Paper No. 405

ENERGY DISSIPATION AND EXPANSION OF FLOW IN SHARP DIVERGENCE BELOW A FLUMED FALL

By
S. M. AYOOB AND Ch. MOHAMMAD ALI

A fall is a structure in a canal built to secure the lowering of the water surface of the canal. The energy so liberated has to be dissipated and the subsequent turbulence stilled within the confines of the structure before the water passes into the lower level channel as otherwise the structure will not be safe against tail erosion. The canal fall is sometimes designed with a contracted width and is known as a flumed fall which can economically be built together with bridge and used as a meter or regulator as well.

Before the first Great War 1914-1918 free falls in the form of Notch falls or long crested weirs were in vogue in this sub-continent and the distance between the abutments was almost the same as the width of the canal. Diverging flared out wing walls u/s and d/s of the structure were not required. The discharge per foot run was less and consequently less energy had to be dissipated per foot width of the d/s cistern.

After the first World War, to afford economy new canal falls on Sutlej Valley Project were flumed to the utmost practical extent of 30 to 40% of canal width. The rate of expansions of side walls which was generally 1 to 5 was found to be too abrupt to secure uniform distribution of flow at the standing wave and lower down. The resultant scouring action was very serious. Subsequent maintenance cost outweighed the initial economies resulting from the fluming of the falls. As a result of this experience the designers on the Sukkur Project put in more thought and research to guide them for designing the flumed falls.

Mr. C. C. Inglis, later Sir Claude Inglis, brought out a technical paper (44) in 1933 on the dissipation of energy below falls which is based on the model experiments in respect of Tando Masti Fall on Rohri Canal. The success of the devices recommended by (Sir) C. C. Inglis (1) rehabilitated the flumed fall designs which had suffered a serious set-back after experience on the S.V.P. falls. The main features are shown in Fig. 1 (a, b). The cardinal points of the design are :—

(*a*) **Minimum permissible divergences of the expansion d/s of the fall.**—The divergences recommended for various ratios of B/D (where B is bed width and

D depth of channel d/s of the fall) are given in the table below :—

TABLE I

Minimum permissible side divergence for normal channel	B/D	Equivalent F.S.Q. of channels (Cs.)
1 in 4	2.6	3
1 in 5	3.4	30
1 in 6	4.5	220
1 in 7	5.9	700
1 in 8	8.0	2,400
1 in 9	11.0	6,000
1 in 10	20.0	10,500

(*b*) A baffle wall of requisite height located on the d/s floor perpendicular to line of flow at a correct level (required for the formation of jump at the toe of glacis in the absence of baffle wall) whose function is to deflect the stream flowing down the glacis with hyper-critical velocity so that it is flowing horizontally at the toe of standing wave.

(*c*) **Cistern.**—A long cistern to contain standing wave and to comb out all eddies and turbulence d/s of the standing wave.

(*d*) A deflector built at the end of the d/s floor to deflect any residual high velocity flows towards the surface and generates horizontal low velocity roller d/s of the deflector, which assures that the scour shall take place at safe distance below the masonry work.

Sir Claude Inglis also recommended wave suppressor to smooth out surface pulsations, but these recommendations were not carried out.

Mr. Lacey[2] connected the angle of divergence with the discharge as :

$$\text{Angle of divergence} = \tan^{-1} \frac{0.4}{Q^{1/16}}$$

The angle of divergence for a particular discharge given by Lacey and Inglis is plotted in Fig. 2, which indicates that Lacey recommends lesser divergence angle than Inglis for discharges above 540 cusecs and greater angle for lower discharges.

Both from experimental and analytical studies Rouse & Boota and HSU[3] have obtained most satisfactory boundary form for an efficient expansion in super-critical flow, which is a function of the Froude No., of the depth of flow in the approach channel and of normal channel width which will be discussed later on.

The U.S. Soil[4] conservative service has also made tests on super-critical flow through expansions in conjunction with Saint Anthony Falls stilling basin with straight flaring side walls with the major finding that the maximum permissible side wall divergence is $1 : 3F_1$, if back eddy and cross-waves of excessive height are to be avoided.

The present paper deals mostly with remedial measures to some of the existing structures contracted to different fluming ratios and subject to varying submergences which were misbehaving originally due to sharp divergence d/s of the fall, and which were experimented upon in Irrigation Research Institute to find out economical and practical remedial measures.

Formation of Return Eddies and their cause d/s of Flumed Structures

If the flow does not follow the side walls of an open channel expansion which are sharply diverging out as a consequence of boundary layer growth, the large return eddy with vertical axis forms in the wake of separation of flow from the side walls and the flow concentrates in a narrow width (called jetting flow) which follows the other side of expansion either of which has disastrous action in the form of scour of banks and bed of channel d/s.

Fig. 3 refers to prototype flow conditions of the Panjnad Canal fall at R.D. 1,50,000 (which is flumed to 66%, d/s side divergence is 1/3 and the discharge per foot run is 60 cusecs) which shows that stream filaments get separated from the sides just near the toe of stepped glacis. The back roller on the left bank is much larger in extent than that on the right bank with the result that the jetting flow was confined to a narrow width (resulting in discharge per ft. run more than 100 cusecs) which was set against the right bank. The scour below the structure was 40 ft. as was evident from the contours of the bed. Due to conversion of velocity head to pressure head, the water level at 'B' is higher than water level at 'A', leading to reverse gradient from "B to A". The forward momentum of flow at "A" due to reduced velocity along left side wall is not sufficient to out-balance the adverse pressure gradient. The turbulence along inter-face of forward flow and back flow is not sufficient to transfer (by turbulent inter-mixing and viscous drag) forward momentum to the back-eddy area to overcome the adverse pressure gradient. If the turbulent intermixing is sufficient to overcome the adverse pressure along the interface, the interface

boundary will shift towards the side and *vice versa*. Surging occurs where the high velocity jet is not equally balanced by the d/s water depth and thus there is unsteady and heaving surface which results in lateral movements of the jetting flow and also results in a wave wash.

A hydraulic jump will form in the horizontal rectangular channel if the Froude number F_1 of the flow, the flow depth y1, and downstream depth y2 satisfy the equation:

$$\frac{y2}{y1} = \tfrac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

Hydraulic jumps on horizontal floor are of several distinct types. According to the studies of the U.S. Bureau of Reclamation, these types can be conveniently classified according to the Froude number F_1 of the incoming flow, as follows :—

For F_1=1, the flow is critical, and hence no jump can form.

For F_1=1 to 1.7, the water surface shows undulations, and the jump is called an undular jump.

For F_1=1.7 to 2.5, a series of small rollers develops on the surface of the jump and turbulent mixing is in top layers up to depth depending on F_1. The velocity throughout is fairly uniform, and the energy loss is low. This jump may be called a weak jump. The Froude number below the low drop structures in West Pakistan lies in this range.

For F_1=2.5 to 4.5, there is an oscillating jet entering the jump bottom to surface and back again—with no periodicity. Each oscillation produces a large wave of irregular period which, very commonly in canals, can travel for miles doing unlimited damage to earthen banks and ripraps. This jump may be called an oscillating jump. The Froude number below most of the large falls in West Pakistan lies in this range.

For F_1=4.5 to 9.0, the downstream extremity of the surface roller and the point at which the high velocity jet tends to leave the flow occur at practically the same vertical sections. The action and position of this jump are least sensitive to variation in tailwater depth. The jump is well-balanced and the performance is at its best. The energy dissipation ranges from 45 to 70%. This jump may be called a steady jump.

For F_1=9.0 and above, the high velocity jet grabs intermittent slugs of water rolling down the front face of the jump, generating waves downstream,

and a rough surface can prevail. The jump action is rough but effective since the energy dissipation may reach 85%. This jump may be called a strong jump.

It should be noted that the ranges of the Froude number given above for the various types of jump are not clear-cut but overlap to a certain extent depending on local conditions and are likely to be modified if the standing wave is partially located on the glacis.

The percentage energy loss in the jump is equal to the difference in specific energies before and after the jump and is a function of the Froude number. The energy losses in jump (as determined from the Laboratory Flume) for different Froude numbers is plotted in Fig. 4, which shows that percentage loss of energy in the jump increases with Froude number and is low at low Froude number. Impact blocks increase the percentage loss of energy at lower Froude numbers.

The authors have worked out an alternative method of determining the conjugate depths of hydraulic jump if per foot run discharge, q, is known, u/s floor level, u/s water level and cill level of the hydraulic structure are given and the d/s water level is also estimated. The method is given in Appendix I. The perusal of Appendix I curves will reveal that a jump will form for a particular value of q if the difference between u/s energy line and d/s water level is more than the minimum value given by the curves between different parameters of Appendix I. If the set of conditions for formation of a proper jump are not satisfied the turbulence of jump will not be intense or will not be up to full depth of flow and transfer of momentum in the vertical or horizontal will be limited.

It has been stated in the Publication No. 10 of C.B.I. (Irrigation Canal Falls) that in case of small drops, puzzling scour takes place. This is simply a result of failure to destroy surplus energy under conditions where the standing wave is unlikely to form. In case of low drop structures, the flow does not follow the glacis and an eddy with horizontal axis appears along the bed which makes the flow unstable and unbalanced and does not follow the expansion. In such cases the roughening of bed by blocks is not effective as the blocks do not project into the forward flow.

The bed and side scour have been observed on some of the bridges along L.J.C. and L.B.D.C. It was due to low hump in the bed of the canal which resulted in ground rollers which lifted the flow from the bed and made the flow unstable. Downstream of the roller the flow was either (1) diving and converging resulting in heavy scour in the bed or (2) directed against one of the banks resulting in heavy side action. The stone hump in bed of Ravi river to protect the foundations of piers of old Kot Abdul Hakim bridge resulted in heavy scour in bed of river d/s which resulted from the strong ground roller generated at the stone hump protruding into the flow.

FLUMING RATIO FOR PROPORTIONAL DISTRIBUTION OF DISCHARGE

When large canals are run for long periods at low supplies, the bed u/s of fall shows a tendency to silt which is subsequently picked up at full supplies and deposited lower down and this in turn induces meander in the canal. The hydraulic ideal would be satisfied if the water level over the crest of fall would vary precisely in the same manner as in the canal it contracts.

If the limits of the range of the variation of the water depth in the canal be D_1 & D_2 and H_1 & H_2 be the corresponding difference of canal surface water level (beyond the draw-down range of fall) and the cill level, for practical purposes it will suffice if the range of proportional variations is from full supply depth to the two-third of it

$$\frac{Q_1}{Q_2}=\frac{K D_1^{5/3}}{K D_2^{5/3}}=\frac{CXBXH_1^{3/2}}{CXBXH_2^{3/2}}=1.965$$

where B is the contracted width of canal.

or

$$\frac{H_1}{H_2}=(1.965)^{2/3}=1.57$$

$$H_1-H_2=D_1-D_2=D_1/3$$

or

$$H_1-\frac{H_1}{1.57}=D_1/3$$

or

$$H_1=.9177\ D_1$$

Simons and Albertson[6], of Colorado State University, collected and analysed data from some 70 reaches of operating canals (of capacity 5—9000 cusecs) in Punjab & Sind Irrigation System.

The channel parameters from Analysis of Canals with cohesive banks and sand bed as given by Simons and Albertson are depth of flow in channel

$$=D=1.15\times.43\times Q^{.361}$$

Average width of channel $= W=.88\times2.51\times Q^{.512}$

For proportional distribution of discharge the value H is $.9177\times D$.

$$Q=BX3.09\times(.9177D)^{3/2}\text{ for broad crested fall.}$$

$$=2.638\times B\times D^{3/2}$$

Clear waterway at crest of fall $= B = \dfrac{Q}{2.638 \times D^{3/2}}$

$$= \frac{Q}{2.716 \times (1.15 \times .43 \times Q^{.361})^{3/2}}$$

$$= 1.059 \times Q^{.4585}$$

Now $W = .88 \times 2.51 \times Q^{.512}$

$$= 2.209 \times Q^{.512}$$

Fluming ratio of clear waterway $= \dfrac{B}{W} \times 100 = \dfrac{100 \times 1.059}{2.209} \times \dfrac{Q^{.4585}}{Q^{.512}} = \dfrac{47.95}{Q^{.0535}}$

If Q is 100, 1000, 5000, 10,000, 15,000, 20,000 cusecs, the theoretical fluming ratio works out to be 37.46, 33.12, 30.09, 29.28, 28.72 and 28.22 respectively.

GEOMETRY OF EXPANDING SIDE WALLS (NEW STRUCTURES)

Channel expansion in super-critical flow d/s of flumed falls is very obvious. If such an expansion is made to diverge too rapidly, the flow will fail to follow the side boundaries and the separation surface between forward flow and back; eddy will deviate away from the flank wall causing regions of negative or zero velocities along the side wall. These conditions result in convergence of flow together with excessive concentration which usually causes dangerous pits to develop which if not checked could often cause a total collapse of the structure. Proper design for the expansion is, therefore, of high importance. The best way to study any particular channel expansion is by cut-and-try investigation through model tests.

The appropriate divergence proposed by C.C. Inglis and Lacey for different discharges as determined from model experiments is given in Fig. 2. The maximum permissible side wall divergence according to tests of U.S. Soil Conservative Service is $1 : 3 F_1$. The side divergence given by these methods however, is not economical for other than small structures.

From both experimental and analytical studies Rouse, Bhoota and HSU have obtained the following equation for satisfactory boundary form for an efficient expansion :

$$\frac{y}{b} = \frac{1}{2} \left(\frac{x}{b.F_1} \right)^{3/2} + 1/2$$

where b is channel width, x is longitudinal coordinate (from beginning of expansion) and y is lateral coordinate measured from centre line of channel. The boundary represented by the above equation will diverge indefinitely but from economic and hydraulic jump, consideration may be limited to a point beyond 1/2 length of jump and battered in the d/s direction in a slope of 1 : 2 or may be followed by a reverse curvature as shown in Fig. 5.

It may be stressed that the beginning of the expansion should be sufficiently gradual but rate of expansion can be increased beyond half the length of hydraulic jump as is given out by Mr. Hunter Rouse in his Paper No. 3006 on "Turbulence Characteristics of the Hydraulic Jump" submitted to the American Society of Civil Engineers in February 1958 that "In view of the customary practice of considering the length of the jump to be the distance between the limiting sections of uniform flow, it should be noted that the roller extends about one half this distance, and that one half the energy of the turbulence is produced within the first half length of the roller because of both the convective effect of the mean flow and the diffusive effect of the turbulence itself, this energy is not wholly dissipated at the point that it is produced but rather some distance away both vertically and longitudinally. In other words, there is a distinct lag between the loss of energy to the mean flow in the form of turbulence and its ultimate dissipation in the form of heat".

Turbulence in the jump was measured in the Hydraulic Laboratory of Irrigation Research Institute by means of strain gauge and is plotted in Fig. 5(a) which shows that the turbulence in the vertical becomes almost uniform after the middle of the jump.

It was evident from a number of studies conducted in the Irrigation Research Institute that the divergence if constructed along a parabolic curve (with asymptote in line of abutment) starting in between the front of jump on the glacis and the toe of glacis, the flow follows the expansion if the coefficient in the parabola equation ($y=CX^2$) is properly selected keeping in view the Froude No. and the fluming ratio.

The fall at R.D. 45,500 of Begari Sind Feeder which is a flumed fall with fluming ratio of 80%, with a drop of about 10.0′, discharge per foot run of about 94 cusecs and $F_i=2.7$ was tested on the model. The experiments showed that to spread out and to attain an even distribution of flow at the end of the structure the equation of side wall parabola is

$$y=\frac{1}{6}x^2 \qquad \text{(Fig. 6)}$$

and the flare ratio of side walls is 1 : 4.

The vortex silt ejector of D. G. Khan Canal (Q=14720 cft, B=300 ft. D=12.99) was constructed in 2/3rd width of the canal (fluming ratio of 66%) and the model tests showed that a parabola fitted in a flare of 1 : 5 was appropriate and had an equation of

$$y=\frac{1}{10}\cdot x^2 \qquad \text{(Fig. 7)}$$

This tends to indicate that the most appropriate divergence is about 1 : 5 with flared out walls constructed to follow a parabola. Subsequent studies showed that the length of parabola at the d/s end can be continued beyond the mid-point of d/s cistern and then sloped down at a top slope of 1 : 2 but in this case the pitched bank of canal will be continued behind the parabola wall and curved round to join the back side of the parabola wall. See Fig. 31 (*a*).

The proposed Kalri Lake Escape regulator (Fig. 8) was designed for a capacity of 10,000 cusecs and a maximum drop of 34′ which could occur at site when the river Indus would be almost dry. Even though the fluming was 48% and divergence of 1 : 2.5 was adopted, the model showed that the side eddy was extinct if the d/s divergence was parabolic and a baffle wall of suitable height at optimum placement and distance from the toe of the glacis was provided to permit the formation of an additional jump u/s of the baffle wall.

It was noticed in case of subsidiary weir d/s of Islam Headworks that even if there is no contraction at the crest and the side walls are sloping an eddy appears due to break in side slope at the toe: Fig. 9 (*a*, *b*). The eddy was eliminated by means of a short divide wall d/s of the crest at the toe of the wing wall : Fig. 9(*c*).

BAFFLE WALL TO REMOVE RETURN EDDY IN SHARP EXPANSION

The baffle stabiliser consists of a raised step built transversely across the line of flow on the downstream floor. The functions of the baffle wall are :—

(*a*) Owing to the impact of the flow on the baffle, pressure is created in the upstream direction which varies as $V^2/2g$ and chances of separation of flow from the diverging outside wall get minimised due to higher water level generated at and above the baffle wall. Thus the baffle stabiliser induces the flow to follow the expansion.

(*b*) To cause converging flow u/s which has a stabilising influence on the expanding flow.

(*c*) To encourage energy dissipation by impact of flow on the wall.

(*d*) To deflect the stream flowing down the glacis with a hyper-critical velocity so that it is flowing horizontally at the instant at which impact occurs in the standing wave.

The baffle wall was tried in the laboratory for its efficiency in fanning out the otherwise converging and jetting flow under sub-critical and hypercritical flow conditions with Fr. No. less than 4.5.

I. Expansion of flow by means of deflector wall in sub-critical flow

The specific example is that of expansion of flow at the exit end of L.B.D.C. aqueduct over the S. M. Link Canal (Fig. 10) which was designed by Messrs Tipton and Kalambach for a maximum discharge of 1496 cusecs. The aqueduct is 22 ft. wide and the flow leaving the aqueduct at Fr.=0.53 was supposed to expand from a bed width of 22 feet to a bed width of 48 feet in a length of 45 feet in a straight expansion with the divergence ratio of 1 : 3 and then follow the flank walls abruptly rounded off to join the banks of the 80 feet wide d/s channel. The original test on the model showed that the jet emerging out from the aqueduct played from side to side with back circulation on either side which resulted in a deep scour hole and heavy side erosion (Fig. 11a). A series of tests showed that circular baffle of height equal to D/2 (where D is depth of water in the absence of baffle) and radius of 52 feet at a distance of 20 ft., from the exit end of the aqueduct was efficient to disperse out flow to the full section all along the expansion. The radius of the baffle wall was approximately equal to the 2/3rd width of channel and was adjusted slightly so that the baffle wall meets the wing walls at right angle. The maximum scour (on the model) at the end of test with circular baffle wall was only 10% of the scour in the base test: Fig. 11(a, b).

II. Circular baffle wall for fanning out of flow in sharp expansion in supercritical flow at Froude No. 1.7-2.5.

Aurangabad Syphon at R.D. 145350 of U.J.C. (Fig. 12a) was originally designed for 10,000 cusecs. This syphon was washed away in 1960 at a discharge of Q 16,900 cusecs and it was reconsturcted after model tests. The Fr. No. at the exit end of the redesigned syphon consisting of 5 No. barrels (12 × 12) was about 2.0 and the flow had to be expanded from 66 feet at the exit end of barrels to 260 feet in an economical expansion with flare of 1 : 2. The expansion was aligned circular with gradual splay from vertical to a slope of 1/1. A series of tests were conducted to fan out the flow to follow the expansion on a model and it was recommended :—

(*a*) To lay the d/s floor at an optimum level which was 1.1 times D_2 below the water level generated at the maximum discharge and of length equal to 150 feet.

(*b*) To construct a baffle wall of radius 100 feet with a height equal to $D_2/2$ placed 55 feet from the end of glacis.

(*c*) To provide 2 No. apertures of size 3′ × 2′ at the bottom of baffle wall along the flanks to increase the flow along the flank walls and to dewater the cistern above the baffle wall for any subsequent inspection.

(*d*) To align 2-rows of blocks of height $D_1/5$ on an arc of a circle to act as impact blocks for the standing wave which would form downstream of baffle wall.

(*e*) To provide the deflector blocks at the end of the structure to lift the high velocity flow along the bed towards the surface.

The bed contours at the end of the test (Fig. 12b) show almost identical scour all along the tail and wall of the structure. The velocities are plotted in Fig. 12(c) which shows normal velocity distribution below the baffle wall. It was observed that proper standing waves formed u/s as well as d/s of baffle wall, Fig. 12(d).

The Irrigation Research Institute have noticed that the formation of standing wave in an expansion is one of the most effective means of spreading out the flow, by transfer of forward momentum of flow along sharp divergence through intense turbulence of standing wave.

III. Baffle wall in expansion of flow where the bed width is reducing and the side walls are wrapped.

The transition between the fall at R.D. 29000 of Munda Branch Fig. 13(*a*) and the lined section of canal is such that the expansion at the wall surface being sharper than at the bed, the flow results in expansion at bed (and rising at the sides) and surface flow separation from sides is evident from velocity distribution at different sections in transition on model resulting in high velocity jet impinging against left wing wall. The flow was corrected by a baffle stabiliser as shown in Fig. 13(*b*). The height of the baffle was 2.5 feet and radius of 25 feet. There was only a single hydraulic jump forming on the d/s glacis and there was no surface disturbance as a consequent of baffle wall at any stage. The baffle wall helped by deflecting some flow towards sides and towards surface.

IV. Mangla Dam spillway stilling basin with a baffle wall.

The features of final design of main spillway of Mangla Dam were studied at St. Anthony Falls, Minneaopolis (U.S.A.). The ultimate recommendation of St. Anthony Falls was to have two stilling basins with sloping sides on 1 : 1.75 and high free standing walls (up to 100 feet) in upper and lower stilling basins which were introduced to eliminate severe reverse roller formation. The details of the final design are given in Fig. 14a. The special features of the design are:—

(*a*) A breast wall upstream of the gates of the head regulator to converge the flow in vertical form from surface to bottom.

(*b*) The discharge per foot on the glacis (400′ in width) was as high as 2500 cusecs.

(*c*) The Fr. No. at the toe of the chute glacis into the upper stilling basin was about 4.

(*d*) Deflector wall 39 feet high with tapered faces to deflect the flow towards the surface in the upper stilling basin.

(*e*) Groynes in the upper stilling basin to check eddy formation.

(*f*) Curved guide wall to converge the flow towards lower weir. (700′ wide at crest) and vertical free standing wall along two flanks of d/s glacis of lower weir.

(*g*) Side groynes in the lower stilling basin to check back eddy.

(*h*) Baffle blocks to corrugate the flow and Rehbock deflector in the lower stilling basin.

(*i*) Side sloping walls of the two stilling basins at the slope of 1 : 1.75.

The spillway was tested at Hydraulic Research Station Nandipur at 1/100 natural scale which was larger than 1/150 scale at St. Anthony Falls. The model was run for 4 hours (40 hours on the prototype) with 5,00,000 cusecs and then four hours with 9,00,000 cusecs. The bed contours at the end of test are shown in Fig. 14(*b*). The maximum scour was observed to be 33 feet, below the floor level of lower stilling basin. There was also substantial cutting on the right bank. The flow pattern is given in Fig. 14(*c*) which shows that the return eddy in the upper stilling basin is removed whereas the side rollers downstream of the lower stilling basin are not completely eliminated. Series of tests were conducted and it was found that an alternate arrangement is possible to eliminate the side roller without modifying the geometry of the stilling basins but dispensing with walls, groyness and palliatives in the two stilling basins.

(*a*) A deflector wall curved in plan, radius 500 feet with top at R. L. 945 tapering to ends at R.L. 940, was sufficient all alone to disperse out the forward flow to full basin width and to eliminate the return eddies in the upper stilling basin.

(*b*) A straight 12.5′ high baffle wall with upstream vertical face was introduced at the toe of the glacis into the lower stilling basin. This baffle wall was split into three parts the central 460 feet long part was kept at the toe of the glacis whereas the side portions of the wall in length of 152 feet were shifted downstream by 50 feet.

(*c*) The Rehbock cill at the end of the d/s floor of the lower stilling basin was replaced by 2 rows of deflector blocks of height equal to 16 feet.

The modified design is shown in Fig. 15(*a*). The flow conditions in the two stilling basins at a discharge of 500,000 cusecs and 900,000 cusecs are shown in Fig. 15(*b,c*). The scour at the end of usual test was observed to be 15 feet only (Fig. 15-*d*). The velocity distribution in the lower stilling basin is shown in Fig. 15(*e*), which indicates even distribution of velocities.

However, in view of the fact that the design and drawing work on the spillway had reached a stage where the above alteration could not be introduced without causing an acceptable delay, further experiments to determine various parameters *i.e.* pressures, cavitation area, effect on uplift were not carried out.

VANES: TO REMOVE RETURN EDDY IN A SHARP EXPANSION

Vanes form physical obstruction to the formation of return eddy in a sharp expansion. Sir C. C. Inglis recommended splayed vanes in extension of piers extending d/s as far as the baffle wall. The alignment of these vanes when extended divided the bed width of the canal at the end of the expansion from side to mid stream at 1.26 R, 1.09 R, .97 R, .88 R and .8 R where R is ratio of canal bed width and No. of spans. It was claimed that the vanes were so successful that the expansion length could be reduced to half.

Experiments in the Research Institute showed that the return eddy could be satisfactorily eliminated if only the side piers were extended as two end vanes. These vanes caused a natural expansion of flow between the remaining piers. The following examples are discussed to show that only end vanes are required; if the vanes are diverged less sharply than radial vanes.

(*i*) Panjnad canal fall at R.D. 15,000

The Panjnad canal fall has a fluming ratio of 66% and d/s side divergence is 1/3. The discharge per foot run between the piers is 60 cusecs. The d/s glacis is stepped. The head across the fall ranges from 5 to 3.5 feet as the discharge varies from 2,500 cusecs to 7,250 cusecs. This fall, just after its commissioning, developed a heavy tail erosion due to : (*i*) separation of flow from the sides, (*ii*) substantial decreases in area of forward flow occupied by return eddies and (*iii*) concentration of converging flow. Sir Claude Inglis as a result of model tests at Poona, recommended a perforated straight baffle wall with its top 1'-6 ' above the cill level of the fall. This baffle wall was constructed at the d/s end of impervious floor as shown in Fig. 16. At the same time the concrete block apron in a length of 26.3 feet was converted into a pucca floor by filling the joints of the blocks and raising the floor by 0.8 feet by means of bricks on edge. The flow over the baffle wall had a reduced intensity and standing wave formed upstream as well as d/s of the baffle wall. This baffle wall along with the major portion of the cistern both u/s and d/s of the baffle wall failed on 31-5-1971 after about 36 years. The restoration of supplies in the

canal was a matter of extreme urgency. It was decided by the Secretary, Irrigation and Power to remove the debris from the scour pit and the intact portion of the baffle wall near the sides and replace the damaged impervious floor by stones in 'Tarangar' of size $3' \times 5' \times 2'$ over one foot stone ballast as the exit gradient even at the toe of the d/s glacis was safe for the head across of the order of 6 feet. As is evident from the curve of stone size against velocity (Fig. 17) the size of Tarangar was sufficient to cater for the velocity of the order of 18 feet/second while the velocity at site was of the order of 16 ft./sec. The subsequent running of the fall during the summer supplies showed that the stone Tarangars were intact and the flow conditions are shown in Fig. 3. This shows that scour of the order of 40 feet had taken place and that the right bank of the canal was damaged due to the right-sided set of the jetting flow and a large return eddy along the left bank.

A model of the fall was constructed on 1/12 natural scale at Hydraulic Research Station, Nandipur to study and correct the flow conditions. The recommendations of the laboratory shown in Fig. 18 are :—

(1) To replace steps of d/s glacis except the end step by the regular glacis on 1/5. The steps were not effective as these did not intercept the gravity parabola of flow and in fact these slopes were found to introduce vertical instability to the flow. The d/s step should be retained to break the jet as it enters the d/s cistern. The junction of the crest with d/s glacis should be curved to avoid separation of flow.

(2) To construct longer vanes in continuation of end piers along with smaller vanes (Fig. 16) in side bays to direct flow into the back circulation along side walls and thus force the return eddy to move out.

(3) To provide three rows of cubical blocks $2.5' \times 2.5' \times 2.5'$ staggered with one another in the cistern.

(4) To provide an end cill 2.5 ft. in height with d/s face vertical and u/s face of slope of 1/2 to encourge low velocity ground roller at the d/s of the end cill to protect the flexible apron lower down. It was noticed that if the flexible apron level was at higher level, the formation of ground roller was vitiated and the d/s scour increased and thus the flexible apron level should be such that the formation of ground roller is not obstructed.

It was noticed that the head across the vanes did not exceed 0.25 feet. The flow conditions and bed scour at the end of a long duration test is shown in Fig. 19 which shows that the side eddies were extinct and scour of the order of

3.5 feet was confined to the central portion along the axis of the channel. The velocity distribution in different sections is also plotted in Fig. 20 which shows normal distribution of flow at section 750 feet from the crest, and an even distribution of flow at the end of pucca structure.

(*ii*) Fall at R.D. 15000 of Rohri Canal.

This fall has a fluming ratio of 74% and side divergence is about 1/2.5 and head across of 7 feet. It was observed that without vanes some slack flow existed along the side walls near the d/s end. It was found that the flow was corrected when the vanes were provided in continuation of the end piers inclined outward with an angle of 10° to the axis at the piers then the velocities at the end of the structure were normal and evenly distributed. (Fig. 21).

(*iii*) New Tando Masti Khan fall on Rohri Canal at R.D. 120,000

The fall at R.D. 120,000 of Rohri Canal was an Inglis type (Fig. 1). This fall worked satisfactorily till 1956 when the fall developed cracks in its crest and piers which developed and ultimately resulted in complete failure of the work. The new design, Fig. 22(*a*), is Montague type glacis profile with abrupt contraction at inlet, and d/s divergence at 1 : 3 starting at the d/s end of crest. The fluming ratio of the fall is 62% and it is designed for a discharge of 10950 cusecs with an initial drop of 9.9 (till the construction of another fall higher up) and ultimate drop of 4.9. The discharge per foot run is 73 cusecs and F_1 is equal to 2.4. The depth of flow on the d/s floor is 14.9 against Ef2 of 13.45 and the length of floor is 5 Ef2. The I.R.I. type of two rows of impact and deflector blocks are provided. The flow was corrected by vanes alone. The flow conditions without vanes is as shown in Fig. 22(*b*) which shows breaking off of forward flow from the flank walls over the glacis causing regions of negative velocity along the sides. The velocity of flow along the bed at the line passing the end of the divergence *i.e.* 190′ from the d/s end of crest was of the order of 5.0.′ The fanning out of flow on the expansion was achieved.

(*a*) Guide Vanes on the d/s Floor

The vanes 15′ in height in continuation of the line of the side piers starting from toe of d/s glacis and ending at the deflector rows of blocks and aligned parallel to the toe line of divergence were found to be effective to force the flow in the side bays towards the negative velocity pockets and to allow the flow of intermediate bays to expand in between the vanes, Fig. 23(*a*). Three apertures of 3′ × 2′ in each vane reduced the intensity of flow along the flankside of the vanes and increased the velocity of flow in low velocity area in the wake of the vanes. The scour at the end of test run along the line of vanes was only 16%. Whereas the scour along the axis of channel was 53% of the base test indicating

some concentration of flow along the axis of channel, Fig. 23(*b*). The maximum velocity along the bed was 3.1 f.p.s. against 5 f.p.s. in the original test.

(*b*) **Guide Vanes on the Glacis**

I. One guide vane in continuation of the flank piers and one along the axis of the side bays aligned parallel to the toe line of the flank at a height of 3′ and ending at the impact rows of blocks were effective in forcing the flow to follow the flank walls, Fig. 24(*a*). The reduction of scour in the line of the vanes was 33% of the base test whereas the corresponding reduction of scour along the axis of flow is only 40%, Fig. 24(*b*). The velocity along the end increased to 3.7′ f.p.s. against 3 f.p.s. in the above test.

II. The above test with vanes on d/s glacis was repeated with addition of two vanes in continuation of the two intermediate piers. The scour along the axis of the channel was only 16% of base test (Fig. 24(*c*). It is evident that the vanes in continuation of the intermediate piers if aligned on correct angle reduce the concentration of flow opposite the central bays while the vanes along the flank piers and vanes on the crest and the glacis of the side bays are only responsible for elimination of regions of negative or zero or slack velocity along the flanks.

USE OF SUBSIDIARY STANDING WAVE u/s OF BAFFLE WALL TO FAN OUT FLOW IN SHARP DIVERGENCE OF M.R. LINK OUTFALL

In normal cases of design of the falls in canals of high discharge the fluming ratio is generally not allowed to be less than 60 to 70%, as otherwise the problem of maintaining a uniform intensity at all points on the glacis and cistern becomes acute. In case of higher contraction either we have to have a very mild divergence or employ additional means to fan cut the flow to avoid concentration of the jet in the centre and creation of rollers on the sides, as these would result in deep scour hole in the centre and excessive bank erosion. When such a design has to be adopted for outfalls requiring a wide variation of down-stream water level, the problem becomes more intricate. The flow follows the sides at normal tail water level, but it gets separated in the upper portion of the flank wall under accreted tail water level, and in the lower portion of flank wall under retrogressed tail water level.

One of the methods adopted (to overcome the problem of flow separation) is to split a single drop into a double drop by a baffle wall to create secondary standing wave, above the baffle wall as shown in figure 25. The turbulence generated in the standing wave and high water level generated above the baffle wall help in the lateral spreading of the flow above the baffle wall. Moreover

any asymmetry of flow due to angular approach to the fall becomes rectified and the flow gets evenly distributed due to intense turbulence and mixing of flow in the cistern u/s of the baffle wall. The conjugate depth D_2 of flow d/s of the hydraulic jump is given by relation $D_2 = q^{1/2} H_L 1/6$. The discharge per foot run q, as well as the drop H_L at the baffle wall get reduced obviously, therefore, the downstream depth to form a hydraulic jump D_2 decreases and outfall structure designed as a single drop structure can withstand greater degree of retrogession especially in the presence of impact blocks in the lower cistern d/s of the baffle wall. Experiments were carried out on 1/36 scale model of M. R. Link outfall: Fig. 25 (Q=22,000 cs.) with different fluming ratios and different heights of baffle in the cistern of fall designed as a single drop structure with divergence of flank wall at 1 : 2 u/s of baffle wall and 1 : 3 d/s of it.

The main conclusions of this series of tests are :—

(1) The baffle wall fixed on the cistern of the single drop structure is essential to disperse out the flow towards sharply diverging out side wall. The optimum height of the baffle is a function of the range of tail water levels. For normal and retrogressed tail water level conditions the drop at the crest of fall and over the baffle wall should be such that proper and balanced jumps are forming in the upper as well as lower cistern and surface undulations in the two cisterns are avoided.

The curved baffle wall concave to flow spreads it out and reduces concentration of flow opposite the middle bays and induces more flow along the flank wall. The optimum radius of the baffle wall is between 2/3rd to full width of canal. The top R. L. of the baffle should be lower than the crest level.

(2) The vanes in continuation of side piers as indicated in figure 26(*a*, *b*) are helpful to force the discharge from the side bays towards the flank walls. It was noticed that in absence of outer flank vanes the tendency of flow to break from the side walls increases with degree of submergence of baffle wall and so the utility of these vanes increases with submergence of the baffle wall. The function of intermediate vanes is to decrease the degree of concentration of flow opposite the central bays and so can be eliminated at the cost of some increase of scour along the axis of channel which will mean longer length of flexible apron in the middle portion or more dumping of stone during the maintenance period.

(3) The perforations in the baffle wall figure 25(*b*) in two rows each of 1 × 1.5 feet, 3 feet apart horizontally and 1.5 feet apart vertically

contribute to the reduction of scour by about 25% at retogressed tail water level (731) and reduction of scour beyond normal tail water level (736) was almost nil. The flank apertures help in forcing the flow to follow the flank walls.

The openings of the baffle wall for retrogressed tail water levels also help in dissipation due to additional intermixing of flow.

(4) The pattern of flow (Fig. 26-*c*) under accreted and retrogressed conditions improves still further if vanes are replaced by hooks of width equal to 1.5 times the width of piers and of radius equal to depth of flow on the crest and are placed along the right side of piers in the right half and along the left face of piers in the left half of the crest. The maximum scour observed on model with hooks was .4′, 1.8′, 1.4′ and 1.0′ in tests with tail water level at 731, 736, 738 and 741.12 against corresponding scour of 2.4′, 3.0′, 3.5′ and 1.3′ with vanes replacing the hooks. The hooks help in air entrainment and better dissipation of energy in the standing wave. The hooks are, however, likely to catch debris. The choice of hooks as compared to vanes depends on ease of construction and economy of the two devices as the hooks are required to be buttressed on the glacis.

(5) The test showed that the length of floor u/s of the baffle wall should not be less than 40-50% of the theoretical length of the standing wave for a single drop structure. The depth of flow in the upper stilling basin under minimum water level should be of the order of conjugate depth required for formation of a proper hydraulic jump under retrogressed tail water level conditions.

(6) The impact and deflector rows of blocks are part of the d/s floor as impact blocks break the jet and deflector blocks lift up the high velocity flow striking to the bed.

SERIES OF SEPARATE FALL STRUCTURES AS A SUBSTITUTE FOR A SINGLE HIGH DROP OUTFALL STRUCTURE

Chashma Jhelum outfall, a combined canal regulator-*cum*-bridge-*cum*-fall structure, is designed for a discharge of 21,700 cusecs and a maximum head across of 62.5 ft. The fluctuation of tail water level in river Jhelum will be 21.0′.

Due to the large capacity and relatively high fall special attention was given to the selection of a structure which would meet all the requirements of being hydraulically adequate and structurally stable as well as most economical.

Cost estimates were made by M/s Tipton and Kalmbach, Consultants for Link Canals for about a dozen different designs. Poor foundation conditions and permeable, fine sands created difficulties which virtually ruled out a solution with single fall. Hydraulic model studies were undertaken for five alternatives. (Fig. 27).

The alternatives 7-a,b,c were a combination of stepped glacis stilling basin-vertical well box (with a horizontal orifice in the downstream w ll of box) followed by a stilling basin. Alternative 4 is a series of cascade falls with typical crest, a downstream glacis of 1 : 2, a stilling basin of 30 ft. and an upstream floor length of 120 feet. Alternative 5c which is a series of four separate falls, has other advantages such as short sheet piles, thinner concrete, lesser reinforcement and is simple in construction. Comparative estimates of cost for 5 alternatives that were model-tested showed that alternative 5c is about 1.5 million rupees less in cost than the next alternative.

The hydraulic performance of alternatives 7a, 7b and 7c on 1/24 geometrical scale model was unsatisfactory and the observations recorded during the tests led to the following conclusions :—

(1) The steps forming the glacis caused hurdling action of the jet (Fig. 28). There is incomplete energy dissipation in stilling basin following the stepped glacis as standing wave front moved to and fro occasionally sweeping out of basin. Replacement of strpped downstream glacis by straight slope produced improved and stable flow condition.

(2) As the opening of upper well box is not submerged, it is not efficient whereas the lower well box is very efficient due to extraordinary turbulence in the well with proper and highly turbulent standing wave immediately below the well. However, it is a vexed problem to design and position the downstream baffle wall of lower well box for wide fluctuations in tail water level (R. L. 565 to 544).

(3) Replacing cisterns and well boxes by standard type stilling basins made the design less complicated and improved the hydraulic performance of the outfall structure.

(4) Quadrant type downstream wing walls need replacement by straight, flared and tapering down wing walls to provide a smooth transition to the downstream channel.

The alternatives 7a, 7b and 7c as revised as a result of model tests are shown in Fig. 29 which indicates that the "stepped glacis" stilling basin-vertical wall concept is basically replaced by straight glacis and standard stilling basin.

Tests for alternative 4 showed that addition of chute blocks improved the hydraulic jump conditions and lowering of stilling basin helped in moving the jump up the glacis.

The model testing of alternative 5 (which consists of canal regulator-*cum*-fall structure followed by three individual separate falls) led to the following conclusions :—

(1) Piers over the fall structure supporting the bridge when extended into the hydraulic jump results in turbulence in the wake of the tail end of piers entailing increase in scour.

In the presence of deflector at end of d/s floor the d/s stone apron should not slope up as the ground roller gets obstructed by sloping up stone apron resulting in diving down of flow earlier lifted up by the end deflector and consequently increased the scour.

(2) Replacement of end cill by deflector staggered blocks in two rows reduces the scour by about 10 per cent.

(3) A straight, vertical, flared out on 1 : 5 d/s wing wall sloping down at top at a slope of 1 : 2 (Fig. 31-*a*) is hydraulically superior to circular d/s wing walls (Fig. 28) as with tapered down wall the flow after passing over the tapered wall spreads out into the full section.

(4) Fluming beyond 75% of separate falls tends to increase high velocities d/s and should be avoided if the connecting channels are not paved.

(5) The rate of divergence of side walls should be gradual up to mid of cistern and that the rate of divergence beyond mid cistern can be increased but the increase should not be very sharp especially when the flank wall is splayed.

(6) In case of series of separate fall structures the fluming at the tail fall structure should be decreased to avoid too abrupt transition back to the outfall channel.

Single Fall

Submerged slotted roller bucket was found to be capable of working efficiently hydraulically for a wide range of tail water level conditions. The bucket worked as a submerged roller bucket up to the tail water level at which boil of bucket gets submerged. At higher tail water level it acts as stilling basin with the standing wave fully or partially shifted to the glacis and in this case the live vein is lifted up by the teeth in the roller bucket and velocity distribution at end of structure gets normalised. The features of the slotted roller bucket tried on the model are shown in Fig. 30. The width of teeth are .125 R and

spacing is .05R and angle of teeth is 45°. The optimum level of the lip of the bucket is equal to the minimum tail level minus conjugate water depth required for formation of jump for maximum Q and the minimum tail level and the bucket invert level is lower than the lip level and this lowering is function of discharge per foot run, drop in water level, the dimensions of the teeth and the geometry of bucket. It is obvious that the bucket invert level will be lower than the floor level of conventional stilling basin for a single drop and it entails more excavation and more pumping during construction. It was ruled out due to this reason and to poor foundations and uplift considerations as well. Due to the square end of the piers on the crest fins generated at the tail end of piers entrained large quantities of air Fig. 30(*b*). It was noticed that the boil of the roller submerged bucket was very efficient in dispersion of flow as lateral spreading of flow was possible due to higher elevation of surface of boil and high degree of turbulence. The flared out side walls had a vertical face and had divergence of 1 : 5 and tapered down in top slope of 1 : 2 (the taper started at end of 16° sloping up floor of bucket). The concrete block apron was provided d/s of end wall and the series of tests showed that the concrete block apron should taper down and the minimum scour was attained when the taper was 1 : 6.

The final design of outfall for C. J. Link comprised of 4 separate falls (Fig. 31-*a*, *b*) embodied the recommendations of Irrigation Research Institute.

SPREADING FLOOR FLUMED FALLS

The paper contribution by Dr. Mushtaq Ahmad to the Pakistan Journal of Scientific Research in July 1950 deals with a new type design of Canal Falls which is radically different from the conventional flumed glacis type, Fig. 31(*c*). The main feature of this design is that the crest (Horizontal) or sloping down or sloping up to suit medium, large or small drop falls is continued into the divergence and thus the expansion is in supercritical jet and thus takes place in accelerative condition as so long as the accelerative flow continues, separation of flow from the boundary is not likely to occur. The drop at the end of expansion is vertical. The distance X at which the jet would strike the cistern y feet below the crest is given by:

$$X = vt$$

$$y = \tfrac{1}{2} g t^2$$

(or eliminating time)

$$X = \frac{V}{4} \sqrt{Y}$$

where v is the velocity at the end of expanding crest (to be determined by appli-

cation of Bernoulli's theorem at throated section of crest and section at d/s end of expansion).

The length of the cistern for vertical drop was proposed at 6 Ef_2 and depth of flow in the cistern at $1.25 \times Ef_2$ where Ef_2 is calculated from Montague's curves (C.B.I. Publication No. 12) for discharge per foot at end of an expanding crest. The radius of the vertical flank walls of expanding crest is a function of the contraction and the length of the expansion in the direction of flow is obvious from Fig. 31(*c*).

The circular wing walls can be replaced by straight divergence at ratio of 1 : 3 F_1.

Mr. Walter Rand[5] in his paper "Flow geometry at straight drop spillways" submitted to American Society of Civil Engineers in September 1955 has found that the flow geometry at straight drop spillways can be described by function Drop Number (D) which is defined as

$$D = \frac{q^2}{gh^3}$$

where q is discharge per unit width at end of crest of overfall, g is accelaration of gravity and h is height of drop (*i.e.* difference between R. L. of crest at its d/s end and floor R. L. of cistern in case of an expanding crest fall). For a given height h and q discharge per unit width the sequent depths $y1$ and $y2$ and drop length X and pool depth yp under the nappe can be determined by the equations :—

$$\frac{y1}{h} = .54\ D^{.425}$$

$$\frac{y2}{h} = 1.66\ D^{.27}$$

$$\frac{x}{h} = 4.3\ D^{.27}$$

$$\frac{yp}{h} = 1.0 \times D^{.22}$$

Sufficient aeration can be provided to the nappe of spreading floor fall if thin piers with square d/s end are provided at the end of spreading crest. The spreading floor fall needs to be studied in detail for different discharge intensities and a wide range of fall drops.

SCOUR BELOW VERY LOW DROP STRUCTURE AND REMEDIAL MEASURES

1. Fall at R.D. 182+101.8 of Q.B. Link

The fall at R. D. 182+101.8 on Q. B. Link Canal was designed for a drop of .6′ (See Fig. 32) and was tailored to the specifications laid by U.S. Bureau of Reclamation for U.S.B.R. stilling basin IV. (Fr=2.5 to 4.5). The side walls were taken vertical and straight in continuation of the side abutment up to the end of the downstream floor and then turned sharply and flared out.

In case of low Froude No. falls an oscillating jump will be produced in the stilling basin generating a wave that is difficult to dampen. The U.S.B.R. basin IV is designed to combat this problem by eliminating the wave at its source by intensifying the roller which appears in the upper portion of the jump by the use of directional jets deflected from large chute blocks on the toe of the glacis. Basin IV is applicable to rectangular sections.

The chute blocks are a serrated device which are expected to furrow the jet at the entrance to the stilling basin and lift a portion of it from the floor and are also tended to stabilise the jump and improve its performance. But in case of very low drop structure the standing wave does not form and the live jet is lifted off the glacis and the chute blocks do not act as serrated device.

The end deflector is meant to deflect high velocity current towards the surface and create a ground roller in wake of deflector. But in the case of very low drop falls the high velocity current is already on the surface and dives down to the canal bed beyond end of structure. In case the high velocity flow is along bed and is deflected towards surface by deflector, it is likely to fall back towards bed if ground roller is restricted due to high level flexible apron which rises up in adverse slope as is the case at fall 182 of Q.B. Link.

The wave height measurements were made at site by Irrigation Research Institute with the help of electric wave recorder equipment as shown in Appendix II. The results obtained with various discharges and downstream depths are given in detail in the Appendix II. The maximum wave height (varies with discharge) was noted to be of the order of 8 inches.

A deep scour (28′ below the downstream floor level) developed downstream of the loose stone apron (which remained intact) within one month of operation of canal. As is evident from Fig. 32, two scour holes developed symmetrical to the axis of canal.

Due to retrogression in the bed of canal the head across this fall at R. D. 182 has increased to 2.5′ and thus a standing wave (though weak) is now forming at this site and the laboratory tests showed that for the existing conditions at site,

there is separation of flow from flank walls causing a reverse roller and concentration of flow opposite the side bays. It was observed that trapezoidal blocks of 3 feet height (upstream face vertical, width of blocks 5′ at upstream face and 2.0′ at downstream end) placed 5′ apart, Fig. 32(*b*) are effective in deflecting the line of maximum velocity towards the surface (which was not sufficiently lifted up by end deflector alone and its attendant roller). Further experiments are being taken up to improve the scour conditions under all possible working conditions of flow. A guide vane on the downstream floor on each flank aligned at the correct position and of correct length may be desirable to stop the back circulation.

2. Chashma Jhelum Link Canal Fall at R. D. +193+000

The designed discharge of this fall is 21,700 cusecs (q the per foot width discharge is 75.3 cusecs), working head is 0.5, with submergence of 95.5%, fluming ratio is 83.7%, and the side walls are straight in line with the abutment right up to end of stilling basin and then splayed out at a mild angle and dropped at the tail end in a top slope of 1/2.

The coefficient of discharge in the equation ($Q=CBH^{3/2}$) at $Q=21,700$ was observed to be C..2.00 and so it behaves as a headless weir.

The line of maximum velocity at full supply discharge floats near the surface, but a negative velocity of 2.0′ scours near the floor (Fig. 33). The surface roller appears at 3/4th supply when upstream full supply level is maintained and in this case the line of maximum velocity keeps to the floor level and then emerges at the end cill to be deflected towards surface.

It was further noticed on the model that any negligence in the regulation or angular approach to the fall sets up side roller due to instability of flow, introduced by bed roller (with horizontal axis) in the cistern. It was seen that wedge type regulation results in return eddies along flank wall and heavy scour of the order of 30.0′ was observed from bed contours on the model after 10 hours run with full supply discharge (Fig. 34-*a*, *b*). Thus in spite of the fact that flank walls are straight with vertical face right up to end of cistern scour tendencies were noticed with any unequality in distribution of flow in vertical or plain.

The subsequent attempts on the model to even out the flow showed that hydraulic jump stilling basin is not required and that a sloping apron at a mild slope is the answer. The scour profile through the deepest scour hole at the end of usual 10 hours duration model tests with full supply discharge are plotted in Fig. 35-*a*, for the following cases :—

(*a*) Clean sloping apron at a slope of 1 : 3, 1 : 4, 1 : 5, 1 : 6, 1 : 7, 1 : 8, 1 : 9 and 1 : 10 without any horizontal floor.

(*b*) As above but with single row of blocks (3.0′ cube) spaced 3.0′ apart at end of sloping apron.

(*c*) As in item (*a*) above but two rows of staggered blocks at the end of sloping apron.

The maximum scour observed for different slopes of apron, without blocks, with single row of blocks, and with two rows of blocks, are plotted in Fig. 35 which shows that optimum slope of the apron is 1 : 7 and that single row of blocks at the end of apron reduces the scour by about 40% and the two rows of blocks by 60%.

Teeth of submerged roller bucket at the toe of the glacis is another promising alternative for stabilizing the flow d/s of headless falls. Experiments are in progress to determine the optimum dimensions of different elements of such teeth.

BED AND BANK EROSION DUE TO HUMP IN THE BED OF FLUVIAL CHANNEL

In case there is a pucca hump or stone hump in the bed of a fluvial channel, even if the top of the hump is flush with channel bed the flow gets separated at the hump and is lifted up from bed resulting in generation of a ground roller and eddies in the wake of the hump which in turn accelerates any tendency of flow to separate from the flank walls leading to heavy scour in bed of canal and belly action on the sides.

For example—Across L.B.D.C. main line (Q=8100 cusecs) aligned on curve between R. D. 19840 and 24942, a syphon crosses at R. D. 21254. The fluming of canal at the syphon crossing is 88%. The crest of the syphon structure was at canal bed level. The bed contours of canal downstream of the syphon crossing indicated scour of the order of 36′ and a heavy cutting of the banks.

A model of the canal and syphon hump in the canal bed was constructed on 1/6 natural scale. The flow currents observed on the model are shown in Fig. 37. The back eddy on the inner bank is stronger than that on the outer bank and extends over a wider area. The flow converges towards the axis line of the structure.

To eliminate the hump action a stone apron (with top at crest level of hump) 50 feet in length in continuation of the syphon structure and sloped down to a slope of 1 : 1 at the far end was effective in eliminating the bed roller, Fig. 37(*b*). The back circulation along the sides subsided and reduced in area and extent and was completely obviated by providing upstream and downstream banks at the cross structure with stone pitched splays of 1 : 10 and 1 : 7 respectively.

The humps under bridges with piers on open foundations of L.J.C. resulted in heavy bed scour and belly action.

Introduction of stone paving to protect the foundations of piers of the old Kot Abdul Hakim Bridge on River Ravi increased the scour d/s of the bridge substantially and the scour from H.F.L. was of the order of 60.0'.

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APPENDIX—I

EVALUATION OF PARAMETER OF HYDRAULIC JUMP AND DESIGN OF STILLING BASIN

Once the design discharge, crest level, width of hydraulic structure and the bed level of approach channel to the structure are fixed—the value of q the discharge per foot run is known and the total energy line upstream of structure is computed from the equation :

$$q = C\ (\text{T.E.L.} - \text{Cill Level})^{3/2}$$

where C is the co-efficient of discharge.

T.E.L. is total energy line $= H + V^2/2g$

A slightly modified method of calculating the elements of the jump for good dissipation of energy is discussed below. The method takes into consideration the error factor for velocity distribution and a yeloeity coefficient accounting for friction losses.

To attain maximum energy dissipation at the fall structure the jet should be broken up into small vertices as early as possible in the stilling basin downstream of the fall; and this depends on an efficient hydraulic jump which has to be ensured for good energy dissipation. The energy dissipation is proportional to the cube power of the difference of flow depth at the end and the beginning of the hydraulic jump which in turn has a functional relationship with the Froude No. at the front of the jump. The elements of the jump are correlated as below with the characteristic variables with reference to nomenclature of Fig. 1.

H_0 = upstream energy line minus downstream floor level.

D_1 = depth of flow at the beginning of jump.

D_2 = Theoretical conjugate depth of flow at the d/s end of jump required for formation of jump.

D_r = downstream water level minus downstream floor level.

$D_c = (\alpha' q^2/g)^{1/3}$ = Critical depth of flow.

where α' = error factor consequent of use of mean velocity (V) instead of the true actual velocities at different points in the vertical and horizontal section.

$$\alpha' = \int_i V^3 \cdot d\,A/\hat{V^3}.\ A$$

$$\mathcal{E}_0 = H_o/D_c$$
$$\mathcal{E}_1 = D_1/D_c$$
$$\mathcal{E}_2 = D_2/D_c$$
$$\mathcal{E}_r = D_r/D_c$$

at the vena contracta section

$$\alpha q = V_1 D_1 \text{ or } V^2 1 = q^2/\phi^2 D_1^2 \quad \text{(I)}$$

where $\alpha = \int V^2 \cdot d A/V^2.A$

α' is approximately $= \alpha^2$

and ϕ velocity coefficient that accounts for all the friction losses over the crest and the glacis. From Barnouli equation utilizing equation I,

$$H_o = D_1 = V^2/2g = D_1 + \alpha' q^2/2g. \ \phi^2 D_1^2 \quad \text{(II)}$$

Dividing equation II by D_c

$$\mathcal{E}_o = \mathcal{E}_1 + 1/2 \ \phi^2 \ \mathcal{E}_1^2 \quad \text{(III)}$$

Again by Newton's mentum equation

Force = change of mentum/second = mass x change of velocity

$$WD_2^2/2WD_1^2/2 = W.q/g. \ (V_1 - V_2)$$

substituting the value of V_1 or V_2 from equation (I)

$$2 \ \alpha' \ q^2/g = D_1 \ D_2 \ (D_1 + D_2)$$
$$2 = \mathcal{E}_1 \mathcal{E}_2 (\mathcal{E}_1 + \mathcal{E}_2)$$

$$(\text{Froude No.})^2 = \alpha^2 \ V^2/gD_1 = \alpha' q^2/g \ D_1^3 = 1/\mathcal{E}_1^3 \quad \text{(IV)}$$

$$Fr = \text{Froude No.} = 1/\mathcal{E}_1 3/2 \quad \text{(V)}$$

Different values of $\mathcal{E}$ were assumed and corresponding values of $\mathcal{E}_1$, $\mathcal{E}_o$ and F were worked out from equation IV, II and V respectively.

Now $\mathcal{E}_o - \mathcal{E}_2 = H_o - D_2/D_c$ = u/s total energy level minus d/s tail water elevation/critical depth.

$\mathcal{E}_o$- $\mathcal{E}_2$ and $\mathcal{E}_2$ are plotted against F in Fig. 1

Critical depth (D_c) is plotted against q in the same Fig. 4.

The procedure for working out D_2, D_1 and F the three elements of

hydraulic jump will be as follows :—

(*a*) Having fixed the total design discharge and the clear waterway of the structure, work out the design value of q, the discharge per foot width and then read D_c for design q from Fig. 1.

(*b*) Having fixed the cill level of the structure and floor level in the approach channel to the structure, the upstream total energy line is manipulated from the formula,

$q = c$ (Upstream T.E.L. — crest level)$^{3/2}$

(*c*) The water elevation at the tail end of structure worked out from retrogressed and accreted water level curves to be assessed in the manner described above.

Then knowing $H_0—D_2$ (upstream T.W.L. minus tail water elevation) the value of $\frac{(H_0—D_2)}{D_c}$ which is equal to $\mathcal{E}_0—\mathcal{E}_1$ is worked out.

Knowing the value of $\mathcal{E}_0—\mathcal{E}_2$ and assessing the value of ϕ the value of E2 is read from $\mathcal{E}_0—\mathcal{E}_2$ and $\mathcal{E}_2$ curves, the corresponding value of $\mathcal{E}_1$ is read from $\mathcal{E}_1$ and $\mathcal{E}_2$ curves and the value of F_r is read from $\mathcal{E}_1$ and F curves.

(*d*) To work D_2 and D_1 multiply $\mathcal{E}_2$ and $\mathcal{E}_1$ by D_c. Careful perusal of Fig. 1 reveals—

(*i*) No proper standing wave will form if the value of $\mathcal{E}_0—\mathcal{E}_2$ is less than .41. In case of headworks if the intensity of flow is 300,200 and 100 cusecs the value of H_0/D_2 required for proper formation of jump will be 5.98′, 4.55′ and 2.87′ respectively.

(*ii*) For range of value of $\mathcal{E}_0—\mathcal{E}_2$ between .41 and .54 (for ϕ=0.96) the jump will be generating waves due to fluctuations in $\mathcal{E}_2$ from 1.8 to 2.33.

(*iii*) The curves in Fig. 1 cover a very wide range of values of $\mathcal{E}_0—\mathcal{E}_2$ *i.e.* from .1 to 50 and thus are applicable to almost all hydraulic structures.

Example :—

Suppose discharge per foot run is 60 cusecs, the corresponding value of critical depth Dc from fig. 1a is 5.5′.

Now suppose the difference of U/S total energy level and D/S tail water level is 7.5′ or the value of $\mathcal{E}_0—\mathcal{E}_2$ is 1.50. From Fig. 1*b* the corresponding value of λ (=Fr^2) is 10 and the respective value of $\mathcal{E}_2$ from $\mathcal{E}_2 : \lambda$ curve works out to be 1.86′ and the value of $\mathcal{E}_1$ from $\mathcal{E}_1 : \lambda$ curve is .465. Thus the value of D_1 (the depth of flow at the beginning of jump) and the value of D_2 (the conjugate depth *d/s* of jump) is 2.325′ and 9.30′ respectively.

Location of point of wave observations 8 ft. away from water edge on the right bank at R. D. 80 (Taking loose Apron as zero).

Maximum Wave Fluctuations:

(*i*) 3.8 inches.
(*ii*) 3.8 inches.
(*iii*) 4.9 unches.
(*iv*) 3.9 inches.
(*v*) 5.6 inches.
(*vi*) 4.1 inches.
(*vii*) 4.4 inches.
(*viii*) 3.7 inches, 3.9 inches.
(*ix*) 4.8 inches.
(*x*) 4.4 inches.

Results:

Total Time of Recording =30 Minutes.
Max. Wave variation =5.6 inches=.47 ft.
Frequency spectrum of dominant waves =36-52 cycles per minute.

2. Discharge =8950 Cs. U./S. Depth=10.1 ft D./S. Depth=8.8 ft.
Gate opening=4.7 ft. Dated=16-4-68 =13 Hrs.

Max. Wave Fluctuations:

(*i*) 3.5 inches.
(*ii*) 3.7 inches.
(*iii*) 4.2 inches.
(*iv*) 3.2 inches.
(*v*) 3.7 inches.
(*vi*) 4.55 inches.
(*vii*) 4.00 inches.
(*viii*) 3.45 inches.
(*ix*) 3.6 inches.
(*x*) 3.8 inches.

Total time of recording=25 minutes.

Results:

Max. Wave variation =4.55 inches=.38 ft.
Frequency spectrum of dominant waves. =36-44 C.P.M.

3. Discharge=9125 cs. U./S. Depth 10.15 ft. D./S. Depth=8.9 ft.
Gate opening=4.8 ft. Date=16-4-68 at 14 Hrs.

Location of point :—4 ft. away from concrete right bank edge in the middle of loose Apron.

Max. Wave Fluctuations:

(*i*) 5.4 inches.
(*ii*) 5.6 inches.
(*iii*) 5.85 inches.
(*iv*) 4.5 inches.
(*v*) 5.85 inches.
(*vi*) 4.8 inches.
(*vii*) 5.0 inches.
(*viii*) 4.55 inches.
(*ix*) 4.9 inches.
(*x*) 4.4 inches.

Total time of recording=20 minutes.

Results:

Max. wave variations =5.85 inches=.49 ft.
Frequency spectrum of dominant waves =30-47 C.P.M.

4. Discharge=9125 cs. U./S. Depth=10.2 ft. D./S. Depth=8.9 ft. Gate opening=4.8 ft. Date..16-4-68 at 15 Hrs.

Location of points :—In the middle of pocket wall 4 ft. away from concrete edge of right bank.

Max. Wave Fluctuations:

(*i*) 6.4 inches.
(*ii*) 8.0 inches.
(*iii*) 7.6 inches.
(*iv*) 7.2 inches.
(*v*) 6.7 inches.
(*vi*) 5.7 inches.
(*vii*) 5.6 inches.
(*viii*) 4.85 inches.
(*ix*) 4.6 inches.
(*x*) 6.0 inches.

Total time of recording=20 minutes.

Results:—

Max. Wave variation =8 inches=.67 ft.
Frequency spectrum of dominant waves. =24.32 C.P.M.

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