Discussion.

The President. The PRESIDENT, in moving the vote of thanks to the Author, remarked that the application of the Author's theory of silt and scour was mainly concerned with canals. Mr. Kennedy, on whose work the Author's Paper was based, had been an intimate friend of the President's in India many years ago. Mr. Kennedy's work in connection with the theory of silt and scour in canals had been of great service and advantage to the Government of India; it really had resulted in the canals of India being now so constructed that, generally speaking, there was no scour and no silt. It would be of great advantage to those who had to build river bridges if they could get some idea of the extent to which they should allow for scour in sinking the piers below the river-beds. So far as his own experience went—and he thought it applied to others—it had been a matter of basing his decision on his judgment, upon the facts ascertainable as to velocity and scour, height of flood, depth of water, and so on. At the best it had perhaps been but an intelligent guess. No definite rule had governed it. If, by any working out of the theory of silt and scour, an engineer could be enabled to design piers so that they would be absolutely safe from scour, then those who had to build river bridges would feel themselves in a very much safer position. He hoped the discussion would bring out something useful, not only in regard to irrigation, but also to those who, like himself, had to deal with the mighty bridges of India.

Sir Thomas Ward. Sir THOMAS R. J. WARD remarked that the Author consulted him about the Paper 2 or 3 years ago, and since then he had greatly improved it. Any work intended to rationalize practice merited the encouragement of The Institution. Although the Paper did not deal directly with any constructional work, it was, as the President had pointed out, full of interest to those who were responsible for designing. The Author based his reasoning on Mr. Kennedy's well-known formula.[1] Sir Thomas Ward had on two previous

[1] "Hydraulic Diagrams," Edinburgh, 1907, p. 4.—"The first four or five miles of a distributary should also be given full V_o, the length allowed depending inversely on the number of outlets taking off—all at bed level. Below this, the fraction of V_o may be gradually decreased, till at the tail it may be about $0 \cdot 85 V_o$. Similarly, minor distributaries taking off well down the main distributary, may have less than V_o at their heads, decreasing towards the tails. There is no object in grading for V_o for the whole length of a distributary . . . a lower velocity makes more certain of clay side berms and fine sediment on the bed, thus decreasing the seepage loss."

Sir Thomas Ward.

occasions discussed[1] the Kennedy formula, and it would suffice now briefly to notice that Mr. Kennedy was placed in charge of a division on the Upper Bari Doab canal in the early nineties, and he had to remodel a number of its channels. There had been a very definite practice in the Punjab in those days for remodelling canals, but it had not been rationalized. Mr. Kennedy made a few pertinent observations and founded what was now called the Kennedy law, which was given in the Paper. Very soon after that Mr. Kennedy was placed in charge of a division for the construction of the Lower Gugera Branch of the Lower Chenab canal; and Mr. Sidney Preston, M. Inst. C.E., who was his Superintending Engineer, asked him to design the channels according to diagrams which he had worked out. Sir Thomas Ward had designed channels in the division above that—in about 1897 or 1898—also in accordance with those diagrams. It was soon discovered that, although the rule worked well for the head reach of the distributary, farther down the distributary Nature threw up very broad berms which reduced the width and increased the depth of the cross section to what had been the practice before Mr. Kennedy published his diagrams and formula. That difficulty had been surmounted by the introduction of the "critical velocity-ratio"—by reducing the coefficient as one went down the channel. Knowledge of that ratio was a matter of practice, and it could only be learned by long experience in that particular class of designing. As an instance, Mr. Kennedy remodelled, after that, the whole of the Western Jumna canal. Sir Thomas Ward had charge of the canal in 1910—about 10 years after Mr. Kennedy had left. Since then, some over-enthusiastic disciple of Mr. Kennedy had remodelled a series of long distributaries near Delhi, which took out from the main canal about 70 miles from its head in the river. Sir Thomas Ward had been shown small channels of about 10 cusecs capacity which had been widened to about 4 feet of bed-width and about 1 foot depth, and the Executive Engineer, who was a very experienced officer, had said that those channels were not giving satisfaction to the farmers, and that he proposed to make them still wider and shallower! What had really happened was that the distributary was so far away from silt that Nature could not correct the mistakes which had been made; when such a channel had again been reduced to its original dimensions—about a foot or 18 inches of bed-width and about the same depth—the farmers were satisfied once more. More recently, on the Lower Bari Doab canal of the Triple Canal project, a very

[1] Minutes of Proceedings, Inst. C.E., vol. ccxvi (1923), pp. 221, 352.

Sir Thomas Ward.

large canal of several thousand cusecs capacity had been taken out from the Ravi river. Not long after it had been opened it began to scour. There were probably two reasons for that scouring: the canal was very much farther down the river than any of the previous works from which experience had been gained of the proper coefficient to use; and the head works had been greatly improved from the point of view of the power of excluding silt. Since he left India a number of large canals had been taken out along the Sutlej river, and he understood that considerable discussion had arisen as to the right coefficient to use in the formula. It would be seen, then, that a great deal of experience was needed in order to apply the formula, and, considering the very great interest which was now being shown in the question (the formula was used in America and doubtless in the Dominions, and he believed it was used all over India), it would be worth while, he thought, for the Governments concerned to elucidate definitely the theory underlying the critical velocity-ratio. Dr. W. C. Unwin, Past-President, in his article on Hydraulics in the *Encyclopædia Britannica*, showed[1] that the equation for the bed-slope of a river descending from mountains to the sea was parabolic. Observation, too, showed that river-channels were broad and shallow where they were in sand, but in the delta, where only muddy water had to be dealt with, they were narrow and deep. It was much the same on the canals in India. The Kennedy formula, with the critical velocity-ratio now used, made a channel somewhat broad and shallow at the head, the section becoming narrower and deeper the farther it was from the head. Sir John Benton, M. Inst. C.E., pointed out[2] in his Paper on the Triple Canal project that in different parts of the river different coefficients had been used. The Author had taken one formula, with a coefficient varying according to some law, and had used it to discuss further formulas for dealing with other works which arose in practice connected with the design of canals. In one of the instances, the Author stated, in high flood the bed rose, although he mentioned elsewhere that in certain reaches in high flood the beds deepened; but he explained that this tendency of certain reaches to deepen their beds during the period of a high flood applied where the normal width of the river was unduly contracted, either naturally by clay buffs or artificially by spurs. Mr. J. R. Freeman described before the Engineering Conference in 1921 some interesting diagrams and charts relating to the Yellow

[1] Eleventh Edition, vol. xiv, p. 78.

[2] Minutes of Proceedings Inst. C.E., vol. cci (1916), p. 24.

Sir Thomas Ward.

river. Mr. Freeman had found[1] that as the Yellow river flood rose the bed deepened, and as the flood fell the bed rose. With a rise of 5 feet of flood there was practically a 5-foot deepening, and with a rise of 10 feet the river had taken a new depth, in some cases 10 to 15 feet below the depth at low water. The Mesopotamia irrigation-engineers had taken observations on the Tigris and had prepared charts which bore out the same facts, but those charts, unfortunately, had been destroyed by fire. In Punjab river practice it was a well-known phenomenon. It was true, from the Author's point of view, that the observations on the Yellow river and the Tigris were made in their deltas, where the width of their channels was restricted by the very hard clay of which the banks were composed. But, on the other hand, with regard to the Author's cross sections of the Jumna, the river at the place in question was coming out of a very confined channel in the hills and spreading out towards the plains on its way to the sea, thus forming a fan or cone with very steep slopes. The Author mentioned a slope of 1 in 140, and, a little lower down the river, 1 in 300. Apparently there had been a vast flood in 1924, exceeding all previous records, and it was quite understandable that in such a flood a very large mass of material would come down, which was more than could be got away by the river in the ordinary manner. It was hardly likely that a formula based upon the study of a canal in regime (Mr. Kennedy had been in search of a law for designing regime channels) would hold in a case where a river came out of a mountain range and spread itself out into what was, for the time being, really a basin; because it was flowing into a comparatively empty river-bed, which it had to fill to this new high flood-level. The late Mr. P. à M. Parker[2] referred to some interesting experiments made by Mr. J. A. Seddon in 1886, in which water was allowed to erode sand until no sand was carried forward.[3] Mr. Seddon found that the channels became shallower and broader as the slope increased, while the mean velocity remained constant for all slopes, that was, $v = C\sqrt{rs}$. Therefore r, and (very approximately) d, varied as v^2/C^2s, where v was practically the velocity required to move the sand. That, of course, as Mr. Parker pointed out, was a very different rule from Mr. Kennedy's; but it had been applied to very different circumstances, and in applying such rules as the Author had evolved it was essential to know the circumstances on which those rules were based. There again, he thought that those corporations

[1] Inst. C.E., Proceedings of Engineering Conference, 1921, Section II, p. 64.
[2] "The Control of Water," p. 770, London, 1913.
[3] Journal Assocn. Eng. Societies, vol. 5 (1886), p. 127.

Sir Thomas Ward. who owned large irrigation-works would be quite justified in incurring the expenditure necessary to undertake the measurements required in order to make the formula applicable throughout the length of a river or canal system.

Mr. Woods. Mr. F. W. WOODS observed that the Author had discussed, within the compass of a few pages, a subject of very wide scope. There was much in the Paper with which he was in agreement; but, as it was necessary for him to be brief, he would confine himself to criticisms of a few points which seemed to him to need reconsideration. The Author relied greatly on Mr. Kennedy's formula, which was consistent with hydraulic data found by Mr. Kennedy on a limited range of artificial earthen canals; but the Author extended its application to the conditions of great rivers in unstable regime, presenting every kind of irregularity of flow. Such extension of application would be questionable, even if the formula were correct for canals in stable regime. Reference was often made to Mr. Kennedy's theory of the silt-transporting power of a current of water. As a matter of fact, Mr. Kennedy had enunciated no original theory, but had merely deduced, from data collected by him, a useful empirical formula, in support of theories already current in India, which had been enunciated[1] 30 years earlier by the late Mr. Thomas Login, M. Inst. C.E., one of the engineers of the Ganges canal; and Mr. Login, in enunciating those theories, was merely confirming, from his own experience of the Ganges canal, theories suggested by Mr. J. Dupuit[2] in 1848. The Author had adopted Mr. Kennedy's formula, based on "observations of certain Punjab irrigation-canals." That formula had been a useful guide to the design of canals for 30 years past; but it was well to bear in mind its limitations, and certain defects of its structure. Mr. Kennedy neither measured the water-surface slope of any of his twenty-two sample channels, nor calculated their value of Kutter's coefficient of rugosity. Mr. Kennedy assumed in one part of his Paper[3] that N was equal to 0·0225, and in another part 0·02375; whilst he based his first book of "hydraulic diagrams" on $N = 0{\cdot}025$, and the second edition on $N = 0{\cdot}0225$. He measured neither the velocities nor the discharges of his channels, but evaluated the discharges from discharge tables, and deduced the mean velocities by dividing discharge by area of cross section. Even his determination of area of waterway was not quite satis-

[1] Minutes of Proceedings Inst. C.E., vol. xxvii (1868), p. 471.

[2] "*Etudes théoriques et pratiques sur le mouvement des eaux courantes*," Paris, 1848.

[3] Minutes of Proceedings Inst. C.E., vol. cxix (1895), p. 281.

factory, for he expressed it in terms of bed-width and depth, without specifying the inclination of the side slopes. In most cases, especially those of the larger canals, the bed-width stated appeared to be identical with mean width of channel, indicating vertical sides, whilst the figures indicated no side slopes flatter than $\frac{1}{4}$ to 1. Even if the figures were accepted as correct, they did not represent typical conditions of channels in stable regime. He thought that Mr. Preston would confirm the opinion that "regime channels" did not generally exhibit vertical sides, or sides steeper than $\frac{1}{2}$ to 1. Mr. Kennedy had produced a very useful formula; but when the application of that formula was extended beyond the range of Mr. Kennedy's data, as in the present Paper, it had to be borne in mind that it was not scientifically exact. Mr. Kennedy had inferred[1] from his data that "apparently the bed-width had no place in the equation connecting the depth d and mean velocity V_o"; but when those data were plotted graphically against a base scaled to show mean widths of channel w, it was seen at once that d and V_o both varied with w. In later years Mr. Kennedy came round to the opinion that for every value of V_o there must be a corresponding normal ratio of w/d; and it followed from that that the formula[2] $V_o = cd^m$ must be defective, owing to the omission from it of w. The Author might well bear that in mind. The late Sir Thomas Higham, M. Inst. C.E., when he was Chief Engineer, Irrigation Works, Punjab, brought Mr. Kennedy's Paper of 1895 to the notice of Punjab engineers in 1896. He wrote[3]: "This theory (Mr. Kennedy's) may be briefly stated to the effect that the silt-carrying capacity of a channel varies directly with some function of the mean velocity, and inversely with some function of the depth of the supply." That was, as a matter of fact, Mr. Login's theory. But mean velocity varied directly with the depth; and Mr. Woods had endeavoured to account for the apparent paradox of V varying both directly and inversely with d. He had experimented on a small canal which used to suffer seriously from deposits of silt. It had been designed to carry 3 feet depth of water, and a discharge of about 18 cusecs; but it used to silt up, so as to reduce its water-depth to 1·5 foot, and its discharge to 8 cusecs; its surface slope increased, incidentally, cent. per cent. While the stream was 1·5 foot deep its surface and bottom velocities were measured. Mr. Woods.

[1] *Loc. cit.* p. 283.

[2] The Author uses the notation $V_o = cd^n$; but it is better to adhere to Mr Kennedy's notation $V_o = cd^m$, so as to avoid confusion with Kutter's coefficient n.—F.W.W.

[3] Punjab Irrigation Branch Paper No. 7 (1896).

Mr. Woods. Then the silt was excavated, the stream being restored to its original depth of 3 feet, and the surface and bottom velocities were again measured. It had been expected that comparison of hydraulic data would throw light on the inability of the deep stream to carry its silt. Contrary to expectation, the bottom velocity (which was really the velocity at an elevation of 3 inches above the bed) was found to be the same in the two cases; whilst the surface velocity was somewhat higher in the deep than in the shallow stream. But the ratio borne by the *difference* between surface and bottom velocities to the water-depth was 60 per cent. greater in the shallow stream than in the deep one; and, in the light of Mr. Dupuit's theory—that the transportation-power, T, was due to the differences of velocity between adjacent filaments of current, a unit distance apart—the variation of T inversely with d seemed to be accounted for. Those conclusions were presented to Sir Thomas Higham and to Mr. Kennedy, both of whom accepted them. Mr. Dupuit had shown experimentally that if sand or gravel was placed in a vase with water and the vase was caused to rotate on a vertical axis, the heavier bodies were carried up in the water in proportion as the velocity was increased. Mr. Dupuit had stated that the difference of velocity of different filaments of water, in making unequal currents pass to right and left of the solid body, engendered a force which pushed it to the side of the faster filament. It was evident from that experiment that in an open channel horizontal eddies, caused by lateral friction, had silt-suspending power as well as erosive power; and therefore that a stream, in order to be in good regime, neither silting nor scouring, must have a certain ratio of width to depth, varying with its discharge. That conclusion was subsequently accepted by Mr. Kennedy. The ratio w/V was greater than d/V, because particles of matter which might be too large and heavy for a given current to suspend, though not too heavy to be rolled by it, would roll down a side-slope till they rested on the stream-bed; thus raising the bed, reducing the depth, and increasing the ratio w/d. The Author had recognized that, and he would probably agree, therefore, that a rational formula for T should express the relationship of V to w as well as to d. The Author quoted a table of the "bottom velocities" stated by Mr. Parker to be sufficient to move certain materials. That expression "bottom velocity" was a misnomer which ought to have died out ere now in discussions of hydraulics. It originated from experiments of the Chevalier L. G. Dubuat (1786–1816) who caused little balls, of specific gravity slightly greater than that of water, to be rolled by the current along the bed of a channel, and took their velocity to represent "bottom velocity." But Dubuat's own data showed that that velocity varied, in any given current, with the magnitude of the ball rolled. From numerous velocity-measurements which Mr. Woods had made, with a Darcy-Pitot hydrometric tube, in the period 1902–1906, on earthen canals ranging from 8 to 80 cusecs in capacity, he had inferred that the true "bottom velocity" must be zero. Professor Grove K. Gilbert had arrived[1] at a similar conclusion from exact experimental observation of the flow of sand-laden currents in troughs of wood or iron less than 1 foot wide, and having a discharge of less than 1 cusec. Professor Horace Lamb, also, had given[2] mathematical and other reasons for the same conclusion, which certainly simplified discussion of the problem. The Author stated that in consequence of the relation between velocity and depth, the path of greatest depth in a river-bend followed that of greatest velocity; but he did not explain why the greatest velocity round a bend tended to hug the outer, or concave, bank? Dupuit's experiment with the rotating vase was relevant. Flow round a bend necessitated faster flow of the peripheral elements of the current than of those nearer to the centre of the circle, just as when troops wheeled in line, those on the pivot flank remained nearly stationary, whilst the others moved with velocities proportional to their distance from that flank. The propinquity of the maximum velocity of a current to the outer bank of a bend set up severe differences of velocity there (since the "bank velocity" must be zero), and relatively insignificant differences of velocity at the inner, or convex, bank. Hence there was erosion at the outer bank and deposition at the inner. There again the width, and the location of the point of maximum velocity relatively to the width of section, were more relevant to the equation of erosion and silt-deposition than was the relation between mean velocity and depth. The Author stated (p. 244) that, with regard to the bed of a river, the laws governing scour, silt-transportation, and silt-deposition were identical, for all velocities sufficient to detach particles of the material forming the bed; but he seemed to be inconsistent when he stated (p. 249) that the law governing the transportation of rolling boulders was entirely different from that relating to silt in suspension. Mr. Woods thought that there was no such difference in the law of the subject, the law applying being that stated by Mr. Dupuit. Under any given conditions of current there were always some bodies too large and heavy to be kept Mr. Woods.

[1] "The Transportation of Débris by Running Water." U.S. Geological Survey: Professional Paper No. 86, p. 155. Washington, 1914.

[2] "Hydrodynamics," Cambridge, 1924.

Mr. Woods. suspended in the water, though not to heavy to be rolled along the bed. When there was any appreciable reduction of velocity, the rolling body would cease to roll, whilst other lighter bodies, which had been in suspension, would fall on to the bed and begin to roll along it. On the other hand, if there was an increase of velocity, the rolling bodies would rise in suspension, whilst other stationary bodies would begin to roll. After the great flood of 1910 in the Gomal river, on the North-West Frontier of India, he had seen a boulder, about 4 feet long and 3 feet in diameter, lodged in the steel lattice-work of a pier of a suspension bridge at Murtaza, at a level at least 10 feet higher than the surface of the river in the low-water season. That boulder must have been whirled about during the flood, suspended in the current like any grain of sand; though, under floods of less intensity it would be either rolling along the bed or else quite stationary. The Author's experience led him to the opinion that there was a tendency for the beds of great rivers to rise during high floods, and to fall during smaller ones. At first sight that opinion did not seem sound. If the bed and sides of a river were hard enough to resist all erosion, any increase of flood-discharge would be indicated by rise of water-level alone; but it was difficult to believe that rivers in soft alluvial soil would not, in high flood, scour their beds deeper or wider; or that, on the contrary, they would reduce their waterway by depositing silt. The records of the Indus River Commission showed that the river Indus deepened its bed in proportion to the magnitude and intensity of its floods. The shoaling observed by the Author might have occurred on the subsidence of high floods, and been measured by him after such subsidence. Or it might have resulted from loss of discharge by flood-spill. The Author mentioned the occurrence of shoaling upstream of the Bhimgoda weir on the Ganges, which caused heavy shingle deposits in the off-taking canal. According to Punjab experience, such shoaling could be prevented by maintaining an afflux of several feet through the weir-sluices in the flood season. At Bhimgoda, during high floods, the afflux was only 1 foot. In the Punjab, an afflux of 2 feet had been found necessary to prevent shoaling with coarse sand; and to prevent shingle deposits at Bhimgoda an afflux of probably 4 feet would be required. The Author observed that Mr. Kennedy had anticipated variation of the value of m, in the formula $V_o = cd^m$. That did not seem correct. On the contrary, Mr. Kennedy wrote[1]: "On different canal-systems, the values of c and m . . . may vary slightly, though . . .

[1] Minutes of Proceedings Inst. C.E., vol. cxix (1895), p. 284.

any such variation will be but small." It was the ratio (p) of suspended silt to the volume of water carrying it, in which he anticipated variations. Different authorities had taken great liberties with Mr. Kennedy's constants. His formula $V_o = 0{\cdot}84d^{0{\cdot}64}$, had been modified by Sind engineers to $0{\cdot}63d^{0{\cdot}64}$; in California to $0{\cdot}98d^{0{\cdot}64}$; by Madras engineers to $0{\cdot}67d^{0{\cdot}55}$; by those of Burma to $0{\cdot}95d^{0{\cdot}57}$, and in a recently published book it had been asserted that the "most probable value" was $0{\cdot}95d^{0{\cdot}57}$. Amidst that diversity of choice it might be well to point out that in Mr. Kennedy's Bari Doab canal data the ratio ($r^{0{\cdot}50} : d^{0{\cdot}64}$) was nearly constant, and equal to 0·718. Wishing to express V in terms of d rather than of r, Mr. Kennedy substituted $d^{0{\cdot}64}$ for $\sqrt{r}/0{\cdot}718$. But $V = c\sqrt{rs}$: so that, according to him, $V_o = 0{\cdot}84d^{0{\cdot}64} = c\sqrt{s} \times 0{\cdot}718d^{0{\cdot}64}$, or $c\sqrt{s} = 1{\cdot}17$. Unfortunately he neither measured s nor determined c. The ratio r/d of his experimental channels ranged from 0·86 in the largest to 0·65 in the smallest. The ratio $r^{0{\cdot}5}/d^{0{\cdot}64}$ ranged from 0·714 in the largest to 0·721 in the smallest; and the mean value 0·718, given above, would be more divergent from actuals in the case of channels very much smaller than 26 cusecs or very much larger than 1,700 cusecs. By eliminating the hydraulic mean depth $r = (wd/w + 2d)$ Mr. Kennedy arrived at a convenient empirical formula, accurate enough within certain limits; but he eliminated the element w which was essential to a more widely practical formula. The Author applied his theories to practice in respect of the design of bridge-foundations and river-training. With regard to the latter, which was concerned mainly with controlling lateral erosion, or lateral shoaling, it seemed necessary that he should reckon with a formula embracing w as well as d. With regard to bridge-foundations, the conditions of flow causing erosion, or deposition, were governed neither by $V = c\sqrt{rs}$ nor by Kutter's or Kennedy's formula, but by $V = c\sqrt{2gh}$. The formula stated by the Author (p. 245), namely $V = C\sqrt{2grs}$, required explanation. Mr. Woods hoped that his references to Mr. Kennedy's formula would not be taken to mean that he did not place a very high value on that eminent hydraulician's work. But it was necessary to obviate interpretations of Mr. Kennedy's researches of which Mr. Kennedy himself would certainly not have approved. Mr. Woods.

Lieut.-Col. E. P. ANDERSON, R.E., observed that, having been for some time connected with some of the great bridges over the rivers of the Punjab on the North-Western Railway, he was interested in the Paper, not merely from the point of view of the construction of new bridges, but also from the point of view of the extensive work which had been going on lately in connection with the Lieut.-Col. Anderson.

Lieut.-Col. Anderson. strengthening of some of the old bridges built long before river-training works were thought of. Many bridges built before the great work of the late Mr. J. R. Bell and Sir Francis Spring, MM. Inst. C.E., on which the designs of most of the railway-bridge training-works were now based, had been carried out, were extraordinarily long and had very shallow foundations. When those bridges had to be reconstructed in order to take modern loads, it was of great economic importance to reduce the number of spans as far as possible; but in order to do that the engineer required a reliable method of estimating the deepest possible scour; for naturally if the waterway were restricted in width, the river would scour more deeply. The present methods of arriving at that scour were, as the President had said, largely a matter of trial and error and the result of considering a very large number of observations, made very often over an extended period and under climatic conditions of considerable difficulty. The Author gave engineers a method of obtaining the information they required from one set of observations on one high flood. Were his formulas reliable? The Author suggested on p. 260 a method of answering that question, and Colonel Anderson was sure that ample data for doing so could be found scattered among various engineers' offices in India. Unfortunately, very few suitable data were available in London, but he had been able to extract a certain number of figures concerning three bridges, of which he had personal knowledge, from Papers by Mr. F. J. Harvey,[1] Mr. F. C. Pavry,[2] and Mr. A. I. Sleigh.[3] Mr. Harvey dealt with the Beas bridge at Wazir Bhular, which had been built originally in the sixties, with thirty-two spans of $111\frac{1}{2}$ feet and two spans of 131 feet, centre to centre of piers. The girders having become weak, a new bridge was built in 1912, 300 feet upstream of the old one, consisting of nine spans of 211 feet between centres of piers. From the data available in the Paper referred to, which were not altogether complete and therefore not too reliable, it would appear that the mean depth in flood under the old bridge had been about 14 feet. The mean depth under the new bridge, calculated by the Author's method, was 21 feet, and the maximum scour 34 feet. Mr. Harvey recorded the maximum scour under the new bridge as only 28 feet; but that was some years ago, and Colonel Anderson knew there had been some very big floods in recent years, with possibly greater scour. The next case was the North-Western Railway bridge over the Jumna, between Kalanour and Sarsawa, about the lower reaches

[1] Proceedings of the Punjab P.W.D. Congress, Lahore, 1914, vol. 2, p. 149.
[2] *Ibid.*, 1920, vol. viii, p. 33.
[3] *Ibid.*, 1915, vol. 3, p. 1.

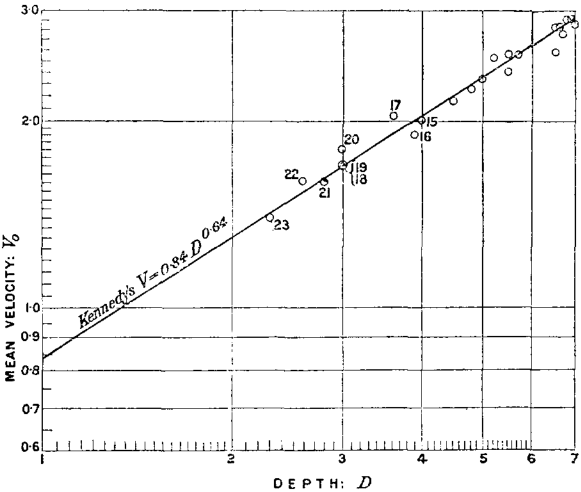
of which river the Author had given some details. The length of that bridge had been originally 2,663½ feet, but it had been replaced by Mr. Sleigh just before the war by one of seven spans of 211 feet between centres, or a total of 1,477 feet. The mean depth in flood under the old bridge appeared to have been about 18 feet, and, calculated by the Author's methods, it should be 26 feet under the new bridge, with a maximum scour of 46 feet. In 1915 a scour of 34 feet was recorded, but he did not know what the scour had been during the great flood of 1924. The third example was the Alexandra bridge over the Chenab, near Wazirabad, which had been reduced from twenty-eight to seventeen spans of 132 feet. The mean depth under the old bridge appeared to have been about 25 feet, and calculated by the Author's methods, the mean depth under the new bridge would be 34 feet, with a calculated maximum scour of 57 feet. Mr. Pavry had actually recorded a scour of 47 feet. Those results were all satisfactory from one point of view, in that they were all safe; and foundations calculated on the basis of the Author's figures would err, if anything, apparently on the right side. The excess of the calculated over the recorded scour was the same—21 per cent.—in the Beas and Chenab bridges, but it rose to 37 per cent. in the case of the Jumna bridge. There might be some connection between that difference and the fact that the Jumna bridge was relatively very much nearer to where the river emerged from the hills than was either of the other two bridges mentioned. The discrepancy might, of course, also be accounted for in some such way as the Author suggested. Engineers in India were a little inclined to work in watertight compartments; and he had to admit that he, for one, was not very familiar with the work that had been done and was being done by irrigation-engineers on this subject. On the other hand, irrigation-engineers were perhaps equally unaware of the extent of the records which were available in regard to floods at some of the big railway-bridges; and he suggested that, in order to put the theory to a definite practical test, old records of floods should be looked up, and calculations made more accurately than he had been able to do, so that by pooling their knowledge engineers might be able to advance to a much more thorough understanding of a subject which was of such great economic importance to all of them, whether engaged on railway or irrigation work. Lieut.-Col. Anderson.

Mr. SIDNEY PRESTON observed that Mr. Woods had suggested that he (Mr. Preston) would confirm the statement that Mr. Kennedy had not allowed for the side slopes in calculating cross sections. The canals in question had been designed and built with 1½-to-1 side slopes, but after a very short time the sides had always Mr. Preston.

Mr. Preston. silted up, so that, eventually, they had become very nearly vertical, perhaps about $\frac{1}{2}$ to 1. [Mr. Woods remarked that, although the slope was $\frac{1}{2}$ to 1, with which, no doubt, Mr. Preston would agree, Mr. Kennedy's data showed it vertical. There was a difference between vertical and $\frac{1}{2}$ to 1.] He agreed that there was a slight difference, but the ultimate side slope, if not absolutely vertical, was nearly so. He was sorry that he was not able to enter into the scientific discussion of the Paper. He had had a good deal to do with Indian irrigation more than 25 years ago, but the records he had obtained were in Government offices, and he had not been able to bring them away for reference.

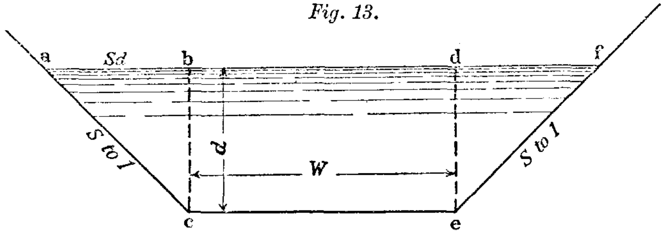
The Author. The Author, in reply, remarked that the main objection raised by Sir Thomas Ward and Mr. Woods to his theory was that it was based on Mr. Kennedy's law ($V = C\,d^n$), which law was not necessarily exact and was not applicable to the cases under consideration. The reasons given were that this law had been framed as a result of observations of a number of artificial channels, and that in framing it Mr. Kennedy had not given consideration to the effect of the sides (which would depend on the ratio of width to depth of the channels observed, which was not taken into consideration). The Author was glad that this important point had been raised. As a result he had re-examined the evidence on which Mr. Kennedy based his law. Mr. Kennedy derived his general equation by plotting the observed depths and mean velocities of a number of irrigation-channels carrying silt-laden discharges. These channels, as a result of numerous remodellings and by silting their beds to form steeper slopes when necessary, had arrived at a state of equilibrium, and were carrying their silt-laden discharges without further silt or scour action. Thirty sites were chosen, but as six of these had not quite reached equilibrium they had been omitted from Table III (p. 278). The depths D and mean velocities V_o were plotted in *Fig. 12*, the curve giving Mr. Kennedy's equation $V_o = 0{\cdot}84\,D^{0{\cdot}64}$. The widths B (column 3) were stated to be to the nearest foot; these had been corrected (column 6) to give exact values of mean widths of the section, as calculated from the discharge, divided by the product of mean velocity and depth. Mr. Kennedy contended that horizontal eddies had no silt-supporting value, and whereas the velocity of flow was a function of the ratio of area to rubbing surface, the critical non-silting velocity (which depended on the vertical eddies) would be a function of the area divided by that length of rubbing surface only which created vertical eddies, or its equivalent. Considering any canal cross section (*Fig. 13*) with horizontal bed of width W and side slopes of S to 1, it would be accepted that a rubbing surface created eddies normal to its plane. If the section were rectangular, therefore, the width of the effective rubbing surface creating vertical eddies would be W, and the critical velocity would be a function of $Wd/W = d$ the depth as taken by Mr. Kennedy. The effect of the inclined sides would be to create eddies in an inclined plane normal to the surface, which might be resolved into The Author.

Fig. 12.



vertical and horizontal components. In *Fig. 13* the effect of the inclined side ac might therefore be considered as creating vertical and horizontal eddies in the ratio of ab to bc. The equivalent rubbing surface of the section creating vertical eddies therefore was af, and the critical non-silting velocity was a function of

Fig. 13.



$\frac{\text{area}}{\text{af}} = \frac{(W + sd)d}{W + 2\,sd} = D_m$ the mean depth of the section. Irrigation-channels were normally constructed with side slopes of $1\frac{1}{2}$ to 1 or 1 to 1, which bermed up and were subsequently cut to $\frac{1}{2}$ to 1, and the channels under observation had therefore mean sides slopes of $\frac{1}{2}$ to 1. $S = \frac{1}{2}$, so $D_m = \frac{(W + \frac{1}{2}\,d)\,d}{W + d}$.

The Author. Now $W + \frac{1}{2} d = B_1$, the mean width of the section, and writing D for d, $D_m = \frac{B_1 D}{W + D} = \frac{B_1 D}{B_1 + \frac{1}{2} D}$. In *Fig. 14* the values of D_m were plotted against the velocities, and the results agreed with the equation $V_o = 0{\cdot}94\ D_m^{0{\cdot}59}$.

TABLE III.—THE ORIGINAL DATA FROM WHICH MR. KENNEDY DERIVED HIS CRITICAL VELOCITY FORMULA, WITH COLUMNS 6 AND 7 ADDED BY THE AUTHOR.

1	2	3	4	5	6	7
No.	Q	B	D	V_0	B_1	D_m
1	1,700	85	7·0	2·86	84·96	6·73
2	1,700	84	6·9	2·91	84·70	6 61
3	1,700	86	6·8	2·90	86·25	6·55
4	1,250	68	6·7	2·75	67·80	6·38
5	1,500	80	6·6	2·83	80·25	6·35
6	1,250	70	6·5	2·81	68·6	6·2
7	924	55	6·5	2·59	55·0	6·14
8	940	66	5·7	2·55	64·7	5·45
9	940	66	5·5	2·55	67·0	5·28
10	650	48	5·5	2·40	49·1	5·21
11	633	50	5·2	2·52	48·5	4·93
12	700	61	5·0	2·33	60·2	4·8
13	390	36	4·8	2·25	36·1	4·5
14	220	22	4·5	2·15	22·7	4·1
15	120	14	4·0	2·00	15·0	3·52
16	142	18	3·9	1·90	19·1	3·52
17	138	18	3·6	2·04	17·7	3·26
18	85	16	3·0	1·70	16·65	2·74
19	75	14	3·0	1·70	14·7	2·72
20	70	12	3·0	1·80	12·9	2·69
21	65	14	2·8	1·60	14·5	2·55
22	65	15	2·6	1·60	15·6	2·40
23	26	8	2·3	1·40	8·07	2·01
24	33	11	2·2	1·30	..	..

Q denotes the discharge of the channel in cusecs.
B ,, ,, width as given by Mr. Kennedy.
D ,, ,, depth.
$_0$,, ,, mean velocity.

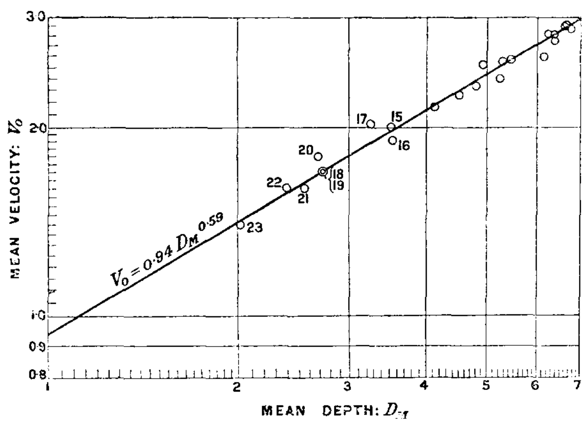
$$B_1 = \frac{Q}{V^o\ D}.$$

$$D_m = \frac{B_1\ D}{B_1 + \frac{1}{2} D}.$$

The Author.

In the light of this evidence, therefore, the Author claimed that the law was a general law and applicable. Exception had been taken to his contention that the unrestricted deep bed of a river flowing under the conditions cited would tend to rise after a high flood and fall after low floods. The examples of the reverse action cited by Sir Thomas Ward, namely, the Yellow river and the Tigris, were rivers not flowing under the required conditions. The Author's deductions were based on the assumptions that a river flowed over a bed of sand having no defined sides, the water being charged with sand in suspension, and that this flow was capable at any point of the wetted perimeter of raising or lowering the bed by depositing or picking up the material in suspension, there being no shear resistance in the material to prevent it. The clay banks

Fig. 14.



referred to by Sir Thomas Ward did not fulfil the conditions; the wetted perimeter obviously could not be considered as unrestricted, or its form definable by any single flood at all points of its cross section. As explained in the Paper, owing to the increased capacity of a river to move rolling silt as the flood-level rose and the depth became greater, there might be a temporary deepening during the flood until this rolling silt again subsided on its termination. The Author's observations of the rise of the Jumna deep bed after the great flood of 1924 were made at two sites, Kalsi and Tajewala, about 30 miles apart. The general equation for the velocity of flow (p. 245), namely, $V = C\sqrt{2\,grs}$ was a combination of two basic equations for flow: (1) Chezy's formula $V = c\sqrt{rs}$, and (2) the equation for free flow $V = c\sqrt{2\,gh}$. He was greatly obliged to Colonel Anderson for the practical

The Author. tests of the theory which he had given. In all three cases the calculations were based on a previously observed mean depth of the old section, from which the maximum depth under the tightened section had been calculated by means of equations (4) and (9). Equation (9) was only advanced tentatively, and in applying it Colonel Anderson had presumably assumed a value for K. Further, the mean scour of the old section was probably based on a higher flood than had occurred after the remodelling, because figures for the great flood of 1924 were not given. By the courtesy of Mr. L. H. Grant of the North-Western Railway, the Author had been able to examine the register of scour of the Jumna bridge between Kalanour and Sarsawa (cited by Colonel Anderson as his second example) for the big flood of September, 1924. During the highest period of the flood, the 29th and 30th September, 1924, the depth could not be observed, owing to the velocity being too high for the sinker. The maximum flood occurred on the 30th September and reached R.L. 881·9. The depth was observed to be 40 feet on the 1st October when the flood-level had fallen by 7 feet to R.L. 874·9. If, therefore, the bed had been at the same level, the day before, under the highest flood-level, the depth of scour would have been 47 feet; it was probably rather greater. The maximum anticipated depth of scour had been calculated from formulas (4) and (9) by Colonel Anderson as 46 feet, which agreed well, since this flood exceeded all past records.

Correspondence.

Mr. Ballester. Mr. Rodolfo E. Ballester, Chief Engineer of the Rio Negro Irrigation Works, observed that in his judgment the Author was not justified in establishing his formulas on the assumption that the Kennedy law (which had been based on the observation of irrigation-canals of small slope) was applicable to all watercourses. Irrigation-canals were dimensioned in such a way that the silt which came from the source of supply could traverse the canal and distributaries to the fields without being deposited in the canal and without being added to by erosion of the bottom or sides. In a river the eternal levelling work of Nature had to be taken into account, and eventually material scoured from the high reaches of the river would always try to find its way to the sea. In consequence, there would not be "equilibrium" or "balance" between silting