*Discussion on Paper No. 6775**

Measurements of bridge scour and bed changes in a flooding sand-bed river

by

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Mr A. R. B. Edgecombe (Project Manager, Kafue Basin Survey, UN Food and Agriculture Organization) wrote that Mr Neill's Paper had given details of measurements of river bed scour in the vicinity of several bridges on the Beaver River.

53. Sir Francis Spring had used this phenomenon in the design of railway bridges in India, where he reduced the span of bridges by allowing the torrent to scour out an adequate waterway during high floods. This had resulted in immense savings in bridge construction on the Indian railways, and it was surprising therefore that Sir Francis' name did not appear in the list of references given in the Paper.

54. In 1958 model experiments had been carried out at the Hydraulic Laboratory of the Central Laboratory of George Wimpey and Co. Ltd for the Development Board of the Iraqi Ministry of Development, to the instructions of the latter's consulting engineers, Messrs Rendel, Palmer and Tritton on the proposed Fallujah road bridge. Here the principle of scour was used to investigate the possible shortening of the bridge span.

55. In this particular case the investigations showed that the choice between a long bridge with an unrestricted waterway and a shorter bridge with training works to restrict it would be determined by the relative costs.

56. The experiments showed that the scour occurring in the vicinity of the bridge pier was of the order expected of an alluvial river with the characteristics of the Euphrates at this point.

57. The consulting engineers drew to the attention of the investigators Research Publication No. 13 of the Government of India Central Water Power Irrigation and Navigation Research Station, Poona, and to Table 8–1 published within it showing scour around the piers of 17 railway bridges in India. This publication relates the scour to the regime depth given by the formula:

$$D = 0{\cdot}473\,(Q/f_d)^{1/3} \quad \ldots\ldots\ldots\ldots \quad (5)$$

where D = regime depth (m)
Q = flood discharge (cu. m/s)
$f_d = 1{\cdot}76\sqrt{m}$ and
where m = median diameter of silt particles (mm).

58. In the case of the 17 railway bridges the maximum depth of scour around the piers was 1·73 to 2·59 the regime depth. In the present case the model tests showed a maximum observed depth of scour of 1·5 times the regime depth calculated from the above formula. Although this was close to the range observed for the railway bridges, the results could not strictly be compared. This was because stone protection was not proposed for the Fallujah bridge, whereas it was known that stone was provided around some, if not all of the Indian bridges (e.g. the Hardinge bridge over the Ganges and the Sutlej River railway bridge near Phillaur[23]).

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Mr M. R. Gourlay (Department of Civil Engineering, University of Queensland) wrote that fluvial hydraulics was a subject with which many engineers were not particularly familiar. It was pleasing to see, therefore, that the Author had presented data of very real significance to engineers concerned with river engineering works and in particular, bridges. The data given in Figs 9, 11 and 13 of the Paper offered conclusive evidence on two points on which there had been considerable misunderstanding. First, the form of a river bed when it was surveyed under low-water conditions following a flood was not the same as during the flood peak. Second, the large depths of scour observed in certain situations such as at bridges or at bends were not representative of the whole length of the river channel.

60. These points were first brought to the writer's notice in a rather unusual manner when he was trying to adjust the bed roughness of a fixed bed model of a river bend with a constriction at its downstream end. Peak flood levels along both banks were available along the section of river reproduced and it was found necessary to concentrate the artificial roughness in a zone just downstream of the constriction if prototype water levels were to be obtained. Sand was known to have built up on the side of the channel downstream of the bend and it seemed logical to assume that this had come from a bar deposited in the main channel under flood flows, which was then flattened and redistributed during the falling stages of the flood.

61. More recently the writer had the opportunity of observing this type of phenomenon in more detail on a movable bed model of the Pioneer River at Mackay, North Queensland.[27] The model included a couple of sharp bends together with four bridges located in pairs at two sites. One pair of bridges was located right on a bend and the scour during the flood and refill during falling stages was very evident. In this case the river turned through an angle of 60° and while the low flows, even those approaching bank-full, followed a relatively large radius, the river at high flows short-circuited the bend and the main current impinged directly on the right bank at the site of one of the bridges. Extreme turbulence was generated, a large scour hole developed and consequently the loss of several spans of the bridge in the record 1958 floods was not surprising. Observations on the model indicated that the bridge piers had relatively little influence on the scour depth.

62. The other pair of bridges was located almost on a crossing, with the piers of one of them aligned approximately at 40–45° to the direction of the current. As the piers were 40 ft long and the span width between pier centres was only 45 ft the inevitability of high turbulence and erosion was obvious and the bridge was in fact severely damaged in 1958. Here measurements after the flood indicated that the scour was greatest at pier 6 while pier 9, which sank 5 ft 8 in., was actually the worst damaged. Peak flood level at the bridge was about RL 19 while normal bed level is RL − 6. Recent borings at pier 9 indicated an undisturbed clay level at RL − 25 compared with RL − 12 before the construction of the bridge and associated river training works. Thus scour up to 19 ft below normal bed level might have occurred and, although some of this might be attributed to the action of a training wall upstream, there was no doubt that the bridge piers were a major factor. Model observations confirmed the general behaviour, in particular showing that scour was greatest between piers 8 and 10 and that the scour hole was filled during the falling flood stages.

63. In general terms the writer's model observations agreed with the Author's field soundings and the earlier observations of Lane and Borland[24]; deep scours developed at bends and constrictions during high flows, with sand being deposited in the crossing or wide section downstream. The latter was then eroded by the succeeding low flows to fill in the hole downstream of it. The bed load in a given reach appeared to be very much a function of channel geometry, in particular the shape of the bend or constriction at the upstream end.

64. Concerning scour at sharp bends and the Author's remark that a multiplying factor on 'average flood regime depth' of nearly 2 would reflect mainly the influence

of the channel bend, rather than that of the bridge structure, the writer was in full agreement. Popovitch[25] had shown from an analysis of river cross sections that $d_{max}/d_{mean}=f(w/R)$ where d_{max} was the maximum depth at a cross section, d_{mean} was the mean depth of the cross section, w was the surface width and R was the radius of curvature of the bend. When w was taken as the width of the active part of the cross section, i.e. portions containing slack or almost slack water or reverse eddies were ignored, d_{max}/d_{mean} generally lay between 1·5 for small w/R and 2 for large w/R. Such results were of the same order as the values 1·7–2·0 quoted by the Author from regime writers. The maximum value of 2 was of course compatible with the development of a roughly triangular cross section at a bend with the deepest point on the outside and zero depth on the inside.[28]

65. The Author's comment on the methods of scour estimation was important. The writer had always felt that the regime method of taking account of constriction by using the local discharge intensity when determining scour depth was basically sound. He had been influenced in this again by the Pioneer River model where scour at one of the bridges was significantly influenced by the concentrating effect of training walls. In this situation the deepest scour occurred where the discharge intensity tended to be greatest.

66. With regard to using a multiplying or an additive factor to allow for the pier effects a recent paper by Larras[30] was important. Larras analysed various data to find that if the channel bed was in equilibrium without general lowering of the bed then the maximum depth of scoured hole directly in contact with a pier was proportional to the 0·75 power of the pier width. The constant of proportionality was dependent only on the shape of the pier and its orientation with respect to the current and was independent of the water depth and the bed material size.

67. If the bed was not in equilibrium, i.e. there was a general lowering of the bed due to factors other than the bridge piers themselves such as constriction or curvature, then Larras indicated that the erosion at the pier would be increased and that the total erosion appeared to exceed the sum of the general bed lowering and the maximum erosion for equilibrium conditions. The phenomenon thus appeared to be basically additive with a secondary multiplying factor.

68. Finally, with regard to the Author's observation of sand waves of the order of 300 ft wavelength and 3 ft amplitude, similar forms were shown on aerial photographs of the dry bed of the Pioneer River after a period of large floods. They occurred most prominently downstream of two rather abrupt bends and similar scaled-down waves were observed in the model where they were formed within the zone of high sediment transport immediately downstream of the scour hole. In form the waves were almost flat on the rear surface over which heavy bed sediment movement occurred; this was similar to that associated with the plane bed movement generally observed near critical depth flow. The forward face was simply a slope at the angle of repose of the material and the wave moved forward as the heavy bed load over its rear surface was deposited in front of it. Every so often a new wave would develop and travel over the almost level rear surface of the preceding wave.

69. Unfortunately, the writer had no qualitative measurements of these waves which were observed under constant discharge conditions, but he speculated that they could be due to a local instability of the flow within the scour hole resulting in a periodic interaction between the bed form downstream and the flow pattern within the bend. A possible analogous situation was the alternation between a surface jet and a plunging jet on an erodible bed downstream of a level apron.[24] In this case the jet tended to plunge and scour a deep hole which produced a bar of scoured material downstream. Then when the bar was built up to a level sufficient to raise the local tailwater, the jet changed to a surface one which then tended to flatten the bar until the water level dropped sufficiently for the plunging jet to reform.

70. It was also of interest that Larras[30] referred to two different erosion regimes: a low velocity one where the erosion depth remained stable with time, and a high

velocity one where the erosion depth varied in a pseudo-periodic manner about a mean position. Such variations in scour depth might be related to wave-like variations in downstream deposit such as was suggested above.

Mr C. Van Beesten (Chief Engineer, Hydraulic Branch, Harra Engineering Co. International) wrote that the Author had presented an interesting Paper with some valuable observations and data on a sand-bed river during and after an exceptionally high flood. Some of the statements concerning comparison between the scour depths observed around bridge piers and computed scour depths using well-known formulae should, however, be analysed more closely.

72. Laursen in his Bulletin No. 4 (not No. 3 as given in the Author's reference 15) had given a thorough treatment on scour around piers, by model studies backed by continuous observations on a prototype bridge pier. He had stressed the fact that a local scour hole around a pier is filled up completely in a matter of two to three days, as shown in Figs 34, 35 and 36 of the above mentioned bulletin.

73. The Author had described the method of observation with an echo-sounder and the timing of these observations. These were carried out six days and one day after the flood peak for the La Corey bridge and the Beaver Crossing bridge respectively. It is probable that depths recorded after such delays could have been substantially less than those predicted by Laursen.

74. Another point concerned the characteristics of the echo-sounder used in this survey. The usual angle of the wave 'beam' was of the order of 30°. In the case of a relatively small hole, the rebound might occur from the sides of the hole or side of the pier if the path of travel was shorter than that to the deepest point. Special narrow-beam equipment should be used for such purposes.

75. Estimating scour with the 'regime' method was applicable only to the wide alluvial river plains which occur in such places as India and Pakistan. It was a 'rule of thumb' based on some prototype observations and failures of old bridge piers. The multiplication factor 2, i.e. $D_5 = 2 \times 0{\cdot}9\ (q^2/f)^{1/3}$ ft below high-flood level, was used to allow for several factors of major importance. In the wide, meandering rivers, flow directions could change during one flood season, so that the angle of attack on piers could change substantially. Moreover, the flow was often curved, giving rise to further deepening in addition to the scour around an obstruction. The above rule was therefore not applicable to the Beaver River, where conditions were entirely different.

76. Concerning the use of models to establish scour depths, the Author concluded that an acceptable method for correct scaling of model scour had not been found. Laursen[15] showed that the values obtained on a prototype bridge pier agreed reasonably well with depths observed on geometrically similar models, using the model scale as a multiplying factor. The slightly smaller prototype depths could be due to a flood peak duration too short to develop equilibrium depths.

77. Information on distorted scale models was scarce. However, some work had been done on this aspect by Mushtaq Ahmad.[23] From a series of tests on distorted scale models (with various distortion ratios) of the River Sutlej at Islam Headworks, it was concluded that scour depths could be scaled up in accordance with the vertical scale, if the distortion ratio was less than 5. For greater distortion ratios an additional multiplication factor was required to account for the limited angle of repose of submerged bed material. The slopes were apparently adjusted so that a relatively wider and shallower scour hole was formed.

78. The writer thought that formulae (1a) and (1b) in § 19 should read:

$$\text{mean depth} \propto Q^{1/3}$$
$$\text{width} \propto Q^{1/2};$$

and in § 33, the bank-full discharge should read:

$$Q_b = 5000 \text{ cu. ft/s,}$$

which checked with

$$w_s = 2{\cdot}55\ Q^{1/2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

but not with the value of 5500 cu. ft/s given in § 5.

Dr D. M. McDowell (Consultant) wrote that the regime equations could be taken to apply to rivers and canals flowing through alluvium that were free to adjust their main dimensions (width, depth and slope) to flow rate, silt charge and grade, provided that these varied, over a period of time, about average values that showed long term stability. Lacey's definition of regime was given in his 1946 paper[4]:

> 'The term regime is readily understood in the sense that a regime channel is stable, and neither silts nor scours. A regime silt charge requires definition. The Author defines regime silt charge as the minimum transported silt load consistent with full activity of the bed. This activity is by nature random, but nevertheless it must be such that any reduction would lead to partial immobility and, therefore, quasi-rigidity of the channel.'

80. Two conclusions which follow from this definition had a bearing on the Paper. First, that the discharge that was able to maintain a regime channel with a fully active bed was not the smallest discharge that was able to transport bed material. Second, that the regime equations, being based on stable channel conditions, could not be expected to apply without qualification to non-regime conditions such as floods or dry-weather flows.

81. The Author had used the bank-full discharge as though it were the dominant discharge in his analysis of the Beaver River channel. In § 34 he gave the results of calculation of the bed shear stress using the overall slope and also using a slope reduction factor of 1·8 to allow for the effect of losses at the very sharp bends at the boundaries of the meander belt. He showed that these tractive forces were much larger than the critical tractive force required for channel stability according to the U.S. Bureau of Reclamation design figures. The Author suggested that either the bed was active at stages far below the bank-full, or the tractive force criterion was invalid. There were, however, other possibilities. There was no suggestion in the regime theory that the bed became inactive at flows smaller than the regime flow, though it might not be fully active; but the real point was that the bank-full discharge used by the Author was evidently not the dominant discharge. The result of soundings taken along the centre line of the river channel on 13 July, 2 August and 24 September were shown in Fig. 13. If it was assumed for the moment that these soundings were sufficient to define the mean depth of the channel, it would be seen that the mean depth increased from 11·4 ft to 12·5 ft between the first two dates when the flow was near bank-full (see Fig. 6) but was 12·6 ft on 24 September. It was worth noting that the bed had completely changed its profile between the last two dates, showing, as expected, that the bed was active at flows well below the bank-full flow. It seemed unlikely that the bank-full discharge, with its return period of four years, could be the dominant discharge for this river.

82. The other topic dealt with in the Paper was that of scour round bridge piers due to the action of flood flows. It was possible that a regime-type equation could be used to estimate part of the scour effect—that caused by constriction of the channel —because of the regime flow in the river, but it was not possible to use such an equation to estimate the maximum scour effect due to a flood because the data on which the regime equations were based all referred to stable channel conditions. Even if it was assumed that there was a stable flood-discharge state, the writer suggested that the corresponding average flood intensity should be used rather than the maximum flow values shown in Table 1.

83. The Author had drawn attention to the difficulties that still existed in interpreting the results of model studies. It had been known for a long time that friction effects required correction by the addition of roughness in distorted models. This

correction could be made quite easily in the flow channels as a whole, either by the addition of roughness elements in a fixed bed model or with the aid of gross rippling of the bed in mobile bed models. It was not always realized that correction of the average flow would only result in correct mean water levels and discharges and that the local flows, particularly along the banks and around obstructions such as bridge piers, also needed local correction of roughness. This was difficult; the amount of roughening that was required might be so large that the roughness elements obscured important details of the flow. They were usually omitted from mobile bed models and consequently it was not possible to rely on distorted models to give accurate information about scour patterns due to local obstructions. Laursen's work with undistorted models was much more likely to yield useful results but there was still a long way to go before it could be applied to real problems; the effect of friction was only one aspect of model scaling.

84. The Paper, based as it was on limited observations on a single river, could not be expected to lead to a significant advance in the knowledge of flooding sand-bed rivers. However, it had drawn attention to the difficulty of applying present knowledge to real problems. It would have served a useful purpose if it led to renewed efforts to conduct systematic research, including field measurements, into the problems of scour in alluvial rivers.

Mr K. Arunachalam (Assistant Engineer Consultant, Ministry of Transport (Roads Wing), New Delhi, India) wrote that the observations of the Author on the bed changes in the river and at bridge piers was extremely interesting. The writer proposed to confine his contribution to that part of the Paper which dealt with scour at bridge piers.

86. Prediction of scour at bridge piers was a complex matter, and there was no universally accepted procedure to solve the problem. The writer thought that the problem could be seen from two angles. First, it was possible to predict scour depth for known conditions with a reasonable degree of accuracy with the aid of scale models. In a model where the horizontal and vertical scales were not the same, the number of spans should be reduced in ratio to the vertical exaggeration. The thickness of the pier would then be in accordance with the vertical scale and the ratios of flow depth to width of pier, and flow depth to span width, would be unaltered.

87. The second consideration was the scour depth in the design of bridge piers, for which accurate estimation was rather difficult. This was because it was difficult to predict changes in river characteristics, and especially the angle at which the current attack might take place during the anticipated life of the bridge. It was not uncommon to see bridges, originally constructed for axial approach, with an angle of current as high as 90° caused by the movement of meanders. A factor of safety appeared to be necessary in the estimation of scour depth.

88. According to Indian practice, scour depth below high-flood level (HFL) was taken as twice Lacey regime depth. The factor (2) had been obtained as a result of statistical analysis of scour data at 17 bridges in India. The relationship worked out to be:

$$D = 2 \times 0{\cdot}473\,(Q/f)^{1/3} = 2\,D_L \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where D was the depth of scour below HFL, Q the total discharge and f the silt factor. From the concept of regime theory, the multiplying factor could be considered as covering the effects of bridge geometry and the angle of current attack.

89. The writer did not agree with the Author's dismissal of the Poona equation as erroneous. The equation had been evolved as a result of model experiments on Hardinge Bridge (now in East Pakistan) and was never recommended for general application. The writer[27] had shown that this equation could be converted in the following non-dimensional form:

$$\frac{d}{b} = \frac{y}{b}\left[\frac{1{\cdot}95}{(y/b)^{1/6}} - 1,\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where d was the depth of scour below the upstream bed, y the normal flow depth and b the width of pier. The value of y in equation (7) should be one which occurred at the river stage when scour depth was measured, and not the one obtained from fair weather survey. This value was difficult to obtain and further analysis had shown that it could be obtained by the relation:

$$y = 0{\cdot}9q^{2/3}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where q was the discharge per foot run.

90. Equation (7) was non-dimensional and compared with Laursen's design curve[28] shown in Fig. 16. The conformity was reasonably close which proved that the Poona equation evolved during 1938 was not very far different from the recently developed Laursen relationship.

91. A controversy had developed as a result of the publication of Laursen's design[28] procedure. It surrounded Laursen's contention that the velocity of flow did not have any effect on the scour depth; this had been questioned[46] by supporters of regime theory. The writer maintained that there was no actual difference between the two theories and the apparent difference in the basic conception appeared to be due to the difference in the reference point from which the scour depth was measured—from the upstream bed by Laursen and from HFL by supporters of regime theory. The scour depth below HFL could be divided into two parts—the normal flow depth, and the depth of the scour hole below the upstream bed. If the first of these could be expressed as a function of velocity or discharge by means of a regime equation, and the second as a function of flow depth and pier geometry, the resulting solution would satisfy the concepts of both the theories and the controversy would cease to exist. The writer's suggested procedure as shown in equations (7) and (8) was based on the same reasoning, the accuracy of which had been verified.

92. However, it should be mentioned that equation (7), as in the case of the Poona equation, would give absurd results when $b=0$. But this condition was imaginary, since the equation had only been developed in order to find the scour depth at bridge piers with a definite width. What was required was a reliable procedure which would apply to conditions normally found in practice. Equation (7) had been verified to give results within 10% of the actual prototype and model observations over the ranges: width of pier from 6¼ ft to 37 ft, total discharge from 40 000 to 225 000 cu. ft/s, and a y/b ratio from 1·5 to 5. The writer did not hestitate in proposing this procedure for application within verified limits. In his original article[24] the writer had also indicated the procedure used in the evaluation of effects of pier shape and the angle of current attack.

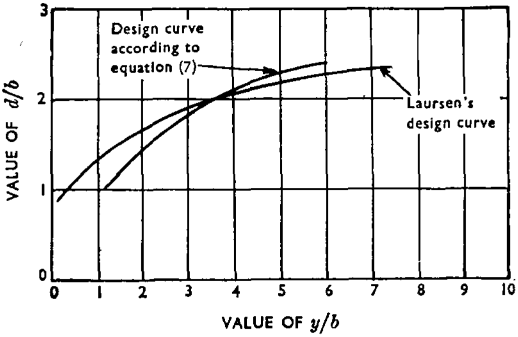


FIG. 16: MR ARUNACHALAM'S VALUES

93. The discussion would not be complete if the writer's procedure was not compared with the Author's observations. The comparison was good except for the peak-flow day. But the discrepancy could be explained by the presence of an inerodible medium. This point had already been mentioned by the Author. In Fig. 16, values of y and b were obtained from Figs 8 and 11 except for the y value of the peak flow day. For this, y was obtained by equation (8).

TABLE 2

S. No.	Date of observation	Flow depth (y) *ft*	Scour depth (D) by Eqn (2) *ft*	Scour depth by actual measurement *ft*
	BEAVER CROSSING			
1	13 June	24·5	36·6	32·0
2	27 June	13·0	21·5	20·5
3	2 August	11·0	18·8	17·5
	LA COREY BRIDGE			
4	13 June	23·0	34·8	28·0
5	19 June	17·0	27·0	24·0

Dr D. I. H. Barr (Senior Lecturer, Department of Civil Engineering, University of Strathclyde, Glasgow) wrote that he was interested in hydraulic model studies and in hydraulic similitude generally. The rationalization and prediction of the behaviour of alluvial rivers and channels represented a most interesting similitude problem and one which was far from being fully solved, as the Author had shown.

95. Dr Barr considered that the Author had been somewhat hasty (in § 17) in dissociating the regime method from the fluid mechanics method and, indeed, in stating that the regime method '. . . tends . . . to pass over the detailed mechanics of alluvial processes'. Supposing a natural relation between streams of different size were to be sought, two conditions for partial similarity which might be put forward were:

(i) that velocities be proportional to the square root of the depth, because the rate of propagation of a small surface disturbance follows this law. This was, of course, the condition observed by Reynolds, which had been applied to all tidal model studies, and to most other channel flow model studies ever since;

(ii) that correspondence in the trajectory of the same silt particle be obtained in both systems assuming the particle to be moving vertically at the still water settling velocity. This condition had been suggested by Lacey and led to the 'two-thirds rule' in circumstances where the first condition is imposed and where the horizontal and vertical scale relations are presumed to be free to differ, as had been recently demonstrated by the writer.[31] The two-thirds rule was commonly quoted in the form $y = x^{2/3}$, where $d_1/d_2 = 1/y$ and $l_1/l_2 = 1/x$; l and d being representative horizontal and vertical lengths respectively, and suffices 1 and 2 distinguishing between two comparable systems.

Taking these two conditions and indicating flow by Q and velocity by V,

$$\frac{Q_1}{Q_2} = \frac{l_1 d_1 V_1}{l_2 d_2 V_2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (9)$$

but

$$\frac{V_1}{V_2} = \sqrt{d_1/d_2} \quad \text{condition (i)} \quad \ldots \quad (9a)$$

therefore

$$\frac{Q_1}{Q_2} = \frac{l_1 d_1^{3/2}}{l_2 d_2^{3/2}} \quad \ldots \quad (9b)$$

Now

$$\frac{l_1}{l_2} = \frac{1}{x}, \frac{d_1}{d_2} = \frac{1}{y} \quad \text{and} \quad x = y^{2/3} \quad \text{condition (ii)} \quad \ldots \quad (10)$$

therefore

$$\frac{l_1}{l_2} = \frac{1}{y^{3/2}} = \left(\frac{d_1}{d_2}\right)^{3/2} \quad \ldots \quad (10a)$$

Substituting in (10)

$$\frac{Q_1}{Q_2} = \left(\frac{d_1}{d_2}\right)^3 \quad \text{or} \quad d \alpha Q^{1/3} \quad \ldots \quad (11)$$

alternatively

$$\frac{Q_1}{Q_2} = \left(\frac{l_1}{l_2}\right) \quad \text{or} \quad l \alpha Q^{1/2} \quad \ldots \quad (11a)$$

in this case the horizontal length of concern is the breadth b

therefore $b \alpha Q^{1/2}$.

Further if s is the uniform flow bed (and surface) slope

$$\frac{s_1}{s_2} = \frac{d_1/l_1}{d_2/l_2} = \frac{d_1/d_2}{l_1/l_2} = \frac{(l_1/l_2)^{2/3}}{(l_1/l_2)} = \frac{1}{(l_1/l_2)^{1/3}} \quad \ldots \quad (12)$$

$$\therefore \frac{s_1}{s_2} = \frac{1}{(Q_1/Q_2)^{1/6}} \quad \text{or} \quad s \alpha 1/Q^{1/6} \quad \ldots \quad (12a)$$

96. Equations (11), (11a) and (12a) were of the general form of the Author's equations (4a), (4b) and (4c) and presumably (11) and (11a) are of the form intended for his equations (1a) and (1b), which seemed to be at variance both with his equations (4a) and (4b) and with the statement in § 20.

97. The foregoing showed that the regime theory might well be based on simple fluid mechanics principles—it might represent a simple natural law for the existence of a stable regime. The Author had raised the question of dominant discharge and the amount and nature of the sediment load. The concept of a series of partially similar systems was thereupon more or less forced upon us. At simplest, uniform sand grains of the same size, shape, and density should be present in each system and it seemed most likely that it would be found that there would exist a unique relation between the flow and the sediment load for a stable straight regime with a given sediment. Now this was, not surprisingly, quite closely related to the fluid mechanics method. In discussion of a recent paper by Ackers,[32] Dr Barr had attempted to demonstrate[33] that a development of basic similarity reasoning (which he had called the method of synthesis to distinguish it from the method of dimensional analysis) allowed the whole range of possible forms of general similarity equation relating to sediment transport in a straight channel with defined sides to be comprehended very simply and the significance of the various groupings of variables to be better understood than when a set of the groupings were obtained by *a* dimensional analysis. Dr Barr did not think it necessary to repeat this previous discussion,[33] but requested the Author to read it. Taking the Einstein bed load equation as typical of the 'fluid mechanics method', its dependence on the submerged Froudian dynamic velocity of the typical sediment particle was marked. Thus it appeared to Dr Barr that there was a possibility that the 'regime method' and the 'fluid mechanics method' both started from more or less the same point, but then proceeded in differing directions,

As understanding grew, it seemed possible that both methods would be assimilated into an overall theory, and that much of the efforts of the protagonists of the supposedly alternative methods to keep them separate will be shown to have been misguided. The obvious first modification to the general similarity equations appropriate to straight channels in seeking to make them then apply to meandering channels was to add a term s'/s where s is the surface slope in an equivalent straight channel and s' the average surface slope of any channel ($s'/s=1$ if the channel was in fact straight).

98. The Author had rightly emphasized the role of sand dunes or waves in forming the boundary roughness of channels which conform to the relations given by the regime method. This apparently conflicted with the concept of sediment fall trajectory as leading to the two thirds rule. It seemed that one should seek either to reconcile the apparent paradox or to find an alternative basis for the two-thirds rule as a natural law. The recent work of Yalin[34] might be relevant here.

99. In discussing the use of hydraulic models in regime and scour studies, the Author had raised some fundamental issues. Dr Barr had gained the impression that Laursen[17] had not in any way considered models with vertical exaggeration. The writer considered that the most suitable way to study local scour in a long mobile bed channel and at the same time to study the overall sand movement was to use two complementary models, one of the whole channel length, with vertical exaggeration, and one of the local conditions which would have natural vertical scale. There would be sufficient problems in the full model in preventing local obstacles from causing imbalance of the overall regime without expecting that details of the scour patterns could be accurately reproduced. There were analogues here with a rather different situation arising in model studies with exaggerated vertical scale where an effluent was entered,[33] and Dr Barr considered it to be better for the general furtherance of confidence in the results from hydraulic model studies, if their limitations were admitted.

100. He considered that the Author should be complimented for his account of his observations in the Beaver River, a particularly fine example of a meandering sand bed river showing a consistent pattern of behaviour over a considerable length, and for his open-minded examination of the place of his observations within the context of regime studies.

Sir Claude Inglis (Consultant; late Director, DSIR Hydraulics Research Station, Wallingford, Berks) considered that the value of the data contained in the Paper was greatly enhanced because it dealt with a 30 mile length of a river in which natural berms had formed at bank-full stage. In 1941[36] he had defined 'dominant discharge' as 'that discharge which mainly determined the stability of a channel and its meanders. It is considered to approximate to bank-full stage.' In India, however, this could not be satisfactorily verified, because bank-full berms did not form except upstream of weirs, barrages, and other obstructions, which affected the slopes of rivers and hence their regimes.

102. The Paper, however, appeared to go a long way towards confirming the validity of that early concept.

103. In § 27, it had been suggested that the data provided by his book[13] were scanty. He referred the Author, however, to chapter 8, entitled 'Maximum depths of scour at heads of guide banks and groynes, pier noses and downstream of bridges', in which the maximum depths of scour at 24 bridges, mostly across large rivers in India, and seven guide banks had been listed. This could hardly be considered scanty data, and the Poona equation gave a good empirical relationship for the range of data.

104. Again in § 27 the Author had stated: 'The so-called Poona equation makes the erroneous prediction that the scoured depth and hence the depth of flow is zero when the pier width is zero.' The assumption of the Author that an empirical equation is inadequate for a specified range if it does not hold down to zero width of pier

is quite unacceptable to the writer. Indeed, even in the case of a basic relation like the Froude number, it is only dominant for a limited range of velocity when the flow is rough turbulent, the Reynolds number being dominant when the velocity decreases.

105. The Author is correct in stating in § 30 that as the pier presents an appreciable obstruction near the bed, either because of its width or obliquity to the current, scour may be very deep, causing diving flow and spiral rollers. This is true of course whatever the cause of the rollers. This was exemplified in the Rohri Canal in Sind where, as a result of retrogression in the canal, the depth of flow over the pavement of one of the bridges was reduced by some 4 ft, or to two-thirds the regime depth. As a result, not only did a very deep scourhole develop downstream, but enormous 'bellies' also formed on both banks, and even when the pavement was lowered the scourhole persisted. This was eventually eliminated by extending bank groynes out from both banks.

106. The Author's discussion of the data of the Beaver River was very interesting, and his conclusions in §§ 45–47 demonstrate the value of the regime concept and equations in assessing the behaviour of the river.

Mr A. R. Thomas (Consultant, Binnie and Partners), as one of the originators of the Poona equation for depth of scour at bridge piers, wished to correct a misconception appearing in § 27. To predict from the equation that the scoured depth was zero when the pier width was zero was to misapply the equation. The equation originally published was[13]

$$\frac{D_s}{b} = 1{\cdot}7\left(\frac{q_c^{2/3}}{b}\right)^{0{\cdot}78} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (13)$$

or approximately,

$$D_s = 1{\cdot}7\, b^{1/5}\, q^{1/2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (13a)$$

where D_s is the maximum scoured depth below water surface, b the width of pier, and q_c the discharge per unit width upstream of pier, in ft-sec units. It was an empirical equation and therefore should not be applied without proper justification beyond the limits of the experimental data, shown as $q_c^{2/3}/b$ ranging from 2 to 10, which covered most cases met in practice. It was, moreover, said to give 'an approximate indication of the maximum depth of scour around a pier with cylindrical nose in Ganges sand, if the flow is straight and normal, *within the range of . . .* $q_c^{2/3}/b$ *shown*'. (Writer's italics.)

108. The writer preferred to use total scoured depth rather than scoured depth below upstream bed because the latter tended to vary locally and with time, as indeed shown by the Author in the case of the Beaver River.

109. The assumption was made (based on observations with models of different scale ratios) that the equation would apply to full-scale conditions and this appeared to be borne out by the data. The pier widths in the two cases quoted in the Paper were not stated. Scaling them from Figs 8 and 10 as effectively 5 ft in the case of La Corey and 6 ft in the case of Beaver Crossing, and taking q_c upstream of the piers therefore as 120 and 137 cu. ft/s per ft respectively, the comparison between total scoured depths calculated by the Poona equation and those measured was as follows:

	La Corey	*Beaver Crossing*
calculated	29	32 ft
measured	28	32 ft

This degree of accuracy might be fortuitous but the Author would perhaps concede that the Poona equation provided in these cases remarkably good estimates of the depth of scour, better than given by any other method quoted by him.

Dr T. Blench (Professor of Civil Engineering, University of Alberta) wrote that few colleges had instructed engineers in hydrology and fewer still in 'evolution of the river bed'—effectively 'river regime'—which the International Hydrologic Decade now accepted as an important section of hydrology.[36,37] It was not surprising, therefore, that the only co-ordinated set of bridge scour observations over a wide enough range of variables for analysis was that of Inglis,[13] and that formulae and ideas used by engineers for estimating bridge scour might be inadequate, inconsistent and sometimes fallacious.[39] The Author's scientific approach to bridge scour came at an appropriate time. He presented and discussed, judicially, the few available significant field and laboratory observations and ideas of specialists. He did not offer instruction or advocate a particular approach, but introduced the reader easily, effectively and without bias to the scientific essentials of the subject. His own observations on the Beaver River illustrated how a scientific quantitative research into the bridge scour problem must be made via knowledge of the laws behind river self-formation and the phenomena consequent on them. Paragraph 31 summarized the reasons for this kind of approach; in short, scour at a bridge pier was a local enhancement of regime depth occasioned in the pier's neighbourhood by constriction and curved flow.

111. As the writer's publications on the use of formulae of the type (2) had been mentioned some explanation was appropriate. He had advocated a multiple of a regime depth as a practical expedient for bridge pier scour *design*, but agreed with Mr Neill that, basically, the usage was illogical. The pros and cons of this policy were briefly as follows. To produce a formula, based on facts, in terms of scour below a bed compelled recourse to models, for there were no field data specifying bed position along with scour. Now, even the best model work is for different and indefinite phases[40] of approach flow, so cannot be trusted yet for extrapolation to field scale; it deals almost entirely with attack in the direction of the pier axis of symmetry. Moreover a formula based on such models requires the engineer applying it to know the approach depth (which he does not), or estimate it (which he cannot do accurately), and apply a coefficient to allow for orientation of attack, interference of other piers, debris caught on the pier, and possibly stone thrown haphazard during various emergencies. After all this empiricism he still cannot be sure that the original formula applies exactly to the prototype for the simple conditions it purports to represent. The Inglis data for scour round bridge piers (which the Author might like to plot, with his reply to discussion, if only because of their unique information value) indicate that, over an enormous discharge range in certain types of sand rivers, total scour below high flood under believed very adverse approach conditions and attendant circumstances, can be represented with useful accuracy *for design* by a useful range of multiples of regime depth calculated for the whole channel. A designer can perform this calculation, will know a reasonable figure for high flood level, will usually need to design for a meandering channel that will assume a most unfavourable approach to a pier eventually, and can soon gain the experience to modify coefficients favourably when circumstances warrant. Above all he knows that field facts are behind his formula. Actually the writer uses various approaches when making his own estimates, although keeping the original Inglis work as a factual field background that must never be ignored. To explain how to use these various approaches would require a small text, and the writer does not feel he could improve on Mr Neill's recent one[39] which has the added practical advantage that it outlines the observation methods and field observations required for a scientific fluviological analysis of rivers aimed at solving, inter alia, the bridge scour problem.

Dr W. H. R. Nimmo (Civil and Hydraulic Engineer, Queensland, Australia) wrote that the Paper presented valuable data relative to the effects of floods in a stream flowing in incoherent alluvium (sand) and of sufficient width and depth to permit the stream to form its own channel. The Author had concluded that under such conditions the width and depth of the channel related to its discharge at bank-full stage was in good agreement with established regime relationships. He also stated that the

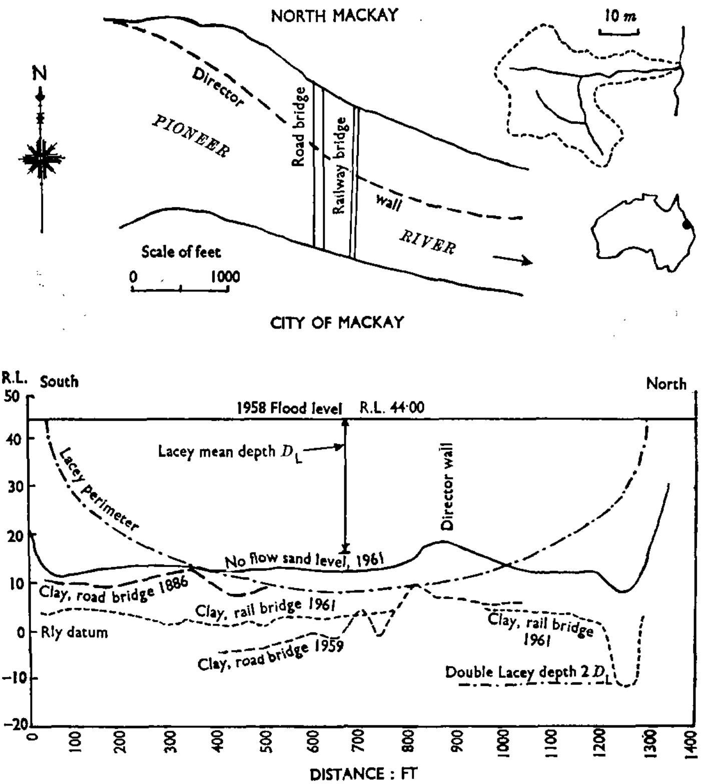


FIG. 17: PIONEER RIVER DATA

main cause of scour appeared to have been the constricting effect of bridges on the flood flow and that local holes at piers appeared to have been small.

113. Measurements obtained by echo-sounding during floods in the large sandy rivers in India, while generally supporting regime theory showed that local curvature of flow at bridge abutments and the ends of groynes might cause scour equal to or exceeding twice the Lacey depth. Of interest in this connexion was the following information on the Pioneer River.

114. The Pioneer River flows into the Pacific Ocean at the City of Mackay, Queensland. The catchment is 715 sq. miles and 45 miles long. The upper portion of the river and its tributaries drain a basin surrounded by mountains with peaks exceeding 4000 ft. The tidal portion of the river extends for 10 miles. At 2 miles above its mouth, at the City of Mackay, the river is crossed by the Forgan bridge carrying a road and almost parallel to it, and about 40 ft downstream is the Mackay Harbour railway bridge. The bed of the river here is entirely of sand underlain by clay (Fig. 17). The latitude of Mackay is 21° S. and intense tropical storms produce large floods in the Pioneer River.

115. The greatest recorded flood occurred in January 1918, when an intense cyclone crossed the coast at Mackay, subjecting it to a rainfall of 55 in. in three days; however, the intensity decreased inland.

116. More detailed information was available relative to the flood of February 1958 (caused by a tropical depression) which produced heavy rainfall, particularly on the headwaters. The average rainfall over the catchment for the four days of the storm was 5·4, 7·3, 20·8 and 3·7 in., a total of 37·2 in. The run-off for the four days was 21·8 in., i.e. 58% of the rainfall, but this value was probably higher for part of the time. At Pleystowe—a gauging station just upstream of the tidal limit—the peak discharge was 248 000 cusec, corresponding to 468 cusec/sq. mile, or a value of Myer's C of 10 800. It was estimated that 209 000 cusec passed over or under the bridges within the banks shown in Fig. 17, and the remainder over or through openings in the long northern approach bank.

117. Dimensions of the Lacey ellipse in Fig. 17 were based upon his 1930 formulae, viz:

width $W = 2{\cdot}67\ Q^{1/2} = 1220$ ft
mean depth $D_L = 0{\cdot}47\ Q^{1/3} = 27{\cdot}9$ ft
maximum depth $D_{\max} = 1{\cdot}273\ D_L = 35{\cdot}6$ ft.

118. Considering the effect of the abutments, it was perhaps not surprising that the area between the 'no flow sand level' and the 1958 flood level (40 000 sq. ft) was about 12% greater than that of the semi-ellipse.

119. The Forgan bridge, completed in 1887, originally consisted of 90 ft truss spans on concrete piers supported on deep cast iron cylinders. Reconstruction completed in 1938 involved the substitution of 45 ft rolled steel joists resting on the existing piers and new intermediate piers of concrete supported on reinforced concrete piles. Similar piles supported an extension of the existing piers to provide for widening of the bridge.

120. The railway bridge, completed in 1938, consisted of 30 ft spans on concrete piers supported on piles.

121. Floods, which were not only large but frequent, scoured out the sand which was replaced as the flow diminished. It appeared that while the clay was uncovered it was partially scoured by each flood, the accumulated depth of erosion being indicated by the difference between the lines of long and short dashes in Fig. 16. Prior to the completion of the rubble director wall about 1924, the curvature of the river would have forced the main current to the north side and turbulent conditions would be accentuated by the current being skew to the piers and abutment.

122. The 1958 flood caused settlement of several piers at the south end of the Forgan bridge. The Harbour Board then carried out exploratory boring along the line of its railway bridge, which revealed that the clay had been scoured to the dotted 1961 line. The maximum depth below the 1958 flood level had reached twice the Lacey depth $2D_L$ near the north abutment.

123. After completion of the director wall, the current was directed towards the south side where the lesser depth of erosion of the clay at the railway bridge compared with that at the road bridge, might perhaps be attributed to longer exposure of the latter to the current deflected by the director wall.

124. Experience at the site of the Mackay bridges indicated that clay—and probably rock that decomposed upon exposure—might be gradually eroded by successive floods, and confirmed the double Lacey depths found in India.

Mr S. V. Chitale (Chief Research Officer, Central Water and Power Research Station, Poona) wrote that estimation of scour round piers in alluvial channels was normally done in India by using the Lacey equation

$$R = 0{\cdot}47\,(Q/f)^{1/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

The multiplying factor adopted was 2 and R was approximated to the flow depth, thus giving an estimated scour depth below water level equal to twice the Lacey regime depth.[13] The Poona equation by which the Author perhaps means the empirical relation

$$\frac{ds}{b} = 1{\cdot}70\,(q^{2/3}/b)^{0{\cdot}78} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (15)$$

wherein d_s = depth of scour below W.L.
b = width of pier; and
q = discharge per foot width upstream of the piers,

was evolved from experiments done specifically for scour estimation at the Hardinge bridge piers and hence its extrapolation would obviously be inappropriate. The work done at the Poona Research Station related to the period prior to 1944 and in the absence of a more reliable and better proven method of scour estimation had served to provide a rough tool to bridge designers. The Blench equation, namely

$$d_s F_{b}{}^{\frac{1}{3}} = 1{\cdot}35\,q^{0{\cdot}74} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (16)$$

given on page 96 of 'Regime Behaviour of Canals and Rivers' had not been popular in India. In recent times at least two additional approaches to the problem had been advocated. One following Laursen and the other Garde.[41] The scour equation developed by Garde was

$$\frac{d_s}{d} = 4{\cdot}0\,\eta_1\eta_2\eta_3\,\frac{1}{\alpha}\,F_r^{n} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (17)$$

wherein d is the flow depth upstream of the piers, and
η_1 depends on the drag coefficient C_D of the bed sand grain,
η_2 depends on length to breadth ratio of piers and Froude number,
η_3 depends on shape of the pier ends,
α is the contraction ratio, and
h depends on the value of C_D.

Though claims have been made about the superiority of one equation over the other, adequate field data for comparison of all the available methods of estimation of scour round piers was not yet available.

126. The Author stated in § 24 that the regime relations were applicable to sand bed channels where boundary friction depended on bed form while tractive force criteria was more correct for coarse materials where grain size effectively determined frictional resistance. The main difference between the regime theory and tractive force method was, however, not one of type of boundary friction. The regime equations were applicable to channels where bed shear was in excess of critical tractive force and thus caused bed movement. The tractive force method was applicable to channels where bed shear was less than the critical tractive force and hence bed movement was absent.

Mr G. Lacey (Consultant, Sir M. MacDonald and Partners) wrote that the Author had endeavoured to interpret observations on the Beaver River in terms of the 'regime concept' with considerable success, notably in respect of the dominant discharge as determined from the bank-full water surface width and the scoured depth at sharp bends. In both these cases the width and depth were due to natural causes. The bridges presented a very different, man-made problem.

128. The Beaver River, in common with great alluvial rivers such as the Ganges, had been self-formed by deposition, the width of the alluvial flats being limited by the glacial trough in which it flowed. The bed consisted of clean (loose) sand; the sides were also sandy but tolerably stable owing to the cementing action of the fine wash load and vegetation. The River Ganges meandered, almost without restriction, within a very wide belt of alluvium which was essentially sand deposited previously. The sand was several hundred feet deep whereas at the La Corey bridge a gravel stratum was encountered; both rivers had sandy beds.

129. Regime theory postulated a minimum *natural* width for alluvial sandy rivers

when in high flood. This would usually be found a short distance upstream of a well defined bend but was not a constricted width in any sense. It was adopted by railway bridge engineers in India, after making allowance for the reduction in waterway due to the piers and omitting the two short land spans. The width having been determined in this way, the river was trained through it by means of appropriate pitched guide banks, the greater length being upstream and heavily protected at the 'nose'.

130. The problem was to estimate the profile of the cross section during a maximum flood and it was on the estimated level of the water surface at the height of the flood that all the scoured depths might be said to 'depend'. It was now common practice to assume that the maximum scoured depth as measured from the high-flood water surface would not exceed twice the mean depth. This implied that the piers would be immersed by twice the mean depth and it was usual to sink the pier kerbs to a total depth of three times the mean depth below high-flood level. This allowance, which was termed 'grip' included the relatively small accentuation of the scour due to the presence of the pier. This well established principle was very simple and preferable to referring scour to an ill-defined, assumed normal, bed level.

131. A remarkable feature of the Beaver River was the regularity of the meanders and the uniformity of the bank-full dominant water surface width. The banks were relatively stable and the river constricted. From Fig. 15 and the numerical coefficient of 2·55 in equation (4a) it was clear that the dominant discharge was 5000 cusec and not 5500 as quoted in § 5. The definition of the mean depth as the area divided by the water surface width (see § 2) was admirable; the later definition of d_m as the area divided by the width was unfortunate and a source of confusion however defined. The Author concluded that an acceptable method of scaling scour depths had not been established. This is incorrect so far as the determination of the cross sectional profile of the river itself was concerned. Piers and man-made obstructions led to scour of a different character due to the local destruction of energy. In this case, the scour, if the flood persisted long enough, depended more on flow pattern than the precise grade of sand.

132. The dominant discharge was 5000 cusec and the maximum discharge 20 000 cusec. That implied, in terms of equation (4a) that the natural water surface width for a record flood was twice the dominant width. The river, therefore, was greatly constricted. The bridges obstructed the passage of a high flood even further because of the unsatisfactory entry conditions. The water surface width of 180 ft was practically constant but the mean depth, as rigidly defined, increased from about 8 to 10 ft in the downstream direction. Since the discharge in unit width was a constant, and the depth steadily increased, it would appear that the size of sediment diminished in a downstream direction. The median diameter of the Beaver River sand was taken as 0·50 mm and employing appropriate sediment factors the Author arrived at the equation

$$\text{Average flood regime depth} = 0{\cdot}90\, q_f^{2/3} \quad . \quad . \quad . \quad . \quad (18)$$

It would appear preferable to rely on actual observations on the river.

133. Adopting the data given in § 33 and bearing in mind that the mean depth of 9 ft was rigidly defined, it would be found that the correct equation was:

$$\begin{aligned} A/w_s &= d_m = 9{\cdot}0 \text{ ft} \\ q &= 5000/180 = 27{\cdot}78 \\ d_m &= 0{\cdot}98\, q^{2/3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19) \end{aligned}$$

134. This equation could be applied to the computation of scoured depth at the bridges, the value of q being determined by dividing the flood discharge by the effective net width of the flood water surface. The net water surface width at La Corey bridge could be taken as 200 ft, which gave a value of q of 90; similarly at the Beaver Crossing bridge the net width was 180 ft, giving a value of q of 111. Adopting a factor of 1·50 appropriate to a moderate bend scoured depths were obtained as shown in Table 3.

135. There was very fair agreement between the actual *maximum* scoured depths

TABLE 3: SCOURED DEPTHS

	La Corey bridge	Beaver Crossing bridge
Flood discharge	18 000 cusec	20 000 cusec
Net flood water surface width .	200 ft	180·0 ft
Value of q discharge intensity .	90 sq. ft/s	111·1 sq. ft/s
$d_m = 0{\cdot}98\, q^{2/3}$	20·08 ft	23·11 ft
$1{\cdot}50\, d_m$	30·10 ft	34·70 ft
Maximum scoured depth . .	28·00 ft	32·00 ft
Percentage difference . . .	−7·0	−7·8

and those computed. The river might almost be said to have been flumed through the bridges because of the great difference between the dominant discharge and the actual high flood. There was certainly insufficient width for marked curvature effects to be produced, despite the awkward entries on the left bank at both bridges. The Author drew attention to the increased depths which would have occurred if the bridges had been located near sharp bends and correctly stated that such depths would have reflected mainly the influence of the bends rather than that of bridge structures, presumably piers. It was for these conditions that the Indian bridge engineer provided. On rivers such as the Beaver, narrow in respect of high floods and with relatively stabilized banks, there was little curvature effect and the design of the piers and the obstruction they made was of first importance. Fig. 8 would suggest that the principle adopted in constructing the piers was that of driving the piles down to a firm substratum. It would be noted that the top of the concrete capping of the piles for the La Corey bridge was actually above the approximate original bed instead of being constructed at a lower level.

136. It was self-evident that a slender pier closely aligned with the flow should not cause a serious scour hole but for such piers to be effective it was essential that the river should be trained and the entry conditions upstream of the bridge controlled. If these precautions were neglected (particularly when the bridge was a skew bridge) the slender piers, far from proving advantageous were a menace. If curvature of flow were developed upstream the flow would impinge on the slender piers as if they were plates and very serious damage could occur, and had in fact occurred in the past. In this respect a cylindrical pier had many advantages.

137. The Author contended that it could easily be shown that the regime and tractive force theories conflicted and that numerous objections could be raised to both. This statement arose from a failure to realize that the tractive force and regime concepts applied to two entirely different phases of transport. The 'tractive force approach' assumed a channel which was formed by the boundaries being eroded until they were 'stable'. The boundary thereafter was *dead* and inert. The regime approach assumed the formation of the channel by *deposition* of loose sediment until such time as the enhanced slopes permitted the sediment admitted to the channel to be swept forward. The bed was in constant movement and *alive.*

138. Early canals in India exhibited both phases. The upper part of the Ganges canal, to quote one example, was excavated in 'earth' and the gradients were sufficient to ensure that the sediment was swept forward over a sub-grade of earth. Here the tractive force concept might well apply. In the lower part of the system where slopes were inadequate the loose sediment was thrown down on the bed and on the sides, a 'regime' slope was established and 'regime' channels were self-formed. In the upper part of the system there was a certain degree of freedom in respect of slope, but in the lower part the slope was a dependent variable.

139. Regime theory indicated that a heavy sediment load demanded a channel of greater width. If such an increase in width were not allowed for the dunes would

be created even in a straight channel, with a corresponding increase in the value of the channel rugosity coefficient. As the ripples were moving it was inappropriate to regard them as rigid or to relate them to the Nikuradse roughness. The ripple height appeared to be a function of the size of sediment, whereas the height of the dunes was probably related to the depth of the channel. The conclusions as given in §§ 45–49 were unexceptionable.

140. Very little progress would be made in the field of regime flow until such time as it was recognized that the tractive force and regime theories apply to entirely different phenomena. The tractive force or 'drag' theory, subject to certain simplifying assumptions, lent itself to academic treatment and a 'rational' solution. The so-called regime theory could no longer be dismissed as 'wholly empirical' but presented a challenge which must be faced.

The Author, in reply, thanked all contributors for their comments, which had added considerably to the value of the Paper. As Dr McDowell had written, limited observations on a single river could not be expected to lead to a significant advance in knowledge, but contribution of such small quantities of information by a number of engineers, together with critical discussion, might well do so. It was with this hope in mind that he had prepared the Paper, but he had been surprised at the extent of interest shown.

142. In reply to Mr Edgecombe, the Author was aware of Sir Francis Spring's pioneering work, but not having read the original reference he was reluctant to quote it. It would be interesting to know how actual behaviour at the Fallujan bridge compared with the model predictions.

143. Mr Gourlay had quoted some interesting experiences with river models, and observations of large sand waves in a river. The serious consequences of constructing oblong piers skewed at large angles to the flow, or of allowing rivers to develop large angles of attack on such piers, could not be pointed out too often, as this had been the direct cause of several bridge collapses. Official reports sometimes glossed over this fact and blamed floods of unforeseeable severity and so on.

144. Larras' Paper of 1963, referred to by Mr Gourlay, drew heavily on the experimental data contained in *Etude des affouillements autour des piles de ponts*,[20] which reported the most extensive experimental investigation of pier scour known to the Author. The two different scour-depth versus time relationships, referred to by Mr Gourlay in § 70, corresponded to conditions of (1) zero or very low bed-load transport, and (2) appreciable transport with dunes progressing downstream. Larras' equation for local pier scour was not dimensionally homogeneous, but Laursen's was.

145. Mr Van Beesten had mentioned the timing of the scour observations. The Author had to admit that some filling of the local scour hole at the La Corey bridge pier could have occurred before the observations were made, and that this constituted a weak point in the analysis. In the case of the other bridge the objection was not valid, because when the observations were made the stage of flow was only slightly below its peak value. The effective width of the echo-sounder beam was of the order of only 10° when the instrument was properly adjusted, and tests had indicated no serious source of error from this cause. It was true, however, that it was probably impossible to measure the absolute maximum depth of scour right against a pier by means of the method employed.

146. The Author had not intended to assert that the 'regime' method of scour estimation was originally intended to apply to this type of river. He had merely attempted to check how its predictions compared with observation. With regard to scaling scour depth from models, the Author had studied the data of Laursen and Ahmad, but was not entirely convinced by their conclusions, particularly with respect to scaling of the bed material, although Laursen's comparison of model and prototype was persuasive. He thought it safer to retain some scepticism towards model scour predictions. After obtaining the Beaver River data he had attempted to model the

bridge crossings in the laboratory, using a similar natural bed sand, but had no success in reproducing the observed phenomena.

147. Mr Van Beesten had drawn attention to several unfortunate typographical errors in the Paper. Errata were listed at the end of the Discussion.

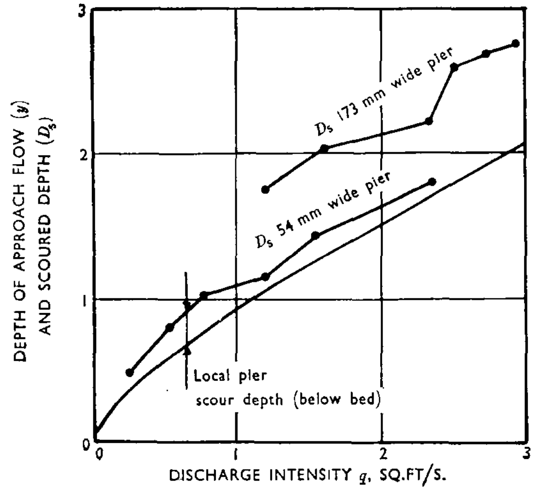
148. Dr McDowell had commented that regime equations could not be expected to apply without qualification to non-regime conditions. The Author had discussed this point in §§ 26 and 27, and indicated his reservations concerning the procedure, which appeared, however, to have been advocated to some degree by Lacey, Inglis and Blench in their writings.

149. The Author had not simply assumed that bank-full discharge was dominant, as Dr McDowell seemed to suggest, but had tested the hypothesis, obtaining the rather remarkable result shown in Fig. 15 and discussed in § 33. Subsequent analysis of channel cross sections for another meandering sand-bed river in Alberta, which had a bank-full discharge of approximately 40 000 cu. ft/s occurring only once every 10 or 20 years, had also yielded points falling within the scatter band of the regime canal data. Considering these facts together with Nixon's analysis,[9] the Author was strongly drawn to the idea of bank-full discharge as dominant in the sense of 'channel-forming'. He had discussed the concept of dominant discharge in § 18, but had avoided a firm definition of the term. Dr McDowell had asserted that the bank-full discharge was evidently not the dominant discharge, but had not defined his use of the term. In fact the concept of dominant discharge could only be a loose one, because there was no reason to believe that any single discharge within the range of a river's discharge cycle, run steadily, would produce the same channel. The Author was inclined to the view that the most significant discharge with respect to slope and bed-material transport might be considerably smaller than that with respect to channel cross section and meander wavelength, and thought that a somewhat similar view had once been expressed by Inglis.

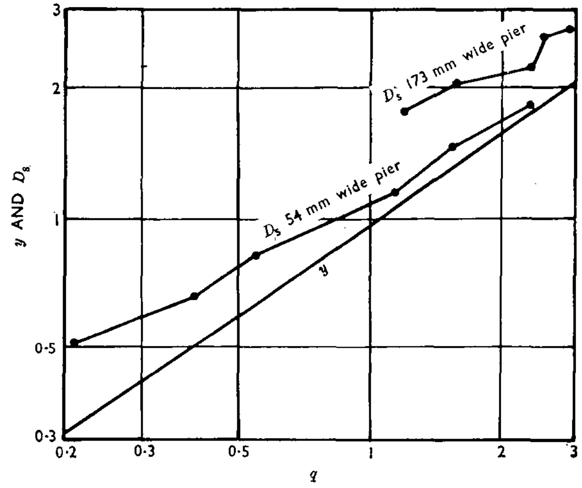
150. Mr Arunachalam had referred to a method of dealing with pier scour in distorted models that seemed to be reasonable, but the Author had insufficient experience of loose-bed models to comment further.

151. The Author's remarks on the Poona equation for pier scour (as quoted by Mr Thomas) had been commented on by Mr Arunachalam, as well as by Sir Claude Inglis, Mr Thomas, and Dr Blench, therefore fuller discussion of it seemed called for. The Author was grateful for Mr Thomas' clarification of its background, and understood that the equation had originally been published with a number of restrictions. Unfortunately it had been quoted in print on several later occasions without these restrictions being stated, and the Author had not previously seen any reference to the particular defect that he had remarked on. In fact, the discussion that followed Laursen's Paper of 1962 (ref. 28) was completely confused on this point, because some contributors had quoted or referred to the Poona equation and other Indian data without making clear that 'scour depth' as used in India and as used by Laursen meant entirely different things; the same symbol was even used for the two different dimensions, one including the depth of flow and the other not. Thus the two groups had in effect been talking past each other. Mr Arunachalam had realized this too and had creditably tried to reconcile these differences in approach; his Fig. 16 was quite persuasive.

152. In an internal report prepared in 1964, the Author had examined the Poona equation more closely. Fig. 18, drawn from that report, showed both arithmetic and log-log plots of some of the Hardinge bridge pier model data, showing depth of approach flow plus depth of local scour at nose of pier versus discharge intensity, for two different widths of pier. The arithmetic plot suggested what the more extensive model data in *Etude des affouillements autour des piles de ponts*[20] established without doubt, namely that above a certain threshold the local depth of scour was virtually independent of discharge intensity (i.e., of velocity) but heavily dependent on pier width, whereas depth of approach flow depended on discharge intensity and not



(a) ARITHMETIC PLOT



(b) LOG LOG PLOT

FIG. 18: ARITHMETIC AND LOG-LOG PLOTS OF SELECTED HARDINGE BRIDGE MODEL DATA REPORTED BY INGLIS[13]; SAND SIZE 0·3 MM.

at all on pier width. Therefore, to express their sum, 'scoured depth', as an exponential function of pier width and discharge intensity was physically and mathematically indefensible, although the log-log plot showed why it was possible to approximate the results in this way over a limited range. The Author had also found that the actual data from the Hardinge pier models accorded well with the much larger body of data reported from the Chatou tests,[20] and with limited model observations of his own.

153. The Author hoped that the foregoing had explained his objection to the Poona equation on logical grounds, and shown the deceptiveness of log-log plots in certain cases. He apologized for being entirely destructive on this point, but it was sometimes necessary to do a certain amount of demolition before a better theory could be constructed. From the contributions by Mr Chitale and Mr Lacey, it appeared that the Lacey-Inglis equation

$$\text{Maximum scoured depth} = 2\times 0{\cdot}47\,(Q/f)^{1/3} \quad . \quad . \quad . \quad . \quad (20)$$

was currently preferred in India to the Poona one.

154. Dr Barr thought the Author had been hasty in his remarks in § 17 on the regime and tractive force methods, but the Author saw no reason to withdraw them. The fact that the regime equations could be shown to satisfy certain similarity criteria did not necessarily conflict with the Author's comment that the theory passed over the detailed mechanics of alluvial processes. For instance, much of the regime literature made virtually no reference to the physical configuration of the bed, to transport rates, or to the details of transport and self-adjustment processes. This was a statement of fact, not a criticism.

155. The similarity criteria worked out by Dr Barr were familiar to the Author from the work of Lacey and Blench. As Dr Barr had pointed out, equations (1a) and (1b) were wrongly printed in the Paper. Dr Barr had raised the question of why similarity of suspended-particle fall trajectories should be maintained in systems where bed or contact load seemed to play a primary role; this was a point that the Author had sometimes pondered. The remarks of Mr Lacey in § 137 might be relevant in this connexion.

156. Dr Barr had suggested that the 'regime' and 'fluid mechanics' approaches might eventually be assimilated into an over-all theory of fluid-solids motion. The Author believed this was inevitable, and thought that the time might not be very far off. He also agreed with Dr Barr's remarks on scour in models.

157. Sir Claude Inglis had quoted his 1941 definition of dominant discharge, for which the Author was grateful. The Author's use of the term seemed to be more in line with this than Dr McDowell's. This point had been discussed further in § 149, in reply to Dr McDowell.

158. The Author could only agree that, considering the general scarcity of data on scour, Sir Claude's compilation of Indian bridge data[13] represented a contribution that was far from scanty, and he was glad to withdraw the comment. Sir Claude's data were discussed further in reply to Dr Blench. He had already discussed his logical objections to the Poona equation in reply to Mr Arunachalam, but could accept Sir Claude's view that they did not necessarily invalidate its use as an empirical design equation within specified limits. Sir Claude's note on experience with the Rohri canal was of much interest.

159. Dr Blench had commented on the comparative neglect of fluvial hydraulics in civil engineering instruction, and had given a reasoned defence of the use of empirical scour equations for design, to which the Author could take no exception. He thanked Dr Blench for his kind comments on the Paper and on another reference.

160. Dr Blench had invited the Author to plot Inglis' scour data[13] with his reply. Accordingly, Fig. 19 plotted scoured depth versus total discharge for the bridge pier, guide bank, groyne and sharp bend sites tabulated by Inglis, 24 in all. (The data for seven cases of scour downstream of bridges were not plotted, because of the Author's uncertainty as to the degree of obstruction by rip-rap cones, etc.) All the data referred to sand-bed rivers, and the Author understood that the bridge sites were for the most part effectively unconstricted. In all but five cases the reported median sand size was between 0·30–0·40 mm. The remaining five sites had finer sand, but as their data showed no systematic departure from the others the Author did not

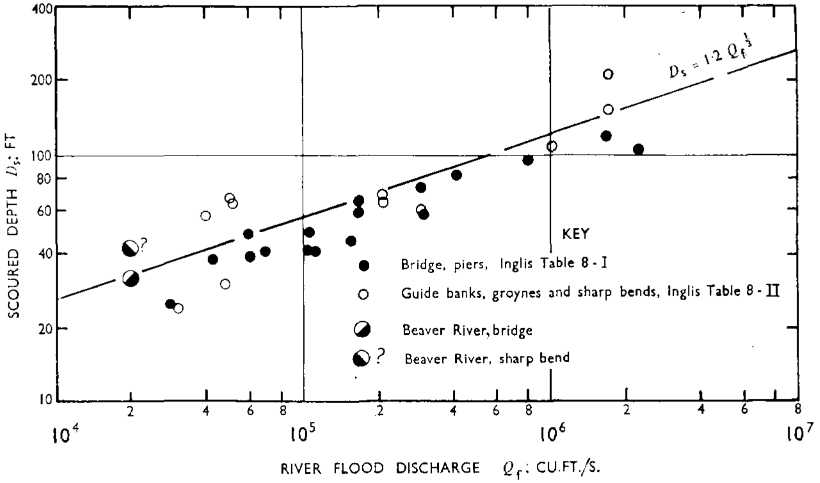


FIG. 19: BEAVER RIVER SCOURED DEPTHS COMPARED WITH INGLIS' DATA[13]

believe that it was of any significance to introduce the 'silt factor' into the relationship. Also shown on the diagram were two points representing the Beaver River data reported; the point for maximum depth at a bend was somewhat uncertain, as discussed in § 13 of the Paper. In the Author's view the diagram gave good support to the application of regime concepts to sand-bed rivers. A tentative upper envelope line for the bridge pier sites was drawn on the diagram, the empirical equation of which was:

$$\text{Maximum scoured depth} = 1{\cdot}2\,(Q_f)^{1/3} \quad . \quad . \quad . \quad (21)$$

161. In reply to Dr Nimmo, the Author was glad to know that echo-soundings were being done on the large Indian rivers, and hoped that findings would be published in due course. Dr Nimmo's data on the Pioneer River bridges constituted a valuable contribution, particularly because they indicated that scour in a material which appeared to be only slightly erodible might in time reach depths similar to those that would be reached quickly in sand. Dr Blench had always maintained this, and would be gratified at Dr Nimmo's factual confirmation. Dr Blench and others had noticed that on many of the entrenched rivers in Western Canada, where the bed consisted of a relatively thin stratum of sand and gravel overlying glacial till, clay or soft bed-rock, maximum depths of alluvium often corresponded closely to expected maximum depths of scour.

162. Mr Chitale, together with Mr Lacey, had disagreed with the Author's remarks in § 24 on the applicability of the regime and tractive force methods respectively. Both contributors stated that the regime method was applicable where the bed was active, and the tractive force method where it was not. The Author could not fully accept this simple view without further proof, for two main reasons. Firstly, most of the published bed-load formulas, starting with Du Boys' of 1879 and including Meyer-Peter and Müller's of 1948, linked the rate of transport closely with the bed shear stress or 'tractive force'. Secondly, the regime equations were derived entirely from data on sand-bed channels, and it had never been shown that they could be applied to gravel-bed channels in the usual engineering range of size without bringing in variable correction factors that in effect destroyed their original form. In fact,

Kellerhals[44] had recently analysed some stable (but not completely inactive) gravel-bed rivers and canals, and had derived regime-type equations, of which only the width equation resembled its Lacey counterpart. The Author agreed with Mr Lacey that the conflict between regime and other theories presented a challenge that must be faced, but this entailed a readiness to re-examine cherished concepts on both sides.

163. Mr Lacey had given a clear description of Indian methods of design for pier scour, which corresponded closely to that given in the recent Paper by Savage and Carpenter,[43] to the discussion of which the Author had contributed. He would not repeat his views on Indian pier design as expressed in that contribution. The crux of the matter, as he saw it, was not whether the method described by Mr Lacey was safe—of that there could be little doubt—but whether it was economical, especially in places other than the Indian alluvial plains. He agreed, however, that if a pier had to sustain severely skewed flow a cylinder might be better than a long slender pier. In the case of wide bridges with piers subjected to skewed flow, model tests indicated that a row of cylinders was better than a long solid pier, but such designs were not popular in Western Canada because of their tendency to clog with timber and ice.

164. The discrepancy in the Paper between bank-full discharge figures of 5500 and 5000 cu. ft/s as quoted in §§ 5 and 33 respectively, to which Mr Lacey and Mr Van Beesten referred, arose from averaging figures over the 30 mile length described, the larger figure applying only at the lower end. Mr Lacey correctly pointed out that different symbols should have been used for different definitions of mean depth. The Author thanked Mr Lacey for his re-calculation of scour depth figures in Table 3.

165. The Author had endeavoured in the foregoing paragraphs to deal in succession with what seemed to be the most significant points raised by the contributors, but apologized for inevitably overlooking some. In the closing paragraphs he would attempt to summarize the more important aspects of the Discussion, and to bring his views on one or two matters up to date, because over three years had elapsed since preparation of the Paper.

166. The Paper's critical appraisal of the regime and tractive force approaches to alluvial channel stability had drawn contributions from several well-known authorities on regime methods, thereby clarifying a number of points which to the Author had previously been somewhat obscure. It was disappointing, however, that there had been no contributions from U.S. engineers, who generally took a rather different view of the subject. The question of dominant discharge, which was of considerable importance in the application of regime methods to river engineering, could be answered conclusively only by more extensive systematic analysis of river data, but the Author's inclination to regard bank-full discharge as dominant for channel cross sections had been supported by Sir Claude Inglis, Mr Lacey and others. Regime equations for scour had received thorough examination, and it had been confirmed that their originators regarded them as purely empirical.

167. It appeared evident that scaling local scour from loose-bed models, whether distorted or not, presented problems that had not been examined very deeply to date.

168. The 'regime' and 'fluid mechanics' approaches to the theory of flow of water-sediment mixtures could not continue to exist indefinitely in more or less separate compartments. There was a need to determine, by means of incontrovertible facts and objective analysis, the ranges of applicability of various equations, and to unify the data into one theory. The situation was until recently analogous to the state of pipe friction theory before the work of the Prandtl school, but recently there had been a number of notable efforts towards unification, of which Ackers' 1964 Paper[46] was a good example. The Author considered that the 'silt factor' or its equivalents constituted a weak point in regime equations. Some 'regime' calculations seemed to involve a circular type of reasoning that was immune to disproof.

169. In connexion with the brief discussion of bed-forms in §§ 43 and 44 of the Paper, a very recent paper[45] might be of interest. This attempted to classify types of bed-forms in alluvial channels generally. It was interesting that a three-part

primary classification was proposed, embracing ripples, dunes and anti-dunes, and bars, which corresponded closely to the Author's observations on the Beaver River and elsewhere.

170. There had been little dissent from the Paper's summary and conclusions with respect to the limited data reported.

REFERENCES

23. AHMAD M. Effect of scale distortion, size of model bed material and time scale on the geometrical similarity of localized scour. *Proc. 6th gen. Meeting Int. Ass. Hydr. Res.*, 1955, Hague.

24. LANE E. W. and BORLAND W. M. River-bed scour during floods. *Trans. Am. Soc. civ. Engrs*, 1954, **119**, 1069–1089.

25. POPOVITCH R. Investigation of the factors affecting the formation of the depths of rivers with movable beds. *Travaux*, 1960 (Oct.).

26. LARRAS J. Maximum depth of erosion in movable beds around river piers. *Annls Ponts Chauss.*, 1963, **133** (No. 4, July–Aug.).

27. ARUNACHALAM K. Scour around bridge piers. *J. Indian Rds Congr.*, 1965 (Aug.).

28. LAURSEN E. M. Scour at bridge crossings. *Trans. Am. Soc. civ. Engrs*, 1962, **127**, 166–207.

29. MEYER-PETER E. Some problems concerning sediment transport in alpine and sub-alpine rivers. *Houille blanche, No. Spec. B*, 1949.

30. GOURLAY M. R. and McKAY G. R. Pioneer River model investigation. *Publ. Dept. civ. Engng, Univ. Queensland, Brisbane*, 1962 (Sept.).

31. BARR D. I. H. Model simulation of vertical mixing in stratified flowing water. *Engineer, Lond.*, 1963, **215** (Feb.), 345–352.

32. ACKERS P. Experiments on small streams in alluvium. *Proc. Am. Soc. civ. Engrs*, 1964, **91**, *J. Hydr. div.*, **HY2** (July), 1–37.

33. BARR D. I. H. Discussion of reference 34. *Proc. Am. Soc. civ. Engrs*, 1965, **91**, *J. Hydr. div.*, **HY2** (Mar.).

34. SELIM YALIN M. Geometrical properties of sand waves. *Proc. Am. Soc. civ. Engrs*, 1964, **90**, *J. Hydr. div.*, **HY5** (Sept.), 105–120.

35. BARR D. I. H. Spread characteristics of a buoyant misable discharge. *Proc. 10th Congr. Int. Ass. Hydr. Res., London*, 1963, **1**, 153–160.

36. INGLIS C. Definition of dominant discharge. *Tech. Rep. cent. Bd Irrig. India*, 1941, **72**.

37. UNESCO. Preliminary suggestions for an international programme in scientific hydrology. NS/NR/17, Paris, 1962 (Oct.). Sections 11 and 111·3.

38. UNESCO. Int. Hydrol. Dec. Intergov. Meeting of Experts. NS/188, Paris 1964 (June), Annex 1, Sections 3 and 4, Annex 11, Section 1.

39. NEILL C. R. River bed scour. *Can. Good Roads Ass., Ottawa, Canada*, 1964, *Tech. Publ.*, **23**.

40. BLENCH T. Mobile-bed fluviology. University of Alberta Bookstore, Edmonton, 1966.

41. GARDE R. J. Local bed variations at bridge piers in alluvial channels. *Res. J. Univ. Roorkee*, 1961 (Nov.).

42. KELLERHALS R. Stable channels with gravel-paved beds. Pre-printed Paper, ASCE Water Resources Engin. Conference, Denver, Colorado, May 1966.

43. SAVAGE C. D. W. and CARPENTER T. G. Indus Basin settlement scheme: T-S-M-B link canal system. *Proc. Instn civ. Engrs*, **32** (Dec.) 1965, 549–571.

44. ACKERS P. Experiments on small streams in alluvium. *Proc. Am. Soc. civ. Engrs*, **90**, *J. Hydr. div.*, no. HY4, 1964, 1–37.

45. ASCE TASK FORCE. Nomenclature for bed forms in alluvial channels. *Proc. Am. Soc. civ. Engrs*, **92**, *J. Hydr. div.*, no. HY3, 1966, 51–64.
46. JOGLEKAR D. V. *et al.* *Ibid.* Discussion of ref. 25.

CORRIGENDA

The following amendments should be made to the original papers:

Page 427, § 19: equations (1a) to (1c) should read

$$\text{width} \propto Q^{1/2} \qquad \text{(1a)}$$

$$\text{mean depth} \propto Q^{1/3} \qquad \text{(1b)}$$

$$\text{slope} \propto 1/Q^{1/6} \qquad \text{(1c)}$$

Page 431, § 33: read 'bank-full discharge $Q_b = 5000$ cu. ft/s'.

Page 435, reference 15: change 'Bull. 3' to 'Bull. 4'.
reference 12: add 'and Dover Publications, New York, 1966'.
reference 22: change 'January 1964' to 'July 1963'.