flood of 18,000,000 cu sec. This was because the flood was sustained and the river scoured its bed as the discharge increased. In the case of rivers with flash floods or with inerodible bed, full afflux has to be allowed for in the design according to standard formulas.

As emphasized by the author, more field experiments are necessary to improve our method of estimating scour at bridge piers.

W. J. BAUER,[29] M. ASCE.—The writer was in attendance at the University of Iowa, when the research described by Mr. Laursen was getting under way, and is, therefore, aware of its pioneering nature. The purpose of this discussion is to consider the application of the results presented by the author to a particular approach to waterway design.

The fundamental purpose of a bridge over a stream is to separate the human traffic and the flowing water. It is commonly accepted that provision for the passage of the maximum conceivable discharge through the waterway beneath the roadway is economically undesirable. It is good practice to provide for the safe, although infrequent, overtopping of the roadway by a rare flood. If the design is adequate, after the passage of such a flood, the highway is immediately ready for service with only minor repairs being required during subsequent routine maintenance.

In order to achieve this objective, the backwater produced by the structure must be small at the stage of incipient overtopping. The writer has used values of 0.5 ft to 1.0 ft as being reasonably small for the backwater at this stage. The design problem then becomes how to provide for the passage of the flood corresponding to the stage of incipient overtopping without exceeding an allowable backwater.

Some of the flow area required will exist beneath the elevation of the low-flow bed of the stream in accordance with the reasoning set forth by Mr. Laursen under the heading "Local Scour at Piers and Abutments: Backwater at Bridge Crossings." The extent to which such area is available to the flow during flood may be controlled by suitable scour protection.

At bridge sites in West Virginia, at which scour protection of broken rock is readily available, the writer has used a design velocity beneath the bridge of between two and three times the typical velocity in the stream during flood. This was accomplished without exceeding the allowable backwater at the stage of incipient overtopping. Such a relationship, between the velocity in the contracted section and that in the approach flow, gives sufficient assurance that the material deposited over the broken rock fill during low flow will be scoured out during flood, provided the depth of scour required is not excessive. Fig. 6 indicates the relationship between depth of scour at an encroaching abutment and a parameter dependent on the degree of contraction of the natural waterway by the structure. Fig. 6 indicates the attaining of depths of scour equal to the depth of flow at relatively minor amounts of contraction. It is, therefore, not difficult to imagine circumstances in which it would be possible to provide as much waterway area beneath the low-flow bed of the stream as above it.

[29] Cons. Engr., Chicago, Ill.

In the instances that the writer has in mind, the waterway area provided beneath the low-flow bed of the stream was about 20% of the total waterway area at the stage of incipient overtopping. The depth of the waterway area

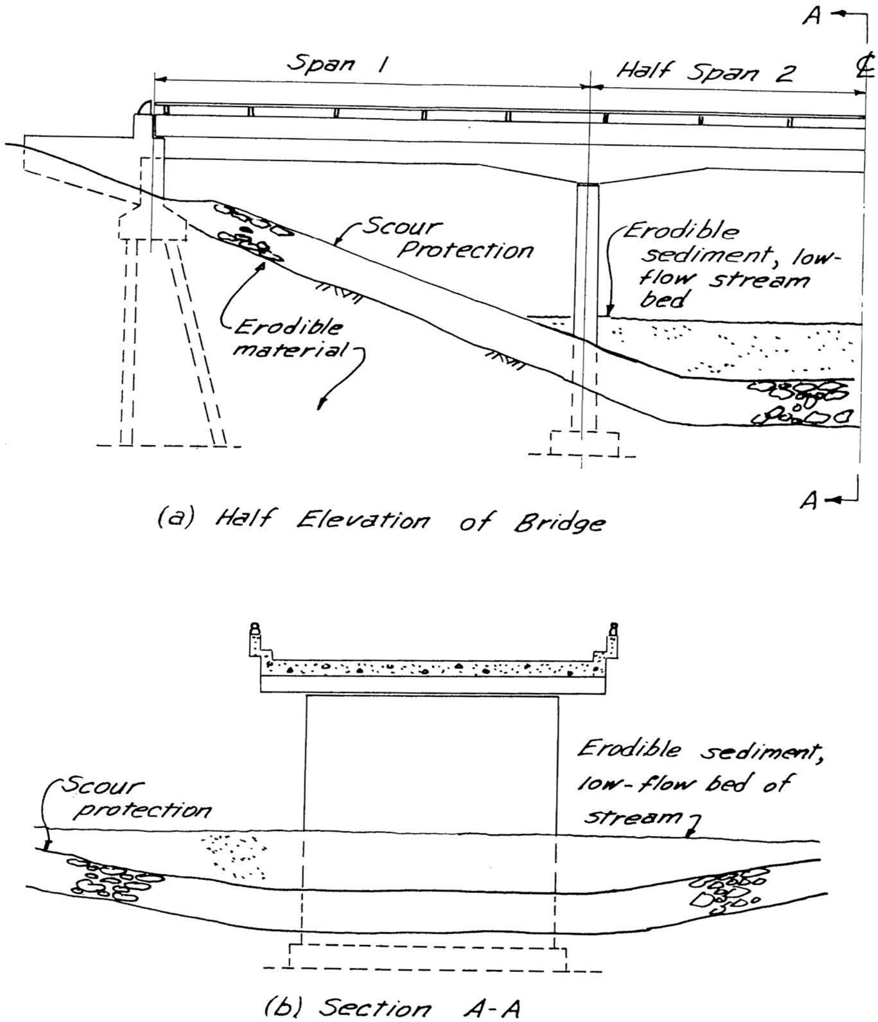

FIG. 12.—PROPOSED DESIGN

beneath the low-flow bed was only about 40% of the nominal depth of flow at the stage of incipient overtopping. Nevertheless, significant cost savings were achieved compared to a longer bridge with equal waterway area entirely above the low-flow bed of the stream. It is possible to imagine circumstances in which the length of the bridge might be out in half, the waterway area remaining the same. This has obvious economic significance.

The natural filling of the scour hole during the recession of the flood tends to minimize the depth of the pool of water that might be left standing under the bridge during low flow. As the beds of such streams are commonly made up of alternating pools and bars anyway, the addition of another small pool beneath the bridge is of little consequence. Fig. 12 shows the type of design proposed.

L. J. TISON.[30]—The relationship proposed by Mr. Laursen for the prediction of scour at piers and abutments for the case in which sediment is supplied to the scour hole, in extremely interesting.

The author presents an analysis in which he compares the flow without contraction and the flow due to a long contraction. He then uses, for both flows, an approximate form of the total sediment load relationship recently proposed by himself and considers a bridge crossing as a long contraction, foreshortened in such an extreme that it has only a beginning and an end.

The writer has found that a scour at piers and abutments also took place when the contraction of the flow was without significance and introduced the idea that the scour at piers may be affected by the curvature of the streamlines around the piers.

When a pier with an arbitrary shape is placed in a stream, it produces (Fig. 13) a first curvature with a center 0_1 and a radius ρ_1, followed by a second curvature 0_2 ρ_2. In the neighborhood of the bank of the river, the pier exerts no action on the direction of the streamlines, cd. But, considering the region of the first curvature in the neighborhood of the surface, a line such AB, tangent in each point to the principal normals of the streamlines, the following relationship may be written:

$$z_B + \frac{p_B}{\gamma} + \frac{1}{g} \int_A^B \frac{v^2}{\rho} ds = z_A + \frac{\rho_A}{\gamma} \quad . . . . \quad (16)$$

in which z is the vertical height, p is the pressure, v is the velocity and γ is the specific weight of the liquid. The first supposition was that the motion is parallel to the bottom.

In the neighborhood of the bottom, another relationship of the same nature gives:

$$z_{B'} + \frac{p_{B'}}{\gamma} + \frac{1}{g} \int_{A'}^{B'} \frac{v'^2}{\rho} ds = z_{A'} + \frac{p_{A'}}{\gamma} \quad . . . . \quad (17)$$

B and B' are on a same line perpendicular on the bottom.

The variation of the pressure between B and B' follows the hydrosatic law,

[30] Prof., Univ. of Ghent, Ghent, Belgium.

so that:

$$z_B + \frac{p_B}{\gamma} = z_B' + \frac{p_B'}{\gamma} \qquad (18)$$

From these three relationships

$$z_A + \frac{p_A}{\gamma} - \left(z_A' + \frac{p_A'}{\gamma}\right) = \frac{1}{g}\left[\int_A^B \frac{v^2}{p}\, ds - \int_{A'}^{B'} \frac{v'^2}{\rho}\, ds\right] \qquad (19)$$

with v greater than v' (v' is the velocity in the neighborhood of the bottom).

A consequence of Eq. 19 is that the motion cannot be parallel to the bottom, the trajectories must dive.

This diving motion will have another consequence : a local attack of the bottom under the influence of the first curvature $0_1 \rho_1$. Evidently, the importance of scour will depend on the value of the vertical component of the diving motion, and this vertical component will depend on the value of the second member of Eq. 19.

For example, small values of ρ will increase the depth of scour. Experiments with different models of piers, with the same length and breadth and with the same discharges and heights, gave a confirmation of this result.

Rectangular piers (reduced values of ρ) produced a scour of 113 mm with a discharge of 301 per sec in a flume with a breadth of 70 cm and a height of 10.5 cm. The length of the pier was 24 cm and its breadth was 6 cm.

The simple rounding of the edges of the piers reduced the scour to 91 mm, whereas a triangular shape reduced it 70 mm.

An aerodynamic shape gave no further reduction, but the shape of a lens produced a reduced scour of 54 mm. (large values of ρ).

The protection realized by a single pile before the lens-shaped pier reduced the scour to 38 mm. This pile produced bigger values of ρ and worked as a reduction of the width-length ratio.

A change in the repartition of the velocity will have an influence on the second member of Eq. 19 and, therefore, on the value of the scour. A higher roughness of the bottom on a distance before the pier (with pebbles on the bottom) is realized. In Eq. 19, v' was reduced whereas v was increased, with the conclusion that the erosion had to be increased. That is what was found with an erosion of 71 mm with the lens-shaped pier (54 mm with the fine sand).

The consideration of Eq. 19 shows that it would be possible to considerably reduce the erosion by adapting the variation of the radius ρ with that of v^2.

Therefore, the repartition of the velocity from the bottom to the surface was measured and the lens-shaped pier was given a variable radii of curvature corresponding with the values of v^2. The result was a non-prismatic pier with a larger base. The erosion around this new model was reduced to 5 mm maximum.

It is easy to see that the second curvature $0_2 \rho_2$ will give a relation (Eq. 19) with a negative second member. A rising motion will, therefore, follow the first diving motion. This was observed on all the models. For the rectangular pier, the small radii of curvature around the upstream edges produced a quasi-vertical rising motion.

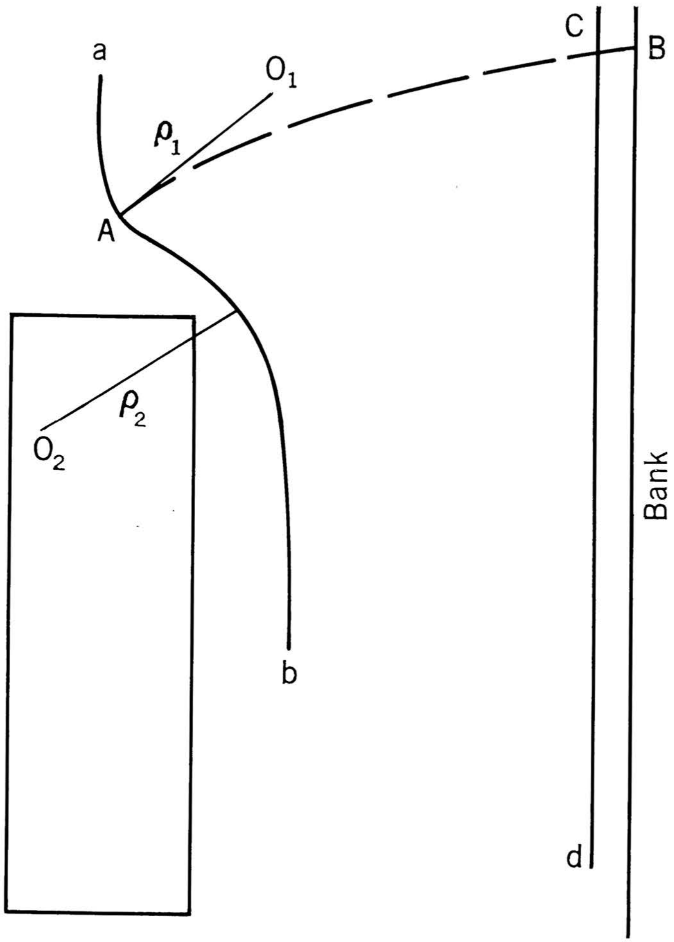

FIG. 13

Behind the pier, with the lens-shaped section, the curvatures of the type 2 was followed by another curvature of type i, and the formation of a secondary smaller scour was to be observed just behind the pier.

Many other results could be deduced from three first considerations and also from the consideration of the spiral motion induced by the diving motion deviated by the reaction of the bottom. Further results can be found in some of the writers publications.

The same theory can be used for the study of the action of groynes and of the motion of sediments in derivations.

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S. V. CHITALE.[31]—In connection with estimation of scour at piers and abutments of bridges, Mr. Laursen has stated that comparison of model and prototype data indicated that the depth of scour could be treated as simply another length and that equilibrium depth of scour obtained in the field. This concept of equilibrium depth is due to the author and more clearly enunciated by the following quotation:[2]

> "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 geometry, i.e. the relative depth of flow, the shape of the pier and the angle of attack. . . velocity of flow and sediment size (and, therefore, rate of transport and intensity of boundary shear) do not influence the equilibrium depth of scour. . . ."

Some investigations in models have been made on the subject of scour depth at bridge piers in India at the Central Water and Power Research Station, Poona which the writer thought would be of interest in context with the author's findings.

The first series of experiments were conducted in connection with the Hardinge Bridge over Ganga River and were reported in the Annual Reports of the Station for the years 1938 to 1942. A geometrically similar replica of Hardinge

[31] Chf. Research Officer, Flood Control Div., Central Water and Power Research Station, Poona, India.

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

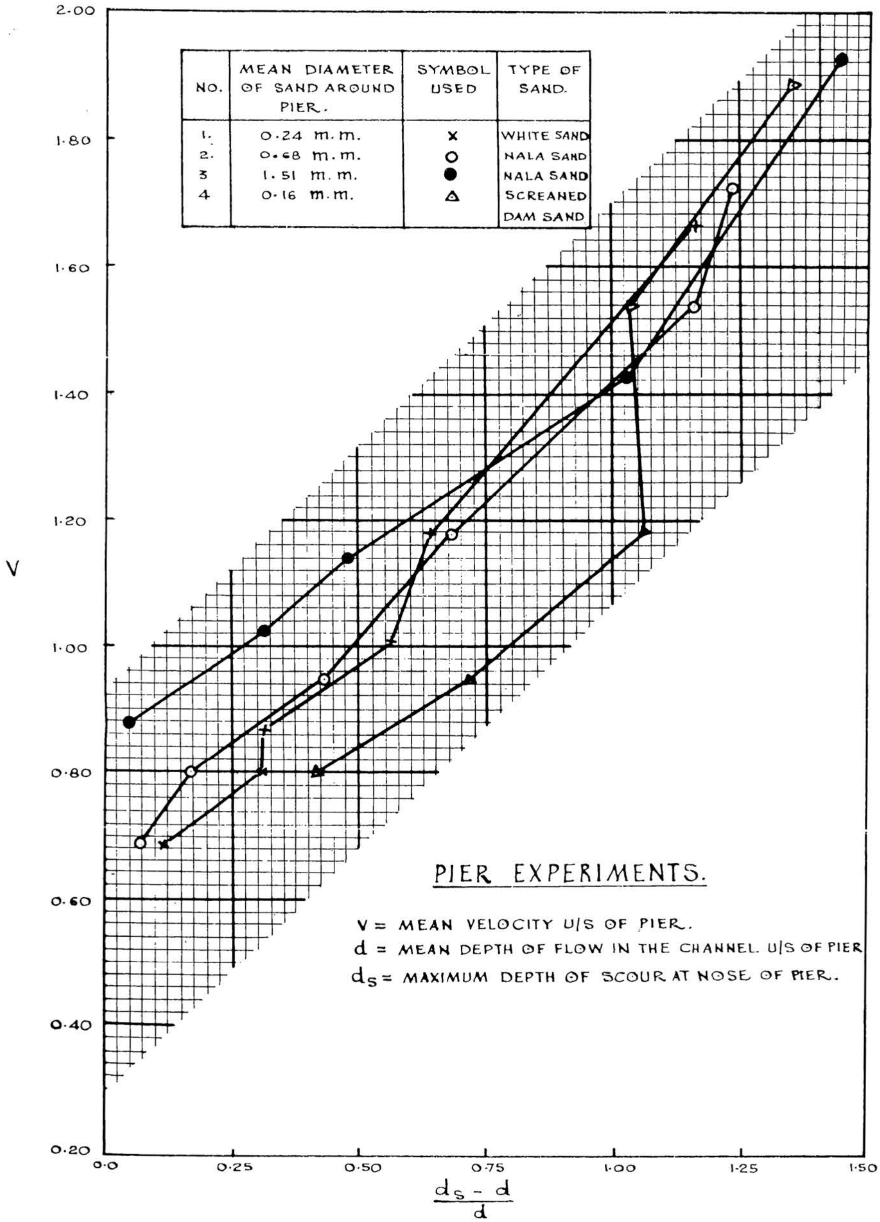

FIG. 14.—PIER EXPERIMENTS

Bridge pier was embedded in Ganga sand of mean diameter of 0.29 mm in a parallel sided channel. The results of these experiments gave the relation

$$\frac{ds}{d} = 1.70 \left(\frac{q_c^{2/3}}{b}\right)^{0.78} \quad \ldots\ldots\ldots\ldots\ldots \quad (20)$$

in which ds = maximum depth of scour below H.F.L.; d = depth of flow in the flume; b = width of pier; and q_c = discharge per foot run upstream of the piers.

TABLE 2.—RESULTS OF EXPERIMENT

Experiment No. 1	$V = \frac{q}{d}$ in ft per sec 2	ds in ft 3	d in ft 4	$\frac{V}{\sqrt{gd}}$ 5	ds - d in ft 6	$\frac{ds - d}{d}$ 7	Mean diameter of sand around pier, in mm 8
1	0.69	1.60	1.45	0.1005	0.15	0.103	0.24
	0.80	1.63	1.25	0.1261	0.38	0.304	
	0.87	1.51	1.15	0.1428	0.36	0.313	White
	1.01	1.55	0.99	0.1789	0.56	0.563	V
	1.18	1.40	0.85	0.2254	0.55	0.647	sand
	1.67	1.30	0.60	0.3799	0.70	1.170	
2	0.69	1.35	1.45	0.1005	0.10	0.069	0.68
	0.80	1.45	1.25	0.1261	0.20	0.160	
	0.95	1.50	1.05	0.1631	0.45	0.428	nala
	1.18	1.43	0.85	0.2254	0.58	0.682	sand
	1.54	1.41	0.65	0.3366	0.76	1.170	
	1.73	1.30	0.58	0.4002	0.72	1.240	
3	0.88	1.13	1.08	0.1490	0.05	0.046	1.51
	1.03	1.28	0.97	0.1841	0.31	0.320	
	1.14	1.30	0.88	0.2140	0.42	0.478	nala
	1.43	1.42	0.70	0.3012	0.72	1.030	sand
	1.93	1.28	0.52	0.4762	0.76	1.460	
4	0.80	1.77	1.25	0.1261	0.52	0.417	0.16
	0.95	1.80	1.05	0.1631	0.75	0.715	
	1.18	1.75	0.85	0.2254	0.90	1.060	
	1.54	1.33	0.65	0.3366	0.68	1.040	
	1.89	1.25	0.53	0.4575	0.72	1.360	

Pier experiments: In the 8 ft channel with sand of m = 0.32 mm

Scale of pier: 1/65 b = width of pier = 0.57 ft ≡ 37 ft
l = length of pier = 0.925 ft ≡ 63 ft
with constant discharge q = 1 cfs per ft
ds = maximum depth of scour round the pier
d = flow depth in the flume

This relation is not dimensionally correct and cannot therefore be adopted for general application.

Basic experiments were subsequently conducted in 1941 with the object of testing the influence of upstream depth and sand diameter on scour round piers. The pier tested in these experiments was also 1/65 scale model of that of the Hardinge Bridge. It was rectangular in section, of length 1 ft, width 0.6 ft and semicircular cut and ease waters. The bed of the flume in these experiments was laid with sand of 0.32 mm while the following materials were used just around the pier in succession.

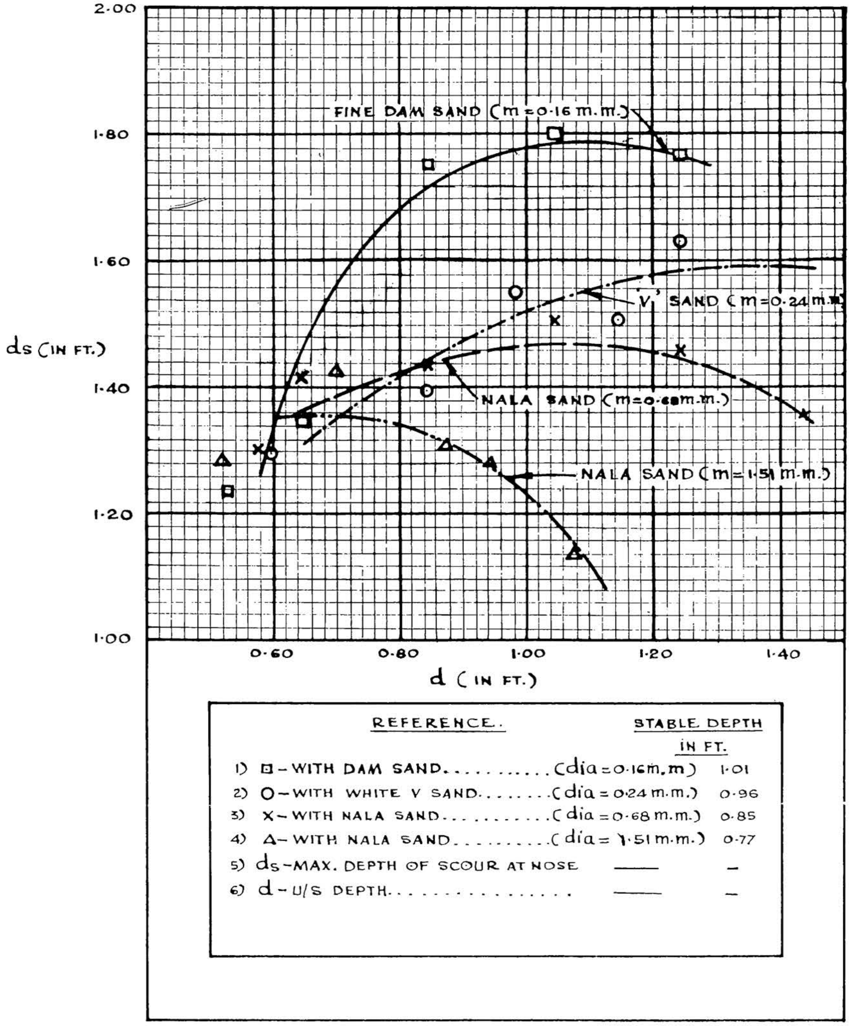

FIG. 15.—1/16 SCALE PIER EXPERIMENTS, ds : d FOR q = 1 cfs

Sieve number	Type of sand	Mean size, in mm
1	screened dam sand	m = 0.16
2	White 'V' sand	m = 0.24
3	Nala sand	m = 0.68
4	Nala sand	m = 1.51

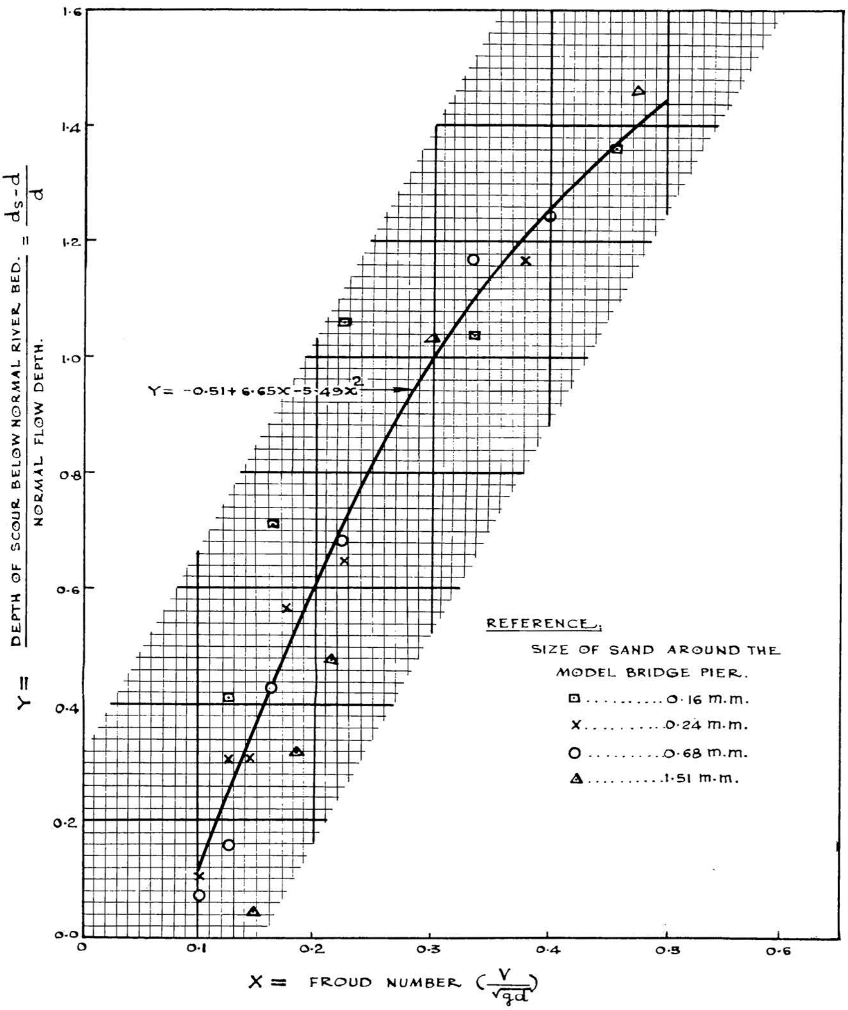

FIG. 16.—RESULTS OF BASIC EXPERIMENTS

The sand round the piers was laid flush with the upstream bed level. A constant discharge of q = 1 cfs per ft was run and water level was adjusted to get a particular depth, the depths varying from 0.5 to 1.45 ft. Each experiment was continued until final maximum scour was obtained round the pier.

In a few tests in which the upstream depth was less than stable depth the upstream bed scoured and blanketed the scour pit around the pier. In such cases, the maximum depth of scour at the nose was measured just before deposition in the scour hole of sand from upstream occurred.

In experiments when sand around pier was coarser than the bed material upstream, the bed around the pier was laid higher than upstream level, to get scour round the pier for the upstream depth laid.

Table 2 shows the results obtained important conclusions, as follows:

1. With axial flow, maximum depth of scour was always at the nose of the pier, scour at sides being less by 5 to 15%.
2. The ratio of scour at the nose and depth of flow in the channel bears a simple relation with the approach velocity in the channel (see Fig. 14).
3. The depth of flow on the upstream has also an influence on scour at the pier nose (see Fig. 15).

It will be seen that correlation of depth of scour either with upstream depth (Fig. 15) does not appear to be satisfactory. The writer, therefore, further analyzed the experimental data and found that the Froud number provides a better criterion. Fig. 16 shows plot of depth of scour against the Froud number. Although some scatter of points is evident in this plot, the general trend is remarkable, statistical equation obtained over the range of experimental data being

$$y = -0.51 + 6.65\,x - 5.49\,x^2 \quad \text{. . . . .} \quad (21)$$

It is thus seen that experiments at the Research Station do not lend support to the author's equilibrium scour theory, and it appears that further investigations both in the field and in the laboratories are desirable before it can be accepted unreservedly and with confidence.

Grateful acknowledgment is made of the kind permission given by M. G. Hiranandani, Director, Central Water and Power Research Station to refer to experiments previously conducted at the Station and also for useful suggestions offered by him.

A. RYLANDS THOMAS,[32] F. ASCE.—In Fig. 17, the author's design curve for piers (Fig. 9), with a factor of 0.9 for application to piers with semicircular nose form, is compared with results of experiments with models of piers carried out by the writer in association with Sir Claude Inglis.[33,34] These piers were models of the piers of the Hardinge Bridge over the Ganges River (now in East Pakistan) which were 37 ft wide and 63 ft long, including the semicircular nose and tail. The model piers were fixed in a parallel-sided channel with a bed of incoherent Ganges sand about 0.3 mm mean diameter and the pro-

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32 Cons. Engr., London, England.

33 Annual Reports (Technical), Central Irrigation and Hydrodynamic Research Station, Poona, India, 1938-39, p. 39; 1939-40, p. 33; 1940-41, p. 35.

34 "The Behaviour and Control of Rivers and Canals," by C. C. Inglis, Central Waterpower Irrigation and Navigation Research Station, Poona, India, 1949, p. 327.

cedure was to run a constant discharge without sediment load until scour had ceased.

It will be seen from Fig. 17 that the author's design curve, and still more his curve from the Iowa data, Fig. 3, lie well above the Poona results. The difference appears to be too great to be explained by sediment load,[21] though this cannot be ruled out as the upstream depth may be reduced more by sediment load than is the depth of scour at the pier. It would be of value if the author presented the data on which he based his design curve, showing particularly the effect of load.

It is also most desirable to compare small-scale results with observations made under full-scale conditions. Such data are difficult to obtain because of

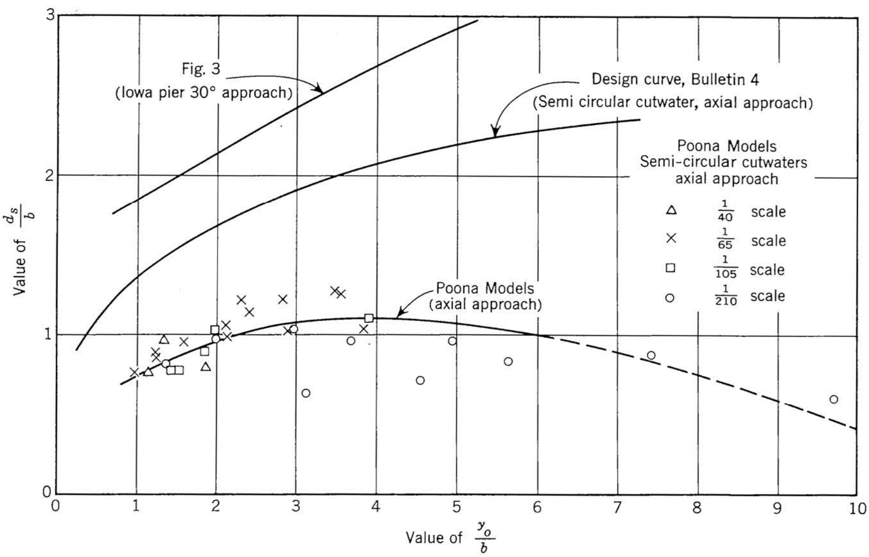

FIG. 17.—SCOUR AT BRIDGE PIERS

the need to take observations during high floods which are very often of short duration. It is easier to measure depth of scour at the pier than in the channel upstream, where a number of observations must be taken to average out the effect of bed waves. Even when this is done it is not certain that the bed was in equilibrium at the time of observation. The Poona experiments showed that a reduction in channel depth, due for example to resistence to scour, would increase the depth of scour at the nose of the pier.

The relationship between depth of scour at the nose of the pier and the depth of channel upstream is, therefore, perhaps not the most suitable one for comparison of full-scale data. Nor is it generally the most suitable for practical application of a design formula, because the upstream depth during a maximum flood is not often known beforehand and would have to be calculated. An error in assessing this depth would lead to a corresponding and greater error in estimating the level to which scour is liable to occur.

A more basic relationship is that between depth of scour below water level, D_s, width of pier, b, and upstream discharge per unit width, q. The relationship indicated by the Poona results is

$$\frac{D_s}{b} = 1.70 \left(\frac{q^{2/3}}{b}\right)^{0.78} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

in ft-sec units, applicable to scour in fine sand within the limits of $q^{2/3}/b$ or y_o/b at least from 1 to 6 (see Fig. 17) and not greatly affected by the grade of sand. It is necessary to take into account the effect of obliquity of approach flow, which may be done by using for b the projected width of pier on a plane normal to the direction of approach. A factor of safety should also be used in design.

The discharge per unit width to be used for design must, in most cases, be greater than the mean value because in rivers the deep channel meanders from one bank to the other. It is therefore advisable to use, at least as a check, the relationship proposed by Inglis[34] giving the maximum depth of scour around a pier as approximately twice the Lacey regime depth, that is,

$$D_s = (Q/f)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

in which D_s is the depth of scour in feet below water surface, Q is the discharge in cu sec, and f is the Lacey silt factor (approximately $1.8\sqrt{m}$ where m = mean diameter of bed sand in mm). Eq. 23, derived from data of several bridges in India and Pakistan, takes into account obliquity of approach and concentration of discharge but not the width of pier, which is clearly an important factor. The data related to normal widths of pier and to depths of scour ranging from 25 ft to 117 ft.

MUSHTAQ AHMAD.[35]--The problem of correct estimation of depth, shape, and extent of localized scour is important from the point of view of establishing sound design practice and safety of hydraulic structures such as bridge piers, abutments, spur dikes, groins, or pitched islands. This problem is much more important for hydraulic structures in alluvial rivers of West Pakistan where fine sand is found for hundreds of feet in depth, and scour depths of 40 ft to 80 ft below water level are common. The author has, therefore, dealt with a subject of special importance and great utility to this region.

It is proposed to discuss the author's approach to the problem with special reference to field and laboratory experience in West Pakistan. The author has assumed the depth of scour as another length and has related it to the normal depth or the width of a pier, and maintains that the depth of scour does not depend on the degree of concentration until scour holes around neighboring obstructions overlap. He maintains that there is equilibrium or limiting depth and believes that for a given mode of sediment movement, the depth of scour depends only on the geometry of contraction and the approach depth. He holds that the effect of velocity and sediment size can be neglected as that of secondary order. The writer agrees that there is an equilibrium and limiting depth of

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[35] Dir., Irrigation Research Inst., West Pakistan.

scour and that the effect of sediment size can be neglected. Tests made on depth of localized scour at spur dikes[36] also showed that localized scour depth does not vary with grain size in the range (0.1 m to 0.7 m) usually met with in alluvial plains of West Pakistan. However, this concept may not be valid for the entire range of bed material sizes ranging from fine sand to gravel and boulders.

The author's view that scour depth is another length connected with normal depth or width of pier implies that it varies with the discharge, or more correctly with the discharge per foot run. The author's Eq. 8b and c holding for rivers and streams in West Pakistan are:

$$\frac{d_s}{y_1} = \left(\frac{B_1}{B_2}\right)^{.64} - 1 \text{ for } \frac{\sqrt{g\ y\ s}}{w} = 1 \ldots\ldots\ldots\ldots (24a)$$

and

$$\frac{d_s}{y_1} = \left(\frac{B_1}{B_2}\right)^{.69} - 1 \text{ for } \frac{\sqrt{g\ y\ s}}{w} < 2 \ldots\ldots\ldots\ldots (24b)$$

where the exponent varies between 0.64 to 0.69, or on the average the relation can be written as

$$\frac{d_s + y_1}{y_1} = \left(\frac{Q/B_1}{Q/B_2}\right)^{.64-.69} \sim \left(\frac{q_2}{q_1}\right)^{2/3} \ldots\ldots\ldots\ldots (25)$$

$$\frac{y_2}{y_1} = \left(\frac{q_2}{q_1}\right)^{.64-.69} \sim \left(\frac{q_2}{q_1}\right)^{2/3} \ldots\ldots\ldots\ldots (26)$$

and

$$\frac{y_2}{q^{2/3}} = \frac{y_1}{q^{2/3}} = K \ldots\ldots\ldots\ldots (27)$$

in which K may be a function of boundary geometry of the contraction at the bridge, abutment shape and thickness and shape of the nose of the piers, and so forth. Because the depth is a dependent variable on discharge intensity, the latter may be used in preference to depth. Here only lies the difference in approach between the author and the writer in studying this problem. As shown previously the two approaches are not very different. The writer prefers to study the variation of $D_s/q^{2/3} = K$ as a function of boundary geometry, shape of pier nose, and abutment, characteristics of bed material, and distribution of velocity in the cross section at the piers representing the concentration of flow. The functional relation for study may be of the type:

$$\frac{D_s}{q^{2/3}} = K f\left(\theta, \frac{V^{1/2}}{V}, \frac{\sqrt{g\ y\ s}}{w}, S, \frac{B}{B_t - nt}\right) \text{ etc.} \ldots\ldots\ldots (28)$$

in which θ is the angle between the current and the spur dike, an abutment or a pier, $V^{1/2}/V$ is the ratio of mean velocity in half the channel width on the side of spur dike or abutment to the mean velocity in full section. It will be a

[36] "Experiments on Design and Behaviour of Spur Dykes," by Mushtaq Ahmad, Proceedings, Minnesota Internatl. Hydr. Convention, September 1-4, 1953.

measure of concentration of flow; W is fall velocity of bed material; B is the river width upstream of the bridge; B_t is distance between the abutments; n is the number of piers,; and t is thickness of each piers.

The writer does not agree that the depth of scour does not depend on the degree of concentration. It has been found that by changing the concentration or

TABLE 3.—VALUES OF $V_{\frac{1}{2}}/V$ AND K

Approach condition (1)	$V_{\frac{1}{2}}/V$ (2)	K (3)
Scour below a severe bend on the concave side accompanied by a swirl on the convex bend.	1.25	2 - 2.25
Moderate Bends.	1.15	1.5 - 1.75
Straight obstruction placed at any angle of 30° to 90°	1.0	1.2 - 1.5
Straight obstruction placed at an angle of 90° - 150° to the flow	1.0	1.5 - 1.75

TABLE 4.—MAXIMUM SCOUR DEPTHS OBSERVED AT BRIDGE PIER AND ABUTMENTS ON DIFFERENT MODEL STUDIES AT IRRIGATION RESEARCH INSTITUTE, LAHORE.[b]

River	Site	Q Max, in cu sec	q	Scour depth			K = $\frac{Ds}{q^{2/3}}$	Y_o	Thickness of pier (b)	$\frac{Y_o}{b}$	$\frac{ds}{b}$	D_s = ds + Y_o
				Observed	Scale distortion factor.	Corrected scour depth.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Ravi	Shahdara Bridge	332,000	237	52	1.23	64.0	1.68	26.4	10	2.64	2.06	47
Jhelum	Jhelum bridge[a]	600,000	120	30	1.3	39	1.60	23	10	2.3	1.98	42.8
	Maagla New bridge	275,000	429	90	1.0	90	1.60	28	abutment	..	...	..
Deg	S-harakpur	40,000	200	34	1.34	45	1.34	21.5	6.25	3.45	2.22	35.5
Rohi	Kasur	35,000	147	30	1.3	39	1.4	14	6.25	2.25	1.97	26.3
Sohan	Dhok Pathan	150,000	172	36	1.17	42	1.36	18.6	9.0	2.07	1.94	36.1
Indus	Thatha Sujjawal proposed bridge.	1,100,000	275	52	1.31	68	1.6	25	7.0	3.57	2.25	40.8

[a] Computation from Fig. 9, Bulletin No. 4.
[b] Bed material - five sand.

velocity distribution at bridge site by training works upstream, the scour depth can be considerably reduced. The variation of scour depth as a result of change in velocity and flow distribution due to the training works is explained in Figs. 18, 19, and 20. Figs. 19 and 20 depict a model of the Ling Stream showing a bridge below a curve. The concentration of flow near the left flank is notice-

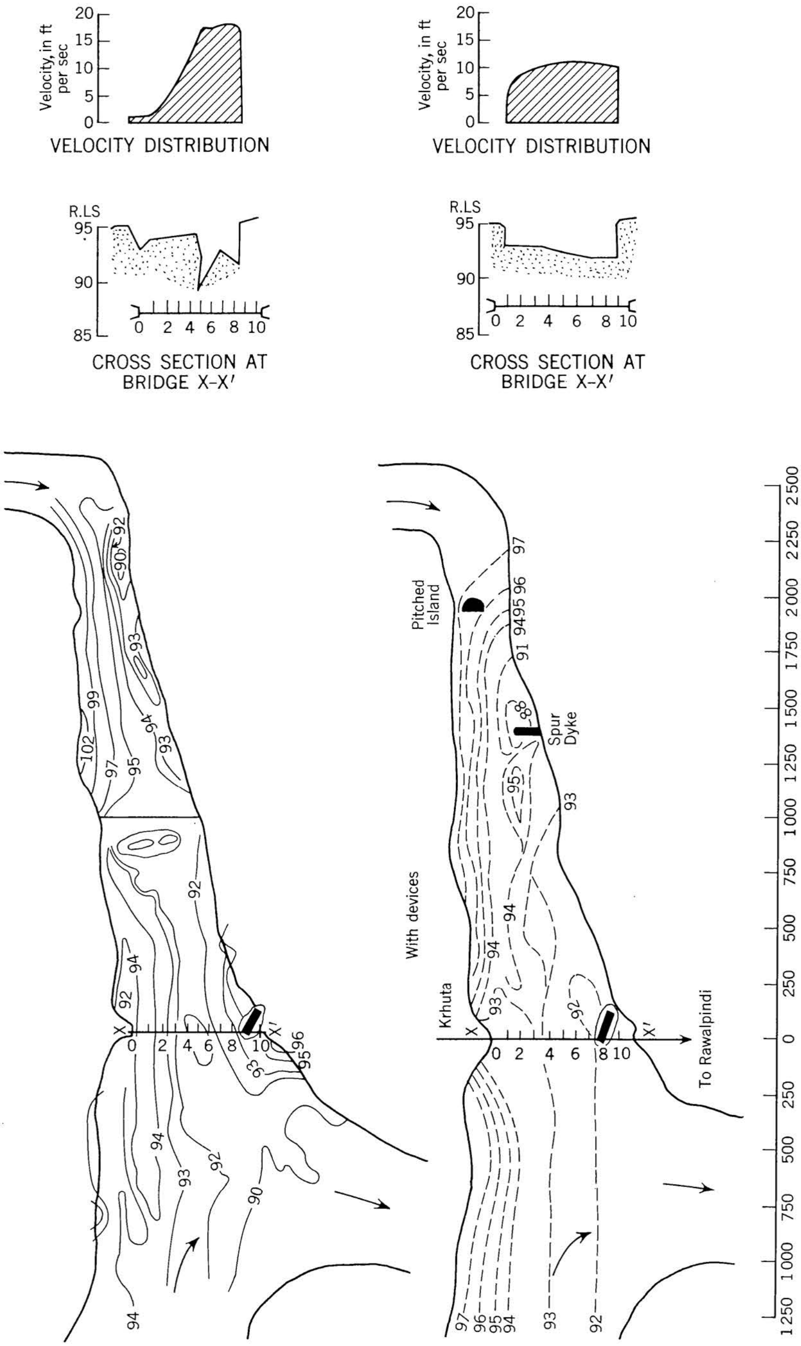
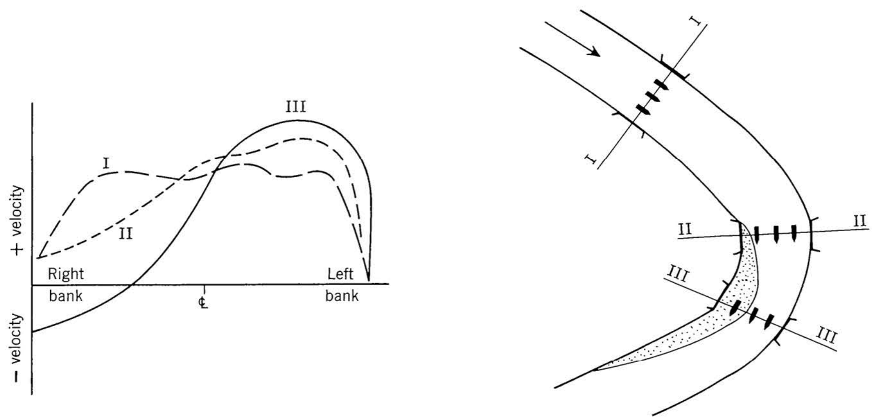


FIG. 18.—LING NALLAH BRIDGE

FIG. 19

FIG. 20.—FLOW AT BRIDGE SITE



(*a*) VELOCITY DISTRIBUTION AT BRIDGE SITES I, II, AND III

(*b*) THREE BRIDGE SITES IN A CURVE

FIG. 21.—THREE BRIDGE SITES IN A CURVE

FIG. 22.—ABNORMAL SCOUR AT A GUIDE BANK HEAD

able in Fig. 19. A pitched island and a spur on the left bank fans out the flow at the bridge site to obtain more uniform velocity and lesser scour as shown in Fig. 20.

In the method of plot adopted by the author in which scour depth in relation to U/S depth has been studied, the non-dimensional scour perimeter may not have significant relation with velocity, for, in open channels as the velocity or

TABLE 5.—SHAHDARA RAILWAY BRIDGE ON RIVER RAVI[a]

S. No.	Year	Q	H.F.L.	Gauge B	Water-way at the bridge B, in feet	q	Max. Scour R.L.	Max. Scour Ds	$K - \frac{DS}{q^{2/3}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1.[b]	1948	86000	688.0	11.7	12 x 90 = 1080	79.5	651.14	36.86'	1.989
2.[c]	1949	52000	686.0	9.7	8 x 90 = 720	72.2	657.14	28.86	1.664
3.[c]	1950	1,93000	692.0	15.7	15 x 90 = 1350	142.9	651.2	40.8	1.49
4.[c]	1951	44000	686.3	10.0	7 x 90 = 630	69.8	656.14	30.16	1.77
5.[c]	1952	56000	687.2	10.9	9 x 90 = 810	69.13	655.0	32.0	1.63
6.[c]	1953	83000	689.1	12.8	11 x 90 = 990	83.75	651.64	37.46	1.95
7.[c]	1954	1,72000	691.8	15.5	15 x 90 = 1350	127.5	655.0	36.8	1.453
8.[c]	1955	250,000	695.0	18.7	15 x 90 = 1350	105.18	660.0	35.0	1.078
9.[c]	1956	87000	688.3	12.0	12 x 90 = 1080	80.5	657.22	31.08	1.66
10.[c]	1957	1,92000	692.3	16.0	15 x 90 = 1350	142.2	654.54	37.66	1.383
11.[c]	1958	1,52000	691.2	14.9	14 x 90 = 1260	120.61	662.64	28.56	1.17

[a] Part of the water way is marked by a semi erodable island at the bridge and hence the difference in the water way for different years. [b] No training works a river bed u/s. [c] A spur constructed u/s to current approach.

discharge increases so does the depth, although not necessarily by erosion of the bed. For a uniform approach velocity, a mean depth can be assumed for use in the author's relations. But, generally, abnormal scour depths at piers or abutments are associated with concentration of flow resulting from the variation of velocity and depth in the approach section and the selection of representative approach depth for use in the author's relation and the estimation of discharge distribution in the channel and on the berm becomes very difficult.

In fact, the author's term $(Q_c + Q_o)/Q_c$ in Eq. 9 is, also, a measure of concentration of flow. However, the estimation of Q_c, Q_o, or Q_w is difficult for practical use in the computation of maximum scour depth in case of a curved approach. The writer has shown[37] that change in flow concentration above a spur dike, can be depicted in terms of velocity distribution and common types are depicted in Fig. 21 by I, II and III. The type III velocity distribution gives the maximum concentration on the outside of a bend as due to negative velocity on the inside, a part of the water way is blocked by reverse flow. For the three types of velocity distribution depending on different types of approaches, the values of $V_{\frac{1}{2}}/V$ and K as determined from writer's studies on spur dikes (which may be applicable to abutments), are given in Table 3. Since maximum scour can occur on any one or a group of piers in a meandering alluvial channel, the depth of foundation of piers has to be computed from the values of K as fixed by the river curvatures likely to occur, keeping in view the restraints imposed on river by the upstream training works. The method commonly used in West Pakistan supported by Laboratory studies and field observations consists in working out $q = \frac{Qmax}{B}$.

At the bridge site, the value of K can be selected from the maximum curvature likely to occur. This will give scour depth below water level, and the depths below bed level can then be worked out. Maximum depth of scour from model studies at bridge piers and abutments of different rivers and streams, after correcting the results for the effect of scale distortion on geometry of scour,[37] are summarized in Table 4. In this table, scour depth has been computed from the author's design curves of Bulletin No. 4, in Fig. 9. The scour depths obtained after correcting for the model scale distortion are generally higher than those of the author. In computing D_s value by his method, the actual mean value of Y_o has been taken from the cross section above the bridge. The value of K does not exceed 1.7 because severe bends are not possible in these cases due to the presence of guide banks.

Fig. 22 shows abnormal scour at a guide bank head. A heavy embayment is noticeable on the right and construction due to silting caused by roller on the left. The K value of 2.3 can be obtained in such a case.

The heavy embayment resulting in abnormal scours illustrated in Fig. 22 with K value greater than 2 are possible only on spur dikes or guide bank heads and are not to be allowed on bridge piers or abutments. In fact, for alluvial rivers or streams of West Pakistan, guide banks at least equal to the length of the bridge, with curved heads and expanding water way on the upstream side, are provided to shift the maximum embayment and abnormal localized scour to the guide bank heads instead of allowing it to occur near the bridge with short abutments. Under these conditions, the values of K for estimating the maximum probable depth of scour can safely be taken between 1.7 and 2.0. In case of a short abutment length before the conditions for abnormal scour where K greater than 2 are obtained, the approach roads or railway will be threatened and cut by the embayment formed by an alluvial meandering river. It is therefor necessary to design pier and abutment depths for scour calculated from K 1.7 to 2.0 and provide proper guide bank to keep the road or railway ap-

37 "Effect of Scale Distortion, Size of Model Bed Material and Time Scale on Geometrical Similarity of Localized Scour," by Mushtaq Ahmad, Proceedings, Internatl. Assoc. of Hydr. Research, The Hague, Netherlands, 1955.

proaches safe from embayment and to keep abnormal scour away from the main bridge crossing. Actual scour depths observed on the railway bridge on Ravi near Lahore and the computed values of K as recorded in Table 5 also supported the recorded values of K.

PIER LUIGI ROMITA.[38]—The author's long lasting efforts to cast light upon the phenomenon of scour around bridge piers and abutments are to be greatly commended because of the great practical importance of the problem. The clear and condensed presentation in this paper of the results of these efforts is a substantial step towards a generalized solution of the problem, and will be of great use to designers and to the administrations responsible for the construction and maintenance of roads and railways. To this end a coordinated effort should be made, in order to insure the necessary verification on prototypes of the proposed relationships.

There is no doubt that one of the most important factors of scour, when piers have a sufficiently high length per width ratio, is the angle of attack between the pier and the flow. Even small deviations of this angle from the zero value are responsible for fast increases of the scour; this usually represents a much greater danger to the stability of the pier than any underevaluation of the maximum possible flood. The collapse of many piers during floods is, in fact, to be ascribed more to the angles of attack, which may occur due to unusual cross-currents and deformations of the river bed, than to the unexpected entity of the discharge.

In view of all this, the writer carried out, some years ago, a systematic model investigation of the influence of the angle of attack upon the depth of scour. The experiments were carried out in a glass-flume of the Hydraulic Laboratory of the Polytechnic Institute of Milan, supplied with clear water, using uniform sand of 1 mm diam as bed material, and in such conditions that there was no general bed movement but only localized scour around the pier. The shape of the pier was not particularly studied, and it reproduced a rather widely used type of pier; its length per width ratio was about 5. Two series of tests were carried out with different values of the discharge, while the angle of attack was varied from 0° to 90°.

The data obtained at a zero angle of attack are in rather good agreement with the curve representing Eq. 11 Fig. 9, but for the fact that they consistently lie a little below this curve. An increase by 60% in the discharge (which corresponded to an increase by 20% in the average velocity of flow) brought around only a 20% increase in the maximum depth of scour, at the same angle of attack.

The tests with varying angle of attack have shown the basic importance of this factor. For an angle of 15° the maximum depth of scour was 1.9 times that corresponding to a zero angle; for an angle of 30° the increase of scour depth was in the ratio 2.6:1 in respect to the zero angle, for an angle of 45° in the ratio 3.0:1, and for an angle of 60° in the ratio 3.3:1. Increases of the angle of attack beyond 60° and up to 90° did not bring around any further appreciable increase in the scour depth. These values are considerably in excess of those indicated by Fig. 10.

Another interesting observation was that the point of maximum scour depth always occurs very close to the pier wall, so that, if the depth of the foundation is not sufficient, the pier will easily be undermined, and collapse.

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[38] Assist. Prof., Hydr. Structures, Politecnico di Milano, Milan, Italy.

As a conclusion, the fundamental importance of avoiding any angle of attack between the pier and the flow should again be stressed. In order to obtain this, the river banks should be stabilized with adequate measures for a sufficiently long stretch upstream of every bridge crossing. In braided rivers, however, the possibility that an angle of attack occurs should always be taken into account, and the pier foundations designed accordingly.

EMMETT M. LAURSEN,[39] M. ASCE.—The geographical distribution of the discussers and the quality of the discussions indicate the widespread interest in and attest to the import of the problem of scour at bridge piers and abutments and similar obstructions in a stream. Without doubt the crucial issue raised by the analyses is the question of the effect of the velocity of flow on the depth of scour. The position of the writer is that there is a fundamental difference in this regard depending on whether the approach flow supplies or does not supply sediment to the scour hole; that under conditions of no supply, such as a relief bridge, the velocity and the sediment size are important in determining the depth of scour; that under conditions of supply by the approach flow well above the critical tractive force, the velocity and sediment size have little effect except insofar as they determine the mode of sediment movement.

The position of Joklekar, Chitale, Thomas, Ahmad, Blench, and Bradley (either explicitly or by reference) is that the depth of scour is proportional to the two-thirds power of the discharge per unit width. Because the discharge per unit width is the product of the velocity and the depth, for a given geometry (including the depth of flow), the depth of scour should vary with velocity. Alternately, one may rewrite the Poona equation so that $d_s/b = f(y_0/b, F)$. Again indicating that, for a constant geometry, the depth of scour is a function of the velocity. Although the position of Tison and Romita is not entirely clear, one may infer from their analyses that the velocity has an effect on the depth of scour. Interestingly, none of them seemed to stress an effect of sediment size.

The case of the long contraction can be used to illustrate the fundamental difference between clear-water scour and scour in a sediment-transporting stream. The merit of this case for illustrative purposes is that the flow conditions and sediment-transporting competence and capacity can be evaluated with relative confidence and agreement, especially if the complicating features of the zone of non-uniformity and of the ripple and dune formation are disregarded. If one now considers a contraction of some given geometry, two widths and a depth of flow, it is possible for the velocity of flow to be so small that there is no movement of the sediment anywhere. At or below this velocity, that is dependent on the sediment size, there will be no scour, and the flow will behave as if there were a rigid bed. If the velocity is increased, but not to such an extent that there is sediment movement in the uncontracted approach, the contraction will scour. The limit of the depth of flow (or scour) will be a velocity (or tractive force) that will not move the bed sediment. For this case of clear-water flow in the long contraction, it is readily apparent that the velocity of flow and the sediment size, as well as the geometry, will affect the depth of scour. The similar argument for the case of sediment-transporting flow in a long contraction culminated in Eqs. 6, 7, and 8 in which the velocity and depth of flow enter only insofar as they affect the mode of movement.

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[39] Assoc. Prof., Dept. of Civ. Engrg., Michigan State Univ., East Lansing, Mich.