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CANAL FALLS AND THEIR USES AS METERS.

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The following are the primary requirements for an efficient fall :—

1. To destroy excess energy, without causing—
   - (a) damage to the work itself,
   - (b) scour of the canal-bed downstream, or
   - (c) wave-wash attacking the downstream banks.
2. To maintain "normal" supply depths in the canal upstream.
3. To measure the supplies passing.
4. To be cheap in first cost and maintenance.

The earlier forms of fall were Ogee weirs or rapids at the foot of which standing waves formed and destroyed the excess energy : these forms were later superseded by falls in which the flow was not guided but dropped to the lower level. The writer is not aware of the reasons for the change in the principle for destroying energy, and whether they are covered by the points discussed below : but it should be noted that the result of a very thorough investigation as to the best means of destroying the energy of large volumes of water under high head, made by the Miami Conservancy Board, was unequivocally in favour of the standing wave.

There is a possibility that standing waves caused some failures, by the step in pressure at the wave not being allowed for ; if so this should not be a danger now, as the hydraulics of standing waves are now far more thoroughly known. Rapid flow down a glacis is also liable to wear the joints ; velocities far higher than those occurring on these works are quite frequent elsewhere, and all that is needed is more suitable construction : for example, in discussion of a paper on the calibration of the sluices of the Assuan Dam, read before the Institution of Civil Engineers, the writer elicited that velocities up to 60 feet a second (quoting from memory) were dealt with.

With the drop form of fall, there is liability to wear of the floor of the work by the constant fall of a jet of water on to it. Parker, "Control of Water", page 716, quotes the test made in the Panjab by letting bottles float over the fall, and the empiric formula arrived at for the depth of water-cushion presumed to be sufficient from the fact that it saved breakage of the bottles. Bligh, page 117, states that such cushions are no longer considered so necessary, and there is a tendency in modern practice to reduce or omit them.

Assuming that floors are properly constructed, that is, that soft bricks are avoided and joints properly constructed, such damage as existing falls show seems generally to consist of pot-holes ground by brick-bats in more or less stationary eddies : these should be avoidable by rounding off internal angles to a degree sufficient to allow eddies to travel instead of working continuously at one spot.

The importance of bed-scour as an evil should not be exaggerated. Unless extraordinarily deep and extensive, it does not seem to matter as long as it does not threaten to undermine the work itself ; such danger can be avoided by a deep enough toe-wall, or preferably a sloping glacis carried down deep enough : a glacis is preferable to a toe-wall because the sudden expansion of section at the latter causes violent action at one place, while with the glacis the action is more spread and so less violent.

A certain fall in the writer's experience suddenly developed a big scour-hole that threatened the work ; the hole was filled with sand to a fairly steep slope, and on this was laid brick pitching in two layers that broke joint so as to form a flexible mattress : though the pitching did not go as deep as the hole had been, the slope decreased action so that the end of the pitching became covered and was safe from slipping ; the rest settled to a form that took care of the action and ended the trouble.

It has probably not been sufficiently clearly realised that most of the trouble experienced below falls is from wave-wash. Below water level the banks stand at their usual slope, but just at water-level the constant wash and draw forms a little flat beach. The causes of this wash call for investigation ; most channels will show examples of falls at which such wash is bad, and of others which for no apparent reason give no trouble. As far as the writer has been able to observe and form conclusions, the falls that give most trouble are those trapezoidal notches at which the drop is big enough to call for large destruction of energy, but not big enough to allow the fans to spread fully before passing the surface of the downstream reach. It is certainly not the biggest falls that give most trouble.

Such wave-wash sometimes continues for a long distance below a fall ; one note on the file of the Research Section on the subject points out that pitching should not be provided till it is seen how much is needed ; another note records that such pitching need not be made for the full depth of the canal, but only near water-level. In some cases the wash is sufficient to act through the joints of the pitching.

A measure tried at some falls on the older canals, to try to kill the action and save the cost of great lengths of pitching, was to enlarge the canal below the fall into a pool. In this enlarged section there is generally back-flow along the sides and as far as the writer has been able to observe, the measure is ineffective. Another measure often tried, is to use roughened pitching, with the intention of killing velocity along the sides; obviously this will have no effect on wave-wash. The writer once proposed to try a floating boom of sleepers; this came to nothing as he was transferred.

The right course, as usual, would be to attack the cause and not merely the effect: it seems likely that it would improve matters to increase the size of lip of the notch-falls that give trouble, so as to spread the fan of falling water more thoroughly.

The desirability of maintaining "normal" depths in the canal above the fall at all supplies has also, in the writer's opinion, been overdone; he believes that this is becoming recognised. If a raised crest be provided which maintains the "normal" depth for (say) full supply, this heads up to more than "normal" depth for all lesser supplies; the decrease of velocity and the increase of depth both involve a decrease of the critical velocity. Also, it is held by some that a raised crest impedes the passage of silt, even at the rate of supply which it does not head up.

Strong support for this view is found in what Mr. Woods showed of the Jhang Branch Upper, in a paper read before this Congress of 1916. He shows deep and extensive bed-scour, which ceased when the crests of falls and regulators were raised. The facts are indisputable, but the writer feels sure that full knowledge of the circumstances of place and time would show other causes.

Mr. Woods' line of reasoning was as follows. Where supply is passed off through under-shot gates, the filament of maximum velocity on a vertical is much nearer the bed than when it is passed off over a crest, so the relative velocities of adjacent filaments on a vertical are greater, and the silt-suspending power of the current is greater according to the theory of Dupuit. Mr. Woods maintained that this distortion of the natural distribution of velocities was not confined to a short distance above the work, but persisted up to the next work which would affect the distribution of velocities; he had observations made on the point. The Main Line of the Lower Jhelum Canal is free of such works for 36 miles from Rasul to Fakirian, where it is controlled by regulators with two tiers of gates and no raised cill and the supply was always passed under the gates, at floor level. Mr. Woods had the lower tier of gates used as a crest. With both methods of regulation, the curves of velocities on a vertical were observed with twin-floats, about 1,000 feet above the regulators, and at Rasul 36 miles above; the writer believes that Mr. Woods considered that the observations confirmed his contention; but they were in his own opinion too few to be convincing, and the differences too slight to be conclusive. At the same time it must be admitted that scouring of the first few miles of the canal diminished with the change of regulation, though not decisively and continuously enough to afford conclusive proof.

On the other hand, the writer had a raised crest about 3 feet high on a canal about 6 feet deep, with distributaries taking off above which suffered from silt trouble. They were close enough to the fall for it to be possible that the raised crest affected velocity distribution so far ; he tried building an upstream glacis to the crest, in place of its vertical face : there was no observable difference in the trouble in the distributaries and periodical soundings showed no change in the bed-level of the canal above the fall. The writer's principal conclusion at this point was that between the offtakes and the fall, the width of the channel should have been reduced more or less proportionately to the discharge of the offtakes. Altogether, it seems so utterly improbable that the position of an offtake can affect the distribution of velocities even as much as a thousand feet above, that a much greater volume of more conclusive data would be necessary to prove the contention.

In attempting to design a fall to maintain "normal" depths for all supplies, the precision aimed at should not be out of proportion to what is attainable in practice. Even where the fall is not partly drowned, the difference between assumed and actual coefficient of discharge for the fall affects the depths it maintains. The regime depths at which the channel eventually runs will differ from designed depths, and will further vary from time to time. The fall, designed to pass a certain discharge, with the depth "normal" for that discharge plus the discharges of offtakes, will be heading up above "normal" when those offtakes are closed. When the fall is partly drowned, differences due to silting and scouring below the fall cause further departures from design.

Knowledge of the coefficients of discharge of falls has of course always been necessary, so that they could be designed to maintain certain supply-levels with certain discharges but the accuracy hitherto accepted as sufficient, fall short of modern requirements. The realisation is also spreading, that by providing reliable meters at suitable intervals along channels, the trouble of taking discharge observations is saved, and efficient distribution is very greatly advanced. One of the primary objects of this paper is to record the simple measures by which canal falls can be converted into reliable meters of supply.

Economy of first cost and of maintenance do not call for any general remarks.

The standard Punjab trapezoidal notch-fall was evolved by Punjab irrigation engineers for use in construction of the Sirhind Canal : the primary object of this new form of fall was to maintain "normal" depths at all supplies ; the manner of using the trapezoidal notch to secure this was discussed and worked out by a number of the engineers of the department, and is published in Irrigation Branch Paper No. 2 of 1894, which largely follows a draft drawn up by Mr. (later Sir Thomas) Higham. Mr. J. Benton, Assistant Engineer (later Sir John Benton) made a series of experiments for determining the coefficient of discharge, and the form that would most effectively kill action downstream. These were printed up in 1879, but are so little known that it is worth while to repeat their salient points.

Tests were made of a single notch 5·63 feet wide at bottom and 8 feet wide at a height of 6 feet; and of a pair of notches each 3 feet wide at bottom and 4·3 feet wide at a height of 6 feet. Each notch was tested also with the bottom rounded off to a semi-circle. Each notch was tested set back 3 feet and 1 foot from the edge of the drop. Each notch was provided with a lip formed by striking an arc of a circle through a point 2·5 feet from the edge of the drop on the centre line, and the points where the splayed sides reached the edge at the bottom. Figures for the form of the notches are not given. By measurement from the plates in the printed report, the upstream wings made an angle of about 37½° with the line of flow, the downstream wings about 30°, and the radius of the curve connecting them seems to have been about 3 inches. The tests were made with only one rate of discharge, 160·8 cusecs; this discharge was fixed by always running a certain gauge steadily, for which that discharge had been observed before the experiments, and confirmed after them: this discharge gave depths of from 3·45 feet to 3·95 feet at the notches. In the smaller notches, the downstream wings were omitted. A few tests were also made of different slopes of downstream wings, of different sizes of lip, and of ridged lips.

As far as action is concerned, the best result was given by the single notch set only one foot back from the edge; this formed a broad smooth fan, and wave action below the fall was "imperceptible". The pair of notches gave a result not quite as good, but better than when set back three feet from the edge, in which latter case the fan was smooth but spread less.

As the result of these tests, the Chief Engineer, Major Home, R. E., laid down the well-known standard form of the notch for use. The upstream wings were made more oblique to the current, but more rounded off near the notch section; the tests had shown cross currents in the jet, which tended to spoil the even broad smooth fan that was sought: the downstream wings were made less oblique, by a similar rounding. One set of standard dimensions for the form was laid down for canals, suitable for depths of about 7 feet; and another for smaller channels, suitable for depths of about 3½ feet.

In all the tests, downstream water-level was about 4 feet below crest. Though a greater drop means that there is more energy to destroy, it also gives the fan room to spread and thin out: and it is apparently where the fan is not thus able to spread that trouble from action below the fall most frequently occurs.

As to coefficients, these were worked out in four ways, each allowing differently for velocity of approach. Results for the notches with semi-circular crests, and those in which h_a is other than $V^2/2g$ do not concern us. There being only one observation for each case, the single notch set back 3 feet gave a coefficient of 0·662; set back only 1 foot the value rose to 0·667, as would be expected. With the smaller notches, the steeper sides would slightly decrease the coefficient, while the rounding being greater relative to the dimensions of the notch would slightly increase it ; the observations gave 0·671 and 0·676 with the notches respectively 3 feet and 1 foot back from the edge.

The mean of the above coefficients, 0·67, was used for design. In the 1894 pamphlet it is stated at paragraph 13, that "an extended series of observations" on distributary notches shows that the value 0·70 may be used instead, at the same time omitting allowance for velocity of approach. "Not very extensive" observations on the larger canal notches, indicate a coefficient of 0·78, similarly ignoring allowance for velocity of approach. The simplification of the formula also makes a slight difference ; working out Sir John Benton's tests by this gives coefficients about 0·005 less than he gave.

These coefficients, 0·70 and 0·78, were then prescribed for the standard notch, both when free and when partly drowned. The writer's experience would lead him to expect an even greater increase of coefficient from the notch tested by Sir John Benton to the shape laid down as standard : in fact, for a free fall he would expect about 0·90 even without ignoring allowance for velocity of approach ; and with drowned falls conventional excessive allowance for the effect of drowning might mean that values up to 1·0 would be found.

There is no record in the Secretariat of the observations referred to, which are the basis of the change of coefficient : there is only a demi-official letter dated 15th August 1893, from Mr. Farrant to Sir Thomas Higham :—"On the Jagraon Rajbaha I found that a coefficient 0·69 had to be used for a free fall ; and in the case of all rajbahas I should say 0·70 would give more accurate results than 0·67. For canal notches, I did not have time to complete my observations before my transfer ; I can only say the coefficient is nearer 0·80 than 0·67. - - - - - - - - - - -. For the Abohar notches I find that the following formula expresses fairly accurately the law of variation of the coefficient for different depths

$$c=0{\cdot}67\ (1{\cdot}0+0{\cdot}07\sqrt{d}).$$

Thus, for 4·0, 5·0, 6·0 and 7·0 feet depths, the coefficients would be 0·76, 0·77, 0·78 and 0·79." Incidentally, Mr. Farrant contends that the coefficient of one notch will not be the same for all depths : and he is against the practice of attempting to include allowance for velocity of approach in the coefficient, since for different notches on the Abohar Branch the allowance should vary from 9 per cent. to 20 per cent.

In Captain Garrett's book on these falls, Sir John Benton is quoted as writing :—" Observations (subsequent to his of 1879) on the depths attained by known discharges in the numerous notch falls of the Abohar and Bhatinda Branches of the Sirhind Canal of about 80 feet bed-width, went to show that neglecting velocity of approach the coefficient of discharge was about 0·75 : this coefficient applies to the standard type of design. The increase of coefficient over that arrived at experimentally is partly due to the approaches to the notch being more rounded, and partly to the sizes of the vents being large, which resulted in friction being proportionately diminished for the discharge passed."

The writer has not troubled to make observations on notched falls, since he considered them now superseded by the meter form to be described below : but Appendix I gives figures for two falls, which he collected for other purposes.

These show, first a free-fall notch running up to $5\frac{1}{2}$ feet deep. Eighteen observations made for ordinary purposes by different observers during about two years, of which four are useless because the gauge was read only to tenths of a foot, are extremely consistent and give a mean coefficient of 0·888 at full supply. (It looks as if the coefficient decreased to about 0·84 at 2 feet depth ; such a decrease would be in agreement with theory). Absorption loss between the fall and discharge site would make this value about 0·007 too high, leaving it correctly 0·881. But, for comparison with the official coefficient which includes velocity of approach, 0·042 must be added to this, making it 0·923 instead of 0·75, or 23 per cent. higher.

The other fall is slightly larger, drowned by about two-thirds, with the sides therefore less vertical. Nineteen observations are tabulated, made by different observers during some seven years ; downstream gauges are interpolated from the gauge register, and that possibly accounts for the results being a little less consistent than those of the former fall. For five observations the downstream gauge could not be given. The mean of the coefficients is 0·971. To cover velocity of approach, 0·26 must be added, making it 0·997. This means that discharges are 33 per cent. higher than those calculated with the standard coefficient.

The official coefficient may or may not be correct for some falls also of standard form ; but these large departures, proved by figures here published, show that it cannot be used universally with any approach to safety. The figures also show that Mr. Farrant's range for the difference due to velocity of approach must be extended to cover from $2\frac{1}{2}$ per cent. to 20 per cent. : this is appreciably more than permissible under modern standards of accuracy.

For the better understanding of what follows next, it is advisable to enter a little upon hydraulic theory and history. In the search for a constant and reproducible standard form for measurement of discharges, all earlier efforts turned toward the "sharp crest". The essence of this is that flow shall leap from the face at right angles to the flow, with absolutely unimpeded and full natural contractions standardised by nature. Any roughness of that face decreases velocities along the face and at right angles to the main flow, and so decreases the contraction and increases the discharge ; any burring of the "sharp edge" interferes with the free leap from that face, and has the same effect. The sharp-edged triangular notch, the sharp-edged rectangular notch, and the

Cippolletti weir, are all examples of this. The first departure from it and concession to the difficulty of always obtaining full unimpeded natural contraction, was the Bazin weir, which fully suppressed contractions on the sides, and attempted to retain full contraction only on the bottom, subject to the effect of the contraction (and the velocity of approach) of a channel bottom at a stated depth below the weir crest. More recently Mr. Clemens Herschell attempted to standardise the partial contraction along the weir crest as well, by a standard rounding which is one of the features of the Herschell weir. The amount of such perfect contraction, and the discharge of the perfectly contracted weir, are not determinable by pure theory. Various experimenters, as the results of elaborate investigations, have given different formulæ and coefficient for it, which are purely empiric. To avoid a common misunderstanding, it is emphasised that the face from which the flow should leap clear is the vertical face, and not the horizontal top of the crest; the top is assumed not to exist, as if the weir were a thin plate.

Meanwhile, there was also the theory of the "broad-crest weir", given as the first part of Appendix II; this assumes not only infinite width or suppression of the side contractions, but also perfect suppression of the bottom contraction and flow from the vertical face follows a rounding or bell-mouth which it does not leave, and then follows the horizontal top as in a flume.

This is as definite a condition of flow as the perfect contraction of the "sharp crest" and while the latter is difficult of practical attainment, because any departure from perfect smoothness of the face or cleanness of the edge causes a small unknown amount of suppression of the perfect contraction, the broad crest is practically attainable: and the fact that the discharge can be calculated from pure theory is also an advantage. A further advantage is that a broad crest is sturdy in day to day working, while a sharp crest needs care even in a laboratory. Frequently most important of all is that, while a sharp crest must have an absolutely clear fall, with free admission of air under the nappe, the broad-crest weir is unaffected by drowning as long as the afflux does not fall short of a third of the difference in level between crest and upstream still-pond level. By addition to the shape this fraction can be reduced to a sixth or even less, so that this device can be used in many situations in which nothing like the drop needed for a sharp-crested weir is available.

This piece of theory is the basis of the flume or weir form of device introduced by Messrs. Stoddard and Harvey for outlets and larger offtakes, and is also suggested by the latter as a "standard notch" for metering discharges, in a paper read before this Congress in 1919. They, and a number of other observers, have made investigations to find the minimum of requirements that shape must conform to. Some of these observers have taken a standard shape, and when discharges did not follow the theoretical broad-crest formula, have been content to establish an empirical formula for discharges by experiment: such formulæ have contained coefficients up to 3·75 instead of the theoretical ~~3·90~~, 3·09 sometimes increasing with increasing depth so that the exponent had to be raised from 1·5 to 1·6 or so. Other observers have taken the point of view that departure from the broad-crest formula involved departure from a definite natural physical condition and introduced uncertainty; that only when this condition was attained was confidence justified in a discharge formula deduced from observations over a limited range; that any departure from this condition involved changes such that it would not be possible to rely on discharges being independent of fluctuations of downstream water levels.

The unexpected departures from the broad-crest condition are accounted for by a new piece of hydraulic theory, due to Mr. E. S. Crump and given in the second part of Appendix II. What the former theory ignored was that when too sudden a contraction at entry involves too sharp a dip of water surface, flow is in effect part of a vortex about a horizontal axis, with pressures within it reduced below those simply due to the depth below water surface. It is also necessary to make the crest so broad that the natural pressures due to depth below surface are established, before (with a free fall as the extreme case) the existence of atmospheric pressure at the bottom of the flow at the downstream edge begins to reduce pressures along the crest. The new set of conditions now calculated for may be named "narrow-crest" conditions, to distinguish them from "sharp" and "broad" crests.

These "narrow-crest" conditions are not desirable for our practical purposes, as discharge is not independent of downstream fluctuations; but the third part of Appendix II shows from them how definite the broad-crest conditions are when attained.

As the result of experiments which he has not published, Mr. Crump gives as the rule for attaining broad-crest conditions, that the width of crest (or length of parallel flume) must be twice its depth below still-pond level of water, and upstream approaches must be by curves of radius equal to this width. This rule for rounding is established for a crest of the full width of the approach channel and raised above its bed; the writer has not yet seen it established as applying to the approach to a flume narrower than the approach channel, except where the side contraction plays so small a part relative to the raising of crest, that the factor of safety allowed in the rule covers it also. Where the side contraction is relatively appreciable, the rule for entrance curves will have to take into account the width of throat as well as the depth.

On these lines, Mr. Crump has remodelled the fall over which the tail discharge of the Upper Chenab Canal is dropped into the river Ravi above the Balloke barrage. The writer has not been able to obtain a plan of the work, but it is probably very like the plan given with this paper.

In Appendix III, the first table gives discharge observations made at this meter, and works out for each the actual value of the coefficient in

$$D = k \times w \times (d + h_a)^{1\cdot 5}$$

for comparison with the theoretical value of 3·089. The first group of fourteen observations was made with special care to test the meter. Among other things, velocities were corrected for the length of rod (*vide* I. B. Paper class B No. 6). Excluding the first observation in which there is some serious error, the rest of the group give an average of 3·098 for the coefficient. The next group contains observations made in the course of subsequent routine, before the accuracy of the meter was officially accepted. Excluding the second, which is badly wrong, the rest give an average of 3·082 for the coefficient. The third group consists of observations made independently, by the division that regulated supplies arriving. Discharges are reduced by 18 cusecs, the approximate amount of the loss by absorption between the discharge site and the meter. Excluding the last but one, which is bad, the rest average 3·074. The average for the whole series is 3·086 ; better agreement could hardly be expected with the theoretical value of 3·089.

The data give no reason for suspecting any variation of the coefficient over the range of observations, which is from roughly $2\frac{3}{4}$ feet up to 6 feet, *e.g.*, from 2,300 to 7,000 cusecs. At this site there is not always a free fall, and figures at some of the observations, giving the afflux as a percentage of $(d+h_a)$ show that discharges are the same whether there be a free fall, or the fall be drowned up to nearly the two-thirds that is the actual limit.

The discharge table was framed by calculation before the work was built, and it was stated that discharges would be independent of down-stream levels as long as afflux was not less than one-third of the depth. These subsequent observations fully prove what was claimed.

Although the discharge table for such a meter need only be calculated once, it is a little laborious to do so in full from the above formula. The fourth part of Appendix II shows a shorter method, by calculating only two points with allowance for h_a and finding the exponential relation that satisfies these without allowing for h_a for use in calculating other points. In the formula thus obtained for the Balloke meter, the coefficient becomes 3·057 and the exponent 1·54. In the second part of Appendix III, $(d+h_a)$ is calculated for a series of discharges, h_a calculated and subtracted leaving remainder d, and discharges calculated for these values of d by the simplified formula. The comparison shows that the percentage difference is negligible over the range of working discharges ; and where the percentage difference becomes greater it is so small in cusecs that it does not matter.

A similar meter has been built at the tail of the Upper Jhelum Canal, on the lines shown on Plate I. The fifth part of Appendix II shows the very small amount of calculation that was needed, and a specimen calculation of another kind.

The observations made by local officers to test the accuracy of the meter are given in the third part of Appendix III; and on Plate I the coefficients are plotted against $(d+h_a)$, to logarithmic scales. From a similar plot that he made to ordinary co-ordinates, the Executive Engineer (Mr. Dixon) deduced a coefficient increasing with depth, from 2·73 at 1 foot to 3·19 at 5 feet. So much variation is physically impossible with this design. The writer's plot shows how, by ignoring the three higher values of coefficient obtained at low gauges and taking only the lower values obtained there, a similarly rising coefficient is arrived at; but we might just as well ignore the lower values and take the higher, when we should get a coefficient decreasing with depth. If we attach equal weight to all the observations, there is not enough evidence for a variation of coefficient to weigh against the showing of the Balloke observations. Taken separately, the data up to 2½ feet depth give an average value of 3·061, those above 2½ feet a value of 3·151, and the whole series an average of 3·118, which is also not far from the theoretical 3·089. The last three observations shown were made for the writer, with a current meter the rating table of which was checked and revised shortly after the observations. These give a coefficient of 3·083 at 4¾ feet gauge, as compared with 3·18 by Mr. Dixon's curve: and it is significant that the meter observation at 4·69 gauge gave 6,803 cusecs at the same time at which the velocity-rod observation gave 6,976 cusecs, which would reduce his coefficient of 3·18 to 3·10.

In the writer's opinion, this meter, also, discharges according to the theoretical formula. At the same time it should be stated that the gauge well is placed above the point at which a masonry floor rigidly fixes the section and the velocity of approach, and projects into the flow in a manner that produces cross-currents and differences of water level across the canal; it is therefore possible that the gauge is reading wrongly.

On this work the writer has tried to obtain a gauge-reading which should include the velocity of approach. The extensions of existing pier noses were all made hollow with orifices in the upstream point. Results are uncertain, apparently on account of the cross currents referred to: and Mr. Dixon reports that these gauge wells silt up too rapidly to be useful.

The fourth part of Appendix III shows observations made on a smaller meter that Mr. Dixon designed and built on a distributary; the writer calculates an average value of 3·15 for the coefficient, while the Executive Engineer states (but does not show) a value of 3·1. This meter was built so as to head up supplies very considerably; the writer's calculations assume original unchanged bed levels in arriving at velocities of approach but it would appear that the Executive Engineer has taken actual and higher values, which would give his lower coefficient. Local officers have accepted the meter as correct.

At all the sites dealt with above a free fall is available, except that at Balloke where the fall is at times drowned, but never by more than the two-thirds that still fails to affect discharges. Where less head is available, a downstream glacis can be built down from the raised crest, or an expanding flume from the narrowed gullet. In the circumstances under which this note is written, the writer is unable to give specimen designs of such works that he has built ; if necessary these can be supplied later. A slope of 1 : 15 is safe for such a glacis. For an expanding flume a widening of 1 : 10 on either side is usual, but the writer finds roughly that 1 : 5 is practically as effective, and that attempts to find a curve which will secure the best recovery of head are not worth while. After all, if such a curve were to reduce the ratio of afflux needed from a sixth to an eighth, this would only make a difference of 3″ in the afflux needed for a meter flowing 6 feet deep, and proportionately less in smaller meters. In designing a meter for a small available fall, it must not be overlooked that the worst silted conditions downstream must be allowed for. In some cases in which such a meter is wanted for regulating at the head of a branch canal, a fall within a mile or two of the head can be used ; this need only be read every two or three days ordinarily, intermediate gauge readings being given from the variations of the ordinary regulating gauge. As a further point of detail, the writer generally has a gauge graduated to read cusecs direct instead of feet, to fix above such a work.

In the specific works referred to above, the crests are so long and depths so small that no special provision is needed against action downstream. Where the work is to consist of a set of narrower deeper notches, the writer proposed to add expanding flumes in which flow would become broader and shallower. With relatively high levels downstream, a standing wave would form within the flume ; with a free fall a relatively thin sheet would shoot out horizontally from the edge of the floor. The idea has not been tested practically so that nothing can be said of its effectiveness ; and *prima facie* it does not promise to be a cheap design.

Mr. C. C. Inglis has been experimenting at Poona, with a type of fall the upper part of which is built to conform to the requirements of a broad-crest meter, flow from which is carried down an Ogee glacis at the foot of which a standing wave is formed ; the principal object is believed to be an economical design, it being suggested that the glacis need only be an apron of reinforced concrete. It is hoped that he will contribute an account of his results to the discussion on this paper.

Another form of fall has been suggested by Mr. F. H. Burkitt ; the upper part conforms to the requirements of a broad-crest meter ; the flow from this is dropped on to a reinforced concrete slab, rising down stream and with a parapet at the downstream edge. It passes to the underside of the slab at the upstream edge, and then flows downstream through the expanding tube formed by the canal bed and the rising slab : Mr. Burkitt suggests that this expanding tube is likely to be the most effective way of killing wave action.

An investigation has recently been made by Mr. Darley, for a fall that would effectively kill action, for use on the Sarda Canals. The types tested were :—

(1) a raised crest, of length=bed width + supply depth ;
(2) a raised crest, of length=bed width only ;
(3) a standard Punjab notch ;
(4) a "re-entrant" notch ;
(5) a "re-entrant" raised crest ;
(6) a contracted gullet.

The "re-entrant" falls are suggested by Mr. Hickey : the crest seems to be a V in plan with the point upstream, forming in effect two weirs discharging into each other : the notch seems to be similar, except that the raised crest also rises from the point of the V. The tests were made on a channel 12 feet wide and 3 feet deep, and consisted of forming a level canal-bed of sand downstream (after $9\frac{1}{2}$ feet of pakka floor), and noting the scour after a period of running. Of the first three items, the longest crest was best and the deep notch by far the worst. The re-entrant falls were "extraordinarily successful," especially the notch, which even allowed a deposit of silt along the sides of the canal. The last appears to have been a "Harvey Flume", and did not do well in the tests ; apparently the central velocity persists through the expansion flume. These tests, made in November 1922, did not complete the investigation. Among other things it was proposed to calibrate the re-entrant notch that was most successful. It is hoped that this paper will elicit some account of these further tests.

Regarding them, the writer would however observe, first, that the re-entrant notch can be calibrated for a free fall, but will have to be given a free fall ; it will thus have a more restricted field than the broad-crest meter fall : second, attention seems to have been concentrated on bed scour, which it is easy to protect against, while the more troublesome wave-wash does not appear to have been watched.

* * * * * *

This note is written on board the steamer, going home on leave. Anyone who has tried to do a job of work under these circumstances knows that they are not propitious ; and with the utmost forethought it is impossible to avoid gaps in the material set aside to be available in the light baggage. The writer therefore asks a little extra indulgence in judging this piece of work.

APPEN

OBSERVATIONS ON

BHOWANA FALL.

2 notches, l= 2·70, n= 0·364. Canal bed at crest level, 23 feet wide.

D	d	h_a	$d+h_a$	s	e
432	5·40	0·15	5·55		0·880
436	5·35	0·16	5·51		0·900
429	5·35	0·15	5·50		0·888
428	5·31	0·15	5·46		0·896
435	5·33	..	..		..
417	5·27	0·15	5·42		0·886
417	5·25	0·15	5·40		0·864
418	5·20	0·15	5·35		0·908
414	5·20	0·15	5·35		0·900
397	5·2?	..	..	Free Fall.	..
388	5·2?	..	..		..
374	5·2?	..	..		..
393	5·15	0·14	5·29		0·870
401	5·10	0·15	5·25		0·900
377	5·05	0·13	5·18		0·868
367	4·82	0·14	4·96		0·908
351	4·82	0·13	4·95		0·870
88½	2·15	0·04	2·19		0·845

DIX I.

TRAPEZOIDAL NOTCHES.

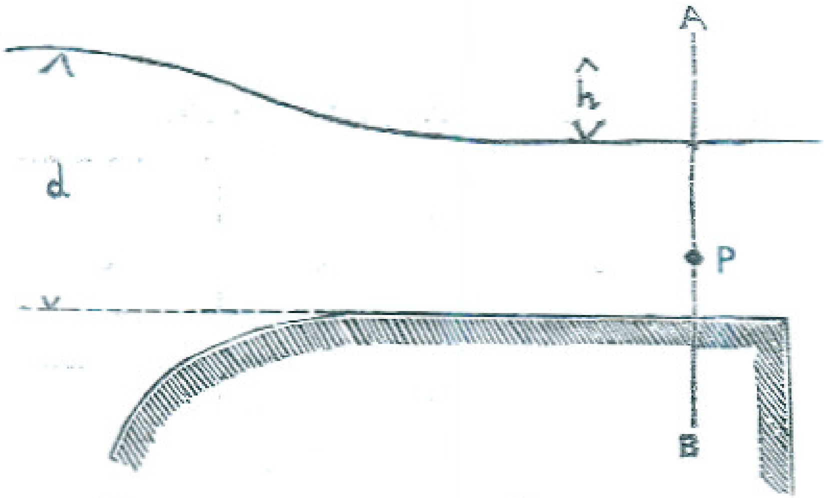
TATNIKHA FALL.

6 notches, l = 2·45, n = 0·543. Canal bed at crest level, 123 feet wide.

D	d	h_a	$d+h_a$	e	c
1,679	6·47	0·07	6·54	4·20	0·980
1,690	6·45	0·07	6·52	4·35	1·016
1,593	6·45	0·07	6·52	4·20	0·936
1,698	6·43	..	..	?	..
1,682	6·43	0·07	6·50	4·05	0·975
1,634	6·42	0·07	6·49	4·29	0·986
1,660	6·38	..	..	?	..
1,670	6·37	0·07	6·44	4·19	0·992
1,656	6·36	..	..	?	..
1,582	6·31	0·07	6·38	4·08	0·964
1,646	6·30	..	..	?	..
1,716	6·27	0·07	6·34	4·04	1·055
1,509	6·03	0·06	6·09	3·67	0·972
1 450	6·03	0·06	6·09	3·65	0·940
1,382	5·89	0·06	5·95	3·36	0·908
1,377	5·855	0·06	5·91	3·525	0·934
1,375	5·73	0·06	5·79	3·40	1·006
1,211	5·41	..	..	?	..
1,236	5·24	0·06	5·30	3·46	0·954

APPENDIX II.

1. *Broad-crest Weir.*



Let d be the elevation of still pond water surface above weir crest.

Let h be the difference of water levels above and below the weir.

On a vertical section A B through the stream, at any point P whose height above crest = y,

$$v^2/2g + p/w = d - y$$
$$p/w = d - h - y$$
$$\therefore v^2/2g = h \text{ or } v = \sqrt{2gh}$$
$$\text{So } D = (d - h)\sqrt{2gh}$$

differentiating with respect to h and equating to zero,

$$\frac{(d - h)}{2\sqrt{h}} - \sqrt{h} = 0$$
$$d - h = 2\,h$$
$$h = d/3$$

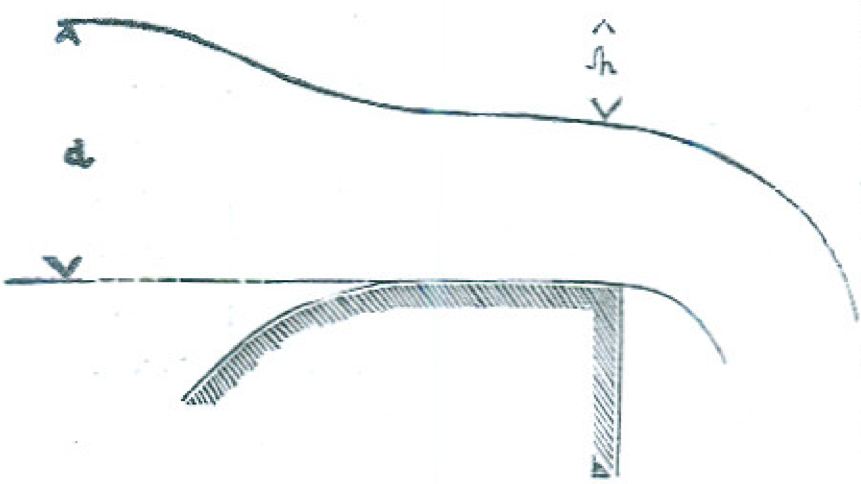
So D is a maximum when h = d/3 : if h be greater than this, the discharge will obviously not be less than that ; but there will be a drop of only d/3 upon the weir crest, which is converted into velocity and a drop of h—d/3 from the crest, which is wasted.

This maximum $D = 2/3\ d \times \sqrt{2g\ d/3}$

$= 3{\cdot}089\ d^{1{\cdot}5}$

2. *Narrow-crest Weir.*

(Due to Mr. E. S. Crump.)



Let d be as before, and h be the drop of water surface before leaving the weir crest, under free fall conditions.

At the surface vertically over the weir crest,

$p = 0$ and $V_s = \sqrt{2gh}$,

At the d. s. edge of the weir crest,

$p = 0$ and $V_c = \sqrt{2gd}$

Between these two points, on a vertical section pressures and velocities for equilibrium must be distributed as in a free vortex; that is, velocity is inversely proportioned to the distance from the centre of the vortex.

Let $y_{,}$ be the depth of that centre below weir crest;

Then $V_c/V_s = (y_{,} + d - h)/y_{,} = 1 + (d - h)/y_{,}$

But from the first two equations,

$$V_c/V_s = \sqrt{d/h}$$
$$\therefore (d - h)/y_{,} = \sqrt{d/h} - 1 = (\sqrt{d} - \sqrt{h})/\sqrt{h}$$
$$\therefore y_{,} = \sqrt{h}\,(\sqrt{d} + \sqrt{h})$$

At any point at height y above the vortex centre,

$$V = V_c \times y_{,}/y$$
$$= \sqrt{2gd}\ \sqrt{h}\,(\sqrt{d} + \sqrt{h})/y$$

So the total discharge in unit width,

$$D = \sqrt{2gdh}\,(\sqrt{d} + \sqrt{h})\int_{y_{,}}^{y_{,}+d-h} dy/y$$
$$= \sqrt{2gdh}\,(\sqrt{d} + \sqrt{h})\ \log (y_{,} + d - h)/y$$
$$= \sqrt{2gdh}\,(\sqrt{d} + \sqrt{h})\ \log \sqrt{d/h}$$

Putting f for h/d.

$$= \sqrt{2g}\, d^{1 \cdot 5}\sqrt{f}\,(1 + \sqrt{f})\ \log \sqrt{1/f.}$$

Differentiating with respect to f and equating to zero.

$$\log \sqrt{1/f}\left(1 + \frac{1}{2\sqrt{f}}\right) + \sqrt{f}\,(1 + \sqrt{f})\ \sqrt{f}\,\frac{\sqrt{f}}{2}\,\frac{-1}{f^{2}} = 0$$

$$\log \sqrt{1/f}\left(1 + \frac{1}{2\sqrt{f}}\right) = (1 + \sqrt{f})/2\sqrt{f}$$

$$\frac{1 + \sqrt{f}}{1 + 2\sqrt{f}} = \log \sqrt{1/f} = -1/2 \log f$$

When $f = 0 \cdot 22$, both sides of the equation $= 1 \cdot 515$.

Therefore, the maximum discharge takes place when

$$h = 0 \cdot 22\ d$$

and then $D = 4 \cdot 184\ d^{1 \cdot 5}$

3. *True broad-crest weir.*

By part 1

$$D = 3 \cdot 089\ d^{1 \cdot 5}$$

By part 2

$$D = \sqrt{2g}\ d^{1 \cdot 5}\sqrt{f}\,(1 + \sqrt{f})\ \log \sqrt{1/f}$$

This is satisfied when $f = 0 \cdot 55$. So, when the weir satisfies the conditions explained in the note, the drop of water surface on the crest is $0 \cdot 55d$ with a free fall, as compared with $0 \cdot 333d$ at the minimum that will pass the broad-crest discharge. That is, in the part of the flow in which pressures are not sensibly less than those due to depth below water surface, surface levels in the two extreme cases do not differ much.

As long as h/d is between 0·29 and 0·38, the theoretical discharge by part 1. is not reduced by more than 1 %; and when h/d is 0·22 or 0·46 the reduction is only 5 %: such reduction of pressure as there is, increases velocities. So these two tendencies are in opposite directions.

The result is that many works discharge near enough to the broad crest formula for practical purposes; if they are provided with a downstream glacis, or if the ratio of free fall to depth is somewhere between ¼ and ½.

4. To find a simplified formula for discharge of the Balloki water fall, see table 2 of Appendix III.

$$D = K\,d^{n}.$$

As the range to be fitted with reasonable accuracy, take for 8,000 to 750 cusecs.

$$\left(\frac{6{\cdot}410}{1{\cdot}367}\right)^{n} = \frac{8{,}000}{750} \qquad 4{\cdot}690^{n} = 10{\cdot}66$$

taking logs, $0{\cdot}671 \times n = 1{\cdot}028$ $\quad n = 1{\cdot}532$

This may be simplified to 1·53, if the method of working used makes this simpler to use.

If discharges and depths were plotted to logarithmic scales, the points for 8,000 and 750 cusecs would be almost in one line for $n = 1{\cdot}53$; between them discharges lie on a curve to one side of this line; it is required to find a value of K that will give roughly equal extreme plus and minus errors.

$$8{,}000 \div 6{\cdot}410^{1{\cdot}53} = 8{,}000 \div 17{\cdot}16 = 466{\cdot}3$$
$$4{,}500 \div 4{\cdot}411^{1{\cdot}53} = 4{,}500 \div 9{\cdot}70 = 464{\cdot}0$$
$$2{,}400 \div 2{\cdot}93^{1{\cdot}53} = 4{,}500 \div 5{\cdot}18 = 463{\cdot}3$$
$$750 \div 1{\cdot}367^{1{\cdot}53} = 750 \div 1{\cdot}615 = 464{\cdot}2$$

(Three points are enough; two intermediate points are taken here to show that the geometric mean of extremes is a better point to note than the arithmetic mean).

As a suitable mean take $K = 465$.

The discharges in the last column of table 2 of Appendix III are worked out by the formula $D = 465 \times d^{1{\cdot}53}$

and agree with actual figures better than necessary for ordinary purposes.

5. Specimen calculations for a meter.

Existing fall has a crest raised 2·5 feet above u. s. floor.

10 bays of 20 feet. Bed-width 236 feet. F. S. L. for 8,000 cusecs to be 7·7 feet above floor.

(*a*) Keeping existing length of crest.

$$8{,}000 = 3{\cdot}09 \times 30 \times 20 \times 5{\cdot}51^{1{\cdot}5}$$

so for 8,000 cusecs, $(d - h_a) = 5{\cdot}51$

$$8{,}000 = 7{\cdot}7 \times (236 + 7{\cdot}7) \times 4{\cdot}26$$

$$\frac{4{\cdot}26^2}{2\,g} = 0{\cdot}285 = h_a$$

$$7{\cdot}70 - 5{\cdot}51 + 0{\cdot}285 = 2{\cdot}475$$

is the height of crest required: but it is not worth while lowering the existing crest by 0·025, so it may be retained.

(*b*) Velocity must be sufficient to keep u. s. floor clear of silt, so that section and velocity at the gauge well are defined and constant.

$v = 4 \cdot 26 \qquad d = 7 \cdot 7 \qquad v = 1 \cdot 36 \ V_o$

which is more than ample.

If V_o were specified, the crest would have to be raised or the floor lowered by 1·57, making depth on floor d = 9·3.

(*c*) As an alternative to (*a*) assume that "normal" depths are to be maintained: then, crest must be (7·7÷10) feet above floor.

$$8{,}000 = 3 \cdot 09 \times 10 \times 13 \cdot 35 \times (7 \cdot 7 - 0 \cdot 77 + 0 \cdot 28)^{1 \cdot 5}$$

so each bay must be reduced to a notch 13·35 feet wide.

APPEN

1. UPPER CHENAB CANAL

Crest, bays of = 150 feet, Canal bed 3·1 feet

Discharge.	depth.	h_a	$d+h_a$	K.	Free Fall.
4,083	3·77	0·139	3·909	3·520	..
3,238	3·46	0·096	3·556	3·220	..
3,050	3·39	0·090	3·480	3·125	..
2,816	3·22	0·078	3·298	3·130	..
4,779	4·47	0·156	4·626	3·200	..
4,529	4·47	0·140	4·610	3·050	..
5,047	4·68	0·164	4·844	3·165	..
5,231	4·85	0·169	5·019	3·100	..
4,823	4·775	0·146	4·921	2·945	45·7%
4,790	4·71	0·146	4·856	2·980	44·8%
4,956	4·70	0·157	4·857	3·085	43·6%
5,263	4·84	0·171	5·011	3·135	44·2%
7,051	5·89	0·237	6·127	3·100	100%
6,706	5·78	0·220	6·000	3·045	..
6,174	5·47	0·201	5·671	3·050	100%
5,669	5·59	0·165	5·755	2·740	88·7%
6,066	5·35	0·200	5·550	3·090	92·5%
5,262	4·83	0·171	5·001	3·140	90·0%
5,333	4·90	0·173	5·073	3·110	96·0%
4,866	4·76	0·149	4·909	2·980	85·3%
5,926	5·28	0·194	5·474	3·090	86·7%
5,947	5·23	0·197	5·427	3·140	100%

DIX III.

TAIL-METER OBSERVATIONS.

below crest, 192 feet wide.

D.	d.	h_a	$d+h_a$	K.	
5,683	5·25	0·180	5·430	2·990	62·8%
5,568	5·045	0·182	5·227	3·105	..
6,214	5·45	0·204	5·654	3·065	..
4,406	4·35	0·136	4·486	3·090	..
2,878	3·33	0·079	3·409	3·045	..
3,881	4·04	0·116	4·156	3·050	..
5,733	5·06	0·191	5·251	3·180	..
5,624	5·03	0·186	5·216	3·150	..
3,786	4·05	0·113	4·163	2·970	..
2,292	2·73	0·061	2·791	3·105	..
5,139	4·91	0·160	5·070	3·000	..
5,150	4·70	0·170	4·870	3·190	..
5,476	5·00	0·177	5·177	3·100	..
6,088	5·42	0·198	5·618	3·050	..
6,274	5·48	0·207	5·687	3·085	..
5,859	5·30	0·188	5·488	3·050	..
5,427	5·25	0·164	5·414	2·870	..
6,394	5·56	0·210	5·771	3·075	..
5,981	5·29	0·197	5·487	3·100	..
6,498	5·44	0·224	5·664	3·210	..
6,373	5·17	0·230	5·400	3·370	..
5,664	5·06	0·187	5·247	3·150	..

APPEN

2. DISCHARGE CURVE OF UPPER CHENAB CANAL

D.	$d+h_a$	h_a	d.	D.	
8,000	6·681	0·271	6·410	7,975	..
6,000	5·513	0·197	5·316	5,980	..
4,500	4·551	0·140	4·411	4,502	..
3,300	3·701	0·095	3·606	3,307	..
2,400	2·993	0·063	2·930	2,408	..
1,800	2·471	0·042	2·429	1,807	..
1,350	2·040	0·028	2·012	1,355	..

3. UPPER JHELUM CANAL TAIL-

Crest, 10 bays of 20 feet = 200 feet,

D.	d	h_a	$d+h_a$	K.	
510	0·95	0·006	0·956	2·730	..
1,200	1·45	0·025	1·475	3·350	..
1,040	1·47	0·019	1·489	2·862	mean of 2
1,150	1·54	0·022	1·562	2·945	..
2,080	2·11	0·053	2·163	3·265	..
2,166	2·17	0·058	2·228	3·257	..
2,760	2·66	0·076	2·736	3·048	..
3,500	3·09	0·104	3·194	3·064	..
4,760	3·75	0·154	3·904	3·082	..

4. METER ON DISTRIBUTARY 14-R. OF

Crest 25 feet long, Canal bed,

29·2	0·52	0·001	0·521	3·10	mean of 2
79·3	1·00	0·007	1·007	3·14	,, 2
148·7	1·50	0·018	1·518	3·20	,, 4

DIX III.

TAIL-METER ($D=465\times d^{1.53}$).

D	$d+h_a$	h_a	d	D	
1,000	1·670	0·018	1·650	1,002	..
750	1·378	0·011	1·367	750	..
560	1·134	0·007	1·127	558	..
420	0·937	0·004	0·933	418	..
320	0·781	0·003	0·778	317	..
240	0·645	0·002	0·643	237	..
180	0·532	0·001	0·531	177	..

METER OBSERVATIONS.

Canal bed, 2·5 feet below crest and 236 feet wide.

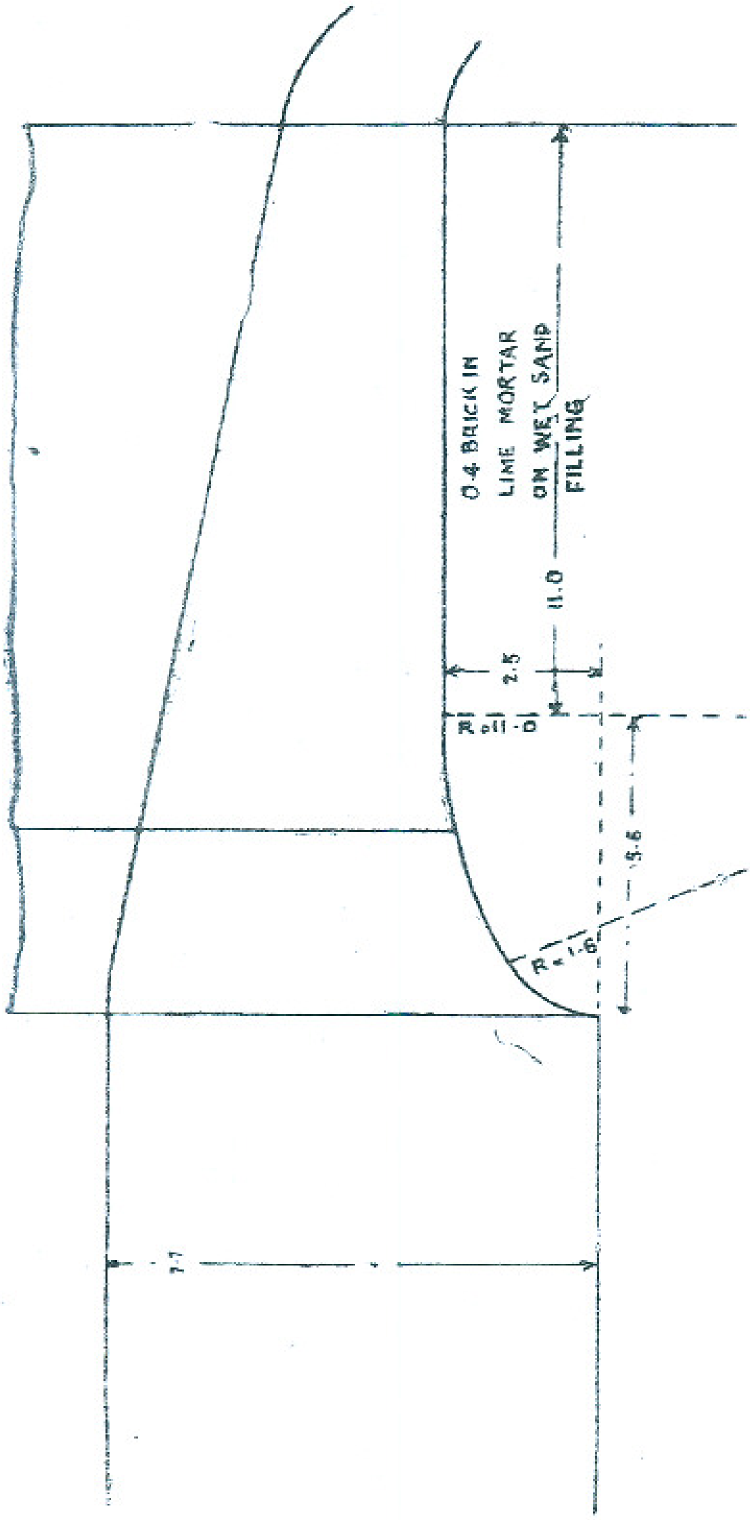
D	d	h_a	$d+h_a$	K	
5,240	3·88	0·179	4·059	3·202	mean of 2
5,170	3·91	0·173	4·083	3·133	,, 2
5,400	3·95	0·186	4·136	3·213	—
6,580	4·44	0·237	4·677	3·252	..
6,976	4·69	0·248	4·938	3·180	..
7,090	4·74	0·252	4·992	3·180	..
6,803	4·69	0·235	4·925	3·110	..
6,881	4·77	0·235	5·005	3·070	..
6,878	4·77	0·235	5·005	3·070	..

UPPER JHELUM CANAL.

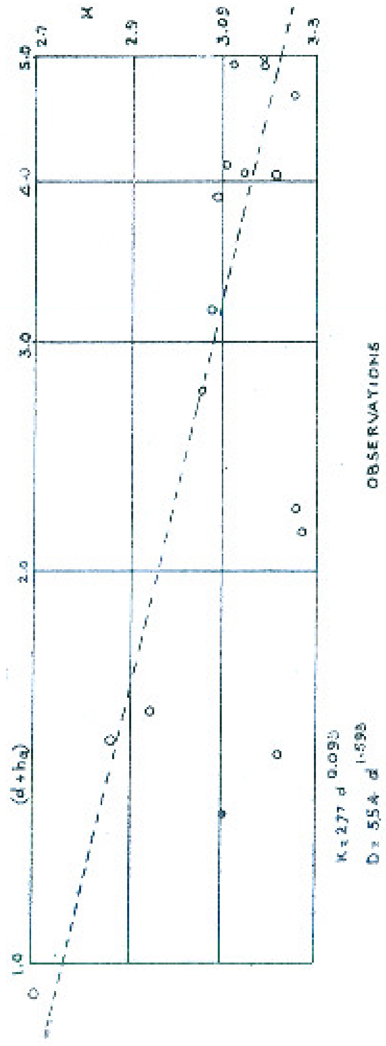
2·87 feet below crest and 27 feet wide.

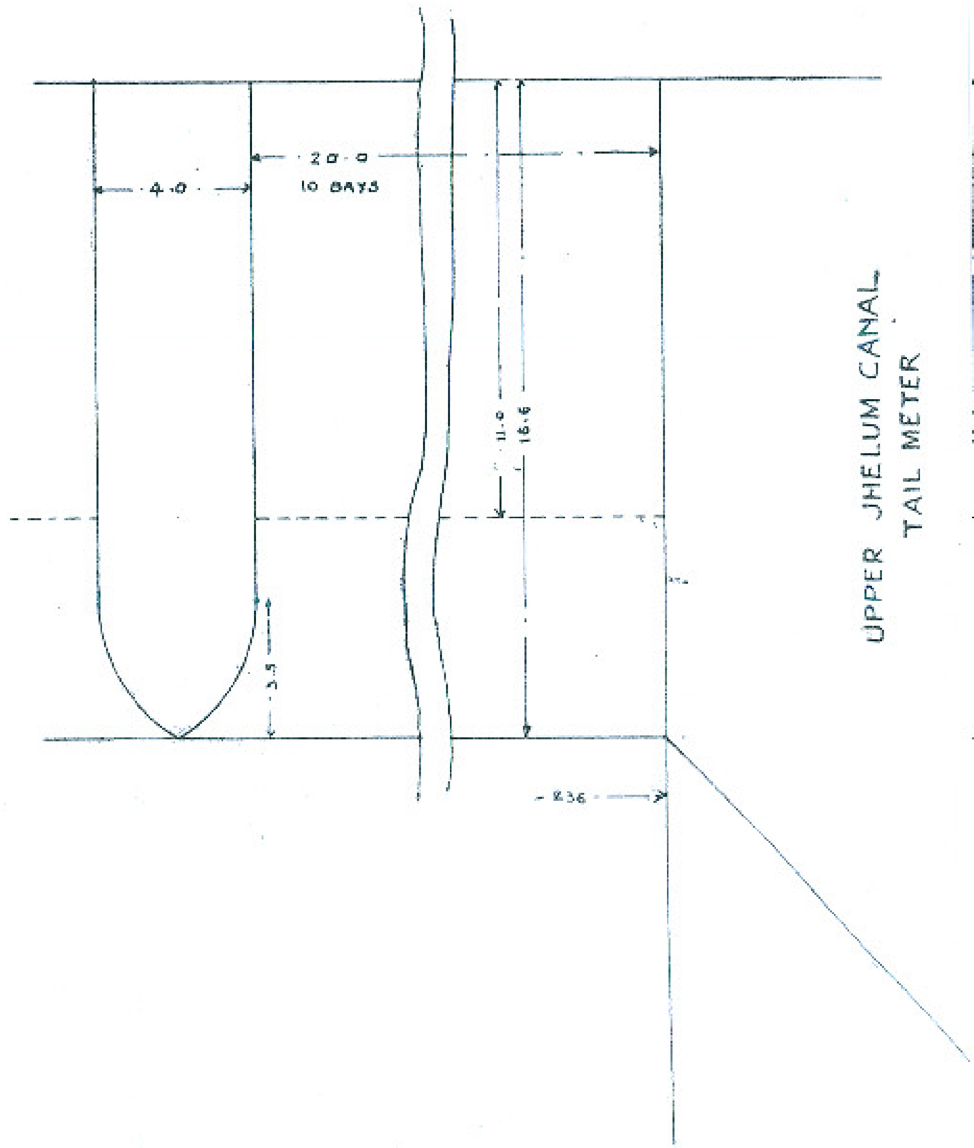
176·8	1·69	0·024	1·714	3·14	mean of 2
208·8	1·90	0·030	1·930	3·11	,, 2
..	—	..	..	..	..

PLATE 1.


0.4 BRICK IN
LIME MORTAR
ON WET SAND
FILLING
11.0
2.5
R=11-0
15-6
R=1-6
7.7

Lindley-Canal Falls and Their Uses as Meters





DISCUSSION.

In the absence of Mr. Lindley, the paper was introduced by Mr. Colyer who at the same time gave his own views on the subject.

He wished to emphasize the practical importance of pages 81 and 82 of the Paper and recommended their careful study.

Even when bed-scour had been provided against, wave-wash or pulsation usually remained, and was the chief cause of damage below falls, flumes, and bridges.

He thought the reason for abandoning the ogee form of fall, was that people hesitated to provide for the high velocities developed thereby.

The standing-wave or 'Hydraulic-jump' was undoubtedly the most efficacious single means of dissipating such energy, but attention was drawn to the error made by some of the best theoreticians in concluding that this phenomenon could, by itself, abstract the whole of the energy. Experiments which he had made on small models, of from one to two cusecs' discharge, showed quantitatively that whereas in one case the depth of cistern theoretically necessary to produce a standing-wave, was only 6 inches, the provision of a cistern 2 feet deep considerably further reduced the surplus destructive energy, both of velocity and of pulsation.

Various methods of destroying surplus energy below falls had been tried. Roughened, or cellular pitching of the side-slopes had not proved effective in any but the smallest channels, on account of the relatively small proportion of the total discharge which was affected thereby.

Side-expansions of the channel with bottle-neck egress, with the sides and part of the bed protected, had been tried by him in several cases and found eventually ineffective, as the smooth swirl-chamber so provided, did not abstract the energy from the eddies, which therefore persisted and attacked the channel farther downstream. Mr. Routh, however, has found these 'bottle-neck' expansions effective.

The difficulty of combating wave-wash and smooth persisting eddies was that, although the small amount of energy contained therein could do considerable damage to earthen banks, it was extremely difficult to abstract it.

As Mr. Lindley remarked, the only satisfactory treatment was to attack the cause. The speaker had attempted a solution as follows :—

The trouble consisted mainly of three parts : high scouring-velocity, destructive eddies both smooth and turbulent, and pulsation or wave-wash. It therefore seemed sound in principle to take this velocity and, keeping it well in hand, lead it by an easy path to form a standing-wave, and then turn it back upon itself, and by giving it a negative direction, to counteract its kinetic energy ; and, to prevent this from merely being converted into potential energy, to cause as violent turbulence as possible for the abstraction of energy in eddies and heat ; and then to screen out the large eddies and pulsations, before once more gradually leading the effluent to the normal channel at a normal velocity.

The first phase was accomplished by gentle side-expansion and a sloping glacis, in place of the usual sudden expansion and abrupt fall; the standing-wave forming somewhere on the glacis, and abstracting part of the energy. (See plate 1).

To effect the next abstraction in the best manner, experiments were made on various forms of deflector (vulgarly, 'biff-wall'). A quantitative comparison of the three shapes shown in Plate 3, resulted in section C giving an improvement over section A of :—23 per cent. reduction of effluent velocity, and 50 per cent. decrease of effluent pulsation or wave-wash.

An unexpected result was that section B which was a rectilinear approximation to section C, was not nearly as effective as the latter; the local turbulence caused by the rectangular corbels, seeming to affect adversely the reversing action which was more completely achieved by the smooth curves of section C.

Similar experiments showed that the most effective height of this 'biff-wall' was such that it should just act as a weir with a free fall. (See Plates 1 and 2.)

This also was a surprise, for it had been thought that to again create turbulence would re-establish wave-wash and scour. The experiments, however, definitely showed that both the velocity and the pulsation evils were greater under drowned-weir conditions than under well-ærated free-fall conditions.

To obtain a maximum screening effect, this weir should be as long as practicable, and, where the downstream F. S. L. was comparatively high, its breadth should approximate to twice the depth of water over the crest, so as to ensure a free-fall.

The third and last stage, that of delivering the effluent at the normal channel section as a smooth and eddyless stream of normal velocity, was found to be best accomplished by warping or 'winding' the side-protection from the vertical into the designed normal side-slopes, in the maximum available length, and in the case of previously eroded beds, to run the floor down as a glacis, to at least old-scoured-bed, at a slope of from 1 in 3 to 1 in 10.

Experience showed that the side-pitching needed to be 'pakka' until it reaches a slope of 1 in 1, and, after that a short length of smooth (not cellular) pitching laid on ballast, followed by bats roughly strewn on at least nine inches of ballast, formed an ideal protection.

A suitable rate of side-expansion was 1 in 10 on each side.

Some people had been afraid of this design on the score of first cost, but after independent investigation and strict scrutiny of cost compared with other methods, it has been accepted by the Chief Engineer, Sutlej Valley Project for use on new works.

Another possible objection which had been advanced was its liability to attack by eddying bricks and stones, and consequent formation of pot-holes. Actually, it was less liable to this defect than any other existing form of fall, and where such action was anticipated, it could be obviated by the provision of fillets or splays of masonry to ease the abruptness of rectangular changes of direction of flow. (See Plate 2.)

Naturally, with a device of this kind, the best materials and workmanship were essential, and this is where the Sub-divisional Officer came in.

As examples of the efficacy of a sloping glacis for the treatment of bed-scour below falls and bridges, he mentioned the bridges at R. D. 8,000 and R. D. 307,000. Lower Bari Doab Canal, where the provision of a short extension of the loose stone apron, sloped down at 1 in 5 or 1 in 10, caused silt-deposit in the place of scour.

He agreed with Mr. Lindley that the desirability of maintaining normal depth at all supplies, had been overdone; in fact, nowadays, in distributaries it was usually enough to legislate for full-supply-depth alone, owing to the excellent custom which had grown up in the last ten years, of running distributaries in turn at full-supply, and closing them after their turn.

He also referred to the somewhat ill-informed notion that a raised crest impeded the passage of silt, thereby causing silting of the bed. People who alleged this condition usually failed to prove it by reliable data of the hydraulic and other conditions both previous to and subsequent to the construction of the crest. Actually, any observed silting was usually caused by the unintentional flattening of the surface-slope, due to faulty design.

He fully endorsed Mr. Lindley's advocacy of the conversion of falls into meters. The cost was slight, and he had found that, especially on distributaries of a length of ten miles and over, the increased control over distribution afforded by the division of the channel into metered sections, was invaluable.

With regard to co-efficients, the bare fact that the co-efficient prescribed for the old Standard-notch changed with a sudden step of ten per cent. at a point dependent upon the judgment of the individual, was enough to condemn it; in any case the results seemed much less reliable than those obtained by using a broad-crested weir, where $Q = KBH^{\frac{3}{2}}$ and H included head due to velocity of approach. In practice it was best to use the form $Q=KBH^{n}$, where H was the crest-gauge reading and did not include velocity-head. K remained constant, and velocity of approach was accounted for by fixing the constant exponent n, from a logarithmic graph in which computed values of head due to velocity of approach at various gauges were plotted.

In the second line of p. 89, the figure 3·90 was a misprint for 3·09.

He agreed with those who held that departure from a broad-crested rectangular weir of standard proportions, involved conditions, which, if not actually uncertain for a given set of data, obviously tended to vary for differing data much more than in the case of the type proposed, which had a practically constant co-efficient over ordinary working ranges.

With regard to Mr. Crump's standard design and co-efficient, unfortunately Mr. Crump did not publish his results and views to the extent which was not only justified by their value, but which was morally incumbent upon persons gifted with an abnormal proportion of grey-matter. He had, however, told the speaker that the factor 2H for the breadth of crest and length of horizontal parallel-sided throat, was such as was found by experiment to contain, for all ordinary conditions, the point of contraflexure, at which parallelism occurs ; a 2H radius for approach curves, both in a horizontal, and in a vertical plane, was an empirical ratio sufficiently approximating to the 'bell-mouth' to ensure stream-line entry.

The plan of the Upper Chenab Canal Tail Meter was very similar to Mr. Lindley's Plate 1.

Another unpublished but important point was that Mr. Crump tested the 3·09 co-efficient on this meter by measuring the actual 'contracted' water-surface, at or near the point of contraflexure, which was also the locus of the 'two-thirds-depth,' and found that the discharge computed from the observed drop in water-surface and the contracted area, was practically equal to the full theoretical discharge.

Careful tests showed that the variation from the 3·09 co-efficient, was within one-half per cent. for the wide range of working gauges of 2·75 to 6·0 feet.

Mr. Lindley's figure of 1 in 15 for the slope of a glacis was over-generous. Actual tests showed the percentage recovery for different slopes to be :—

Abrupt	66 per cent.
1 in 5	80 per cent.
1 in 10	85 per cent.
1 in 15	88 per cent.

The value of gradually expanding sides, 'warped' from vertica to normal side-slopes, and of a special dissipator, for preventing wave-wash, had already been mentioned in detail.

Mr. W. P. Thompson said that the possibility of utilizing falls on canals and distributaries, as meters had occurred to many irrigation engineers, but they had not put the suggestion into practice except that a ledge at full supply level now formed part of the design of falls.

When the water level was up to this ledge it was possible to know as one passed down a distributary whether full supply conditions prevailed.

The channels, the correct discharges and water levels of which were most important were the distributaries, because the main channels were subject to regulation of supply and of water level during rotational working.

The method adopted, so far, for gauging the supply passing a fall was a gauge in the stream below the fall. This was satisfactory, for the purposes of the Irrigation Department, Punjab, so far—the gauge was read to one-tenth of a foot by the gauge-reading staff without difficulty, and they had been content with the percentage of error which such readings might involve in the indication of the discharge passing; neglecting varying conditions of bed silt, as outside argument.

There might come a time, however, when owing to contracts for supply of water at volumetric rates, it would be important to supply a quantity which would be say, exactly 21 cusecs; and when it would be important to know with exactitude the quantity passing in defect of this, to one cusec, or even less.

Then, undoubtedly, the fall calibrated to $\frac{1}{100}$th of a foot would become a necessity on distributary channels. On larger channels it had shown its value in the calibration of the supply passing at the tail of the Upper Jhelum and Upper Chenab Canals.

Another point, Mr. Lindley wrote "it is held by some that a raised crest impedes the passage of silt" and went on to say "Mr. Woods supported this view in a paper read before the Congress in 1916."

He thought the expression "impedes the passage of silt" was misleading. The effect of a raised crest might be a change in bed regime, but, the fresh regime established, no impedence to the passage of the silt seemed possible.

The Author appeared to be under the impression that as a result of the introduction in 1916 of a method of regulation, whereby the lower gate of the regulator at Faquirian was utilized as a crest, there was a diminution of the scour in the canal.

This was not so, and the scour appeared to be still progressing slowly.

A sheet is appended of the result of monthly observations since 1916.

Average Bed Level of M. L. Lower Jhelum Canal.

	Months.											
	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	January.	February.
9116 ..	695.75	696.05	696.42	96.65	97.05	97.55	96.44	96.30	96.43	96.05	95.65	96.75
1917 ..	95.34	95.98	95.66	95.89	96.13	96.28	96.20	96.28	95.58	96.20	95.65	96.78
1918 ..	95.62	95.80	96.47	97.41	96.92	96.24	96.91	96.08	95.79	95.86	95.57	95.58
1919 ..	95.47	95.77	96.12	96.42	97.05	97.30	96.74	95.96	95.65	95.67	95.36	94.78
1920 ..	95.18	95.02	95.43	95.73	95.09	95.38	95.22	95.15	95.09	94.77	95.22	95.26
1921 ..	94.83	94.72	94.93	95.06	94.74	95.16	94.94	95.35	95.03	94.66	95.22	94.85
1922 ..	94.52	94.43	94.54	94.74	95.04	94.57	94.86	94.75	94.21	94.33	94.56	94.21
1923 ..	93.86	93.54	94.33	94.87	94.87	95.14	94.69	94.29	94.74	94.29	94.23	94.32
1924 ..	93.55	94.33	94.58	94.84	95.19	95.31	95.94	95.43	96.25	94.76	94.08	93.64 (1925)

W. P. THOMPSON,
1-4-25.

SARDAR BAHADUR PRABH SINGH said that the design of the works below falls in his experience had a great deal to do with the amount of side scour. He had invariably noticed that where there were heavy side splays below, there was a tendency to considerable side scour, while in the case of falls in which there were no side splays below, the tendency to side scour was greatly lessened.

On page 83, the Author had quoted the opinions of certain engineers on the effect of raised crests on passage of silt. The speaker pointed out that this question was discussed at full length in his own paper No. 96 on Principles of Design for proportionate flumes at heads of channels read before the Congress at this session.

THE PRESIDENT said that it would appear as if irrigation engineers in the Punjab at the present days were coming back to the solution of the problem of destroying excess energy at drops in canals by means of rapids. A study of old canal records showed that originally in the older canals, this was the method adopted but later on proper falls were introduced. Rapids again were now coming back and probably at some time in the future these might again be superseded by falls. He considered that generally rapids were preferable to falls. He endorsed Sardar Prabh Singh's remarks as to straight exits below falls. On the old Sirhind Canal beautiful ogee curves were provided below falls and had silted up and straight pitching had been laid on the silt at some falls.

MR. D. J. MORRIS said there was one point of practical importance in the design of rapids of the standing wave type and that was the thickness of the floor of the glacis. Very often, there might be a pressure equal to 4 or 5 feet head of water, or even more, tending to lift this floor and it would be very costly to construct the floor to take this unbalanced pressure, and it was possible that this high cost might make the expense of the structure prohibitive.

MR. NICHOLSON said in connection with the remarks in the last paragraph of page 81 that on the Sirhind Canal some years ago an experimental fall was made at Rupar with sides of glass. In this case the greatest damage in the "cistern" took place when 4 inches of water passed over the crest, although experiments were made up to 18 over the crest.

MR. BURKITT said that the factors of stream levels and width of channel came into the design of rapids of the type mentioned by Mr. Morris. He agreed that for wide channels, the cost of strengthening the floor to take the uplift pressure might prove prohibitive, but where the channel was narrow, the cost would not be affected very much.

MR. F. F. HAIGH, said Mr. Colyer, suggested the use of a cistern designed to absorb the energy of the water by turning the stream over on top of itself.

On the Upper Bari Doab Canal, Main Line and Branches, there were many examples of both falls and rapids. The falls had cisterns designed on the old lines and there were two points wherein these were defective.

Firstly, they were very susceptible to damage by drift wood of which considerable quantities entered the canal at certain seasons—sleepers got caught in the cistern and were unable to get out. Some times, if not removed, they remained for two days and did considerable damage to the crest wall and bridge piers.

He had known a three-feet wide pier battered through in this way. It appeared to him that a fall designed as suggested by Mr.Colyer would suffer from the same disadvantage.

Secondly, repairs to the bed and the lower portion of the sides of the cistern could only be done after the cistern was dewatered. The cisterns were usually below spring water level and were by no means water-tight, and consequently the cost of dewatering was frequently four or five times the cost of the repairs.

On the other hand, a rapid was far more accessible.

PUNDIT MOOL CHAND SHARMA referring to the President's remarks with regard to falls versus rapids, suggested that it was preferable to stick to falls, and utilize the energy. Well falls were suitable for this purpose, and he considered that they could be constructed more cheaply for the same head and discharge conditions.

MR. B. DARLEY wrote that in the United Provinces, cisterns below falls had been given up everywhere, and it had been found that when canals were run with the usual supplies, there was sufficient water cushion to protect the floor below from any damage which might be caused by falling water. Supplementary weirs below high dams were being abandoned for the same reason. The usual practice was to give a supplementary weir where the main dam was more than 30 feet high. It had been found in practice that these cisterns below falls and weirs collected debris which caused more damage than if the cistern had not been there. A good example of this could be seen at the Chahia Dam in Benares State. This dam was founded on a sound sandstone base, the layers of which however being rather thin. The dam was 44 feet high and originally a supplementary weir 14 feet high was built below it. The river carried a certain number of trees when in flood and the first year after construction, these collected in the cistern formed by the supplementary weir and were seen to churn about, pounding the rock floor of the cistern. This action evidently continued until the thin layers were broken up, one after the other, and water admitted to a layer below the foundation level of the supplementary weir which then moved downstream in sections, still standing. The following season, a thin layer of cement concrete, one foot thick was laid over the downstream floor for a distance of 35 feet from the toe of the main dam. This had withstood the action of the water falling 44 feet without any cistern for the last 10 years and had given no trouble.

In regard to scour below falls, toe walls below falls and weirs, built on very light or sandy soils were to be avoided as a means of protecting the work from being undermined by scour downstream. The saturated sandy soil would flow under a toe wall with the scour hole below, leaving a cavity under the floor of the work, which would collapse. A flexible mattress of pitching laid on a slope as described by Mr. Lindley was safe and satisfactory as it indicated at once when the work was in danger.

Roughened pitching should be avoided as it only set up eddies which tended to increase the scour.

Notches had been abandoned for years in the United Provinces and raised crests substituted, since they caused much less action downstream. No appreciable silting had been noticed upstream due to water being held up above the normal depth when running low supplies. Such silting in any case, would only continue till a new "regime" channel had been formed; then, as supplies were again increased, this silt would be picked up once more and carried on. The writer had little to add concerning the "re-entrant" falls suggested by Mr. Hickey and about which the Author asked for further information. These falls appeared extraordinarily successful where first tried on a small anal just below Naini Tal which carried enormous quantities of very coarse sand. When tried elsewhere on an ordinary irrigation channel which carried very little silt, they gave results no better than the raised crest fall. Another type designed by Mr. Hickey on the same principle for falls in larger channels was next tried with the same result, *viz.*, it was more expensive and no better than the raised crest fall. The raised crest fall was therefore being adopted everywhere on the Sarda Canal where there were a large number of falls on big channels, one or more gaps 4 to 5 feet wide were being left in the crest. These would always be planked up before the canal was run and could be opened when the canal was closed, to quickly unwater the bed upstream.

On similar canals and channels where falls were far apart, pipes through the crest at upstream bed level drew off all water at once when the canal was closed. Mr. Lindley's paper was most interesting and the writer thought that on the Sarda Canal they should be able to avoid the construction of several expensive discharge sites by calibrating falls near at hand and thus obtain more accurate results than those usually obtained.

Mr. Colyer replied to the discussion on behalf of the Author and said that the time for accuracy in gauging the discharge of channels, mentioned by Mr. Thompson as 'coming' had already come. The chief disease to eliminate was "Hydraulic Appendicitis" or Tail Trouble. To Sardar Bahadur Prabh Singh he said that silting might occur above a raised crest if the channel did not run at 'full supply,' but modern practice was to run a distributary at 'full supply,' or close it, and a weir meter helped to ensure that full supply was being run.

Regarding the President's remarks, two rates of expansion or divergence proposed were not ideal, as they were limited by considerations of economy of first cost.

Mr. Nicholson had mentioned the raising of the maximum velocity of the stream line. The intention of the biff-wall was more, the reversal of direction, and the conversion of energy into heat.

To Mr. Burkitt he said it was not always necessary to keep the supply depth proportional to the discharge, but that in a flume weir, a depression of 0'9 D gave proportionality. He thought Mr. Haigh's difficulty about the sleepers would not arise, if the cisterns were not made too deep, and the biff-wall made about half the depth of the normal channel.

Pandit Mool Chand Sharma had raised the question of cost. It had been shown that for most ordinary canal falls, the "rapid" type was economical.

It was very interesting to have the point of view of engineers of other Provinces. The conditions mentioned by Mr. Darley were very different from the usual Punjab canal conditions, The purpose of the "dissipator," *i.e.*, of the cistern and biff-wall together was not to act as a water cushion but as a deflector to turn the current back upon itself.

In the Punjab there was usually no difficulty owing to trees floating in the canals.

He agreed that toe walls were not good and that it was best to use a sloping glacis.

SUBSEQUENTLY COMMUNICATED.

Mr. C. C. Inglis wrote that in the Bombay Deccan, falls were rarely necessary on main canals, which flowed on a falling contour of about one foot per mile along the sides of the river valleys.

Distributaries, on the other hand, flowed from the canals towards the rivers on subsidiary ridges and the gradient was often of the order of 1 in 100 or even steeper.

Consequently, falls were mostly required for distributaries carrying less than 100 cusecs.

The design hitherto adopted consisted of a notched fall with a vertical drop into a stilling pond downstream, the sidewalls acting as buttresses to the notch wall and also as side retaining walls. The cost of this design was very considerable and the falls were liable to develop leaks or even to be outflanked owing to the subsoil water gradient being of the order of 1 in 3.

The objects aimed at in the new standing wave flume fall design were :—

(1) Reduction of first cost.

(2) Prevention of leaks and breaches, *i.e.*, reduction of maintenance, etc.

(3) accurate measurement of discharges.

(4) Standardization of correct upstream water levels to prevent upstream scour or afflux.

The new design consisted of a Crump type (long throated) standing wave flume which discharged into a flume of the same width as the throat, sloping at 45° to the vertical, which, in turn discharged into a horizontal flume at the bottom of the fall. This expanded to the full bed-width the slope of the expansion depending on the degree of stilling required. The 45° slope was joined to the horizontal bed upstream and downstream by curves of 3 feet radius.

In this design, the contraction was mostly made at the sides to reduce the dimensions and hence the cost of the sloping flume. Special care was taken to insure the formation of a standing wave in the expanding flume under all conditions of flow, a small hump being provided at the downstream end of the expanding flume.

This hump was placed six times the maximum height of jump below the toe of the fall and the hump was so designed that the velocity of the water as it went over the hump was equal to the critical velocity. The primary object of this was to insure the standing wave, but on theoretical ground it should also steady the flow and in the event of scour downstream of the hump, the standing wave would still remain. In the writer's tests, the best results were attained when the water over the hump had just got free overfall into the channel below the hump; but he had not, as yet carried these tests so far to be able to give useful data on this point, and, for Deccan conditions, it was not very important, as the strata below falls was nearly always fairly hard. The writer had heard that Mr. Colyer had been working on the same lines, and as complete stilling was very much more important under Punjab conditions, he expected this question had been tested in much greater detail by the latter.

The cost varied from a little less than, to about 2-3rd that of the old design, according to conditions; and the subsoil water gradient round the fall was reduced to a safe limit for the normal height of fall. In addition, accurate measurements and standardisation of water level were obtained.

THE AUTHOR wrote that the paper attempted to deal with the subject constructively, principally on two points; the metering of supplies and prevention from damage by wave-wash. The first point seemed now to have found acceptance in practice; on the second, the discussion elicited useful information.

One of the more essential features of both Mr. Colyer's and Mr. Inglis' forms of fall was the submerged weir at the exit. An investigation carried out by the Miami Conservancy Board, intended to find the best method of dispersing the energy of considerable discharges issuing from culverts of dams that formed flood-delaying reservoirs, under high head, found the standing wave the best means: and it also found a similar submerged weir necessary to prevent concentration of flow after the wave. (Miami Reports, Part III.) Some of the early meter-falls of the Punjab were made much narrower than the channel and gave trouble through such concentration of flow: this narrowing for the sake of cheapness was possible if a submerged weir were provided, as could be realised from the Miami designs. Mr. Quizumbing of the Phillipine Irrigation Service had also sent the Author a design due to Engineer Louzada in the Dutch East Indies, in which the submerged weir was again a feature.

The effectiveness of this submerged weir could probably be explained theoretically. Osborne Reynolds had pointed out that converging flow was one of the conditions favourable to stream-line as opposed to turbulent flow: the accelerated flow over the weir was converging in this sense and tended to draw out the eddies present in the standing wave, and thus to screen out the turbulence.

The Author had planned experiments to test Mr. Crump's theory of flow over a "narrow-crest" weir: he had only been able to carry out some tests on a laboratory scale. The results were presented in the following statement and diagram.

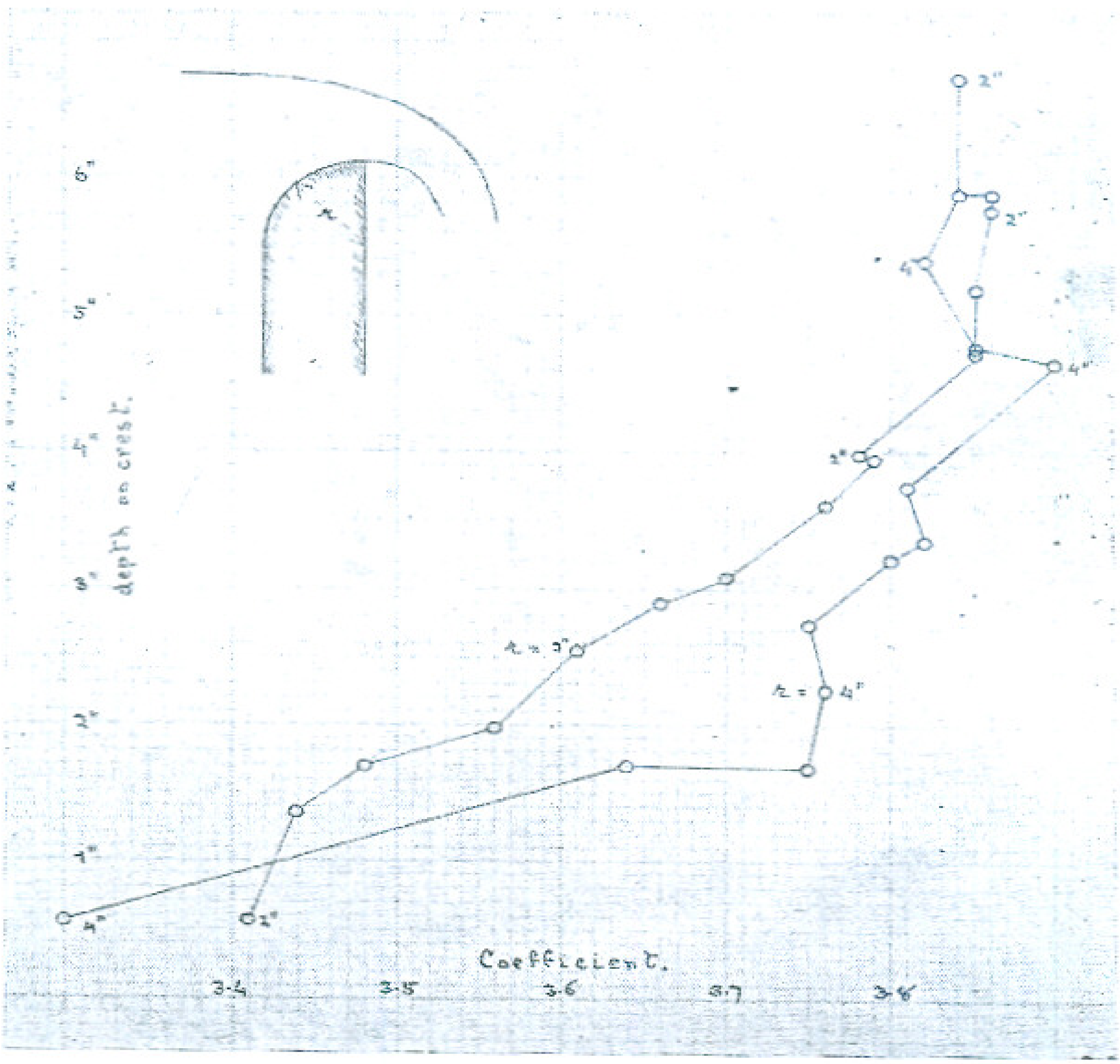
The section of weir tested was shown, tests being made with weirs of 2″ and 4″ radius: the test weirs were 1 feet long with side contractions suppressed by curved wings, set in a flume 4 feet wide: the weir crest was only 8″ above the flume floor, but as bottom contraction was suppressed and velocities of approach were small, this should not matter: the issuing nappe was aerated underneath, and never clung to the downstream face of the weir. The discharge for upwards of two minutes was measured in a carefully calibrated tank, the rate being checked by Venturi meter: the depth on crest was measured with a hook-gauge, and the small head due to velocity of approach had been added to it before calculating the coefficient.

It was to be seen that the coefficient rose with head, and steadied down to a maximum value of about 3·85: this was only 0·92 of the theoretical 4·184, while the theoretical 3·09 of the broad-crest weir was fully realised in practice. It might be observed that the very complete collection of data of weir tests given by Robert E. Horton in U. S. Geological Survey (Water-supply and Irrigation) Paper 200, seemed to indicate a maximum possible weir coefficient of about 3·9, when cases in which the coefficient covered velocity of approach, or probable adherence to a steep downstream glacis increased discharge, were excluded.

There was expectation that the co-efficient would rise to the theoretical value at and above depths of 1·46 times the crest radius, that being the ratio at which the radius fitted the vortex of maximum discharge; but neither weir reached its maximum at that depth, and the discharges of the 4″ weir were greater than those of the 2″ instead of less. So such indication as these few observations afforded was against the theory.

Acknowledgments were due to Professor Inglis and the Cambridge University Engineering School, for facilities and apparatus for these tests: and to Mr. Mallock for help in staging them, and actually making some of them.

Mr. Colyer and Mr. Inglis were good enough to send the Author copies of their remarks: he was not present at the discussion on the paper and had still not seen the report of it: he was therefore not in a position to refer to any more of the discussion, or even to acknowledge other contributions that he had heard were valuable.


depth on crest.
Coefficient.
3·4
3·5
3·6
3·7
3·8

2″				4″			
d″	Q	Ha	C	d″	Q	Ha	C
0·565	0·035	0·000	3·41	1·68	0·196	0·00	3·75
1·345	0·130	0·000	3·44	2·25	0·305	0·00	3·76
1·695	0·185	0·000	3·48	3·21	0·525	0·00	3·80
1·990	0·242	0·000	3·56	4·64	0·940	0·01	3·90
2·550	0·356	0·005	3·61	5·74	1·285	0·02	3·86
2·900	0·438	0·005	3·66	0·56	0·033	0·00	3·30
3·080	0·483	0·005	3·70	1·70	0·194	0·00	3·64
3·595	0·616	0·005	3·76	2·73	0·407	0·00	3·75
3·940	0·714	0·005	3·79	3·35	0·564	0·00	3·82
3·990	0·728	0·010	3·78	3·73	0·663	0·01	3·81
4·715	0·950	0·010	3·85	4·75	0·961	0·01	3·85
5·185	1·098	0·015	3·85	5·39	1·157	0·01	3·82
5·885	1·332	0·015	3·86	5·89	1·330	0·02	3·84
5·89	1·329	0·02	3·84				
6·71	1·620	0·02	3·84				

PLATE № 1

DISSIPATOR

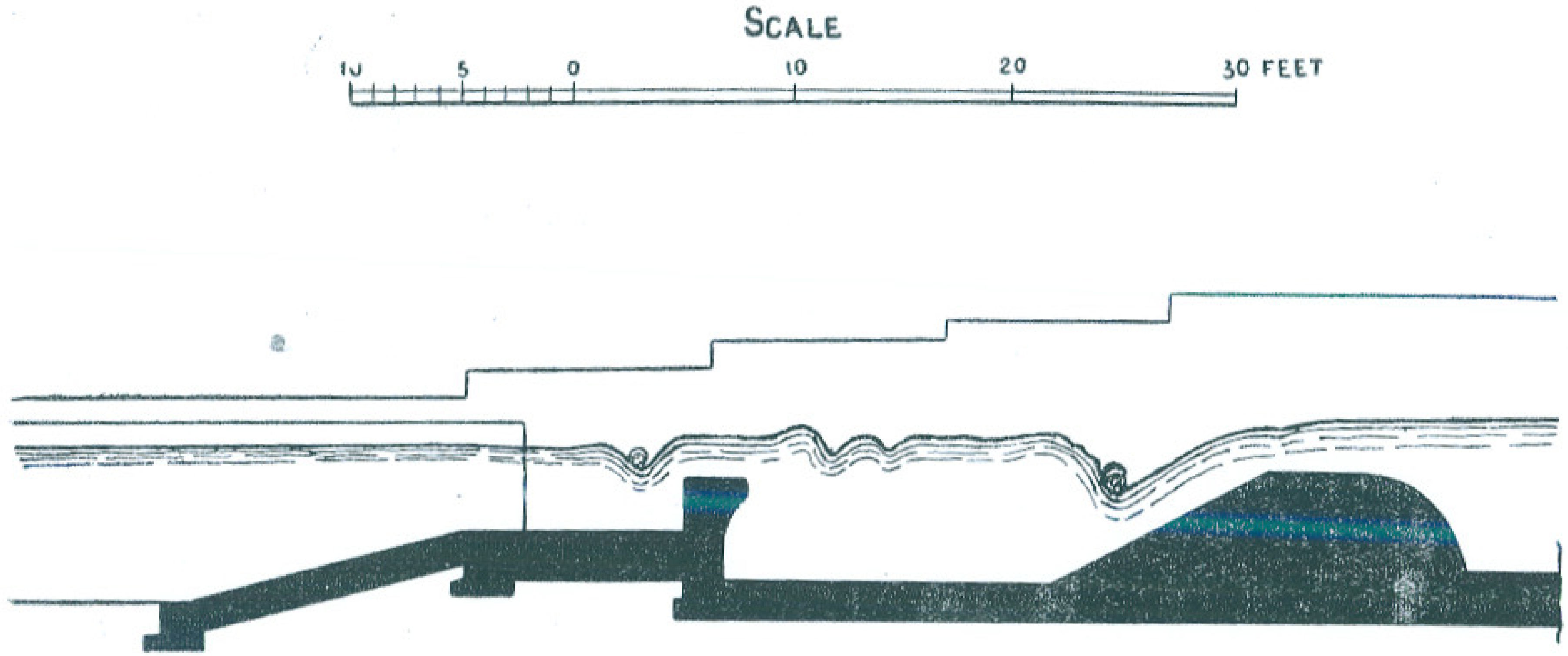


PLATE № 2

DETAIL OF DISSIPATOR

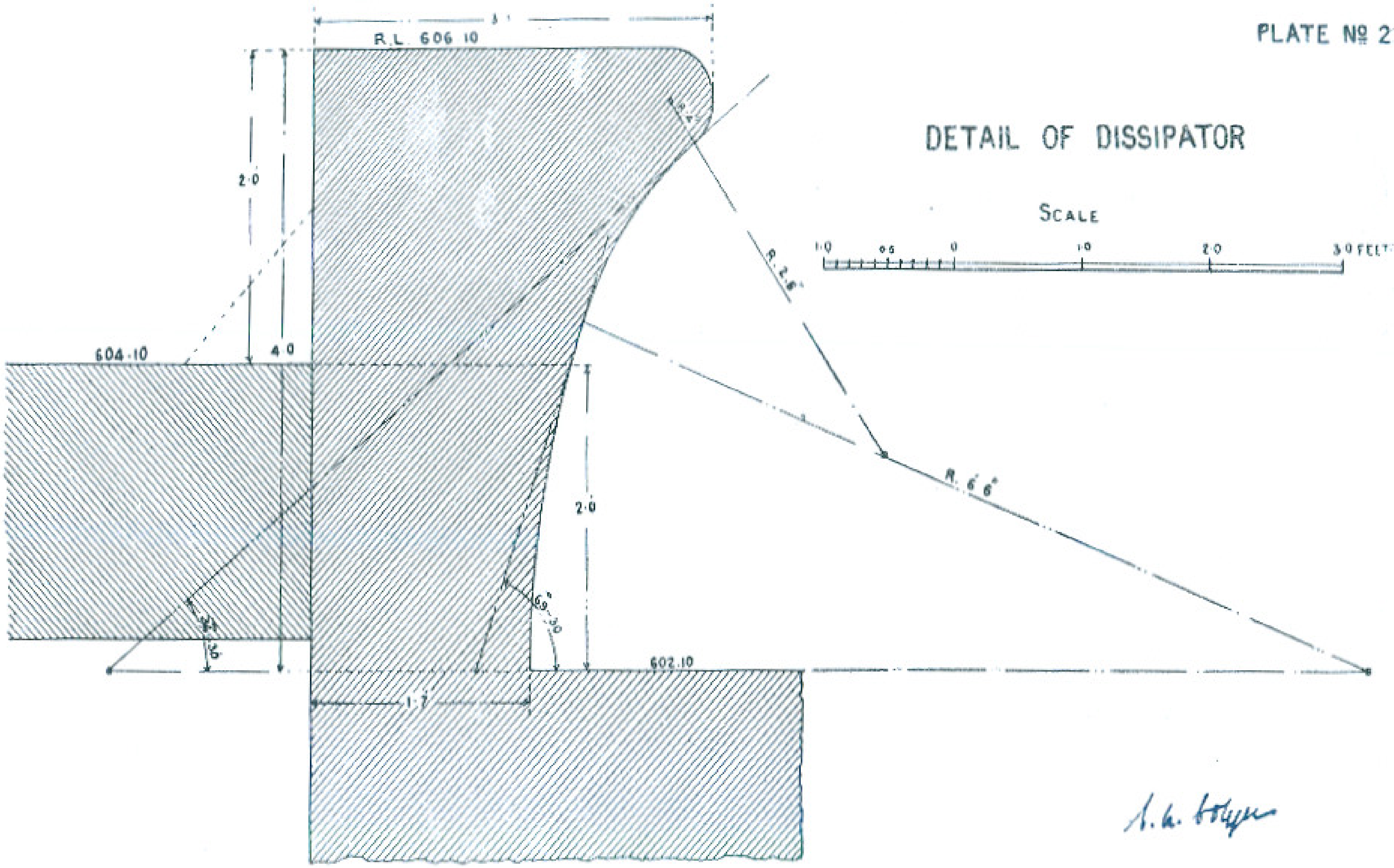


PLATE No. 3.

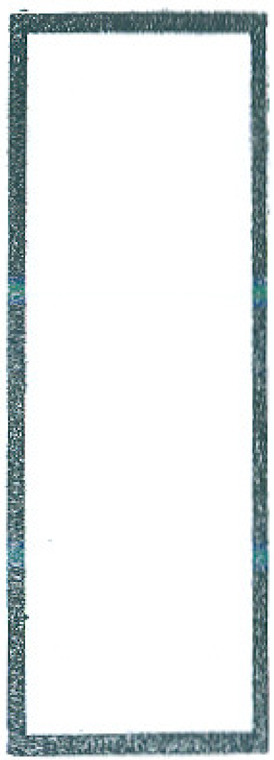


FIG. A.

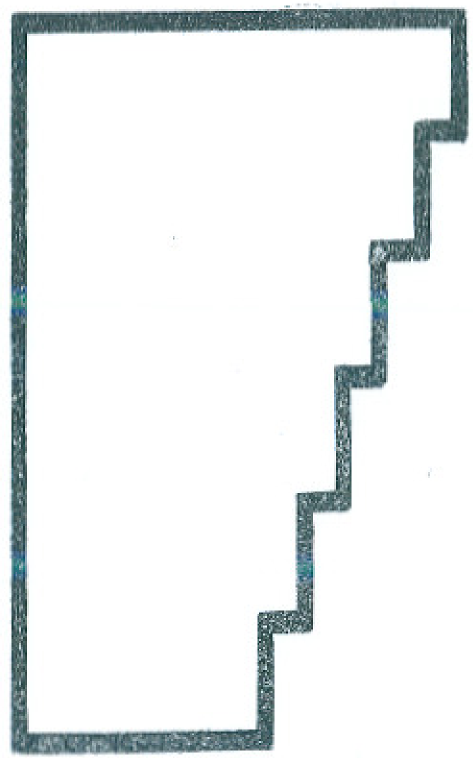


FIG. B.

FIG. C.