ENERGY DISSIPATION DOWNSTREAM OF DAM SPILLWAYS

DISSERTATION SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE.

OF

MASTER OF ENGINEERING

IN

WATER RESOURCES DEVELOPMENT

by

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1967

CERTIFICATE.

Certified that the dissertation entitled "Energy Dissipation Downstream of Dam Spillways" which is being submitted by Shri K.N.Misra in partial fulfilment for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee is a record of candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a period of 9 months since 1st October 1966 to 30th June 1967 for preparing dissertation ~~for Master of~~ Engineering Degree.

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SUMMARY.

In this study an attempt has been made to discuss the important types of energy dissipators below dam spillways. Before discussing the various energy dissipators, a review of the basic characteristics of hydraulic jump on horizontal, sloping and part sloping and part horizontal apron has been made. This is followed by discussion on the choice of energy dissipators to suit different hydraulic conditions. After that hydraulic jump stilling basins, roller buckets, skijump and Trajectory buckets have been discussed in all its aspects.

From this study it is seen that so far no rational formulae have been developed for determining the jump length. As far as the design of stilling basins is concerned, a very accurate determination of jump length is hardly necessary as the basins are rarely designed to contain the full jump length. The existing knowledge regarding energy dissipation indicates that energy dissipation is slightly better in case of horizontal apron than sloping apron. The stilling basin with horizontal apron generally can be used for small and medium height of dams having moderate discharge intensities as jump is very sensitive to tailwater variations and difficult to control. Small difficiency in tailwater can be made up with the help of basin appurtenances. But the use of appurtenances is not favoured in case of high dams where discharge intensities and velocities are high as these cause cavitation.

The stilling basins with sloping and part sloping part horizontal apron are suitable for even high dams and large discharge intensities. In these basins the jump movement due to tailwater variations takes place within fixed range of apron length. The sloping aprons have tailwater requirements lesser than horizontal

aprons. The tailwater requirement decreases as the apron slope is steepened but cost of stilling basin increases. The stilling basin cost gets reduced with the adoption of part sloping and part horiz-ontal apron though the tailwater requirement is slightly more than the corresponding case of fully sloping apron. An effort has been made by utilising the existing data, to bringout quantitatively the tailwater requirements for fully sloping and part sloping part horizontal apron, for different slopes.

The roller buckets are suitable where tailwater is more than the jump requirement. As regards the performance, these have worked well wherever used, but the only point is that these have regular maintenance problems. The roller buckets are cheaper than hydraulic jump stilling basins.

The skijump or Trajectory buckets are suitable where tailwater is inadequate for hydraulic jump and the bed strata consists of fairly good rock free of laminations. These buckets have been extensively used in India and abroad and the performance have been quite satisfactory. As regards the economic aspect, it can be said that skijump buckets are cheaper than hydraulic jump basins.

Lastly it can be said that problems of energy dissipation for any two dams are not exactly similar. Each case needs consideration on the basis of its particular hydraulic, topographical and geological characteristics. The existing knowledge and experience about various energy dissipation arrangements can only serve as a guide for working out of preliminary designs which can be finalised on the basis of model studies.

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CHAPTER I

INTRODUCTION.

The construction of any structure across a stream creates some amounts of energy difference between upstream and downstream of the structure. The water discharged through the hydraulic structure acquires high velocities depending upon this energy difference. This high velocity flow may cause stream bed erosion and if it is not checked, may endanger the structure itself. It is therefore necessary that excess energy should be dissipated to such an extent that the river bed below the structure is not undermined by scouring action so as to endanger the stability of the structure.

The operation of any type of energy dissipator depends largely on expending a part of the energy of high velocity flow by some combination of the following methods.

(i) by external friction between the water and river bed and between the water and the air.

(ii) by internal friction and turbulence.

The energy dissipated by external friction between water and stream bed or water and air is usually negligible. The greatest amount of energy is lost by turbulent flow which dissipates the energy in surging, boiling and eddying, and greatly increasing the internal friction between particles of water. Thus the fundamental process is that the energy dissipator converts Kinetic energy into turbulence and finally into heat energy.

The choice of energy dissipation as stated earliar depends upon hydraulic, topographical, and geological condtions of site. Of all these the hydraulic condition is most predominent in the selection of particular energy dissipation arrangement. The topographical and geological condition generally affect the economic aspect of energy dissipator.

The energy dissipation methods, which are commonly employed for the dissipation of energy below dam spillways, are given below :-

(1) Hydraulic jump type of stilling basins with horizontal or sloping apron.

(2) Roller buckets.

(3) The skijump and Trajectory buckets.

The hydraulic jump type stilling basin is a structure in which the energy dissipation is accomplished by means of turbulent behaviour of well formed hydraulic jumps. The stilling basin may consist of fully horizontal apron or fully sloping apron or part sloping and part horizontal apron. The horizontal stilling basin is generally used where the available tailwater elevation match the sequent depth requirement for the formation of hydraulic jump for all range of discharges i.e. tailwater is adequate for the formation jump at all discharges. In cases where the tailwater depth available is more than required for formation of jump for certain range of discharges and less for certain range of discharges, the fully sloping or part sloping and part horizontal aprons are provided which ensure the formation of jump for all discharges.

The roller bucket type of energy dissipators are employed where the tailwater depth is more than required for the formation of jump. The roller bucket consists of bucket like apron with concave circular profile of appropriate radius and a lip which deflects the high velocity flow away from the stream bed. The sheet of water is deflected upwards forming two rollers i.e. a surface roller or high boil on water surface on the bucket and a ground roller downstream of the bucket. The surface roller is effective in dissipating energy while the ground roller provides the reverse flow in the upstream direction and prevents erosion immediately downstream of bucket. The bucket may be either solid type or slotted type. The slotted type of buckets were evolved as an improvement over the solid types. The slotted type of bucket has an advantage over the solid type as the provision of slot in bucket ensures self cleaning action and the material inside the bucket sweeps out through the slots.

The skijump or trajectory type of bucket are used where the tailwater is inadequate for the formation of hydralulic jump and the bed strata consists of fairly good rock. These types of buckets are designed to throw the high velocity flow leaving the spillway with such an angle that the water jet strikes the river bed at an adequate distance away from the structure. The striking jet of water scours the river bed and a pool gets created after sometime which acts later as cushion for the falling jet of water. The energy dissipation does not take place in the bucket itself but is brought about by dispersion of the jet in the atmosphere, resistance of air, diffusion of jet in the tailwater and finally the impact against the river bed. The important consideration for this type of energy dissipator as stated earliar, is that the bed strata should consist of fairly good rock so that the scour caused due to falling jet of water does not progressively move upstream so as to endanger the structure itself.

Besides the above types, some energy dissipators employing the principle of diffusion, and interacting jets have been developed. These types of dissipators have been used successfully in moderately high dam. However the fact remains that these types cannot be used in high dams where high discharge intensities are involved.

The present study has been limited to main type of energy dissipators discribed earliar. In the initial two chapters that follow, basic characteristics of hydraulic jump on horizontal and sloping apron have been discussed. Later the hydraulic considerations for the choice of specific energy dissipators have been dealt in detail. After that each type of energy dissipator has been discussed in detail in all its aspects.

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CHAPTER 2.

HYDRAULIC JUMP ON A HORIZONTAL APRON.

Among the various methods available for dissipation of energy, the hydraulic jump is one of the most effective means of dissipation of energy. Before discussing the extent of energy dissipation which can be accomplished through hydraulic jump, it is proposed to discuss the basic characteristics of the hydraulic jump.

The hydraulic jump is defined as the sudden and turbulent passage of water from a low stage below critical depth to high stage above critical depth, during which the velocity changes from supercritical to subcritical . The jump is accompanied by great turbulance and energy dissipation. The relation between supercritical depth and subcritical or sequent depth can be established with the help of Newton's second law of motion. The well known formulae showing the relationship between supercritical depth and subcritical depth is briefly derived here to clarify discussion.

The assumption made in the analysis are :-

(1) The channel is rectangular in shape with parallel sides.

(2) The floor is horizontal.

(3) The friction loss is negligible.

(4) Streamline flow before and after the jump.

The change of momentum/sec. between section 1 and 2 (Fig. 2-1) = $\rho q (V_2 - V_1)$ where ρ is mass density and q is discharge intensity.

According to Newton's second law of motion, change in hydrostatic pressure must be equal to rate of change in momentum.

$$P_1 - P_2 = \rho q (V_2 - V_1) \qquad \text{---------------------(1)}$$

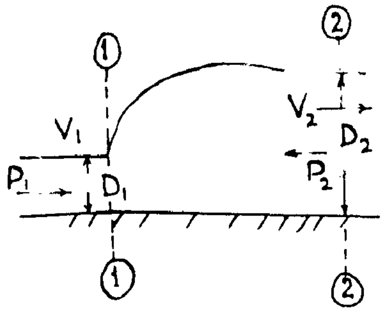
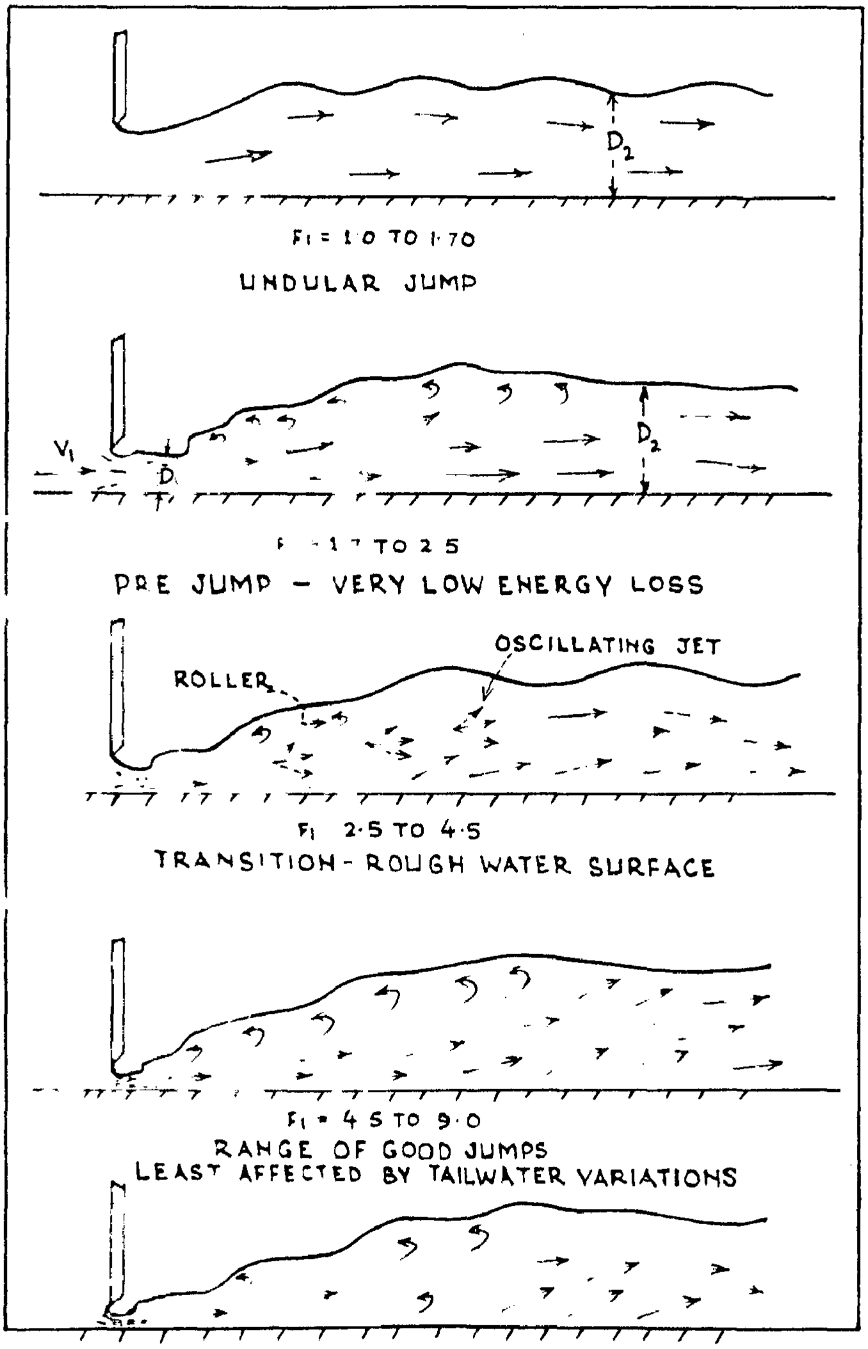


FIG. 2-1



F = 9.0 — UPWARD
EFFECTIVE BUT ROUGH
JUMP FORMS

FIG. 2-2

By making suitable substitution for P_1, P_2, V_1, V_2 in terms of D_1, D_2, q and solving for D_2 we get the well known expression.

$$D_2 = \frac{D_1}{2} + \sqrt{\frac{2q^2}{g\,D_1} + \frac{D_1^2}{4}} \qquad \text{-----------------------(2)}$$

As $\frac{V_1}{\sqrt{g\,D_1}} = F_1$ (Froude Number) the eq. (2) can be written as

$$D_2 = \frac{D_1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

or

$$\frac{D_2}{D_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) \qquad \text{-----------------------(3)}$$

FORMS OF JUMP.

The jumps have been classified according to Froude Numbers based on extensive experiments conducted by Bureau of Reclaimation[1] (Fig. 2-2). For $F_1 = 1$, Flow is critical and no jump takes place.

(i) $F_1 = 1$ to 1.7, the water surface shows a series of undulations and the jump is termed as undular jump.

(ii) $F_1 = 1.7$ to 2.5, a series of small rollers develop on the surface of the jump but the downstream water surface remains smooth. The velocity is generally uniform and the loss of energy is poor and the jump is termed as weak jump.

(iii) $F_1 = 2.5$ to 4.5, the entering jet osscillates from bottom to surface and back again with no regular period. The jump is termed as oscillating jump.

(iv) $F_1 = 4.5$ to 9, the downstream extremity of the surface roller and the point at which the high velocity jet tends to leave the floor occurs at practically the same vertical section. This jump is well balanced, least affected by tailwater variations. The jump is termed as steady jump. The energy dissipation ranges from 45 to 70%. The discussion on energy dissipation aspect of jump is done later in this chapter.

(v) $F_1 = 9$ and above, slugs of water rolling down the front face of the jump, intermittently fall into high velocity jet generating waves downstream and a rough surface can prevail. The energy dissipation may reach upto 85 % and the jump is termed as strong jump.

The limits of the Froude number given above for various forms of jump are not definite values but overlap some what depending upon the local factors. Very little is known about jumps having Froude numbers 16-20 because if physical size of the jump is increased sufficiently to provide reliable test data the jump is too costly to produce and study. The higher number jumps require large test facilities. Fast moving water and heavier measuring equipment which are seldom available.

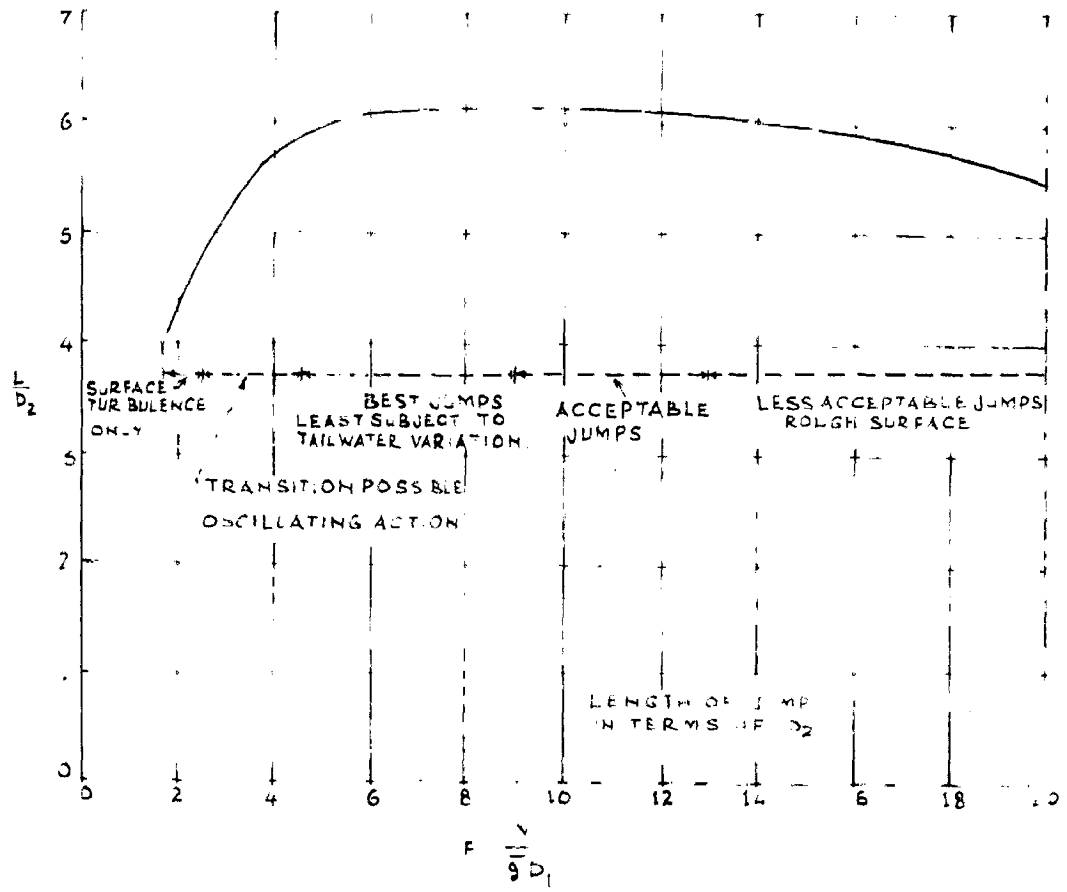
LENGTH OF JUMP.

The length of jump has not been evaluated analytically so far. All the formulae so far developed by various authors are emperical based on their own set of experiments. The formulae given by some authors are given in Table 2-1. In the last column the length of jump in terms of D_1 for a value Froude number equal to 6, have been worked out to show the difference due to application of various formulae.

Extensive experiments were carried out by Bradley and Peterka[2] for the length of jump for a good range of Froude numbers. The results of these tests have been presented in the shape of charts showing the plot between $\frac{L}{D_2}$ and F_1. The Fig, 2-3 shows that the value $\frac{L}{D_2}$ varies from 4 to 6.15 for various values

TABLE 2-1.

Author	Date	Formula	$\frac{L}{D_1}$ for $F_1 = 6$ ($D_2 = 8D_1$)
Safranez	1927	$\frac{L}{D_2} = 5.2$ (Approx.)	41.6
Bakhmeteff and Matzke	1932-33	$\frac{L}{D_2 - D_1} = 5$	35
Smetana	1934	$\frac{L}{D_2 - D_1} = 6$	42
Kinney	1935	$\frac{L}{D_2 - D_1} = 6.02$	42.14
Posey	1941	$\frac{L}{D_2 - D_1} = 4.5$ to 7	31.5 to 49
Wu	1949	$\frac{L}{D_2 - D_1} = \frac{10}{F_1^{0.16}}$	52.5
Woycicki	1934	$\frac{L}{D_2 - D_1} = 8 - 0.05\frac{D_2}{D_1}$	53.2
Ivanchenko	-	$\frac{L}{D_2 - D_1} = \frac{10.6}{F_1^{0.0925}}$	63
Chertoussov	1935	$\frac{L}{D_2 - D_1} = 10.3\,(F_1 - 1)^{0.81}$	38
Page	1935	$\frac{L}{D_2} = 5.6$	44.8
Riegel, Beebe	1917	$\frac{L}{D_2 - D_1} = 5$ (Approx.)	35
Aravin	1935	$\frac{L}{D_2 - D_1} = 5.4$	37.8
Bradley and Peterka	1957	$\frac{L}{D_2} =$ varies with F_1 (See Fig. 2-3)	49


SURFACE TURBULENCE ONLY
BEST JUMPS LEAST SUBJECT TO TAILWATER VARIATION
ACCEPTABLE JUMPS
LESS ACCEPTABLE JUMPS ROUGH SURFACE
TRANSITION POSSIBLE
OSCILLATING ACTION
LENGTH OF JUMP IN TERMS OF D_2
$\frac{L}{D_2}$

F G 2 3

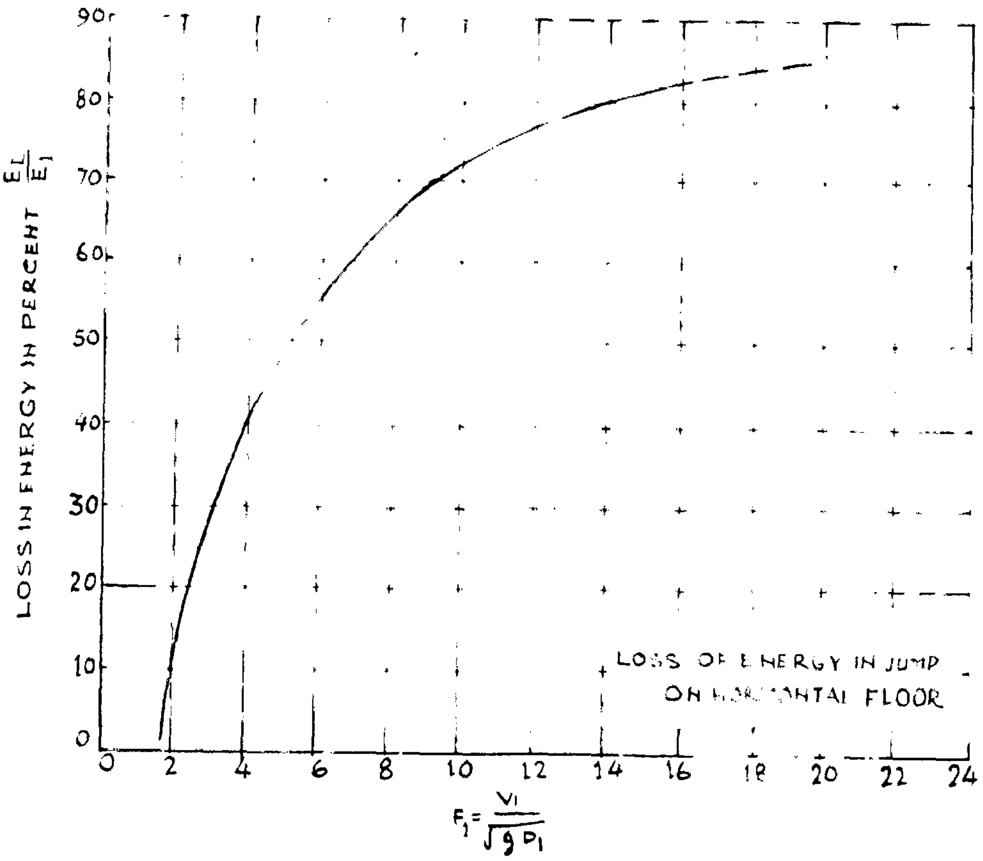

LOSS IN ENERGY IN PERCENT $\frac{E_L}{E_1}$
LOSS OF ENERGY IN JUMP ON HORIZONTAL FLOOR
$F_1 = \frac{V_1}{\sqrt{g D_1}}$

FIG 2-4

of F_1. The value of $\frac{L}{D_2}$ is greater than 6 for Froude number lying between 5 to 15 i.e. range in which jump gives generally satisfactory performance. If this value is compared in terms of last column of table 2-1, the $\frac{L}{D_1}$ comes to nearly 49.

The results of Bradley and Peterka differ from the results of Bakhmeteff and Matzke. It has been stated by Bradley and Peterka that the disagreement in results may be due to scale effect as the experiments by Bakhmeteff and Matzke were conducted in a 6" flume with maximum discharge of 0.7 cusecs and minimum discharge of 0.14 cusecs.

Further attempt has been made to give some rational formulae by Richard Silvester [3] who tried to develop a relation between parameters L/D_1, L/D_2 and $(L/D_2 - D_1)$.

$$\frac{L}{D_1} = \frac{L}{D_2} \cdot \frac{D_2}{D_1} = \frac{L}{D_2 - D_1}\left(\frac{D_2}{D_1} - 1\right) \quad \text{------------------------} (4)$$

In a rectangular channel

$$\frac{D_2}{D_1} = \frac{1}{2}\left(\sqrt{1+8\,F_1^{\,2}} - 1\right)$$

$$\frac{L}{D_1} = \frac{L}{D_2 - D_1} \cancel{x\,(\sqrt{2}F_1 - 1.5)} \left(\frac{1}{2}\sqrt{1+8F_1^{\,2}} - 1.5\right)$$

For small working value of $F_1 = 2$, $\frac{1}{2}\sqrt{1+8\,F_1^{\,2}} = \sqrt{2}\,F_1$

$$\frac{L}{D_1} = \frac{L}{D_2 - D_1} \times (\sqrt{2}F_1 - 1.5)$$

$$\frac{L}{D_1} = \frac{L}{D_2 - D_1} \times 1.414\,(F_1 - 1) \quad \text{------------------------} (5)$$

The author has conducted studies on various shapes of channels and it is seen that value of $\frac{D_2}{D_1}$ vary slightly for different shaped channels at F_1 equal to 2.

Thus the general equation can be put as

$$\frac{L}{D_1} = K\,(F_1 - 1)^{\alpha} \quad \text{------------------------} (6)$$

Where K is constant determined by experiment on a particular channel section. When $F_1 = 2$, $\frac{L}{D_1} = K$. The value of α is determined by the relationship of $(\frac{D_2}{D_1} - 1)$ to $(F_1 - 1)$ and is dependent on shape of channel. For rectangular channel α is equel to 1.01. The author further states that the results of experiments conducted by U.S.B.R. indicate that $\frac{L}{D_2 - D_1}$ is equel to 6.9. If this is substituted in equation (5), then equation (6) can be written as

$$\frac{L}{D_1} = 9.75\,(F_1 - 1)^{1.01} \qquad \text{-----------------------------(7)}$$

This equation gives results close to U.S.B.R. Results for F_1 upto 9 but for higher values of F_1, L/D_1 is greater than U.S.B.R. values.

ENERGY DISSIPATION IN A FREE JUMP.

The specific energy at section 1, $E_1 = D_1 + \frac{V_1^2}{2g}$

The specific energy at section 2, $E_2 = D_2 + \frac{V_2^2}{2g}$

Energy loss $E_L = E_1 - E_2 = \frac{V_1^2}{2g} - \frac{V_2^2}{2g} - (D_2 - D_1)$

$$= \frac{q^2}{2g}\left\{\frac{1}{D_1^2} - \frac{1}{D_2^2}\right\} - (D_2 - D_1) \qquad \text{---------------------(8)}$$

By simplification and substitution

$$E_L = \frac{(D_1 - D_2)^3}{4\,D_1\,D_2} \qquad \text{-----------------------------(9)}$$

The loss can also be expressed as a percentage of initial energy

$$= \frac{E_L \times 100}{E_1}$$

After computing the energy loss for different value of F_1, a plot between F_1 and E_L / E_1 has been prepared by U.S.B.R.[1] and the same is shown in Fig. 2-4. From the plot it is seen that energy loss varies from 45% to 70% for values of F_1 ranging from 4.5 to 9.

ENERGY DISSIPATION IN A SUBMERGED JUMP.

The loss of energy in a submerged jump was investigated by GovindaRao and Rajaratnam[5] and according to these authors it has not yet been conclusively established that whether submerged jump is desireable as an energy dissipator. Smetana has claimed that the energy loss in a submerged jump is greater than the corresponding free jump and loss increases with submergence. The views of GovindRao and Rajratnam differ greatly with that of Smetana.

The expression for the loss of energy as derived by Govindrao and Rajaratnam, is given below. The submergence factor is defined as :-

$$S = \frac{D_4 - D_2}{D_2}$$

Where D_4 is the actual tailwater depth (Fig.2-5).

Let $\psi = \frac{D_3}{D_1}$, where D_3 is backed up waterdepth due to submergence at inlet section.

The energy at efflux section is given as

$$E_1 = D_3 + \frac{V_1^2}{2g}$$

$$\text{or} \quad \frac{E_1}{D_1} = \frac{D_3}{D_1} + \frac{V_1^2}{2g\,D_1} = \psi + \frac{F_1^2}{2} \quad \text{------------(10)}$$

Energy at the end of jump, $E_4 = D_4 + \frac{V_4^2}{2g}$, where V_4 is mean velocity at the end of submerged jump.

Energy loss E_L is given as

$$E_L = E_1 - E_4$$

$$\text{or} \quad \frac{E_L}{E_1} = 1 - \frac{E_4}{E_1}$$

$$= 1 - \frac{\left[(1+S)D_2 + \frac{1}{2g}\frac{q^2}{(1+S)^2 D_2^2}\right]}{E_1} \quad \text{---------- (11)}$$

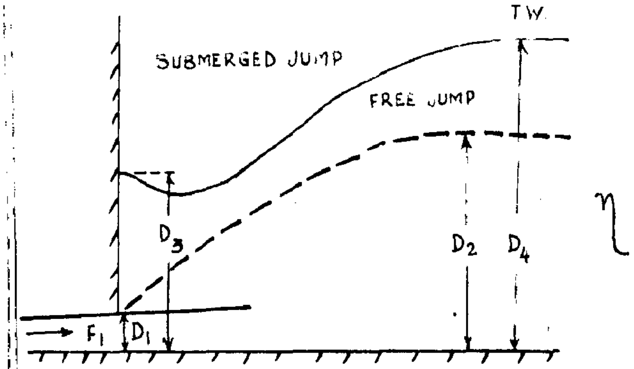
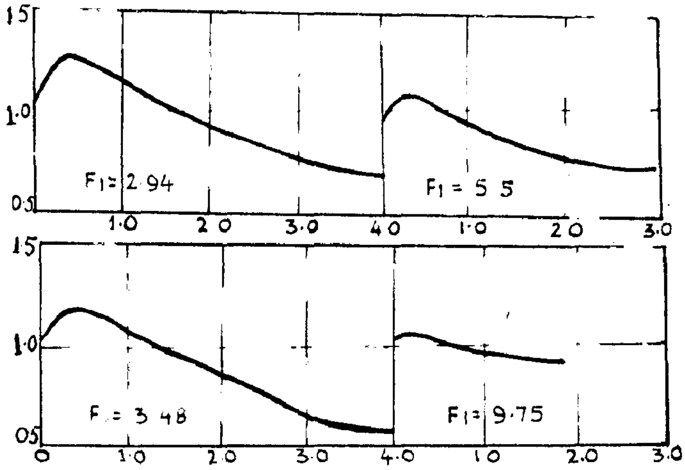


FIG. 2·5



SUBMERGENCE FACTOR

FIG. 2-6

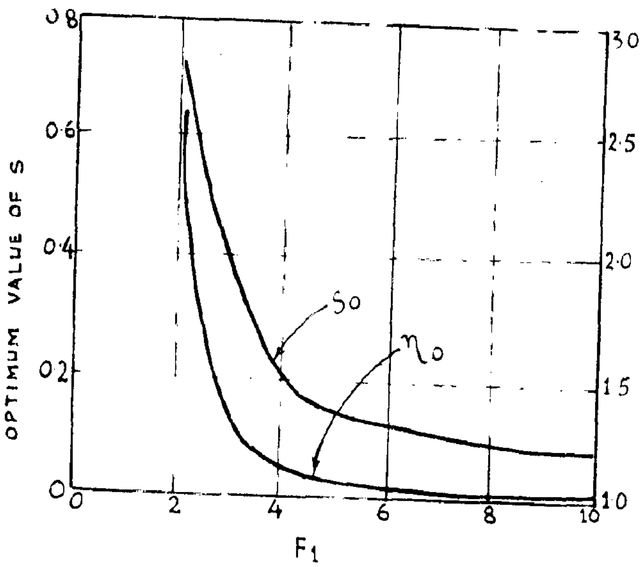


FIG. 2-7

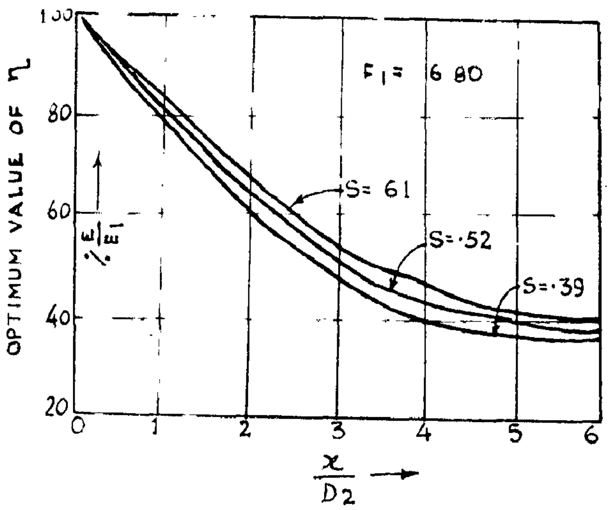


FIG. 2-8

by substituting $\phi = \frac{D_2}{D_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$,

and ~~simplifying~~ simplifying

$$\frac{E_L}{E_1} = \frac{\left[\psi - (1+S)\phi\right] + \frac{F_1^2}{2}\left[1 - \frac{1}{(1+S)^2\phi^2}\right]}{\left(\psi + \frac{F_1^2}{2}\right)} \quad \text{-------(12)}$$

In terms of F_1 and S

$$\frac{E_L}{E_1} = \frac{\left[(1+S)^2 \cdot \frac{1}{4}\left(\sqrt{1+8F_1^2}-1\right)^2 - 2F_1^2 + \frac{4F_1^2}{(1+S)\left(\sqrt{1+8F_1^2}-1\right)}\right]^{1/2} - \left[\frac{1+S}{2}\left(\sqrt{1+8F_1^2}-1\right)\right] + \frac{F_1^2}{2}\left[1 - \frac{4}{(1+S)^2\left(\sqrt{1+8F_1^2}-1\right)^2}\right]}{\left[\frac{(1+S)^2}{4}\left(\sqrt{1+8F_1^2}-1\right)^2 - 2F_1^2 + \frac{4F_1^2}{(1+S)\left(\sqrt{1+8F_1^2}-1\right)}\right]^{1/2} + \frac{F_1^2}{2}}$$

Thus $\frac{E_L}{E_1} = f(F_1, S)$ -------------------------------(13)

In order to verify the eq.(13), experiments were conducted by authors and the values of $\frac{E_L}{E_1}$ were plotted with theoretical values of E_L/E_1 and the correlation was found to be satisfactory.

PERFORMANCE OF SUBMERGED JUMP AS COMPARED TO FREE JUMP.

According to Govindarao and Rajaratnam[5] the energy dissipation capacity of submerged jump is seen to be more or less same as that of corresponding free jump depending upon the value of F_1 and S.

If η is an index of relative energy dissipation then it can be expressed as

$$\eta = \frac{\left(\frac{E_L}{E_1}\right) \text{ submerged jump}}{\left(\frac{E_L}{E_1}\right) \text{ free jump.}} \quad \text{-------------(14)}$$

η could be more or less than unity depending upon F_1 and S. η varies from 0.55 to 1.25 for the experimental range studied by authors (Fig.2-6). The optimum value of S for a given F_1 at which η will be maximum can be found out by partially differentiating eq (14) with respect to S and equating it to zero. The equation so arrived is complicated and have been solved by indirect methods. If S_o is optimum value of S then it is seen that S_o decreases rapidly from 0.7 to 0.1 as F_1 varies from 2.0 to 7.0 and remains constant for higher values of S. If η_o is maximum value of η, it is seen that η_o also decreases rapidly from 2.75 to 1.05 as F_1 varies from 2.0 to 6.0 after that it is constant for higher values of F_1 (Fig. 2-7).

The studies by the above authors have established that energy loss in a submerged jump computed above, occurs in a much longer length than corresponding free jump. In order to study the manner in which energy dissipation is affected by submergence, the energy of the expanding stream was computed at suitable intervals. The effect of submergence on energy dissipation is shown in Fig.2-8 where E denotes the specific energy at any distance X from the start of jump. It is seen that the fall of energy is retarded/continuously by the increasing submergence. From these studies the authors have drawn the conclusion that submerged jump should not be preferred to free jump for energy dissipation purposes unless the submergence factor is less than 10% for Froude numbers higher than nearly 5.

In the above paragraphs the characteristics of hydraulic jump on a horizontal apron have been discussed. The loss of energy in a clear jump, submerged jump and their relative effectiveness have been broughtout. The hydraulic jump has been extensively used for dissipation of energy and design of various types of stilling basins have been developed. The design considerations for different type of stilling basins and suitability for a particular situation have been discussed in subsequent chapters.

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CHAPTER 3.

HYDRAULIC JUMP ON A SLOPING APRON.

All the physical laws applicable to hydraulic jump on a horizontal apron are also applicable to jump on a sloping apron. Only the extra force of gravity of the hydraulic jump body needs be considered for analysing the jump on sloping apron. The jump on sloping apron was earliar studied by Riegel and Beebe in 1917 , Yarnell in 1934, Rindlaub in 1935, Backmeteff and Matzke in 1936. Kindsvater made extensive studies in 1942. Recently Bradley and Peterka[2] have made considerable experimental studies which are of great value in design of stilling basins. The basic characteristics of jump on sloping apron are discussed in this chapter.

FORMS OF JUMP.

There are basically four forms of jumps possible on a sloping apron (Fig.3-1). These are described below-

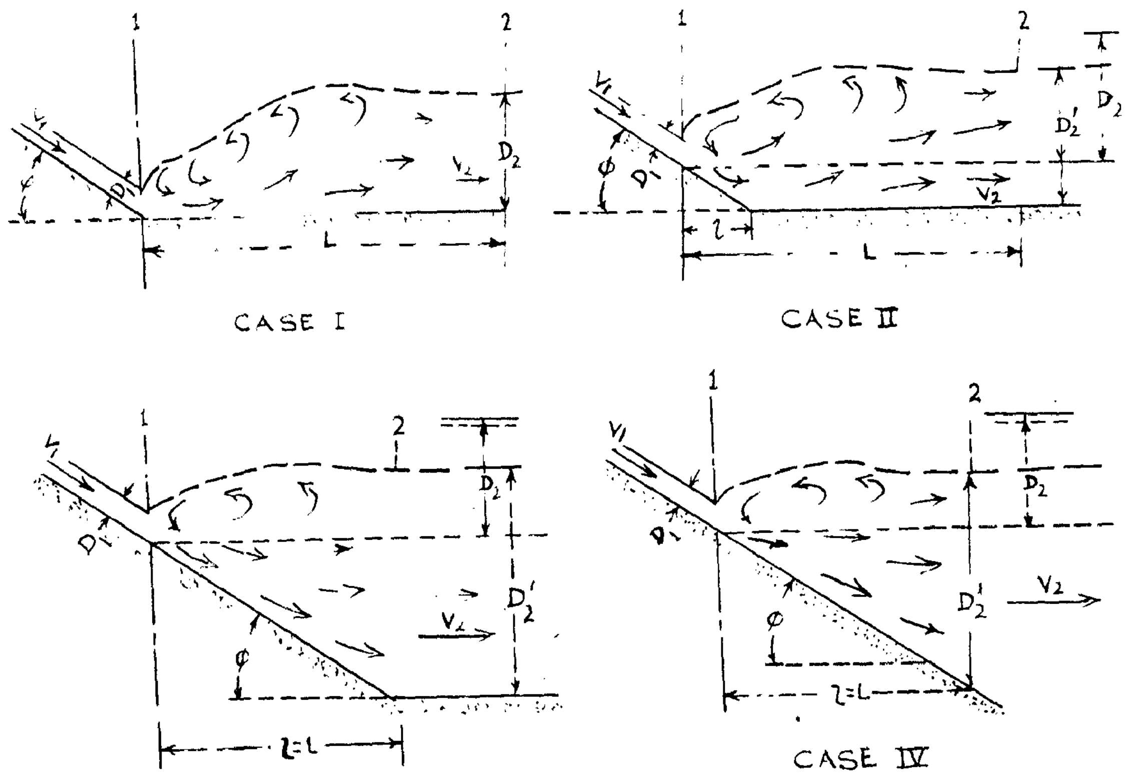
<u>Case I</u> : The jump toe is just at the junction of sloping apron with horizontal apron and jump end lies on horizontal apron.

<u>Case II</u> : The jump toe is on slope and the end on horizontal apron.

<u>Case III</u>: The jump toe forms on slope and the end of jump at the junction of sloping and horizontal apron.

<u>Case IV</u> : The entire jump lies on sloping apron.

In the above classifications it is seen that case I is like a jump on a horizontal apron which has been dealt in detail in the last chapter. Case III and Case IV are practically the same. The studies by above mentioned authors were done mostly for case IV. Extensive studies on case II and IV have been done



SLOPING APRONS
FIG. 3-1

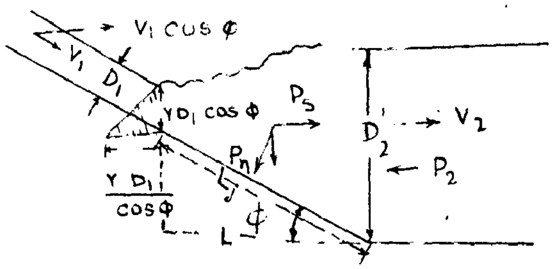
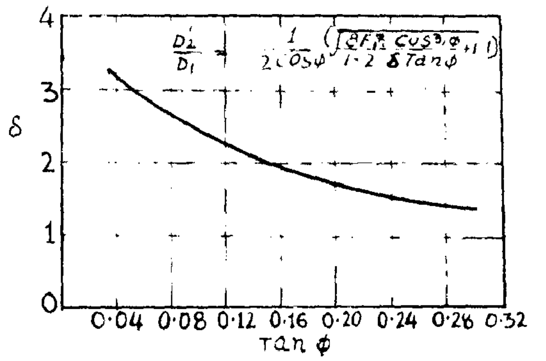


FIG. 3-2



ABOVE CURVE IS BASED ON ASSUMPTION THAT δ IS INDEPENDENT OF F_1

SHAPE FACTOR δ
FIG. 3-3

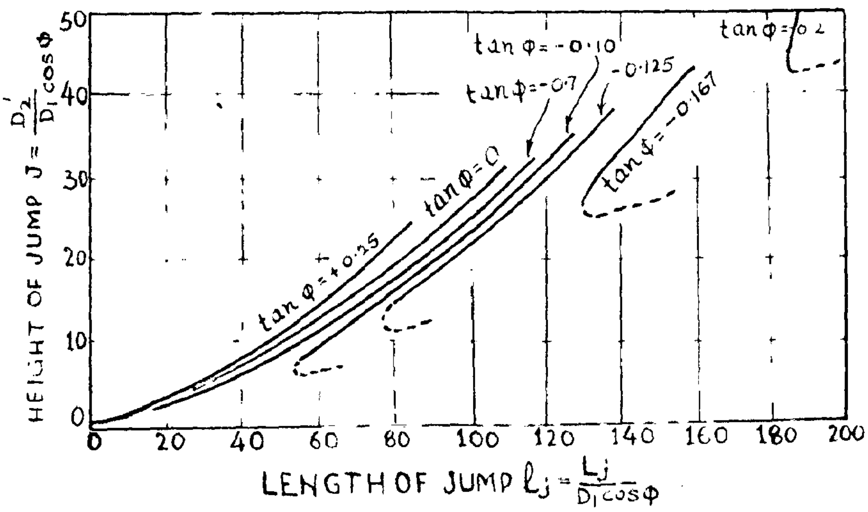


FIG. 3-4

by Bradley and Peterka[2]. The full significance of the studies of these two cases is dealt in a subsequent chapter while discussing the design of stilling basins with sloping apron.

The jump analysis for case IV has been made by Kindsvater[6] after ~~kei~~ making the following assumptions:

(i) There is no appreciable curvature of stream lines.

(ii) The friction at boundary is negligible.

(iii) The velocities before and after the jump are uniform.

(iv) There is no acceleration of stream flow (this is not true for steep slopes).

(v) The air entrainment is neglected.

Applying pressure momentum relationship and referring to Fig. 3-2,

$$P_2 - P_1 - P_s = \frac{\gamma Q}{g}\,(V_1 \cos\phi - V_2) \quad \text{----------------(1)}$$

From continuity $V_1\, D_1 = V_2\, D'_2$

$$V_2 = \frac{V_1 D_1}{D'_2} = \frac{\left(V_1 \cdot \cos\phi \cdot \frac{D_1}{\cos\phi}\right)}{D'_2}$$

The pressure P_1 and P_s for unit width are

$$P_1 = \left(\frac{D_1}{\cos\phi} \cdot \frac{\gamma D_1 \cos\phi}{2}\right) = \frac{\gamma D_1^2}{2}$$

$$P_2 = \frac{\gamma D_2'^2}{2}$$

P_1 can also be approximately written as

$$P_1 = \frac{\gamma D_1^2}{2\cos^2\phi}$$

because the difference due to slope will not be much.

P_s is the horizontal force due to pressure of weight of the jump. The experiments indicate that pressure on sloping floor is proportional to weight of fluid body on the slope. P_s can be expressed in dimensionless parameters

$$P_s = \delta\left[\gamma D_2 - \frac{\gamma D_1}{\cos^2\phi}\right]\tan\phi \quad \text{------------ (2)}$$

According to Kindswater δ varies with Kinetic flow factor i.e. square of Froude number. Eq(1) can be written after substituting the values on P_1, P_2, and P_s.

$$\frac{\gamma D_2'^2}{2} - \frac{\gamma D_1^2}{2\cos^2\phi} - \delta\left(\gamma D_2'^2 - \frac{\gamma D_1^2}{\cos^2\phi}\right)\tan\phi = \frac{\gamma V_1^2 D_1\cos\phi}{g\, D_2'}\left(D_2' - \frac{D_1}{\cos\phi}\right) \quad \text{---(3)}$$

simplifying by dividing both sides by $\frac{\gamma}{2}\left(D_2' - \frac{D_1}{\cos\phi}\right)$

$$\left(D_2' + \frac{D_1}{\cos\phi}\right) - 2\delta\left(D_2' + \frac{D_1}{\cos\phi}\right)\tan\phi = \frac{2V_1^2 D_1\cos\phi}{g\,D_2'} \quad \text{---(4)}$$

$$\text{or} \quad D_2' = \frac{D_1}{2\cos\phi}\left[\sqrt{\frac{8V_1^2\cos^3\phi}{g D_1(1-2\delta\tan\phi)}+1} - 1\right] \quad \text{---------- (5)}$$

Replacing $\frac{V_1^2}{gD_1} = F_1^2 = \lambda$

$$D_2' = \frac{D_1}{2\cos\phi}\left(\sqrt{\frac{8\lambda\cos^3\phi}{1-2\delta\tan\phi}+1} - 1\right) \quad \text{---------- (6)}$$

Thus sequent depth D_2' after the jump can be computed.

PRESSURE COEFFICIENT.

If P_n is normal pressure at any point on the slope then P_n in equal to γy, where y is vertical height of pressure gradient above the floor. If $\bar{y}$ is the average height of pressure gradient over the length of slope $\frac{L}{\cos\phi}$ the total normal pressure per unit width

$$P_n = \frac{\gamma\bar{y}L}{\cos\phi} \quad \text{-------------- (7)}$$

$\bar{y}L$ is the area of pressure Diagram, shown by experiments to be proportional to jump body. Ps the horizontal component of Pn is given by

$$P_s = P_n \sin\phi = \gamma\bar{y}L\tan\phi \quad \text{-------------------- (8)}$$

from eq (2) and (8)

$$\delta = \frac{\bar{y}L}{D_2'^2 - \frac{D_1^2}{\cos^2\phi}}$$

A plot of experimental data from tests on 1 on 6 slope, indicate that δ is a function of Kinetic flow factor λ. The curve in the range of $\lambda = 5$ to 50, is nearly a straight line having an equation

$$\delta = 2.58 - 0.021 \lambda$$

The experiments conducted by G.H. Hickox[7] on 1 on 3 slope also established a similar relation between δ and λ. He utilised the curve of Kindsvater for interpolating the curves showing the relationship between δ and λ for intermediate slopes. For horizontal apron the data of Bakhmeteff and Matzke was utilised.

Later on experiments were conducted by Bradley and Peterka[2] on extensive scale on different slopes. These experiments established that the Froude number has little effect on the value of δ which varies with the slope of apron only. The value of δ was computed from equation (6) by substituting experimental values in the equation. The value of so computed by Bradley and Peterka, indicated striking disagree-ment with the values of Kindsvater and Hickox. This may be due to the fact that the values of δ is dependent on the method used for determining the length of jump. The average values of δ for different slopes are indicated in figure 3-3.

LENGTH OF JUMP.

The length of jump has always been a subject of controversy between different authorities working on this aspect. This controversy persists regarding location of end of jump, i.e. whether the end of jump should be taken at highest water surface or at the point where velocity distribution becomes normal. According to Stevens[8] the length of jump should be so chosen that the energy conversion would be completed as indexed by the maximum height of water surface. In the experiments conducted by U.S.B.R.[1] the end of jump was chosen at the point where high velocity jet began to leave the floor or at a point on level tailwater surface just downstream from the surface roller, whichever was longer.

The length of jump for sloping apron as given by Stevens[8] on the basis of formula of Bakhmeteff and Matzka, is expressed in dimensionless form

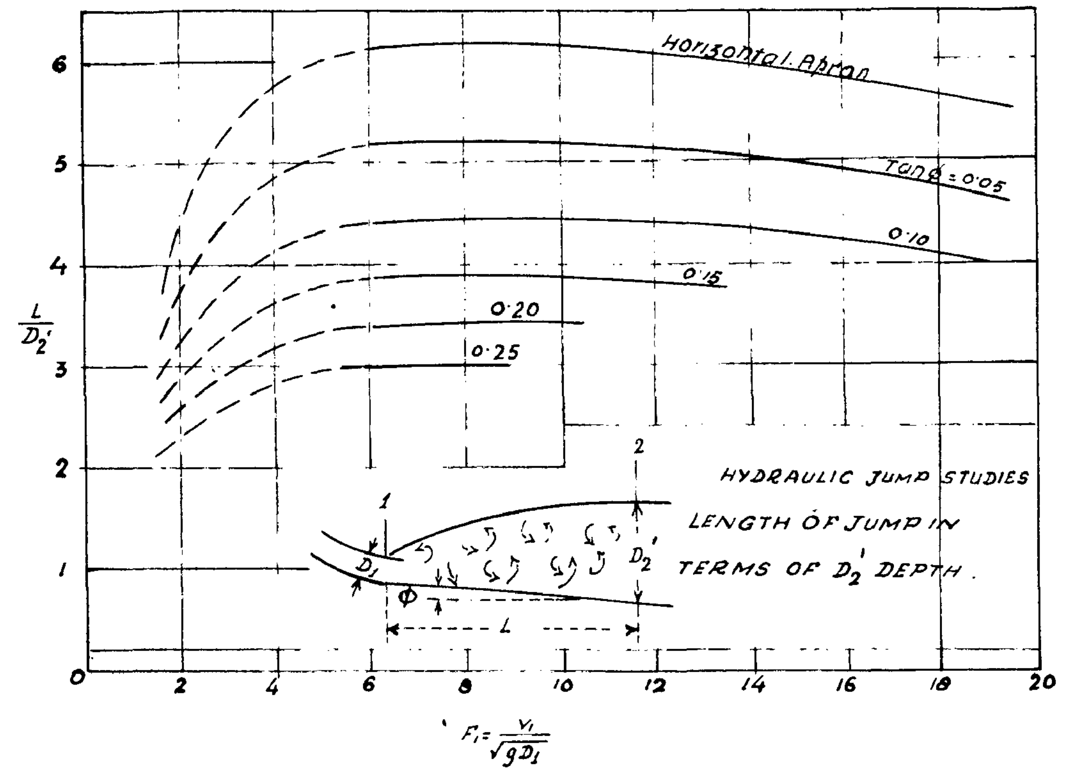
$$l_j = 9.3\, K^{-0.185} \left(J - \frac{1}{\cos\phi} \right) \qquad (10)$$

where $l_j = \frac{Lj}{D_1 \cos\phi}$, $K = \frac{V_1^2}{2g D_1 \cos\phi}$ and $J = \frac{D_2}{D_1 \cos\phi}$

The curves showing the relation between J and l_j for various slopes positive as well as negative, are given in Fig. 3-4.

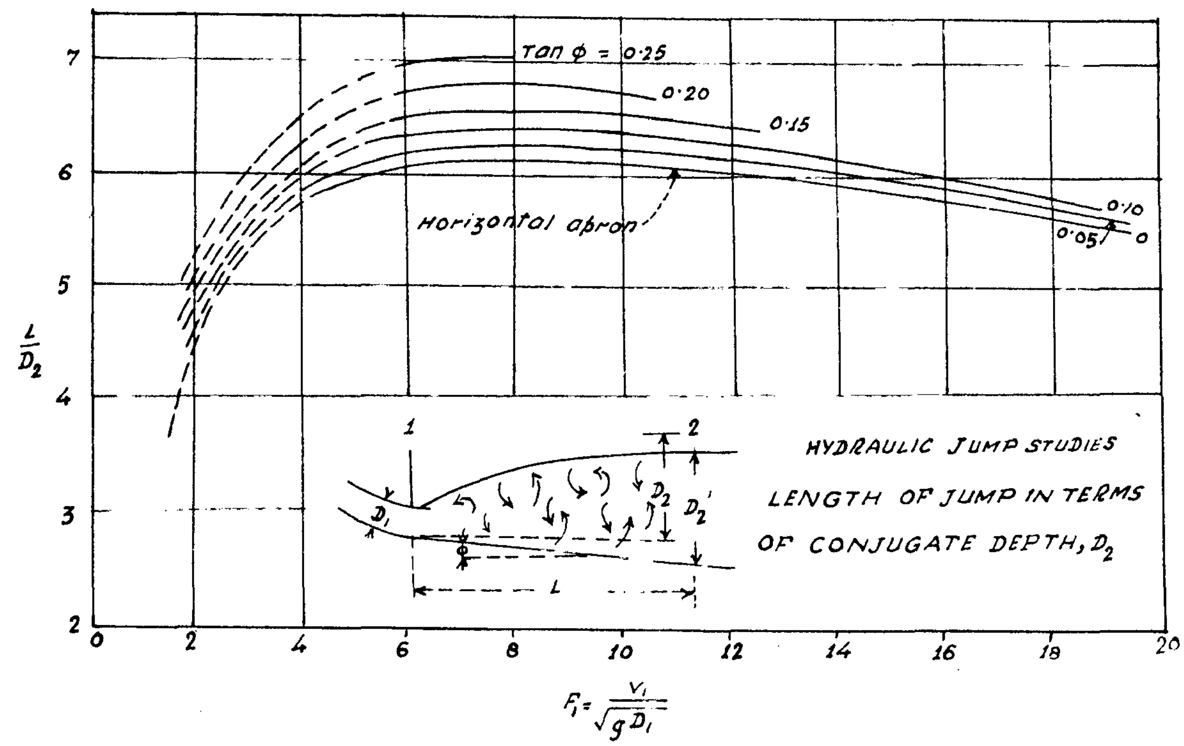
The investigations conducted by U.S.B.R.[1] have established that length of jump is a function of Froude number and slope of apron. The curves showing relation between F_1 and $\frac{L}{D_2'}$; F_1 and $\frac{L}{D_2}$ for different slopes, have been plotted based on the results of experiments and are shown in Fig. 3-5 and 3-6. A study of these curves show that the ratio $\frac{L}{D_2}$ varies from 5.6 to 7.0 for various value of slopes and Froude numbers. The maximum value of $\frac{L}{D_2}$ occurs for all slopes between Froude number 5 to 10 approximately. For lower as well as higher values of Froude number, $\frac{L}{D_2}$ decreases.

Elevatorski[9], on the basis of experimental data of U.S.B.R., gives the following formulaes for the length of jump for different slopes in terms of height of jump. According to author ~~numbers vary~~ these are applicable for Froude numbers varying from 2.9 to 17.9.



Horizontal Apron
Tanφ = 0·05
0·10
0·15
0·20
0·25
HYDRAULIC JUMP STUDIES
LENGTH OF JUMP IN
TERMS OF D_2' DEPTH.
$\frac{L}{D_2'}$
$F_1 = \frac{V_1}{\sqrt{gD_1}}$

FIG. 3-5



Tan φ = 0·25
0·20
0·15
0·10
0·05
0
Horizontal apron
HYDRAULIC JUMP STUDIES
LENGTH OF JUMP IN TERMS
OF CONJUGATE DEPTH, D_2
$\frac{L}{D_2}$
$F_1 = \frac{V_1}{\sqrt{gD_1}}$

FIG. 3-6

Slope.	Length of jump.
Horizontal	6.9 $(D_2 - D_1)$
1 in 20	5.2 $(D'_2 - D_1)$
1 in 10	4.4 $(D'_2 - D_1)$
1 in 6.7	3.8 $(D'_2 - D_1)$
1 in 5.0	3.25 $(D'_2 - D_1)$
1 in 3.6	2.75 $(D'_2 - D_1)$

Another formulae for the length of jump given by Solano Vega Vischi[10] on the basis of study of available experimental data,

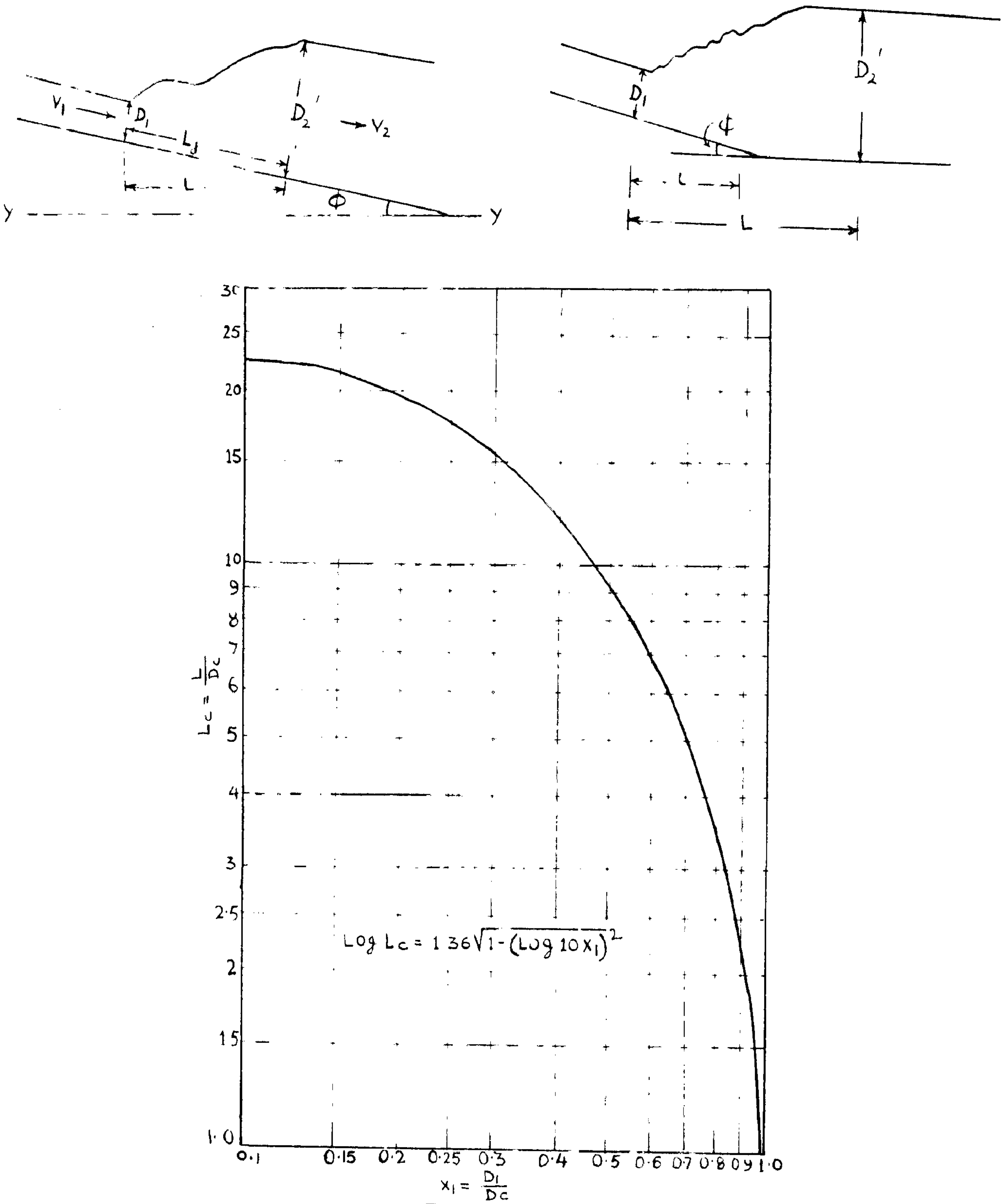
$$\log Lc = 1.36 \sqrt{1-(\log_{10} X_1)^2} \qquad \text{------------(11)}$$

where $Lc = \frac{L}{Dc}$, Dc being the critical depth

$$X_1 = \frac{D_1}{D_c} = \frac{V_c}{V_1} = \frac{\sqrt{gD_c}}{F_1\sqrt{gD_1}} = \frac{1}{F_1}\sqrt{\frac{1}{X_1}} \quad , \text{ or } \quad X_1 = F_1^{-2/3}$$

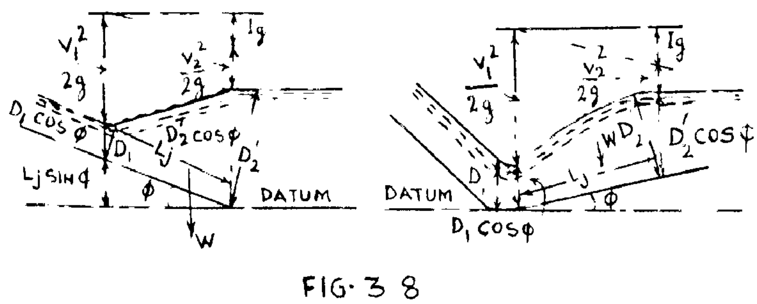
The plot between Lc and X_1 is shown in Fig. 3-7.

The above formulae is applicable for value of X_1 equel to 0.10_0.70, which is generally the range of stable jumps. For X_1 equel to 0.70(F_1=1.7) the jump loses its stability and length cannot be defined. This formulae is true for values of slopes from 0.05 to 0.28. It is seen that this formulae does not contain any variable defining slope which means that the length of jump is independent of slope. The author claims that the results computed by this formulae donot exceed by more than 5.7 % of actual values of jump length obtained in U.S.B.R. tests. This variation is within the permissible range which can be expected in experimental results. Bradley and Peterka[11] have tried to show that the individual results computed from Vega's formulae differ much more from the experimental values than claimed by the later. These authors have also pointed out that



V_1
D_1
L_j
D_2'
V_2
L
ϕ
Y
Y
D_1
D_2'
ϕ
l
L
30
25
20
15
10
9
8
7
6
5
4
3
2.5
2
1.5
1.0
$L_c = \frac{L}{D_c}$
$\text{Log } L_c = 1.36\sqrt{1-(\text{Log } 10X_1)^2}$
0.1
0.15
0.2
0.25
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
$X_1 = \frac{D_1}{D_c}$

FIG. 3-7



$\frac{V_1^2}{2g}$
$\frac{V_2^2}{2g}$
I_g
$D_1 \cos\phi$
D_1
$D_2 \cos\phi$
L_j
D_2'
$L_j \sin\phi$
ϕ
W
DATUM
DATUM
$\frac{V_1^2}{2g}$
$\frac{V_2^2}{2g}$
I_g
WD_2
$D_2' \cos\phi$
D_1
L_j
ϕ
$D_1 \cos\phi$

FIG. 3 8

the Vega's formulae is based on the average curve drawn by utilising the experimental data for various slopes. This is the reason for variation in results computed from Vega's formulae and the actual values obtained by experiments. In view of the above, Bradley and Peterka have questioned the conclusion drawn by Vega that length of jump is independent of slope, as the experiments conducted by them have established that the length of jump does depend on the slope.

ENERGY LOSS IN JUMP.

The extent of loss of energy in hydraulic jump on sloping apron has been evaluated by Stevens[8]. He has given the formulae for both negative and positive slopes of apron. Before proceeding for derivation of the expression for the energy loss, it is necessary to study the formulae derived by Stevens for the solution of hydraulic jump.

In each case whether the slope is positive or negative momentum relationship for unit width of a rectangular channel can be expressed as below (Fig. 3-8).

For negative slopes (downward direction of flow)

$$\frac{q}{g} V_1 + \frac{D_1^2}{2}\cos\phi + W\sin\phi = \frac{q}{g} V_2 + \frac{D_2'^2}{2}\cos\phi \text{ ------ } (12a)$$

for positive slopes

$$\frac{q}{g} V_1 + \frac{D_1^2}{2}\cos\phi = \frac{q}{g} V_2 + \frac{D_2'^2}{2}\cos\phi + W\sin\phi \text{ ---- } (12b)$$

where W is the weight of jump body. The weight of jump body for unit width is given by

$$W = 10.6\, \lambda^{-0.185} \times \tfrac{1}{2}\,(D_2'^2 - D_1^2) \text{ -------------- } (13)$$

This weight has been arrived at by considering the straight line profile of the jump surface. The jump surface profile has been assumed a straight line and curvature of surface profile is ignored as the profile is quite uncertain and not much error in weight of jump body is likely to be caused. The length of jump has been assumed on the basis of formulae given by Ivanchenko for horizontal apron

i.e. $L = 10.6 \lambda^{-0.185}(D'_2 - D_1)$

In the above expression $\lambda = F_1^2$. If $K = \frac{\lambda}{2}$ then eq (13) can be written as

$$W = \frac{9.3}{K^{0.185}} \frac{(D_2'^2 - D_1^2)}{2} \qquad \text{------------------------(13a)}$$

Substituting eq (13a) in eq. (12a) and (12b) and simplifying

$$\frac{q}{g}(V_1 - V_2) = \frac{1}{2}(D_2'^2 - D_1^2)(\cos\phi \pm m \sin\phi) \qquad \text{--------------(14)}$$

where $m = \frac{9.3}{K^{0.185}}$ and positive sign to be used for upward slopes and negative sign for downward slopes.

Putting $K = \frac{V_1^2}{2g\, D_1 \cos\phi}$ and $J = \frac{D'_2}{D_1 \cos\phi}$ and substituting in eq (14) and using continuity equation $q = V_1 D_1 = V_2 D'_2$, the equation can be written as

$$J\ (J \cos\phi + 1) = \frac{4K}{\cos\phi \pm m \sin\phi} \qquad \text{----------------(15)}$$

The final depth is given by

$$J = \frac{1}{2\cos\phi}\left(-1 + \sqrt{\frac{16K}{1 \pm m\tan\phi} + 1}\right) \qquad \text{----------------(15a)}$$

The loss of energy in jump can be expressed in two ways:-

(1) specific energy loss with reference to bed of stream.

(2) Geodetic Energy loss i.e. with reference to horizontal datum.

Specific Energy Loss.

The specific energy equation is given by

$$D_1 \cos\phi + \frac{V_1^2}{2g} = D'_2 \cos\phi + \frac{V_2^2}{2g} + I_s \qquad \text{----------------(16)}$$

where Is is the specific energy loss in jump.

$$\text{or} \quad \frac{V_1^2}{2g} = \frac{D_2'^2}{(D_1 + D'_2)} \cos\phi + \frac{D_2'^2}{(D_2'^2 - D_1^2)} I_s \qquad \text{--------------(17)}$$

if Is is expressed in terms of initial potential energy, then

$$i_s = \frac{I_s}{D_1 \cos\phi}$$

Substituting this in eq (17)

$$\frac{V_1^2}{2gD_1\cos\phi} = \frac{D_2'^2}{(D_2'+D_1)D_1} + \frac{D_2'^2}{(D_2'^2-D_1^2)} \cdot i_s \quad \text{-------------(17a)}$$

This equation can be written in terms of K and J as below

$$K = \frac{J^2\cos^2\phi}{J\cos\phi+1} + i_s \cdot \frac{J^2\cos^2\phi}{J^2\cos^2\phi - 1} \quad \text{-----------------(18)}$$

eleminating K using eq (15) and eq (18), i_s can be written as

$$i_s = \frac{(J\cos\phi-1)^3}{4J\cos\phi} \pm \frac{(J^2\cos^2\phi-1)(J\cos\phi+1)}{4J\cos\phi}\tan\phi \quad \text{---(19)}$$

For level apron $\phi = 0$ and equation (19) reduces to

$$i_s = \frac{(J-1)^3}{4J} \quad \text{-------------------------------(19a)}$$

Geodetic Energy Loss.

Let the plane of reference be horizontal plane passing through the bottom of D_1 for positive slopes and at the bottom of D_2 for negative slopes. The geodetic energy equation for positive slope is

$$D_1\cos\phi + \frac{V_1^2}{2g} = D_2'\cos\phi + \frac{V_2^2}{2g} + (Lj\sin\phi + I_g) \quad \text{-----(20a)}$$

for negative slope

$$Lj\sin\phi + D_1\cos\phi + \frac{V_1^2}{2g} = D_2'\cos\phi + \frac{V_2^2}{2g} + I_g \quad \text{-------(20b)}$$

where I_g = Geodetic energy loss.

Substituting the value of Lj in terms of $(D_2' - D_1)$ i.e. $Lj = m(D_2' - D_1)$ in eq (20) then

$$\frac{V_1^2}{2g} - \frac{V_2^2}{2g} = (D_2' - D_1)\cos\phi \pm m(D_2' - D_1)\sin\phi + I_g \quad \text{----(21)}$$

$$\text{or} \quad \frac{V_1^2}{2g}\left(1 - \frac{D_1^2}{D_2'^2}\right) = (D_2' - D_1)\cos\phi \pm m(D_2' - D_1)\sin\phi + I_g$$

$$\text{or} \quad \frac{V_1^2}{2g} = (\cos\phi \pm m\sin\phi)\frac{D_2'^2}{(D_2'+D_1)} + \frac{D_2'^2}{(D_2'^2-D_1^2)} \cdot I_g \quad \text{--(21a)}$$

Substituting I_g in terms of initial potential energy i.e. $i_g = \frac{I_g}{D_1 \cos\phi}$ and putting in terms of K, we get

$$K = \frac{J^2 \cos\phi}{J\cos\phi + 1}\,(1 \pm m \tan\phi) + i_g \frac{J^2\cos^2\phi}{J^2\cos^2\phi - 1} \quad \text{--------} (22)$$

eliminating K with help of eq (15)

$$i_g = \left(\frac{J\cos\phi - 1}{4J\cos\phi}\right)^3 (1 \pm m\tan\phi) \quad \text{--------} (23)$$

For only when $\phi = 0$, the eq (23) reduces to $i_g = \frac{(J-1)^3}{4J}$ -----(23a)

For any value of K, J can be found out from eq (15a) and knowing J the value of i_g can be computed from eq (23). The I_g can be determined by multiplying i_g with initial potential energy. In case of jump on horizontal apron the specific energy loss and geodetic energy loss are obviously one and the same which is also clear from eq (19a) and eq (23a).

The geodetic energy loss can also be expressed as percentage of initial energy

$$\%\ \text{loss} = \frac{100\, I_g}{L_j \sin\phi + D_1\cos\phi + \frac{V_1^2}{2g}} \quad \text{--------} (24)$$

substituting $l_j = \frac{L_j}{D_1 \cos\phi}$ i.e. length of jump interms of initial potential energy and I_g in terms of i_g

$$\%\ \text{loss} = \frac{i_g . D_1 \cos\phi \times 100}{l_j . D_1 \cos\phi . \sin\phi + D_1 . \cos\phi + K\, D_1 \cos\phi}$$

$$= \frac{i_g \times 100}{l_j \sin\phi + 1 + K} \quad \text{--------} (24a)$$

Fig. 3-9 shows the plot between J and percentage loss of in terms of initial energy for different slopes. It is seen that percentage loss increases for a given height of jump as negative slope diminishes. The fact that percentage loss varies with slope and Kineticity K is clear from Fig. 3-10 which shows the plot between percentage loss against apron slope for various

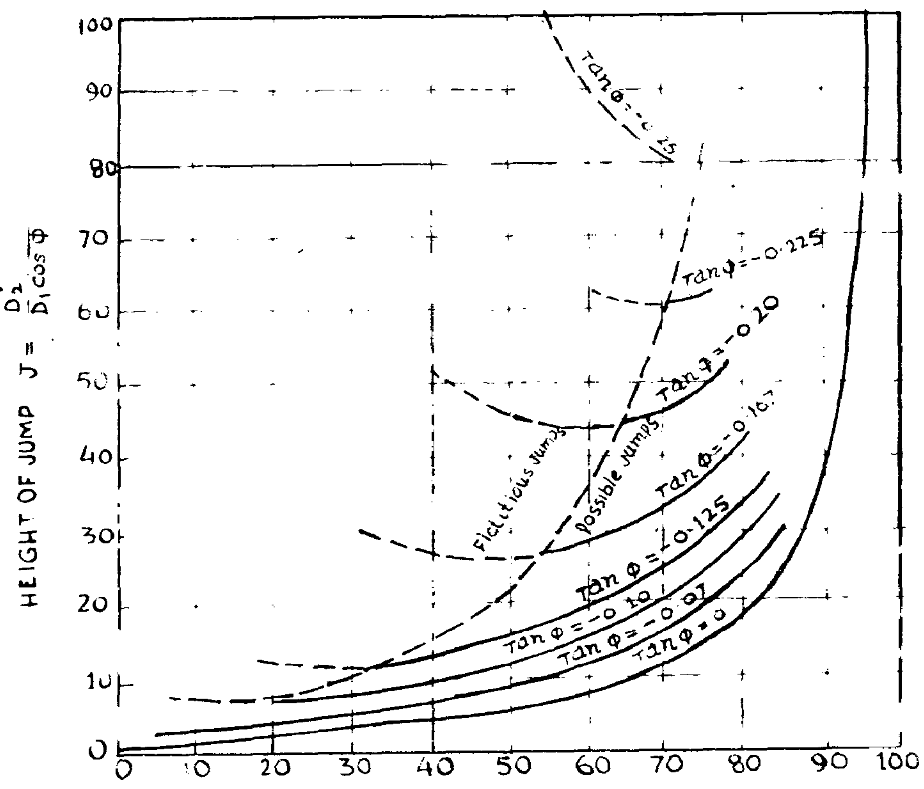


FIG. 3-9

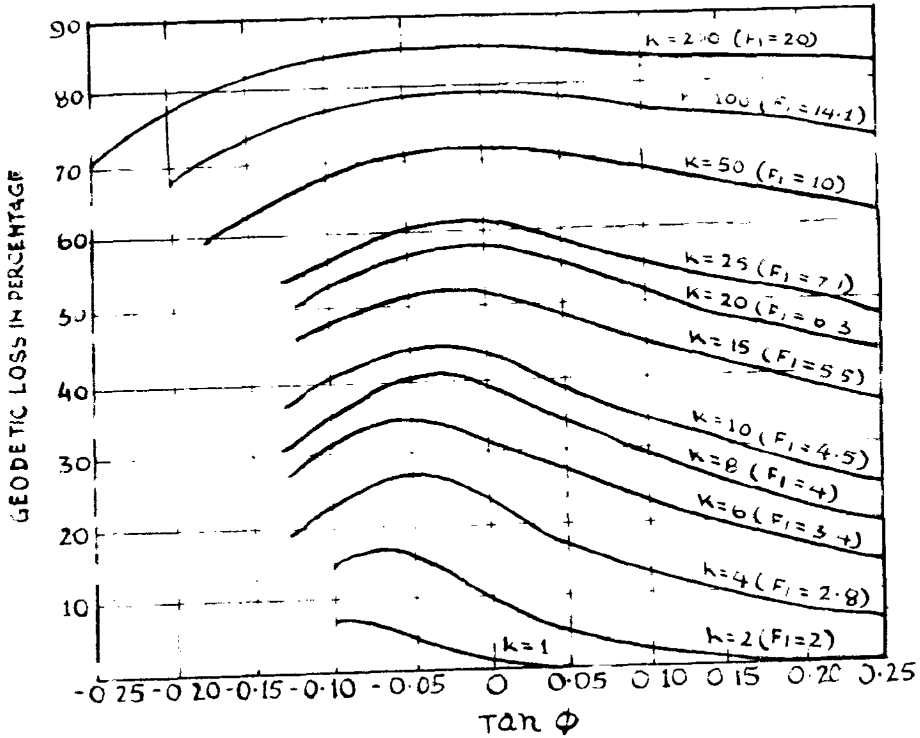


FIG. 3-10

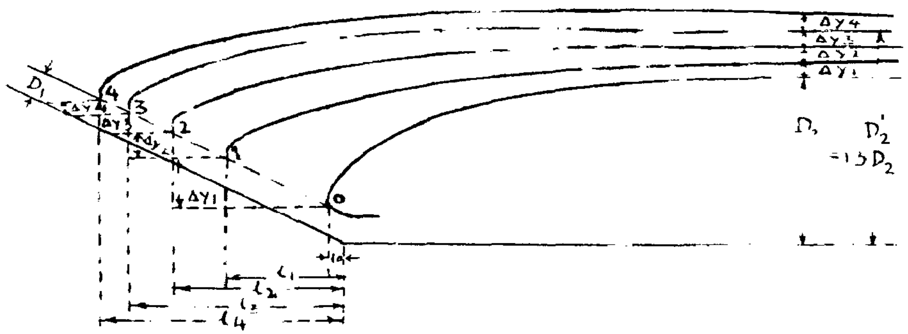


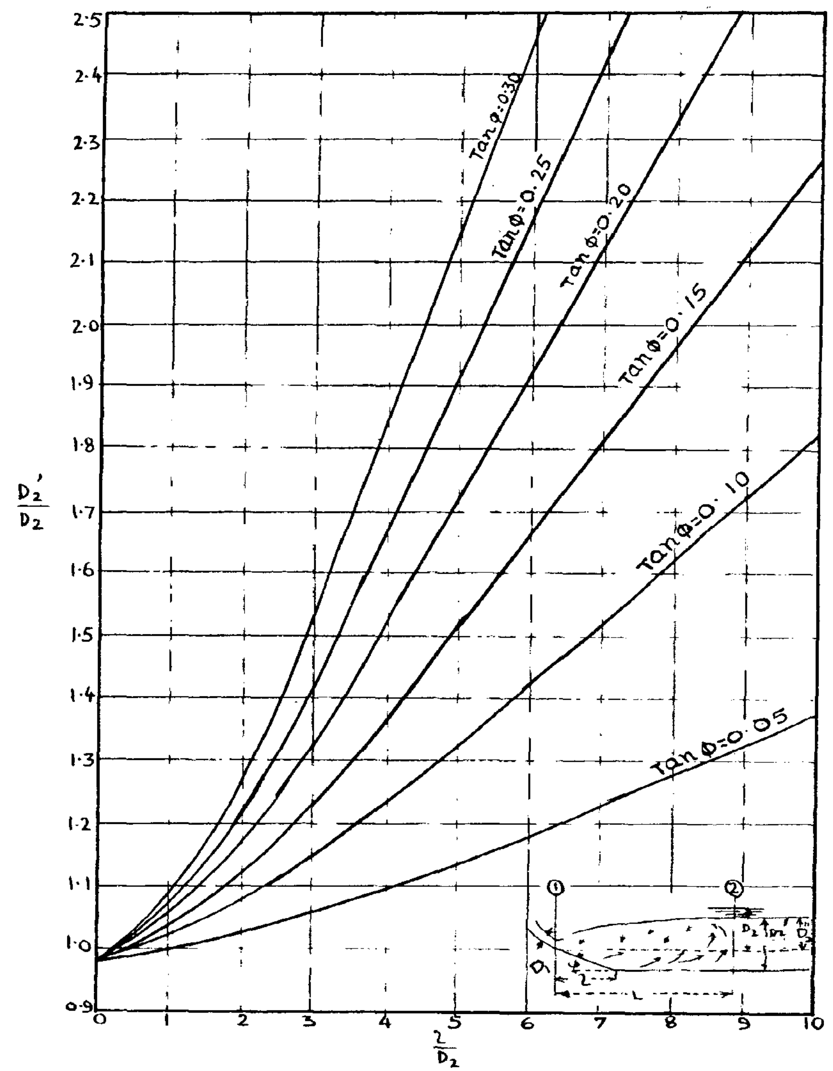
FIG. 3-11

values of K. It is also seen that the apron with negative slope of about 0.05 is slightly more efficient than a horizontal bed in dissipation of initial energy.

Kindsvater [12] has pointed out the formulae for the length of jump adopted by Stevens as given by Ivanchenko is for horizontal apron. The length of jump is dependent on slope of apron. Secondly the assumption that jump body shape is a trapezium for evaluation of volume of jump body, results in inaccuracy. According to Kindsvater, results computed with the above assumption were very much different from experimental results. Kindsvater, therefore believes that the variations in results computed from Stevens analysis and the experimental results, is largely due to above two assumptions.

STUDIES ON JUMP FOR PARTSLOPING AND PART HORIZONTAL APRON.

The studies about jump when it lies partly on sloping and partly on horizontal apron, were carried out by Bradley and Peterka [2]. It is observed that jump front move up the slope when tailwater is increased. The vertical movement of jump on slope is many times more than the increase in tailwater depth (Fig. 3-11). This trend continues till the tailwater depth approaches 1.3 times the sequent depth. Further increase in tailwater depth results in movement of jump front in vertical direction equal to change in depth of tailwater. In case of apron slope being very flat, the horizontal movement of the jump front is even more pronounced. The experiments conducted by above authors were confined to slopes verying from 0.05 to 0.30. The authors have prepared charts showing the plot between diminsionless parameters l/D_2 against $\frac{D_2'}{D_2}$



TAILWATER REQUIREMENT FOR SLOPING APRONS

FIG. 3-12

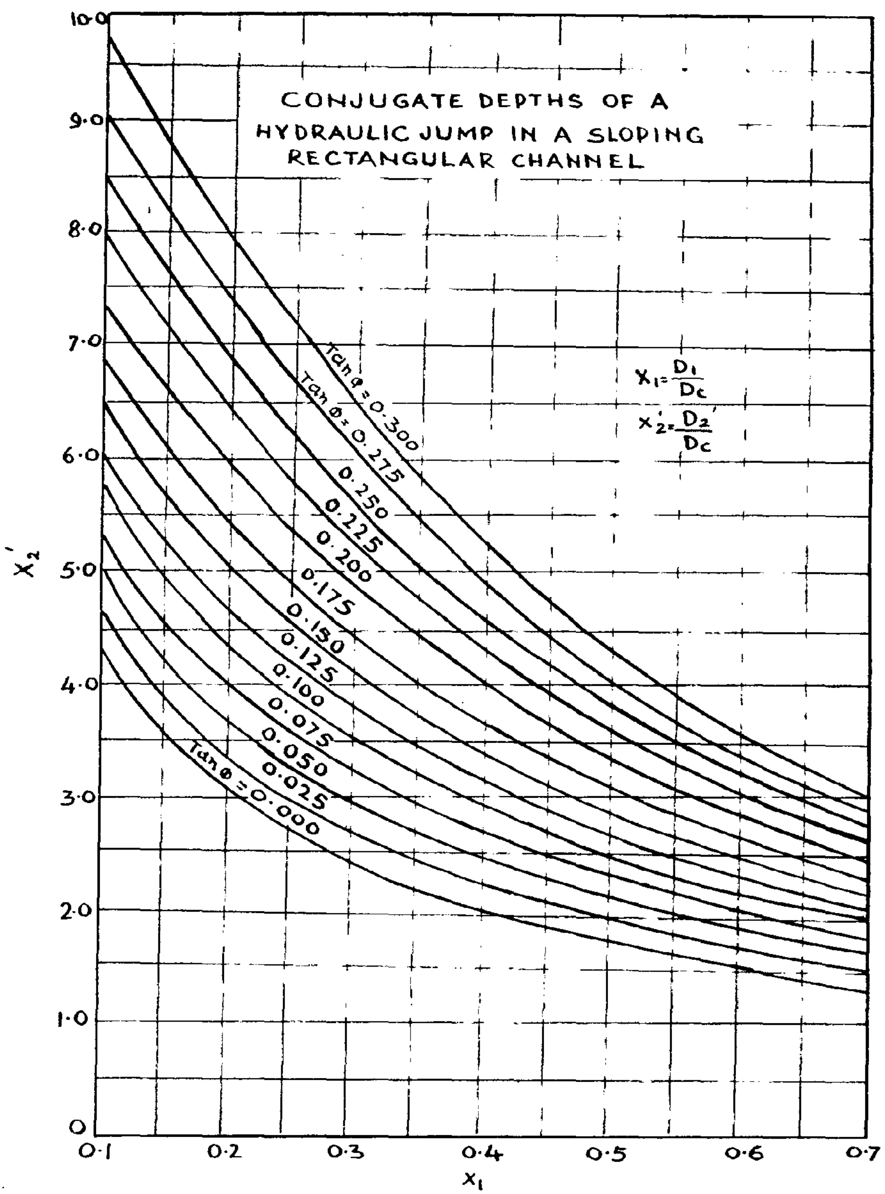


FIG. 3-13

for different apron slopes (Fig. 3-12). These charts show that various lines representing different slopes tend to inter sect at a common point where $\frac{\ell}{D_2}$ and $\frac{D_2'}{D_2}$ are 1 and 0.92 respectively. The change in profile of jump at it moves from horizontal apron to sloping apron is evidenced by the curved portion of the lines. These charts are very useful in the study of tailwater requirements for the formation of jump on different apron slopes.

A formula has also been developed by Solano Vega Vischi[10] for determining the tailwater depth for the formation of jump which lies partly on sloping apron and partly on horizontal apron. This is given by

$$T_w = D_2' \times \frac{\ell}{L} \cos^3\phi + D_2 \left(1 - \frac{\ell}{L} \right)$$

where D_2' = Sequent depth when whole jump forms on fully sloping apron.

D_2 = Sequent depth when jump forms on horizontal apron.

ℓ = length of jump on sloping portion of apron.

The above formula can be expressed in dimensionless form by dividing with Dc (critical depth)

$$\frac{T_w}{D_c} = \frac{D_2'}{D_c} \times \frac{\ell}{L} \cos^3\phi + \frac{D_2}{D_c}\left(1 - \frac{\ell}{L}\right)$$

$$\text{or } X_2'' = X_2' \cdot \frac{\ell}{L} \cos^3\phi + X_2\left(1 - \frac{\ell}{L}\right)$$

The plot between X_2' and X_1 is shown in Fig. 3-13. The author claims that variations in tailwater computed from above formula and observed values on model tests hardly differ by 4.1%.

As far as the energy dissipation characteristics of jump on part sloping and part horizontal apron is concerned, it appears this has not yet been specifically studied. However, it may be added here that the stilling basins utilising hydraulic jump on part sloping part horizontal apron, have been used extensively and the performance have been quite satisfactory.

In the above paragraphs an attempt have been made to discuss the various elements pertaining to hydraulic jump on sloping apron and on part sloping part horizontal apron. The application of above discussion for the design of stilling basins utilising hydraulic jump on sloping apron as means of energy dissipation, is dealt in a subsequent chapter.

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CHAPTER 4.

HYDRAULIC DESIGN CONSIDERATIONS AND CHOICE OF ENERGY DISSIPATOR.

Various types of energy dissipation arrangements below dam spillways have been developed depending upon the hydraulic requirements. The main types of energy dissipation arrangements may be classified under three categories as below-

(1) Hydraulic jump stilling basins.

(2) Roller bucket stilling basins.

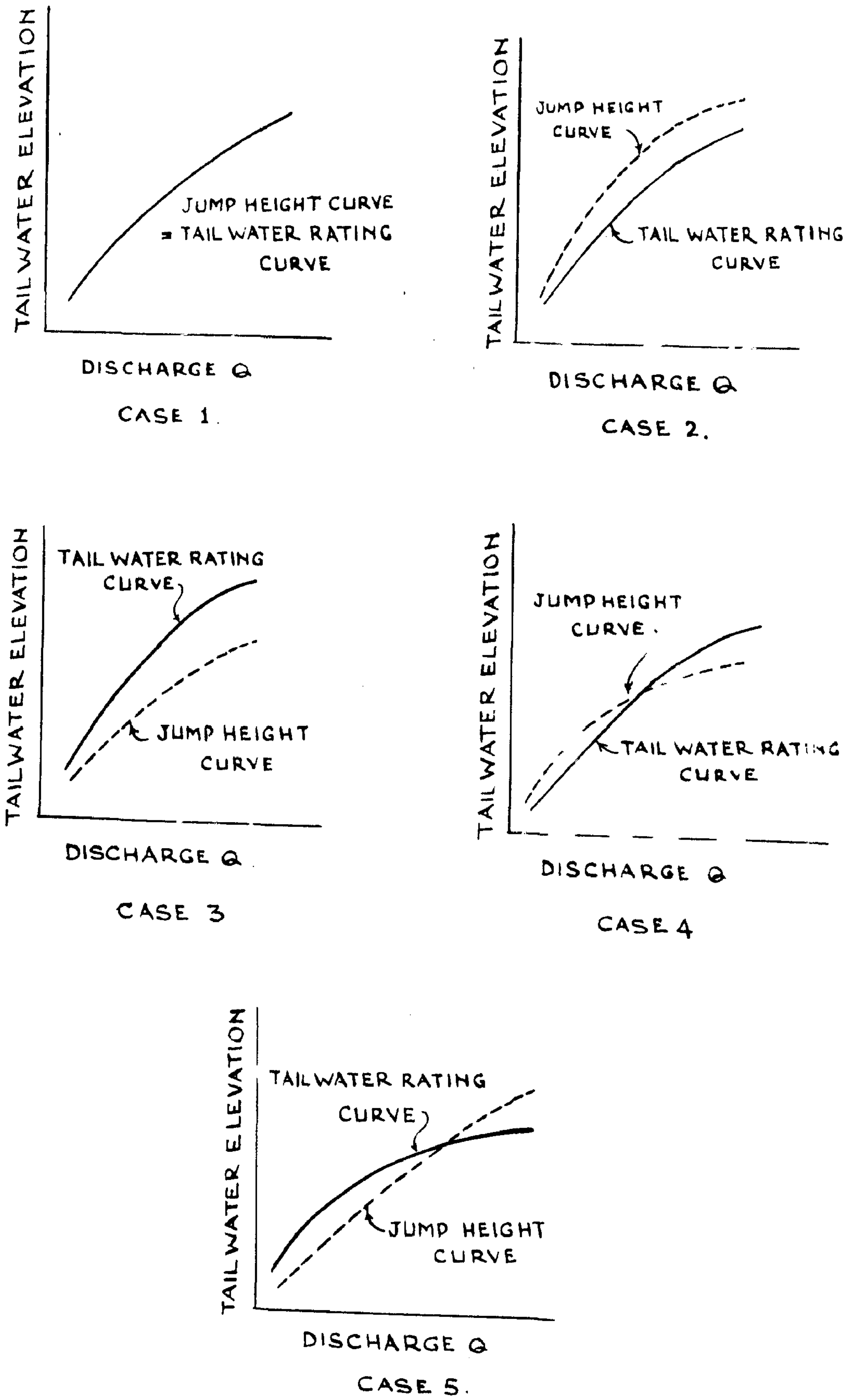
(3) Skijump and Trajectory buckets.

The selection of any particular type of stilling basin depends upon the site conditions viz. topography, foundation strata and hydraulic conditions. Perhaps the most important factor which affects the choice of energy dissipation arrangement is the hydraulic conditions at a particular site. In the following paras these conditions at as well as the choice of particular type of arrangements for energy dissipation have been discussed.

TAILWATER AND JUMP HEIGHT RELATIONSHIP.

The study of tailwater rating curve and jump height curve is most important for choosing a particular energy dissipation arrangement. In all, five conditions (Fig. 4-1) may define the relationship between the depth required for the formation of jump and the tailwater available downstream of the structure . In the whole discussion the jump height is considered with reference to jump on a horizontal apron.

Case I : When the tailwater rating curve coincides with the jump height rating curve. This is seldom possible in actual cases. In this case the jump will form for all range of discharges.



CLASSIFICATION OF TAIL WATER CONDITIONS

FIG- 4-1

Case 2 : In this case the jump height rating curve is always higher than tailwater rating curve. As the tailwater is less than sequent depth required for the jump in the entire range of discharges, the jump will form far downstream. The stream bed will get scoured till sufficient depth is available to permit jump formation.

Case 3 : In this case the jump height rating curve is always below the tailwater rating curve. This condition will cause the jump to move upstream and submerged jump will result. It has already been discussed in Chapter 2 that the submerged jump is not as efficient as a free jump from energy dissipation point of view. The scour will take place downstream of apron as flow with high velocity travels downstream.

Case 4 : In this case, the jump height rating curve is higher than tailwater rating curve at low range of discharges but is at a lower stage for high range of discharges. This will result in movement of jump position towards downstream for low discharges (case 2) and movement of jump towards upstream with a tendency to form submerged jump for high discharges (case 3).

Case 5 : This is reverse of case 4 i.e. the jump height rating curve is lower than tailwater rating curve at low range of discharges but at a higher stage for high range of discharges. This will result in movement of jump towards upstream with a tendency to get submerged at low range of discharges, and the jump will move towards downstream side at high range of discharges. As the jump has a tendency to move towards downstream side at high discharges, the bed will get scoured.

In the cases mentioned above the effect of variation in tailwater on the movement of jump has been broughtout. Since the design of stilling basin is governed by tailwater rating curve, any incorrectness in tailwater rating curve will greatly affect the performance of stilling basin and may even make the basin ineffective. In order to ensure a satisfactory performance of stilling basin, utmost care in determining the tailwater rating curve should be exercised. All the factors which affect the tailwater and are likely to affect it in future will have to be considered.

The tailwater stage discharge relation at the structure may be affected by possible retrogression of the outlet channel, river channel and downstream projects etc. In streams which carry sediment, the construction of structure across the stream results in retrogression of levels downstream of the structure. This takes place because water released through the structure is comparatively free of sediment charge and it picks up the material from the bed of stream causing retrogression in bed levels which ultimately leads to lowering of tailwater level. This possible reduction in tailwater is required to be evaluated in advance and accounted for while preparing tailwater rating curve.

DESIGN DISCHARGE FOR STILLING BASIN.

The knowledge of design discharge which a stilling basin has to cater for, is essential for its design. For flood control projects, the designed discharge can be evaluated from hydrological studies and flood hydrographs of the natural stream. The discharge in the stilling basin comprises of discharge from the overflow spillway as well as discharges through the outlet sluices if these are located in the body of spillway. The discharges from large storage reservoirs are mainly governed by downstream requirements and irrigation demands.

WIDTH OF STILLING BASIN.

The width of stilling basin is fixed from hydraulic as well as topographical considerations. The reduction in width of stilling basin increases the depth of the basin. Sometimes topographical features may favour small width of basin. In determining the final width of basin, it has also to be considered that the uplift pressures on floor slab as represented by the difference in depth before and after the jump, are not excessive.

CHOICE OF PARTICULAR TYPE OF ENERGY DISSIPATION ARRANGEMENT.

As already stated earliar the choice of a particular basin is generally determined from hydraulic, topographic, geological and economic considerations. Out of which the hydraulic consideration i.e. tailwater and jump height relation--ship , is the most dominant factor. The choice of particualr arrangements for different cases of jump height and tailwater relationship, are discussed below.

In case 1 , the choice of stilling basin naturally falls on hydraulic jump type stilling basin with horizontal apron, as the jump forms for all range of discharges.

In the case 2 , the tailwater is deficient for all range of discharges causing the jump to move towards downstream resulting in lot of scour in bed. The jump formation in the stilling basin can be ensured by suitable measures to make up the tailwater deficiency. The possible arrangements are :-

(i) The depression of floor at the start of basin. This will ensure the start of jump near the toe of the dam, and the jump body in all cases will lie generally in the basin.

(ii) The full apron can be suitably lowered below the stream bed to ensure formation of jump.

(iii) The construction of a low secondary dam downstream of main dam, will increase the tailwater depth to ensure the jump formation.

The skijump or trajectory type of buckets can be usefully adopted in such situations where tailwater is not sufficient for the formation of jump. This type of bucket throws away the jet of water in the air which strikes the river bed at a good distance downstream of structure. This type of arrangement is very economical as compared to hydraulic jump stilling basin. Only consideration for this type of arrangement is that the bed material should be composed of fairly firm rock.

In case 3, where the tailwater depth is more than the hydraulic jump requirements for all range of discharges the jump has a tendency to move upstream. The jump formed is of submerged type which results in inadequate dissipation of energy. The clear jump at all discharges can be ensured by providing the following arrangements.

(i) An apron at suitable elevation above the river bed will ensure the formation of free jump.

(ii) A sloping apron above the river bed level will ensure formation of efficient jump at all ranges of discharges. The start of jump front will adjust itself along the sloping apron depending upon the requirement of tailwater for the formation of jump.

The other type of energy dissipation arrangements used in such cases, are roller buckets. The roller buckets may be solid type or slotted type. The slotted buckets are used where the material of river bed is likely to be sucked inside the bucket as a result of ground roller action. The detailed discussion about roller buckets is covered in a subsequent chapter.

In the case 4 , the tailwater is inadequate for low range of discharges while it is in excess at high range of discharges. This results in moving the jump downstream at low range of discharges. While at high range of discharges, the jump has a tendency to move upstream and submerged type of jump is likely to be formed. The submersion of jump at high range of discharges will cause inadequate dissipation of energy and the flow will have high velocities after it leaves the basin which will result in scouring of river bed. At low discharges, river bed is also likely to get scoured, as the jump has a tendency to move towards downstream. In order to ensure the formation of clear jump at all stages and to control the jump position within a certain range, a sloping apron starting from higher level and ending at a lower level than the river bed level, is required to be provided. The position of start of jump will adjust itself along the sloping apron according to availability of tail water. The jump will form on sloping apron portion above the river bed level at high range of discharges and in apron portion below the river bed level at low discharges. In this case a combination of part sloping and part horizontal apron can be used with advantage . The part sloping apron will start at a higher level and will end at a lower level than the river bed level, followed by horizontal apron. This type of arrangement is more economical than fully sloping apron. The detailed discussion about this is done in a subsequent chapter.

In case 5 , the tailwater depth in in excess of jump requirements at low range of discharges while it is inadequate for high range of discharges. This will result in movement of jump towards upstream side and jump will get submerged at low discharges while at high range of dischagges the jump will shift towards downstream side causing a lot of scour in river bed. In this case the later situation is dangerous as the discharge intensities will be quite high resulting in severe scour, while submersion of jump at low discharges will not create much problem. The provision of a similar type of sloping apron or part sloping part horizontal apron as suggested in the previous case, will ensure the formation of clear jump. The difference will be that the jump will form on the sloping portion of apron above the river bed level at low discharges while it will form on the lower portion of apron at high discharges.

The provision of sloping apron below river bed to provide necessary depth for formation of jump, may involve huge quantity of excavation. In such situations a high level skijump or trajectory type of bucket may be more economical provided the bed consists of fairly good rock.

In some of the cases discussed above the jump height curve may differ greatly from tailwater rating curve. The jump height rating curve can be made to agree closely with tailwater rating curve , by proper selection of spillway crest length provided other considerations permit. The change in crest length may result in increased cost of crest gates, spillway portion or other features of dam but this might be more than offset by reduction in stilling basin cost. Therefore it is always desireable to examine the proposals of energy dissipation arrangements from this point also.

The entire discussion on hydraulic jump and categorising them into cases 1 to 5 , presupposes that the discharge is uniformly flowing over the entire length of the spillway. This is only possible when the crest is either uncontrolled or the gates are so operated as to lift equally in all bays. Such operation of gates is hardly possible in actual practice. Thus there is considerable limitation to this in practical operation for crest controlled spillway.

The above discussion mainly deals with hydraulic design considerations for different types of energy dissipation arrangements and choice of particular type of arrangements. In the subsequent chapters, each type of energy dissipator is discussed in detail.

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CHAPTER 5.

HYDRAULIC JUMP STILLING BASINS WITH HORIZONTAL APRON.

The stilling basins, utilising hydraulic jump as energy dissiptor, have been used on large scale throughout the world. Inspite of this, it has not been possible to evolve standard type of designs for such basins as the designers have differed on many points eg. length of basin, permissible variation in tailwater depth, use of appurtenances such as chute blocks, baffle piers and end sill etc. The design of stilling basin involves many factors which cannot be evaluated fully. However such important factors which generally influence the design of stilling basins are described here.

FACTORS AFFECTING DESIGN OF STILLING BASIN.

According to Progress Report of Task Force Committee[16], the main factors affecting the design of basins particularly shape and dimensions are as below-

(i) Absolute size of structure.

(ii) Frequency of operation.

(iii) Durability of river bed downstream of stilling basin.

(iv) Use of appurtenances such as chute blocks, baffle piers and end sill etc.

The size of structure and the discharge which requires to be handled, greatly affects the design of stilling basin. When the structure is big and important, no risk can be taken and ample factor of safety in design is necessary while in small structures some calculated risk can be taken in design as forces involved and are small and repairs can also be effected if necessary. It is imperative to know the range of structure sizes used to develop a dimensionless design data before it can be applied in a particular case.

The frequency of operation has a great bearing on the design of stilling basins. A greater factor of safety is necessary for structure which operate frequently or continuously as carrying out of repairs may be more costly than providing an adequate factor of safety in design. In structures which operate rarely, repairing may be more economical than provision of expensive design in the begining intself.

The durability of river bed, many times affects the provision of necessary length of basin. If the bed material cannot be eroded easily, the basin of shorter length can be provided as compared to the basin where bed material consists of loose material such as sand and gravel etc. No rational relationship between internal velocities in the jump and their effect on heterogeneous bed material, has so far been established. Only guide in these cases is model studies and past experience on similar structures.

The use of other appurtenances such as chute blocks, baffle piers and end sill can be helpful in reducing the length of basin. Baffle piers, unless streamlined excessively, move the jump action towards upstream portion, promote stability, reduce wave heights and prevent early jump sweepout if tailwater depth is inadequate for jump formation. Chute block also help to prevent early sweepout and stabilise the jump position and action. The Task Committee is of the opinion that these when used alongwith end sill may reduce the basin length by one third. The baffle piers are maximum effective if they are square edged and are placed at 1.5 times the sequent depth from the start of the basin. However tests have also indicated that baffle piers produce cavitation if the discharge intensities exceed 200 cusecs and velocities exceed 40 to 50 ft/sec. The higher velocities may be allowed provided discharge intensity is less than 200 cusecs. Thus the use of baffle piers is limited. The detailed discussion on the use of appurtenances is given later in this chapter.

DESIGN OF STILLING BASINS.

The stilling basin may be designed with horizontal apron, fully sloping apron and part sloping part horizontal apron. The horizontal and sloping apron are used depending upon the hydraulic requirements as discussed in the last chapter. In this chapter the design of stilling basins with horizontal apron only is proposed to be discussed.

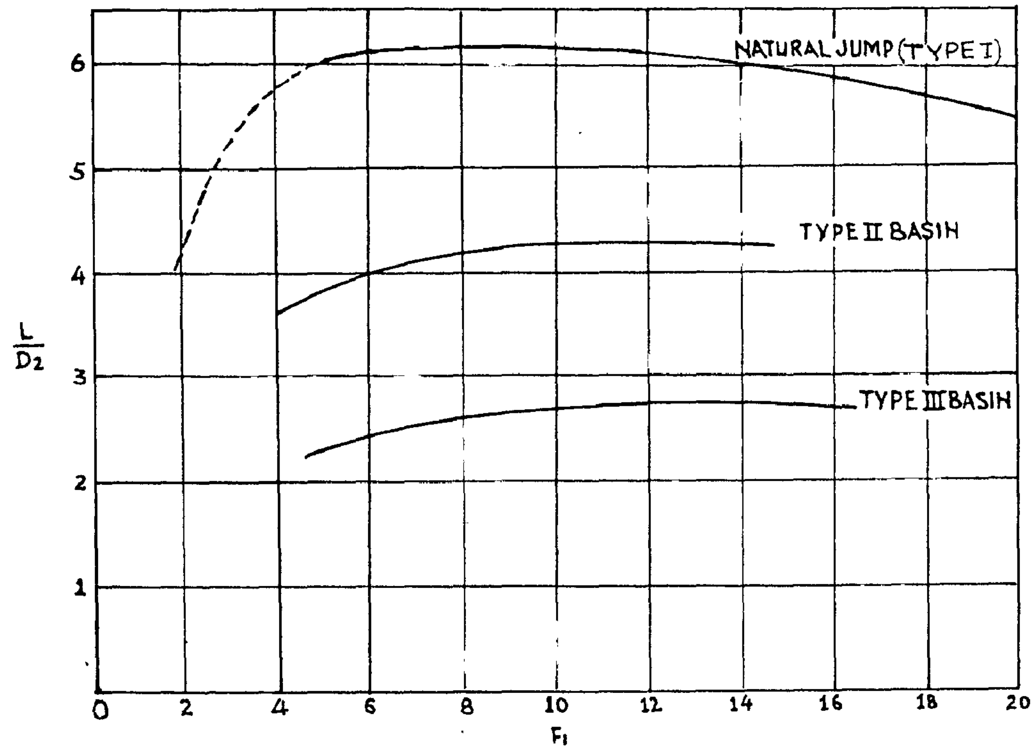
The jump on horizontal apron insures slightly better dissipation of energy than sloping apron provided tailwater depth required for formation of jump is available at all range of discharges. The jump on basin with horizontal apron is very sensitive to tailwater variations. A slight defficiency in tailwater may cause the jump to sweep off the basin. As already stated earliar the problem of defficiency in tailwater to some extent can be overcome by providing stilling basin appurtenances.

Classification of basins with horizontal apron has been attempted by U.S.B.R.[1] depending upon the discharge intensity and velocity involved . The arrangement of appurtenances in the basin as well as the dimensions, also vary according to discharge intensity and velocity. Elevatorski[9] has also suggested the shapes of basins similar to above with some difference in dimensions depending upon the discharge intensity and velocity in the basin.

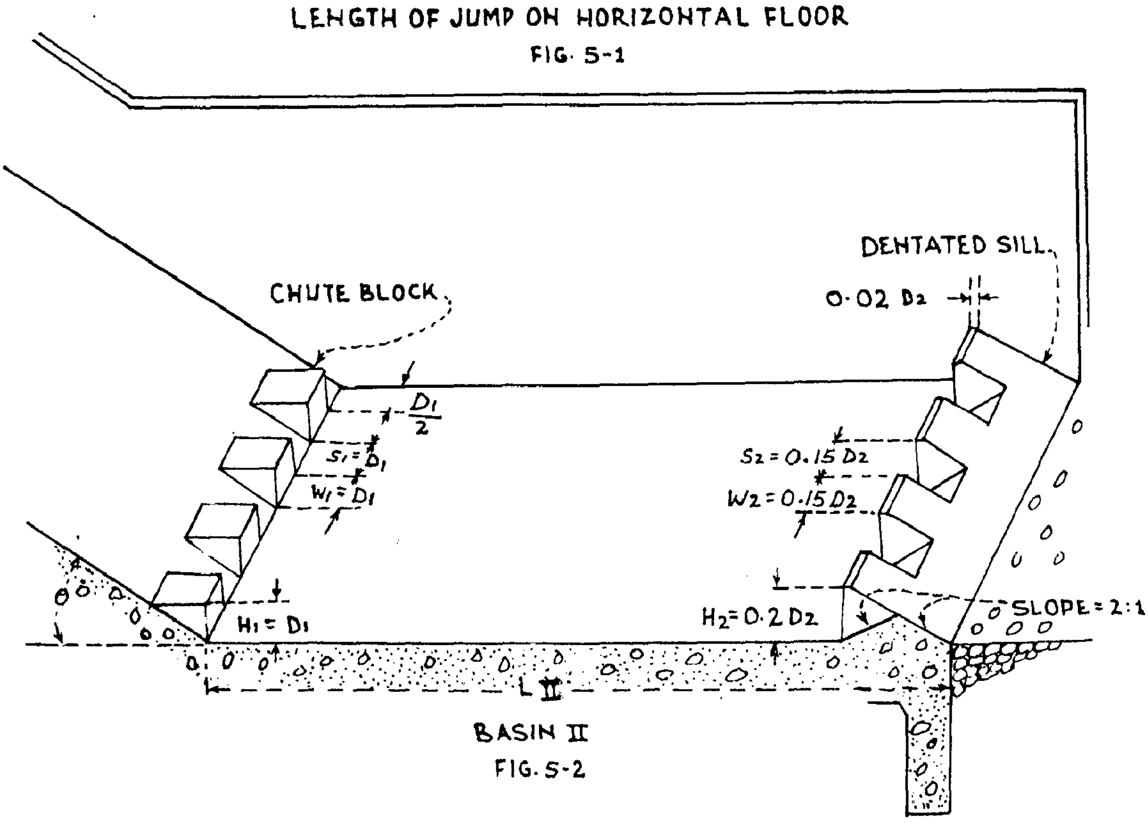
Length of basin.

Basins are seldom designed to contain the whole length of jump on the paved upon apron as this will make the basin very costly. The purpose of energy dissipation can be accomplished by providing shorter stilling basin with installations of suitable appurtenances. Experimental studies conducted on jump characteristics by Rouse, Siao and Nagaratnam[17] indicate that major portion of energy loss takes place in the initial 50 to 60% portion of length of jump for moderate value of Froude numbers. The redistribution of velocities occurs after majority of loss have occured. This is also clear from Fig. 2-8 (Chapter 2) where the effect of submergence on the fall of energy in a jump is shown, that major portion of energy loss takes place in the initial half of jump length.

The condition of bed material also affects the length of basin. The scouring of river bed can be reduced by providing end sill. The length of jumps for various types of U.S.B.R. basins are shown in Fig. 5-1. It is observed that Basin type I (without appurtenances) has jump length as 6 times sequent depth for medium range of Froude numbers. For Type II Basin in which chute blocks and end sill are used, the length of jump is 4 to 4.3 times the sequent depth. In case of Type III Basin which has chute blocks, baffle piers and end sill, the length of jump varies from 2.4 to 2.8 times the sequent depth depending upon the Froude number. Thus the length of jump gets reduced by 1/3rd to 2/3rd with the help of appurtenances which is very significant. The details of Basin Type II and Type III are shown in Fig. 5-2 and 5-3 respectively.



HYDRAULIC JUMP STUDIES
LENGTH OF JUMP ON HORIZONTAL FLOOR
FIG. 5-1



BASIN II
FIG. 5-2

Elevatorski[9] recommends a length of $5(D_2-D_1)$ for basins where the chute blocks and end sill are installed (Fig.5-4). In case of basins which have chute blocks, baffle piers and end sill (Fig.5-5) the length recommended is 4.5 (D_2-D_1).

The following table shows the comparision between length of basins of U.S.B.R. type and suggested by Elevatorski.

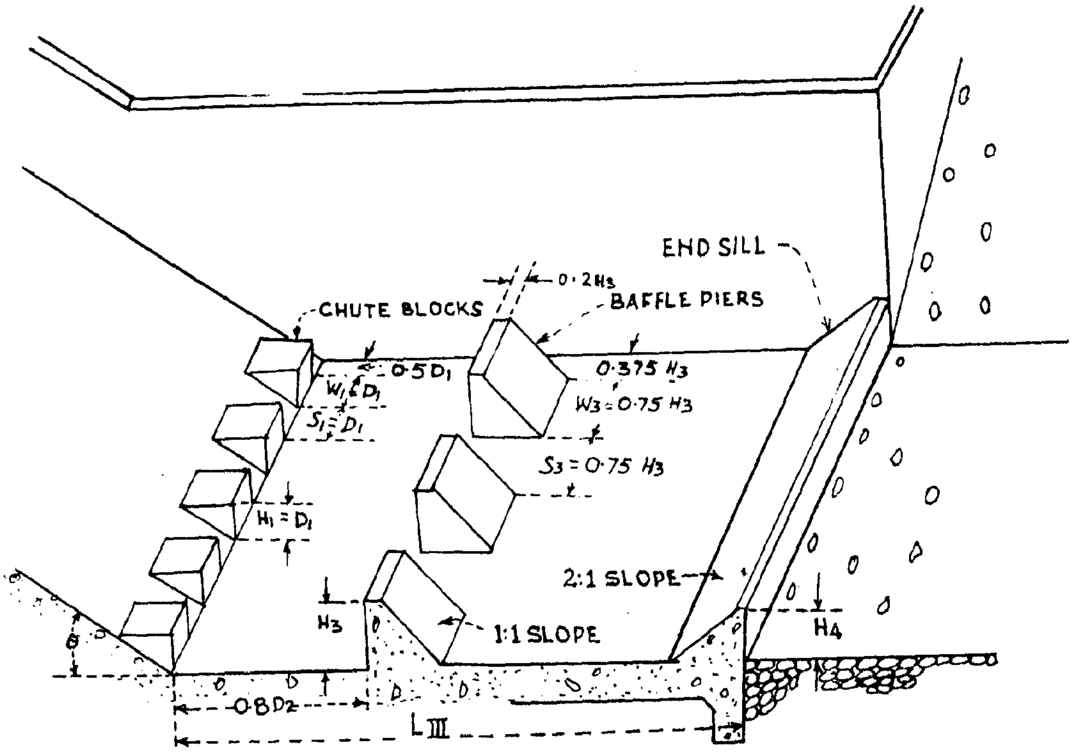
Table 5.1.

F_1	$\frac{D_2}{D_1}$	Length of stilling basin.			
		Basin with chute blocks and end sills. (i)		Basin with chute blocks Baffle pier and end sill. (ii)	
		Elevatorski type.	U.S.B.R. Type II.	Elevatorski Type.	U.S.B.R. Type III
4	5.2	21 D_1	18.8 D_1	-	-
6	8.0	35 D_1	32.0 D_1	31.5 D_1	18.8 D_1
8	10.8	49 D_1	45.3 D_1	44.0 D_1	28.6 D_1
10	13.7	63.5 D_1	59.0 D_1	57.0 D_1	37.0 D_1

From the above table it is seen that length recommended by Elevatorski is 8 to 10% more than the U.S.B.R. in case (i) and 50 to 60% more than U.S.B.R. in case of (ii). The dimensions of appurtenances used in basin also differ slightly. It has already been stated earliar in Progress report of Task Force Committee[16] that the use of appurtenance reduces the basin length by one third. Thus there is marked difference in the recommendations of various authorities regarding reduction in the length of basin due to installation of appurtenances.

Appurtenances used in Basin.

The appurtenances used in the basin are chute blocks,



BASIN III
FIG. 5-3

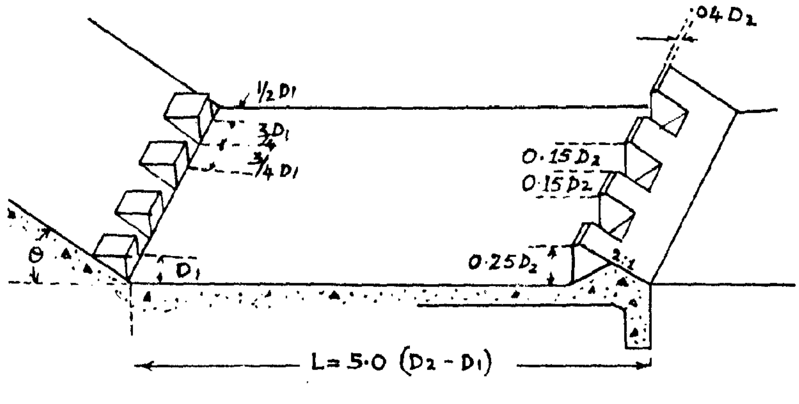


FIG. 5-4

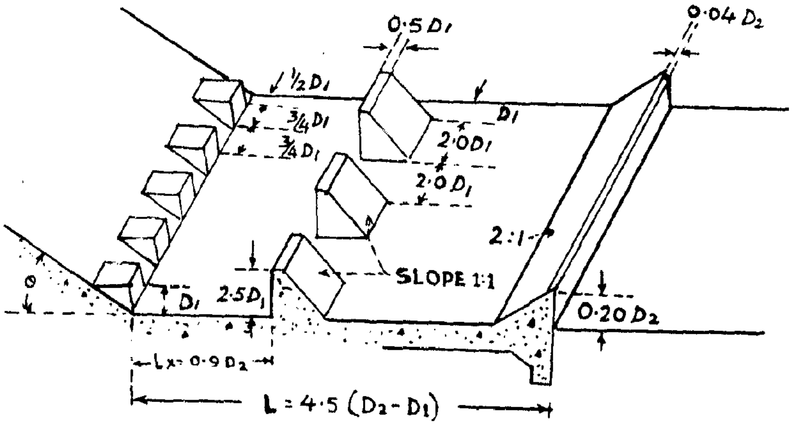


FIG. 5-5

baffle piers and end sill. As stated earliar the use of these helps to improve the performance of stilling basin. These help in stabilising the flow, in increasing turbulence and in distributing velocities more evenly throughout the basin. The basin length can be reduced as mentioned earliar and it can function with lesser tailwater than required for the basin without appurtenances. The extent of tailwater difficiency which can be permitted in various U.S.B.R. basins is shown in fig. 5-6. It is seen that the maximum difficiency in tailwater depth is permissible in case of Type III Basin.

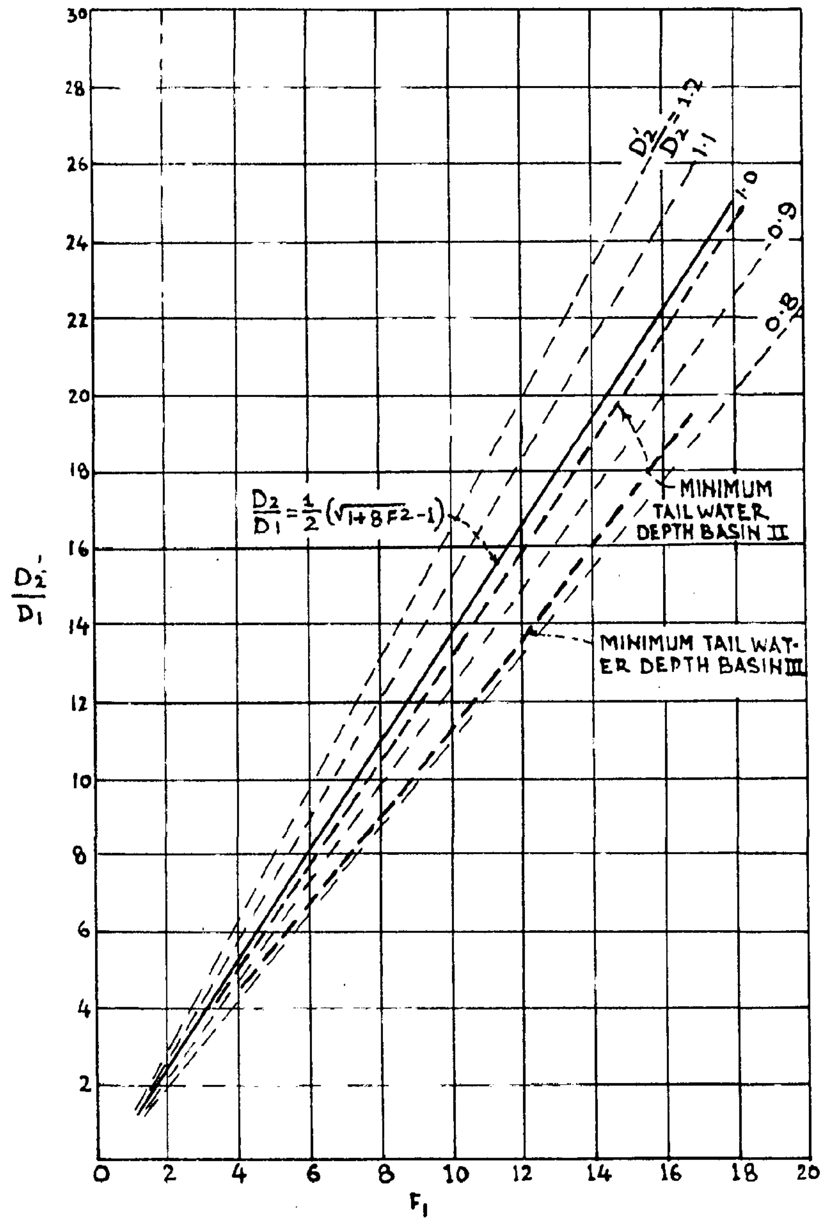
The functions and dimensions of each type of appurtenances used in basins are discussed in following paras.

Chute Blocks.

These are provided at the entrance of basin to increase the effective depth of entering stream, to breakup the flow into numerous jets and to create turbulence required for energy dissipation. The blocks make the jet lift off the floor and thus shorter basin length is possible. The provision of chute blocks may not be desireable when the streams carry large amount of debris and sediments as these may damage the blocks by impact and abrasion. The general dimensions of chute blocks according to U.S.B.R.[1] practice are shown in Fig. 5-2, 5-3. The dimensions recommended by Elevatorski[9] are shown in Fig. 5-4 and Fig. 5-5.

Baffle Piers.

These are provided in basins to stabilise the formation of jump and to increase the turbulence thus helping the energy dissipation when the discharge intensity is low. According to Elevatorski[9] baffle piers help to compensate slight difficiency in tailwater and during flow of high intensity, assist in deflecting



STILLING BASIN I, II AND III
MINIMUM TAILWATER DEPTHS
FIG. 5-6

the flows aways from the bed of river. The basin length also gets reduced as stated earliar. It is seen from Fig. 5-4 and 5-5 that the use of baffle piers causes very little reduction in basin length as far as Elevatorski basins are concerned, while in case of U.S.B.R. Basins (Fig. 5-2 and 5-3) the reduction is quite significant.

The permissible difficiency in tailwater can be evaluated analytically by using momentum equation. The final equation as derived by Elevatorski[9] is expressed as

$$\frac{P/b}{\frac{\gamma D_2^2}{2}} = 1-\left(\frac{D_p}{D_2}\right)^2 - \left(\frac{D_2}{D_p}-1\right)\frac{16\lambda}{(\sqrt{1+8\lambda}-1)^2}$$

where P = force exerted by piers.

b = width of basin.

D_p = Depth after jump with baffle piers.

D_2 = Sequent depth without baffle.

λ = Kinetic flow factor.

If P is known in the above equation then $\frac{D_p}{D_2}$ can be evaluated. P can be determined by means of drag equation.

$$P = C_b \rho A \frac{U^2}{2}$$

where Cb = Drag coefficient of baffle.

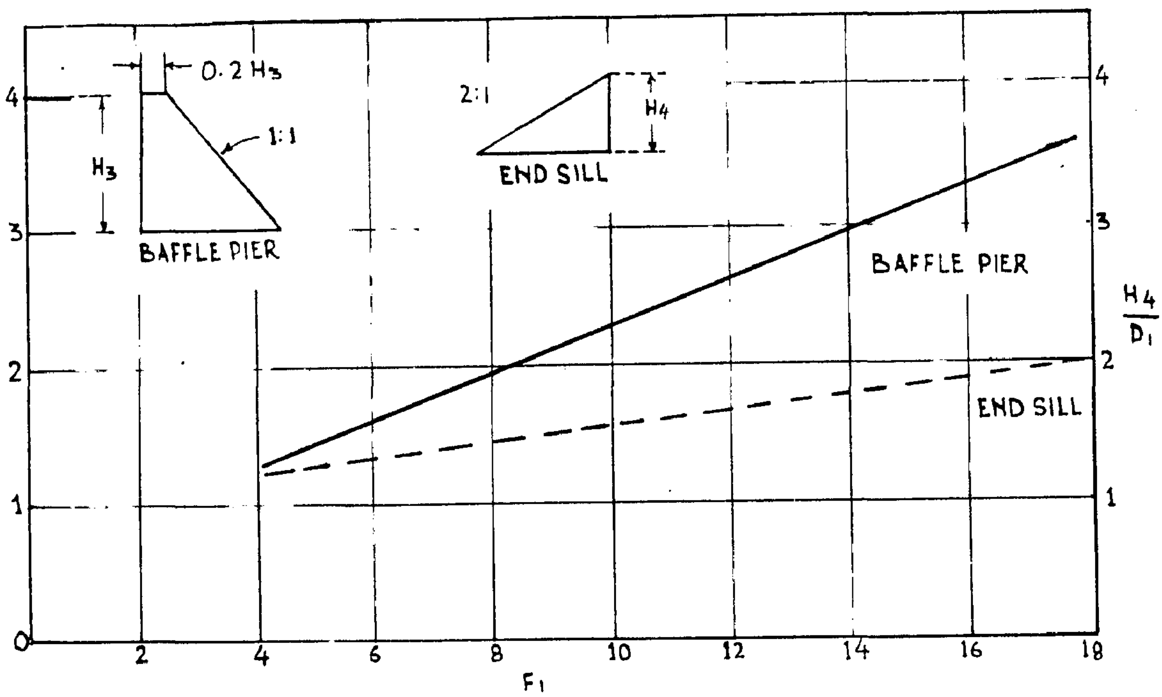
A = area of blocks.

ρ = Mass density.

U = Velocity.

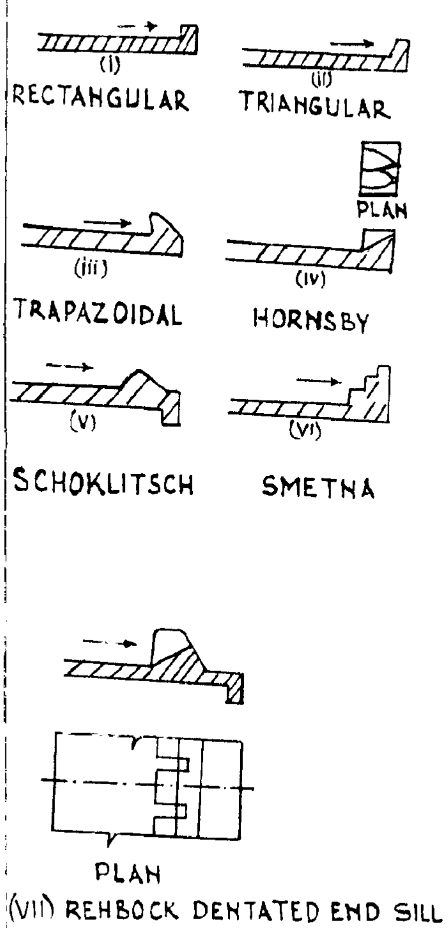
The value of Cb can be determined from model studies in the laboratory for particular shape of piers and spacings.

The spacing and other dimensions of baffle piers are shown in figure 5-3 for U.S.B.R. stilling basin type III. The dimensions recommended by Elevatorski[9] are shown in Fig. 5-5. The U.S.B.R.[1] studies indicate that height of baffle pier H_3 varies from 1 to 3.6 times D_1 for Froude number ranging from 4 to 18 (Fig. 5-7) while Elevatorski[9] recommends H_3 equel to 2.5 D_1 in all cases. Baffles of other shapes have also been used such as



HEIGHT OF BAFFLE PIER AND END SILL.

FIG. 5-7



(VII) REHBOCK DENTATED END SILL

FIG. 5-8

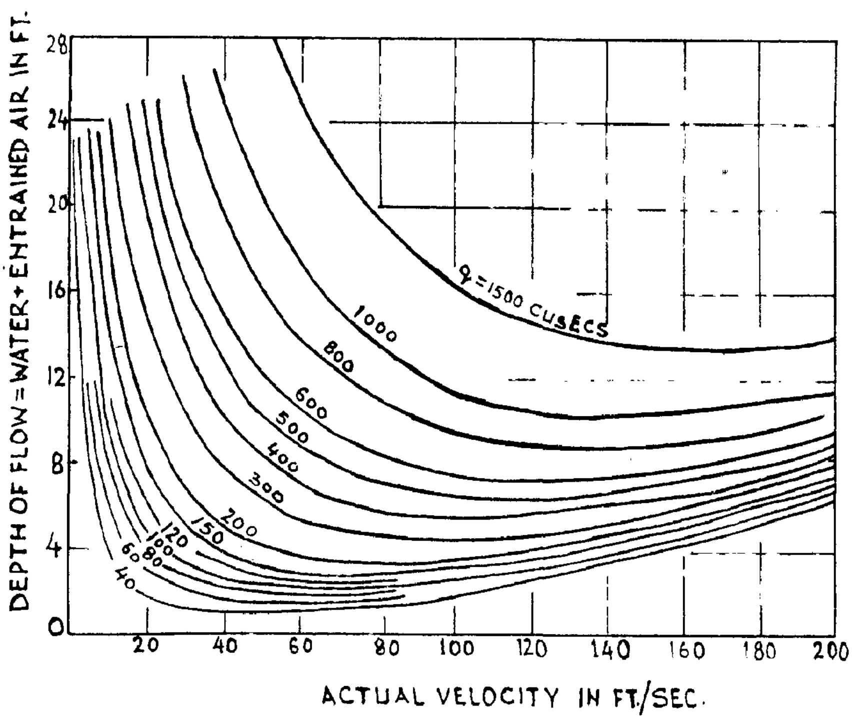


FIG. 5-9

triangular shape. Recent experiments conducted by Pillai and Unni[18] have indicated that wedge shaped baffles with 120° angle at upstream nose gives the minimum length of jump. The results obtained by them for F_1 equel to 7.85 , are given below.

Angle of Block.	Length of jump.	Remarks.
60°	56.0 D_1	
90°	31.5 D_1	
120°	27.0 D_1	Least value
180°	36.0 D_1	

From the table 5-1 it is seen that for F_1 equel to 8 the U.S.B.R. Type III Basin gives the length as 28.6 D_1. Thus it is clear that the least length of jump with 120° angle of block is only slightly less than that of U.S.B.R. Type III basin. The above authors also claim that this shape of blocks reduces the chances of cavitation. Further they have also indicated that energy dissipation improves if the blocks are placed nearer towards the begining of jump than the distance recommended by U.S.B.R. (Type III basin). The authors have not indicated any definite distance which will give better dissipation as the studies probably were not then conclusive.

According to Elevatorski[9] the spacing of baffle blocks should be such that they should occupy 40 to 55% of stilling basin width. If these occupy too much area, sufficient water will not flow in between the blocks and they will tend to act more and more like a sill than individual blocks.

The provision of baffle piers have many advantages discussed above, even then as stated earliar the use of these is not favoured by Task Force Committee[16] and Elevatorski[9], where discharge intensities and velocities are greater than 200 cusecs and 40-50 ft/sec. respectively as these are likely to cause cavitation.

It has been stated above that the use of baffle piers is not favoured where discharge intensities are greater than 200 cusecs because of cavitation. It has not been explained how the discharge intensity affects cavitation. Normally a higher discharge intensity will have greater tailwater depth which will result in more hydrostatic pressure on the block. This will have the effect of reducing the cavitation. Thus the idea of limiting the use of baffle piers upto a particular discharge intensity does not seem to be beyond question.

End Sill.

These are provided to lift the flow off the river bed and create a back current which causes bed material to be transported and heaped up against the back face of the fill. The use of end sill also stabilises the flow, deflects the current from the river bottom and helps in performance of basin with less tailwater depth. The end sill may be vertical, stepped, sloped or dentated wall provided at downstream end of the basin. Some of the prevalent shapes are shown in Fig. 5-8. According to Elevatorski[9] the Rehbock dentated sill is the most efficient type of sill. The standard U.S.B.R. end sill consists of a solid end sill with upstream face as 2:1 and downstream face as vertical. The studies conducted by U.S.B.R.[1] have also established that $\frac{H_4}{D_1}$ where H4 is height of end sill, increases with Froude number (Fig.5-7). The height of sill is quite important dimension as far as performance of basin is concerned. If sill is not high enough the flow may pass over the sill as standing wave or swell and the purpose of providing sill may not be realised. In case the sill is too high, a violent roller will set in motion, which will draw the bed material in the basin. Therefore dimensions of end sill should be decided like other appurtenances after conducting model studies.

STILLING BASIN WALLS.

The design of basin walls depends upon the shape of basin channel. The basin channel may be rectangular or trapa-zodial. According to Elevatorski[9] the model studies have established that vertical or near vertical walls provide better flow conditions in basins than the sloping walls. The sloping walls are more economical to construct but poor performance of jump will offset the economic advantage.

The top of side walls are so fixed that maximum designed discharge is contained in the basin with sufficient free board to allow for spray and air entrainment of turbulent water. The free board recommended by Elevatorski[9] for rectangular channels is as below.

Discharge Q	Free board in Ft.
0 - 100	2
100 - 500	3
500 - 1000	4
1000 - 5000	5
5000 - 10000	6
10000 - 50000	8
50000 - 1,00,000	10
1,00,000 and as above.	1/3rd of sequent depth.

The side walls can be reduced in height when basin is confined by good rock.

In the above recommendation for fixing the free board Elevatorski has related the free board with the total discharge Q which does not seem to be very rational. The free board should have been fixed on the basis of intensity of discharge rather than total discharge. He has also recomended that free board should be 10 ft. for discharges ranges of 50,000-1,00,000 cusecs, and 1/3rd the sequent depth in case of discharges greater than 1,00,000 cusecs. It is not explicit as to what amount of free board be adopted in last range of discharges if the 1/3rd sequent depth comes out to be less than 10'. The minimum free board may be 10ft. in such cases.

Effect of Air Entrainment.

According to Elevatorski[9] the effect of air entrainment upon the depth of flow below spillway should be accounted for in the case of high spillways. He suggests that depth of air entrained flow can be obtained from curves developed by Gumensky[19]. The discharge intensity and actual velocity should be known before using these curves (Fig.5-9). The actual velocity below a spillway can be found out by the usual formulae $V_1 = \sqrt{2gH_1}$, where H_1 is the total drop from head water energy gradient to water surface at the point under consideration, less friction losses which are determined by Manning's formulae with rugosity $n = .008$ to 0.011. Gumensky has indicated that air entrainment only affect the horizontal hydrostatic water pressure acting on the jump and the hydraulic jump itself is little affected. This means that the jump elements will not be so much affected as to require consideration in practical cases. Hence for practical designs he suggests that actual velocity and net depth of water without air entrainment should be considered in jump computation. The effect of air entrainment should be considered while fixing the height of Training walls.

BRIEF DISCRIPTION OF STANDARD STILLING BASINS.

The design aspects ~~for~~ stilling basins have been discu-ssed above. In the following paragraphs the standard type of stilling basins used below spillways which have been developed after experimental studies by various authorities, are discussed briefly.

S.A.F. Basin.

This basin (Fig.5-10) was developed by Blaisdell[21] and according to him it is suitable for Kinetic flow factor λ ranging from 3 to 300 ($F_1 = 1.7$ to 17). The baffle blocks occupy nearly 40 to 55 % of basin width. The spacing of chute and baffle blocks is 3/4times the height of blocks. No effect of air entrainment is considered in this type of basins. The tailwater requirement varies with Kinetic flow factor and is given by

$$\text{For } \lambda = 3 \text{ to } 30, \quad \frac{D_2'}{D_2} = \left(1.10 \quad \frac{\lambda}{120}\right)$$

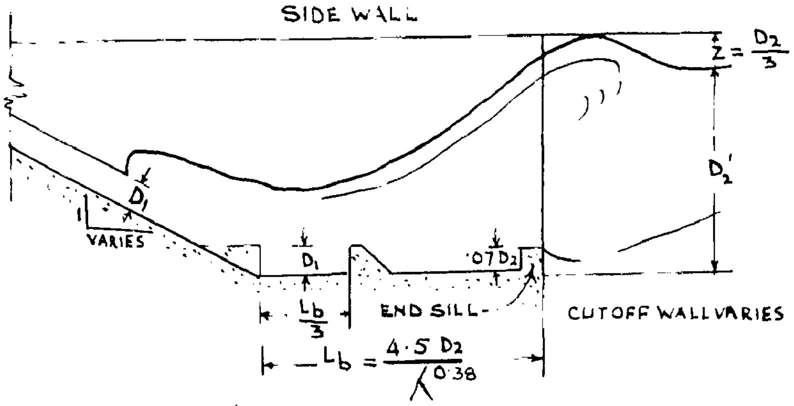
$$\lambda = 30 \text{ to } 120, \quad \frac{D_2'}{D_2} = 0.85$$

$$\lambda = 120 \text{ to } 300, \quad \frac{D_2'}{D_2} = \left(1.0 - \frac{\lambda}{800}\right)$$

where D_2' is tailwater requirement and D_2 is sequent depth after the jump. The basin length is given by the formulae

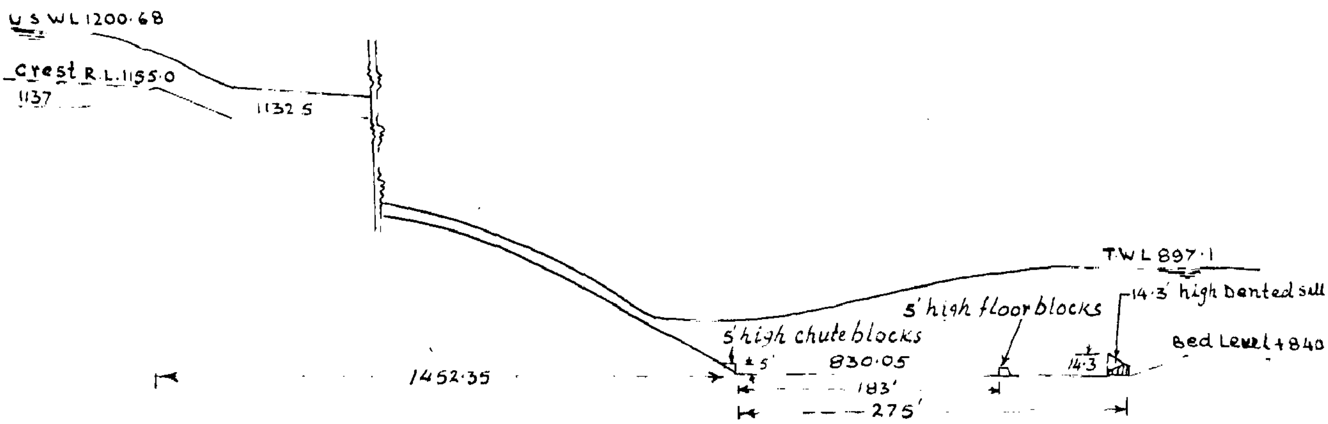
$$L_B = \frac{4.5 D_2}{\lambda^{0.38}}.$$

From the above formulae for basin length, it is seen that for λ equel to 3 the basin length comes to 3 D_2 and for λ equel to 300 it comes to $0.5D_2$. This means that basin length goes on reducing as λ in-creases. Considering the length of natural jump as $6D_2$ the reduction in basin length is more than 90% for λ equel to 300. This anomaly indicates that this type of basin can be used only for low range of Froude numbers. Vente Chow[22] also suggests that this basin is suitable for small spillways, outlet

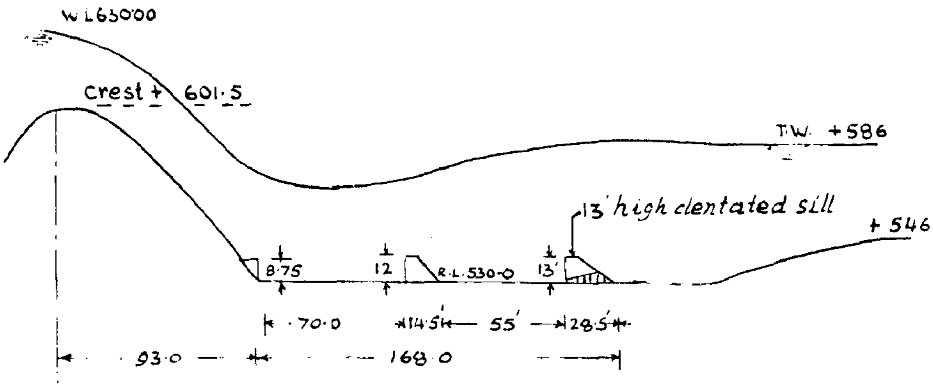


S.A.F. STILLING BASIN

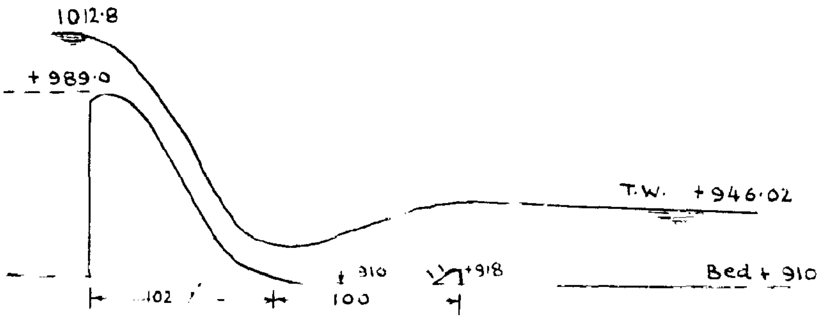
FIG. 5-10



RAMGANGA CHUTE SPILLWAY



OBRA DAM SPILLWAY



MATATILA DAM SPILLWAY

FIG. 5-11

works and canal structures.

U.S.B.R. Basins I.

These are natural stilling basins with horizontal apron without any appurtenances. As the length of jump is too much these types of basins are costly. The jump position is also very sensitive to tailwater variations.

U.S.B.R. Basin II.

These are used for high dam and large outlets. This type of basin is effective for Froude number greater than 4. The appurtenances used are chute blocks and end sills only. Baffle piers are not provided as velocities are generally high. It is not necessary to stagger the chute blocks with respect to sill dentates. For narrow basins which have few dentates the width and spacing may be reduced further than shown in Fig. 5-2. The apron elevation is set to utilise the full sequent depth plus an added factor of safely i.e. nearly 5% is advisable.

In cases where these basins are provided below chutes and if slope of chute is 1:1 or greater, the sharp intersection of chute with basin may be replaced by a curve (Radius $R \geqslant 4D_1$). This basin is suitable upto fall of 200 ft. and discharge intensity of 500 cusecs provided the flow at entry is uniform in velocity and depth.

U.S.B.R. Type III Basin.

This basin (Fig. 5-3) is used for small spillways and canal structures, similar to that of S.A.F. type basin. It has a higher factor of safety than S.A.F. stilling basin. The performance of this basin indicates that jump and basin length can be reduced by 60% with the help of appurtenances. This type of basin ~~were~~ not suitable when velocities are likely to exceeds 50-60 ft/sec. This type of basin may be quite effective for

Froude number as low as 4. U.S.B.R.[1] has not indicated the highest limit of Froude number upto which this basin is suitable.

Basins suggested by Elevatorski.

The basins (Fig.5-4 and5-5) recommended by Elevatorski are suitable for the size of structures similar to U.S.B.R. stilling Basin type II and III. The difference in dimensions of these basins with respective U.S.B.R. Basin has been already pointed out earliar in this chapter.

EXAMPLES OF STILLING BASINS WITH HORIZONTAL APRON.

The stilling basins with horizontal apron has been used and are proposed to be used at some of the projects in our country. Most of the dams below which this type of basin have been used are of low and medium height with moderate discharge intensities. For example, these type of stilling basins have been adopted at Matatila Dam (ht.79'), Obra Dam (ht.71.5') and Ramganga chute spillway (ht.325') having discharge intensities 435, 696, and 594 cusecs respectively. The details of these stilling basins are shown in Fig. 5-11.

In the above paras, the factors affecting the design of basins and design details of basins with horizontal apron have been discussed. The points of difference between the various authorities have been broughtout. The limitations of each type of basins have been indicated. A few examples of stilling basins adopted in this country are also given. The stilling basins with sloping apron in all its aspects will be discussed in the following chapter.

@@££@@

CHAPTER 6.

HYDRAULIC JUMP STILLING BASINS WITH SLOPING APRON.

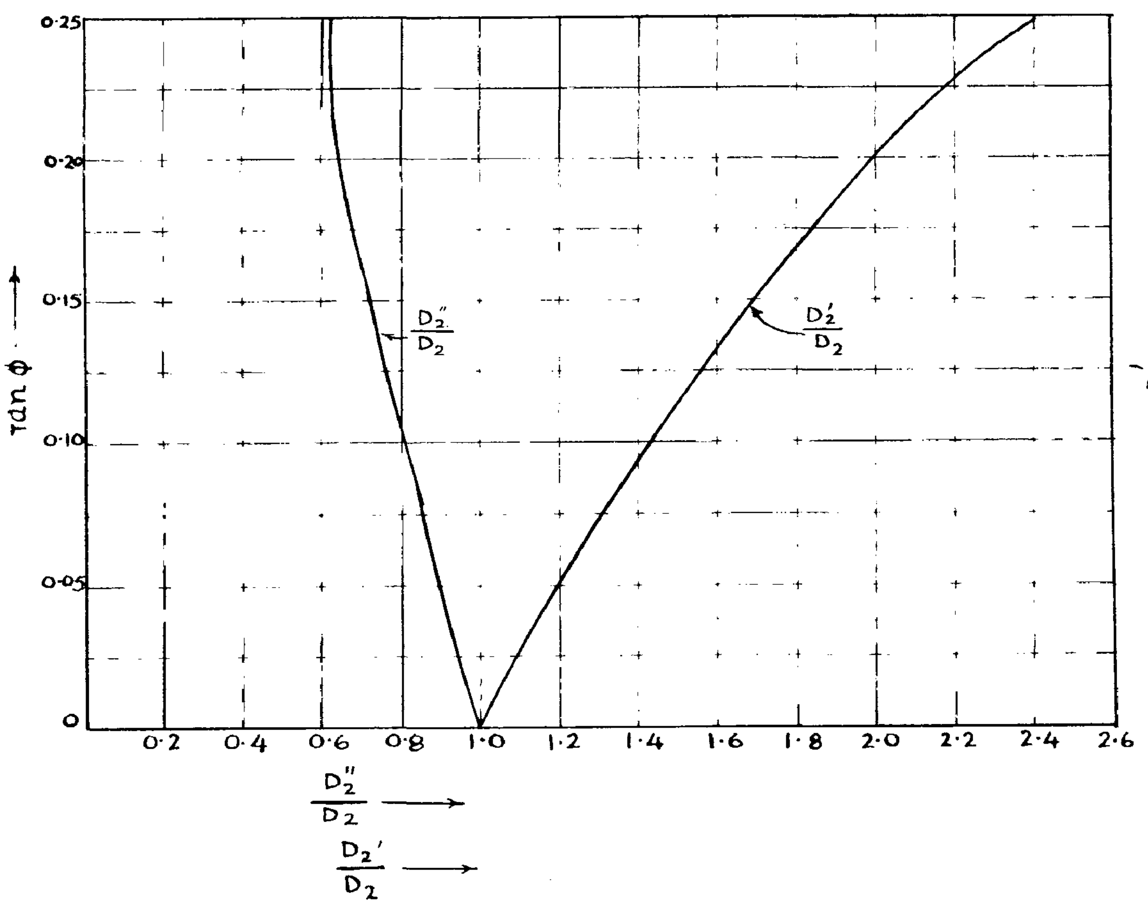
The stilling basins with sloping apron like horizontal apron also utilise the principle of hydraulic jump as means of energy dissipation. The necessity of adopting sloping apron has been discussed in chapter 4 while discussing the choice of specific type of energy dissipation arrangements. The sloping aprons are necessary for the satisfactory performance of basins where the tailwater is in excess or deficient for a good range of discharges. When the tail-water depth is in excess of jump requirements, the sloping apron is provided above the level of stream bed. In case the tailwater depth is less than required for formation of jump, sloping apron below the stream bed level is provided.

The jump on sloping apron takes many forms depending upon the slope, arrangements of the apron, Froude number and discharge intensity. The extent of loss of energy in jump depends upon the slope of the apron and kineticity. According to Stevens[8] analysis which has been discussed in Chapter 3, the energy loss in a jump increases with Kineticity and decreases as apron slope increases (Fig. 3-10). The loss of energy in hydraulic jump on horizontal apron is more than the jump on sloping apron. The difference in loss of energy in jumps on horizontal apron and sloping apron is more pronounced when the apron slope is greater than 10% and Kineticity is low. Inspite of fact that there is slightly less loss of energy in jump on sloping apron as compared to jump on horozontal apron, extensive use of sloping apron has been made. The jump on sloping apron is better controlled and its position is not much affected by tailwater variations unlike jump on a horizontal apron. In case of horizontal apron, the slight deficiency in tailwater will shift the jump towards downstream and even the jump sometimes may form outside the basin resulting in undesireable scouring of river bed. This is the reason that few stilling basins with horizontal apron have been designed for large structures. On the other hand, the jump on fully sloping or part sloping and part horizontal apron, moves up and down the slope according to tailwater variations and the range of jump movement is limited. This ensures better functioning of stilling basins.

The studies on sloping apron and part sloping part horizontal apron, as stated earliar, were carried out by Bradley and Peterka[2]. These investigators have produced valuable data which can be usefully applied to the design of stilling basins.

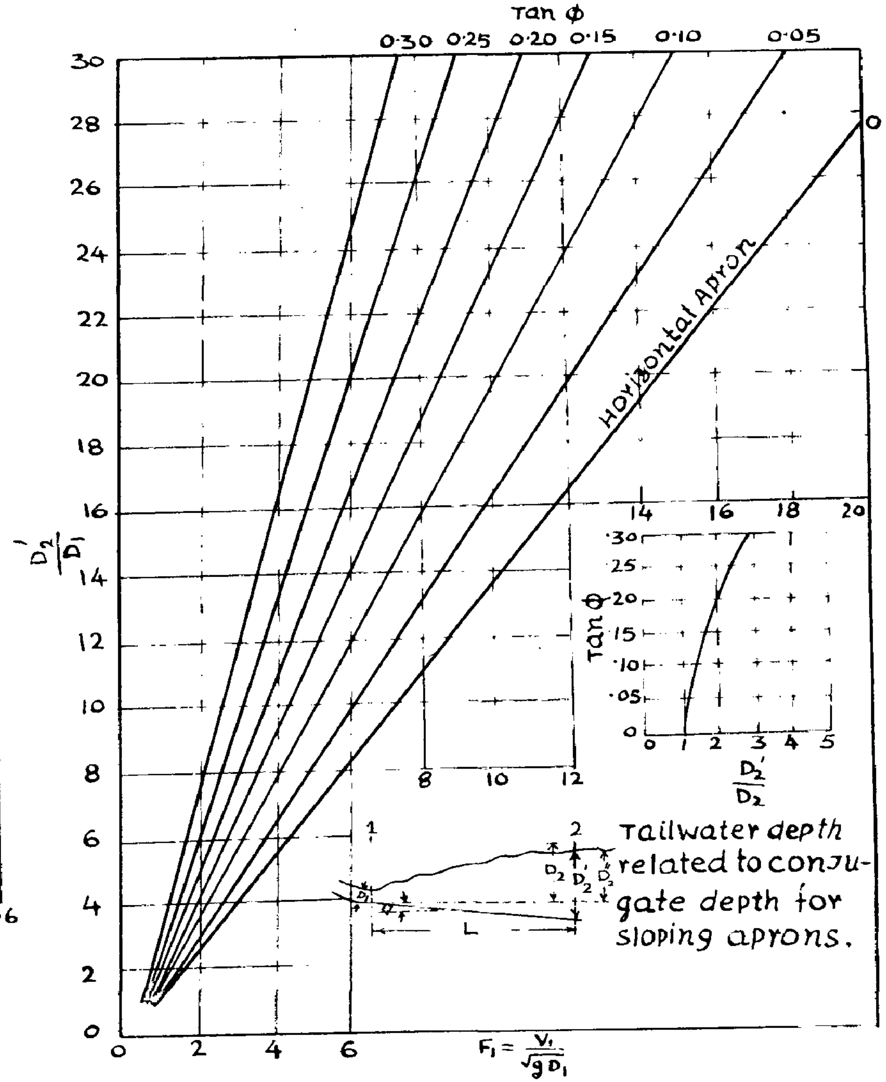
BASINS WITH FULLY SLOPING APRON.

The experimental studies by the above authors have clearly established that tailwater requirement for the formation of jump and the length of jump depend upon the slope of apron and Froude number. The charts prepared by the authors showing the plot $\frac{L}{D_2}$, vs. F_1 $\frac{L}{D_2}$ versus F_1 and $\frac{D_2'}{D_1}$ versus F_1 for different apron slopes are shown in Fig. 3-5, Fig. 3-6 and Fig. 6-1 respectively. The plot between $\frac{D_2'}{D_1}$ and F_1 for different slopes indicates that for same value of F_1 the $\frac{D_2'}{D_1}$ goes on increasing with slope. The $\frac{D_2'}{D_1}$ also increases as F_1 increases for same slope. Another small plot in same figure indicates the relation between $\frac{D_2'}{D_2}$ and slope. It is seen that $\frac{D_2'}{D_2}$ goes on increasing as the slope increases. The plot between $\frac{L}{D_2}$ and F_1, $\frac{L}{D_2'}$, and F_1 show that $\frac{L}{D_2}$ increases with the slope while $\frac{L}{D_2'}$, decreases, with slope for the same value of F_1. The values of $\frac{L}{D_2'}$



tan ϕ
0·25
0·20
0·15
0·10
0·05
0
0·2
0·4
0·6
0·8
1·0
1·2
1·4
1·6
1·8
2·0
2·2
2·4
2·6
$\frac{D_2''}{D_2}$
$\frac{D_2'}{D_2}$

FIG. 6-2



Tan ϕ
0·30
0·25
0·20
0·15
0·10
0·05
0
Horizontal Apron
$\frac{D_2'}{D_1}$
$F_1 = \frac{V_1}{\sqrt{g D_1}}$
tan ϕ
$\frac{D_2'}{D_2}$
Tailwater depth related to conjugate depth for sloping aprons.
L

FIG. 6-1

and $\frac{L}{D_2}$ are maximum for medium range of Froude numbers and reduce for higher and lower range of Froude numbers (Fig.3-5 and 3-6).

The above charts donot clearly bringout the extent of tailwater requirements for the formation of jump on different slopes. From these charts little idea can be had about quantitative require-ments of tailwater for different slopes and Froude numbers. In order to bringout explicitly the effect of slope and Froude number on the tailwater requirement for the formation of jump, an analysis on the basis of these charts, has been attempted and is detailed in the following steps.

(1) Firstly a value of D_1 as 1 ft. is assumed for convenience. This value will not affect the results as charts are in dimensionless form.

(2) A value of F_1 is assumed and $\frac{D_2'}{D_1}$ from Fig. 6-1 is determined. D_1 being known, D_2' can be determined. This is done for all values of slopes ranging from 0.00 to 0.25. At zero slope the value of D_2' is obiously same as D_2.

(3) From Fig. 3-5, $\frac{L}{D_2'}$, for same F_1 is determined. D_2' is already known and so length of jump L can be determined. The value of L is found out for all slopes.

(4) Knowing the value of L, the drop in bed level between start of jump and end of jump i.e. L $\tan\phi$ is calculated for all slopes.

(5) The depth of water D_2'' i.e. above horizontal plane passing through the point on sloping apron where the jump starts (Fig.6-1), is calculated for all slopes by subtracting L $\tan\phi$ from D_2'.

(6) Knowing D_2'' for all slopes for a particular value of F_1 and D_2, the ratio $\frac{D''_2}{D_2}$ is determined.

The above steps are again repeated for different values of Froude numbers ranging from 4 to 18 and $\frac{D_2''}{D_2}$ is determined. The

TABLE No. 6-1

Assume $D_1 = 1$ ft.

S. No.	F_1	tanØ	$\frac{D'_2}{D_1}$	D'_2	D_2	$\frac{D_2'}{D_2}$	$\frac{L}{D_2'}$	L	LtanØ	D''_2	$\frac{D''_2}{D_2}$
1.	4	0.05	6.25	6.25	5.25	1.190	4.80	30.00	1.50	4.75	0.905
		0.10	7.50	7.50		1.430	4.10	30.75	3.07	4.42	0.842
		0.15	8.80	8.80		1.670	3.60	31.50	4.73	4.07	0.775
		0.20	10.50	10.50		2.000	3.15	33.10	6.62	3.88	0.739
		0.25	12.50	12.50		2.480	2.78	34.75	8.70	3.80	0.735
2.	6	0.05	9.70	9.70	8.00	1.200	5.10	49.50	2.47	7.23	0.902
		0.10	11.60	11.60		1.450	4.33	50.30	5.03	6.57	0.821
		0.15	13.70	13.70		1.710	3.80	52.10	7.81	5.79	0.724
		0.20	16.00	16.00		2.000	3.33	53.30	10.66	5.33	0.667
		0.25	19.25	19.25		2.410	2.93	56.40	14.10	5.15	0.643
3.	8	0.05	13.00	13.00	10.8	1.200	5.20	67.60	3.38	9.62	0.890
		0.10	15.70	15.70		1.450	4.40	69.10	6.91	8.79	0.815
		0.15	18.50	18.50		1.710	3.85	71.20	10.68	7.82	0.723
		0.20	21.70	21.70		1.010	3.40	73.80	14.76	6.94	0.642
		0.25	26.00	26.00		2.410	3.00	78.10	19.50	6.50	0.602
4.	10	0.05	16.40	16.40	13.67	1.200	5.15	84.50	4.22	12.18	0.892
		0.10	19.67	19.67		1.440	4.37	86.00	8.60	11.07	0.811
		0.15	23.20	23.20		1.700	3.83	88.80	13.32	9.88	0.723
		0.20	27.16	27.16		1.980	3.37	91.50	18.30	8.85	0.648
		0.25	32.50	32.50		2.380	2.96	96.20	24.05	8.45	0.618
5.	12	0.05	19.75	19.75	16.50	1.197	5.10	100.70	5.03	14.72	0.892
		0.10	23.80	23.80		1.430	4.33	103.00	10.30	13.50	0.810
		0.15	28.00	28.00		1.697	3.80	106.50	15.97	12.03	0.728
		0.20	32.50	32.50		1.970	3.35	109.00	21.80	10.70	0.648
6.	14	0.05	23.17	23.17	19.30	1.210	5.00	115.80	5.79	17.38	0.901
		0.10	27.75	27.75		1.430	4.25	118.00	11.80	15.95	0.825
		0.15	32.67	32.67		1.690	3.70	120.10	18.00	14.50	0.750
7.	16	0.05	26.50	26.50	22.20	1.195	4.88	129.50	6.48	20.02	0.901
		0.10	31.75	31.75		1.430	4.15	131.70	13.17	18.58	0.837
8.	18	0.05	29.80	29.80	25.00	1.190	4.73	144.00	7.20	22.60	0.903

TABLE No. 6.II

S.No.	Slope	F_1	$\frac{D''_2}{D_2}$	Average value.	$\frac{D'_2}{D_2}$	Average value.	Remarks.
1.	0.05	4	0.905		1.190		
		6	0.840		1.210		
		8	0.890	0.896	1.203		
		10	0.892		1.200	1.199	
		12	0.892		1.197		
		14	0.901		1.210		
		16	0.910		1.195		
		18	0.903		1.190		
2.	0.10	4	0.842		1.430		
		6	0.821		1.450		
		8	0.815		1.453	1.438	
		10	0.811	0.824	1.440		
		12.	0.818		1.430		
		14	0.825		1.430		
		16	0.837		1.430		
		18	-		-		
3.	0.15	4	0.775		1.675		
		6	0.724		1.710		
		8	0.723	0.730	1.710		
		10	0.723		1.700	1.697	
		12	0.728		1.197		
		14	0.750		1.690		
4.	0.20	4	0.739*		2.000		
		6	0.667		2.000	1.992	
		8	0.642	0.650	2.010		
		10	0.648		1.980		
		12	0.648		1.970		
5.	0.25	4	0.725*		2.480		
		6	0.643		2.410		
		8	0.602	0.621	2.410		
		18	0.618		2.380	2.420	

* Not considered while averaging.

results are shown in Table 6-1. The value of $\frac{D'_2}{D_2}$ is also found out for different Froude numbers and slopes.

From the table it is seen that values of $\frac{D_2'}{D_2}$ and $\frac{D_2''}{D_2}$ are nearly constant for different Froude numbers for same slope of apron. This leads to the obvious conclusion that $\frac{D_2'}{D_2}$ and $\frac{D_2''}{D_2}$ are independent of Froude number for a particular slope. The value of these dimensionless parameters are averaged and are shown in table 6-II. While averaging some of the values shown in table 6-II, have been left out as these may be inaccurate because they have been obtained by extrapolating the plot between $\frac{L_r}{D_2}$ and F_1. These average values of $\frac{D_2''}{D_2}$ have been plotted against $\tan\phi$ and shown in fig. 6-2. A study of this plot indicates that $\frac{D_2''}{D_2}$ decreases with slope. For $\tan\phi$ equal to 0.25, the value of $\frac{D_2''}{D_2}$ is equeal to 0.62 nearly, which means that the tailwater requirement for the formation of jump gets reduced to 62% of the requirement for the corresponding case of jump on horizontal apron. The value of $\frac{D_2''}{D_2}$ decreases comparatively rapidly upto $\tan\phi$ equal to 0.20. This clearly gives an indication that by providing a sloping apron with slopes ranging upto 0.20 or 0.25 the tailwater requirement can be substantially reduced. On the same figure $\frac{D_2'}{D_2}$ is also plotted against $\tan\phi$ and it is seen that $\frac{D_2'}{D_2}$ increases with slope.

The above presentation gives a better idea about the tailwater requirements for the formation of jump for different slopes. Knowing the availability of tailwater in a particular case the apron slope can be determined without much difficulty from the Fig. 6-2. The basins with fully sloping aprons have been used below many dam spillways some of which are listed in Table 6-V.

BASINS WITH PART SLOPING AND PART HORIZONTAL APRON.

While discussing the basins with fully sloping apron it has been established that tailwater requirement for the formation of jump goes on reducing as the apron slopes increase. Though the maximum reduction in tailwater requirements can be obtained by steepening the slope, but the steepening of slope and continuing the apron at that slope, leads to huge amount of rock excavation in apron, which may make the stilling basin very costly. This lead to the study of tailwater requirements for part sloping and part horizontal apron. The provision of part sloping and part horizontal apron involves less rock excavation than a fully sloping apron.

The effect on tailwater requirement for partsloping and part horizontal apron for different slopes was studied by the same authors through a series of extensive experiments. These authors established that $\frac{D_2'}{D_2}$ increases with the increase in slope for the same $\frac{l}{D_2}$ ratio (Fig. 3-12) where l represents the horizontal length of sloping portion of apron. The ratio $\frac{D_2'}{D_2}$ increases as $\frac{l}{D_2}$ increases for a particular slope. The charts prepared by the authors showing the plot between $\frac{l}{D_2}$ and $\frac{D_2'}{D_2}$ (Fig. 3-12) for slopes ranging from 0.05 to 0.30, provide valuable basis for the design of such type of stilling basins.

As far as the total length of jump is concerned, the authors are of the opinion that the length of jump, in case of part sloping and part horizontal apron, is not much different than the case of fully sloping apron. The exact length of jump is not important as far as design of stilling basins are concerned because these are seldom designed to contain the full length of jump.

From the above mentioned charts of Bradley and Peterka showing the relationship between $\frac{D_2'}{D_2}$ and $\frac{l}{D_2}$ for different apron slopes, little quantitative idea can be had about tailwater requirements for jump formation in comparision with that of fully sloping aprons. How the tailwater requirement is affected if the slope is steepened and length of sloping apron is increased or decreased, is not explicit in the charts. Therefore a study about the variation in tailwater requirements for different slopes and $\frac{\ell}{D_2}$ ratios have been attempted on the basis of these charts, and presented in a manner which may be more convinient to the designer. The study involves the determination of $\frac{D_2''}{D_2}$ where D_2'' is depth of tailwater above horizontal plane passing through the point on sloping portion of apron where the jump starts, for different $\frac{\ell}{D_2}$ ratios and slopes. The study made, is described in the following steps:-

(1) First the value of D_1 is assumes as 1 ft. As already stated earliar, this will not affect the ratio $\frac{D_2''}{D_2}$ as the charts have been prepared in dimensionless form.

(2) A reasonable value of F_1 is assumed and D_2 is obtained from the plot showing the relation between $\frac{D_2}{D_1}$ and F_1 for horizontal apron (Fig.6-1). D_1 being known, D_2 can be determined.

(3) The length of jump is determined from the Fig. 3-6 which shows the plot between $\frac{L}{D_2}$ and F_1. Knowing D_2 the length of jump is determined.

(4) After knowing the length L, ℓ is determined for a particular slope for different values of $\frac{\ell}{L}$ i.e. 0.0, 0.10, 0.2, 0.3, 0.4, 0.5, 0.6, 0.8, 1.0.

(5) The ratio $\frac{\ell}{D_2}$ is determined as ℓ and D_2 and known.

(6) From Fig. 3-12, showing the plot $\frac{D_2'}{D_2}$ versus $\frac{\ell}{D_2}$, the values of $\frac{D_2'}{D_2}$ are determined for all sets of $\frac{\ell}{D_2}$ ratios.

(7) Knowing $\frac{D_2'}{D_2}$, D_2' is determined.

(8) The drop in bed level in sloping apron portion i.e. $\ell \tan\phi$ is calculated for various values of ℓ for a particular slope.

(9) The depth D_2'' is determined by substracting $\ell \tan\phi$ from D_2'

(10) D_2'' and D_2 being known, the value of $\frac{D_2''}{D_2}$ for all values of $\frac{\ell}{D_2}$ ratios are calculated for a particular slope.

Assume $D_1 = 1$

S.De.	$\tan\phi$	$\frac{D_2''}{(D'_2 - \ell\tan\phi)}$	$\frac{D_2''}{D_2}$
1.	0.05	13.43	0.980
		13.20	0.965
		12.98	0.950
		12.77	0.933
		12.55	0.917
		12.47	0.910
		12.39	0.900
		12.19	0.890
		12.10	0.883
2.	0.10	13.43	0.980
		12.89	0.942
		13.36	0.903
		12.03	0.880
		11.78	0.860
		11.54	0.845
		11.47	0.838
		11.37	0.830
		11.28	0.823
3.	0.15	13.43	0.980
		12.49	0.911
		11.81	0.863
		11.32	0.826
		10.83	0.795
		10.67	0.780
		10.56	0.770
		10.45	0.763
		10.45	0.763
4.	0.20	13.43	0.980
		12.11	0.883
		11.16	0.815
		10.47	0.765
		10.14	0.740
		9.98	0.729
		9.91	0.723
		9.90	0.723
		9.89	0.722
5.	0.25	13.43	0.980
		11.83	0.865
		10.53	0.768
		9.65	0.705
		9.30	0.680
		9.20	0.673
		9.20	0.673
		9.20	0.673
		9.20	0.673

Assume $D_1 = 1$ ft.

S.No.	$\tan\phi$	F_1	$\frac{D_2}{D_1}$	D_2	$\frac{L}{D_2}$	L	$\frac{\ell}{L}$	ℓ	$\frac{\ell}{D_2}$	$\frac{D'_2}{D_2}$	D'_2	$\ell\tan\phi$	$D_2'' = (D'_2 - \ell\tan\phi)$	$\frac{D_2''}{D_2}$
1.	0.05	10	13.70	13.70	6.23	85.3	0.0	0.00	0.000	0.980	13.43	0.00	13.43	0.980
							0.1	8.53	0.623	0.995	13.63	0.43	13.20	0.965
							0.2	17.06	1.250	1.010	13.83	0.85	12.98	0.950
							0.3	25.59	1.870	1.027	14.05	1.28	12.77	0.933
							0.4	34.12	2.495	1.047	14.34	1.70	12.55	0.917
							0.5	42.65	3.120	1.067	14.60	2.13	12.47	0.910
							0.6	51.18	3.740	1.087	14.88	2.56	12.39	0.900
							0.8	68.24	4.980	1.140	15.60	3.41	12.19	0.890
							1.0	85.30	6.230	1.195	16.36	4.26	12.10	0.883
2.	0.10	10	13.70	13.70	6.37	87.2	0.0	0.00	0.000	0.980	13.43	0.00	13.43	0.980
							0.1	8.72	0.637	1.005	13.76	0.87	12.89	0.942
							0.2	17.44	1.270	1.030	14.10	1.74	13.36	0.903
							0.3	26.16	1.910	1.070	14.65	2.62	12.03	0.880
							0.4	34.88	2.550	1.115	15.27	3.49	11.78	0.860
							0.5	43.60	3.180	1.160	15.90	4.36	11.54	0.845
							0.6	52.32	3.820	1.220	16.70	5.23	11.47	0.838
							0.8	69.76	5.100	1.340	18.35	6.98	11.37	0.830
							1.0	87.20	6.370	1.460	20.00	8.72	11.28	0.823
3.	0.15	10	13.70	13.70	6.54	89.6	0.0	0.00	0.000	0.980	13.43	0.00	13.43	0.980
							0.1	8.96	0.654	1.010	13.83	1.34	12.49	0.911
							0.2	17.62	1.310	1.057	14.50	2.69	11.81	0.863
							0.3	26.90	1.965	1.120	15.35	4.03	11.32	0.826
							0.4	35.94	2.620	1.185	16.25	5.37	10.83	0.795
							0.5	44.80	3.270	1.270	17.40	6.73	10.67	0.780
							0.6	53.75	3.930	1.360	18.63	8.07	10.56	0.770
							0.8	71.70	5.230	1.555	21.20	10.75	10.45	0.763
							1.0	89.60	6.540	1.750	23.90	13.45	10.45	0.763
4.	0.20	10	13.70	13.70	6.73	92.2	0.0	0.00	0.000	0.980	13.43	0.00	13.43	0.980
							0.1	9.22	0.673	1.020	13.95	1.84	12.11	0.883
							0.2	18.44	1.346	1.085	14.85	3.69	11.16	0.815
							0.3	27.26	2.020	1.170	16.00	5.53	10.47	0.765
							0.4	36.88	2.690	1.280	17.52	7.38	10.14	0.740
							0.5	46.10	3.380	1.400	19.20	9.22	9.98	0.729
							0.6	55.30	4.040	1.530	20.97	11.06	9.91	0.723
							0.8	73.75	5.380	1.800	24.65	14.75	9.90	0.723
							1.0	92.20	6.730	2.070	28.33	18.44	9.89	0.722
5.	0.25	10	13.70	13.70	6.95	96.0	0.0	0.00	0.000	0.980	13.43	0.00	13.43	0.980
							0.1	9.60	0.695	1.040	14.23	2.40	11.83	0.865
							0.2	19.30	1.390	1.120	15.33	4.80	10.53	0.768
							0.3	28.80	2.080	1.230	16.85	7.20	9.65	0.705
							0.4	38.40	2.780	1.375	18.90	9.60	9.30	0.680
							0.5	48.00	3.470	1.540	21.20	12.00	9.20	0.673
							0.6	57.60	4.170	1.720	23.60	14.40	9.20	0.673
							0.8	76.80	5.560	2.075	28.40	19.20	9.20	0.673
							1.0	96.00	6.950	2.430	33.20	24.00	9.20	0.673

TABLE No. IV.

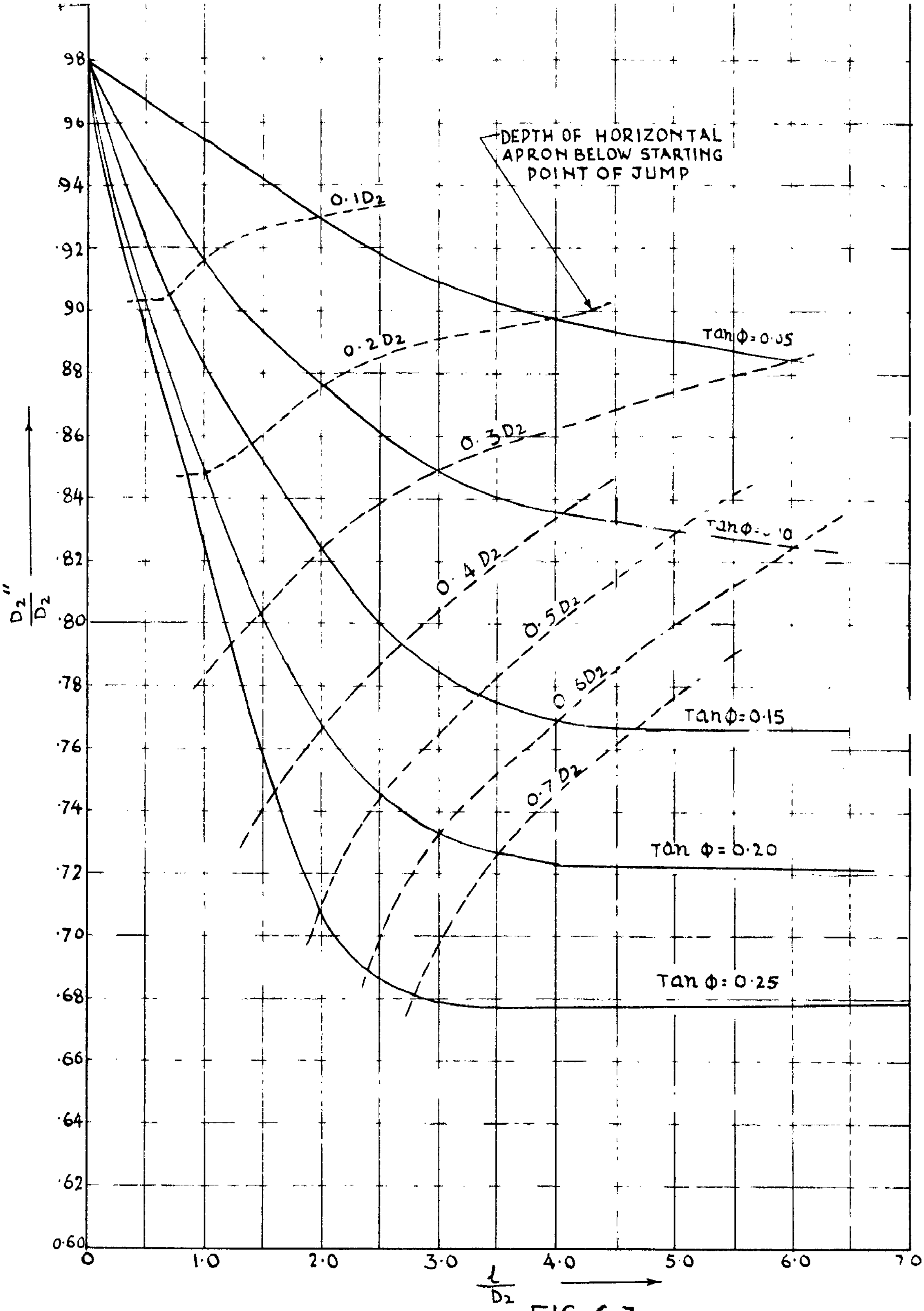
Assume D_1 = 1 ft.

S.No.	$\tan\phi$	F_1	$\frac{D_2}{D_1}$	D_2	$\frac{L}{D_2}$	L	$\frac{\ell}{L}$	ℓ	$\frac{\ell}{D_2}$	$\frac{D'_2}{D_2}$	D'_2	$\ell\tan\phi$	D''_2 $(D'_2 - \ell\tan\phi)$	$\frac{D''_2}{D_2}$
1.	0.05	6	8.0	8.0	6.15	49.20	0.0	0.00	0.00	0.980	7.84	0.00	7.84	0.980
							0.1	4.92	0.61	0.995	7.97	0.25	7.74	0.967
							0.2	9.84	1.23	1.010	8.08	0.49	7.59	0.950
							0.3	14.76	1.84	1.025	8.20	0.74	7.46	0.933
							0.4	19.68	2.46	1.045	8.36	0.98	7.38	0.922
							0.5	24.60	3.07	1.065	8.52	1.23	7.29	0.911
							0.6	29.52	3.69	1.087	8.67	1.48	7.19	0.900
							0.8	39.36	4.92	1.113	9.06	1.97	7.09	0.888
							1.0	49.20	6.15	1.190	9.52	2.46	7.06	0.883
2.	0.10	6	8.0	8.0	6.32	50.56	0.0	0.00	0.00	0.980	7.84	0.00	7.84	0.980
							0.1	5.06	0.63	1.005	8.04	0.51	7.53	0.942
							0.2	10.11	1.26	1.030	8.24	1.01	7.32	0.903
							0.3	15.17	1.90	1.070	8.56	1.52	7.05	0.880
							0.4	20.22	2.53	1.113	8.90	2.02	6.88	0.860
							0.5	25.28	3.16	1.160	9.28	2.53	6.75	0.844
							0.6	30.34	3.79	1.215	9.72	3.03	6.69	0.837
							0.8	40.45	5.06	1.327	10.62	4.04	6.58	0.823
							1.0	50.56	6.32	1.455	11.62	5.06	6.56	0.820
3.	0.15	6	8.0	8.0	6.50	52.00	0.0	0.00	0.00	0.980	7.84	0.00	7.84	0.980
							0.1	5.20	0.65	1.010	8.08	0.78	7.30	0.912
							0.2	10.40	1.30	1.055	8.44	1.56	6.88	0.862
							0.3	15.60	1.95	1.115	8.92	2.32	6.60	0.825
							0.4	20.80	2.60	1.185	9.48	3.12	6.36	0.795
							0.5	26.00	3.25	1.265	10.13	3.90	6.23	0.780
							0.6	31.20	3.90	1.355	10.84	4.68	6.16	0.770
							0.8	41.60	5.20	1.550	12.40	6.24	6.16	0.770
							1.0	52.00	6.50	1.740	13.92	7.80	6.12	0.767
4.	0.20	6	8.0	8.0	6.72	53.6	0.0	0.00	0.00	0.980	7.84	0.00	7.84	0.980
							0.1	5.36	0.67	1.020	8.16	1.07	7.09	0.886
							0.2	10.72	1.34	1.080	8.64	2.14	6.50	0.813
							0.3	16.08	2.01	1.170	9.36	3.22	6.14	0.768
							0.4	21.44	2.68	1.275	10.20	4.29	5.91	0.729
							0.5	26.80	3.35	1.400	11.20	5.36	5.84	0.725
							0.6	32.16	4.02	1.530	12.24	6.43	5.81	0.724
							0.8	42.88	5.36	1.800	14.40	8.60	5.80	0.724
							1.0	53.60	6.70	2.065	16.52	10.72	5.80	0.724
5.	0.25	6	8.0	8.0	6.94	55.50	0.0	0.00	0.00	0.980	7.84	0.00	7.84	0.980
							0.1	5.55	0.69	1.035	8.28	1.39	6.89	0.863
							0.2	11.10	1.39	1.115	8.92	2.77	6.15	0.768
							0.3	16.65	2.08	1.230	9.84	4.17	5.67	0.707
							0.4	22.20	2.78	1.375	11.00	5.55	5.45	0.681
							0.5	27.75	3.47	1.540	12.32	6.94	5.38	0.673
							0.6	33.30	4.16	1.715	13.72	8.34	5.38	0.673
							0.8	44.40	5.55	2.060	16.48	11.10	5.38	0.673
							1.0	55.50	6.94	2.410	19.28	13.90	5.38	0.673

Same procedure i.e. repetition of steps (1) to (10) is followed for determining the value of $\frac{D_2''}{D_2}$ for different slopes varying from 0.05 to 0.25. The table 6-III shows the computations of $\frac{D_2''}{D_2}$ for Froude number equal to 10 with slopes having $\tan\phi$ equal to 0.05, 0.10, 0.15, 0.20, 0.25 . Another similar table is prepared for Froude number equal to 6, having the same range of slopes, and $\frac{D_2''}{D_2}$ is determined. The results are shown in table 6-IV.

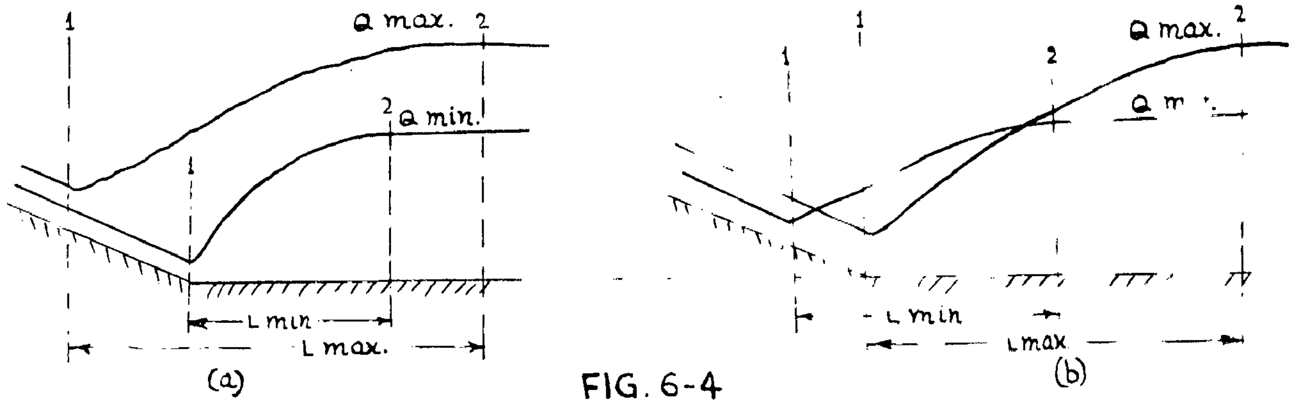
A comparision of table No. 6-III and Table 6-IV shows that $\frac{D''_2}{D_2}$ for a particular value of $\frac{l}{D_2}$, is independent of Froude number and is dependent only on slope. The values of $\frac{D''_2}{D_2}$ goes on decreasing as $\frac{l}{D_2}$ increases for each particular slope. A chart showing the relation between $\frac{l}{D_2}$ and $\frac{D_2''}{D_2}$ for different slopes is prepared (Fig.6-3). The plot between $\frac{l}{D_2}$ and $\frac{D_2''}{D_2}$ indicates that $\frac{D''_2}{D_2}$ decreases as $\frac{l}{D_2}$ increases, less rapidly for flat slopes than for steep slopes. For steep slopes $\frac{D''_2}{D_2}$ decreases very rapidly as $\frac{l}{D_2}$ increases to 3-4 and it is practically constant for higher values of $\frac{l}{D_2}$. This indicates that tailwater requirement goes on reducing rapidly for steeper slopes as length of sloping apron increases to nearly 40-60% of jump length. Any further increase in length of sloping apron causes little reduction in tailwater requirement while quantity of rock excavation increases. This conclusion is very important for the design of stilling basins with part sloping and part horizontal apron. Another fact which is seen from the ~~value~~ figure is that the value of $\frac{D''_2}{D_2}$ goes on decreasing as the slope increases for the same value of $\frac{l}{D_2}$. In other words an apron consisting of part steeper slope and part horizontal, will require less tailwater depth than part flatter sloping apron of same length and part horizontal apron.

In order to study the quantitative reduction in tailwater requirement more clearly with reference to different apron slopes



DEPTH OF HORIZONTAL APRON BELOW STARTING POINT OF JUMP
0·1D2
0·2D2
0·3D2
0·4D2
0·5D2
0·6D2
0·7D2
Tan Φ=0·05
Tan Φ=0·15
Tan Φ=0·20
Tan Φ=0·25
D2''/D2
l/D2

FIG. 6-3



Q max.
Q min.
L min
L max.
(a)
(b)

FIG. 6-4

having horizontal apron at the same level for all slopes, a new set of lines representing same horizontal level below the level of foot of start of jump in terms of sequent depth D_2 are drawn on the plot showing the relation between $\frac{\ell}{D_2}$ and $\frac{D''_2}{D_2}$. These lines are marked on the same figure (Fig. 6-3) as $0.1D_2$, $0.2D_2$, $0.3D_2$ and so on. These lines clearly show the quantitative reduction in tailwater requirement as the slope of apron increases keeping the horizontal part of apron at a particular level. The inference which can be drawn from this study is that by adopting a steeper slope and reaching a level of horizontal apron will require less tailwater depth, than providing a flatter part sloping apron and part horizontal apron at the same level. In other words, for the same tailwater, a part steeper sloping apron will require part horizontal apron at a higher level than part flatter sloping apron with part horizontal apron. Thus for the same tailwater, there can be many combinations of different part sloping part horizontal aprons at different levels. Therefore it cannot be stated with certainty as to which combination will involve the least rock excavation. A few trials with the help of fig. 6-3, will have to be made for determining the most economical combination of part sloping and part horizontal apron for a particular tailwater depth. Thus the figure 6-3 provides a very useful guide for determination of most economical combination of part sloping and part horizontal apron for a particular hydraulic condition.

OPTIMUM UTILITY OF PART SLOPING AND PART HORIZONTAL APRON.

It has already been discussed in chapter 4 that the use of hydraulic jump stilling basin with part sloping and part horizontal apron can be considered for the following hydraulic conditions:-

(1) The tailwater rating curve lies below jump height rating curve for low range of discharges and above for high range

of discharges.

(2) The tailwater rating curve lies above the jump height rating curve for low range of discharges and below for high range of discharges.

In the first case the horizontal part of apron can be so adjusted that the starting point of jump for small discharges is at the begining of horizontal apron. For large discharges the starting point of jump will move up along the sloping part of the apron and it will be at its upper most point at maximum discharge (Fig. 6-4a). In the second case the position will be reverse of the first case i.e. at the maximum discharge, the jump will start from the foot of sloping apron and for smaller discharges jump will start on sloping part of apron (Fig. 6-4b).

From the above, it is seen that in the first case the full length of sloping and horizontal aprons are utilised at maximum discharge condition and the lesser discharges are accomodated within this length of stilling basin. In the second case the jump at maximum discharge lies entirely on horizontal portion of apron requiring longer horizontal apron. It is thus clear that for the first case the basin with part sloping and part horizontal apron will be most suitable and economical. This type of basin can be utilised for the second case also but may not be economical as compared to other type of energy dissipation arrangements.

BASIN LENGTH.

It has already been stated earliar that it is not economical to provide stilling basin length to contain the full length of jump. In such cases the river bed downstream of paved apron also acts as part of stilling basin. The length of stilling basin is dependent on the river bed material. When the bed strata consists of good rock there is no difficulty and the length of paved apron can be considerably reduced. If the bed strata is of poor quality, a longer length of paved apron becomes necessary. A study of existing stilling basin (table No.6-V) indicate that the paved apron length varies from 40 to 80% of the jump length. The average length of basin appear to be 60% of jump length. According to U.S.B.R.[1] the paved apron length as 60% off jump length is sufficient for most of the cases unless the downstream bed strata is very poor. The shorter basin can be used where sound rock in bed exists.

STILLING BASIN APPURTENANCES.

In stilling basins with fully sloping apron or part sloping part horizontal apron, generally no appurtenances are provided except an end sill. In some cases the chute blocks also have been provided. The baffle piers are never provided as high velocities and discharge intensities are generally involved in such type of basins. The table No.6-V shows that velocities in most of cases are more than 50 ft/sec. The chute blocks have been provided in case of Keshwick and Dickinsion Dams where the velocities are comparatively not so high. In basins with high velocities the chute blocks are not favoured.

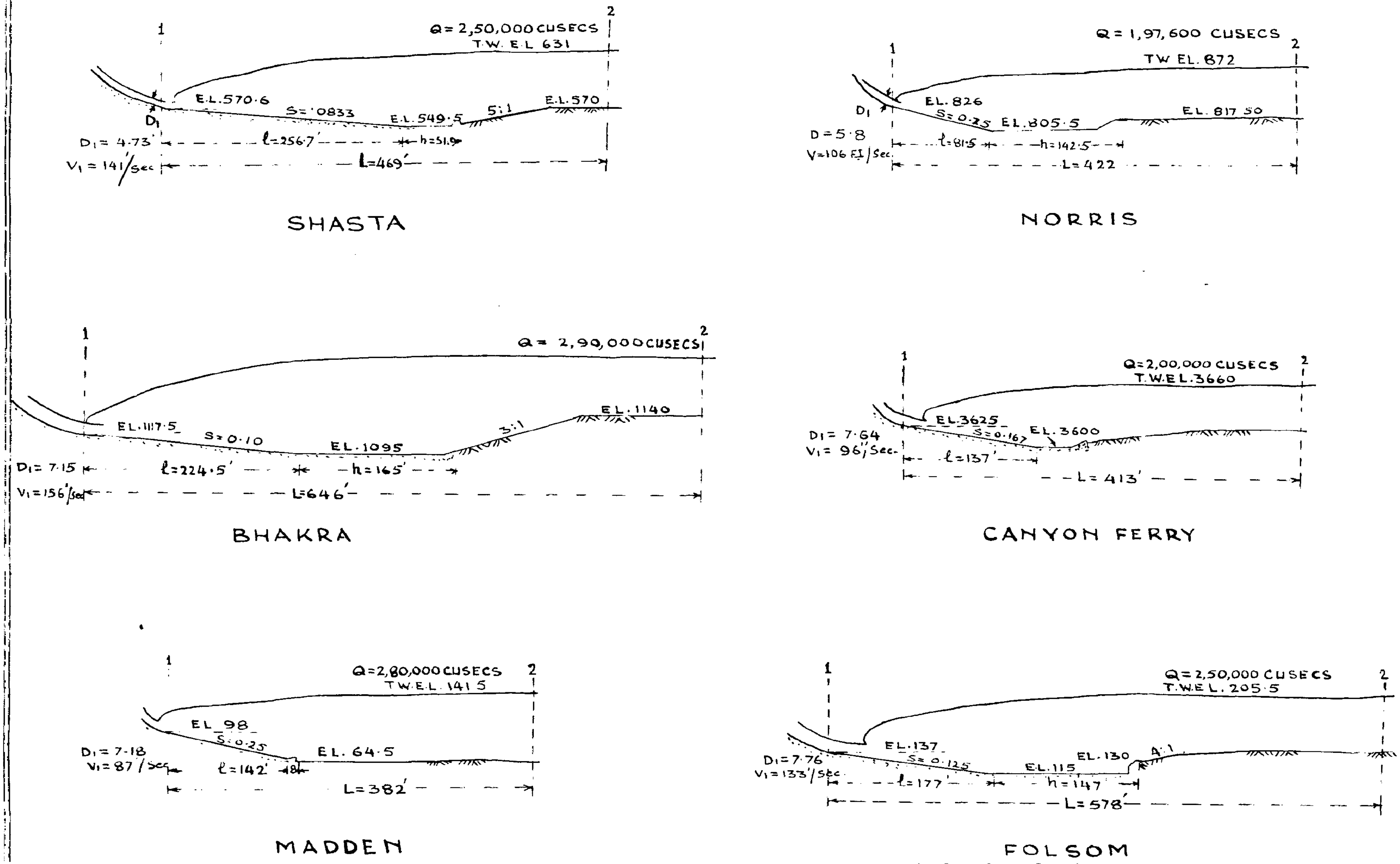
The use of appurtenance in the basin is not favoured also because the jump position is not fixed and it moves up and down according to tailwater availability. The appurtenances if used in the basins can be only effective for a particular position of jump i.e. for a particular discharge and for other discharges these will not be effective at all.

The end sills provided are either triangluar, rectangluar and for low discharges even dentated. According to U.S.B.R. practice[1] small solid triangular sill is only required. It serves to lift the flow as it leaves the apron and thus acts to control the scour. The most effective height is between $0.05D_2$ and $0.10D_2$ and a slope of 3:1 to 2:1 on the upstream face of the end sill, is quite

TABLE 6-V.

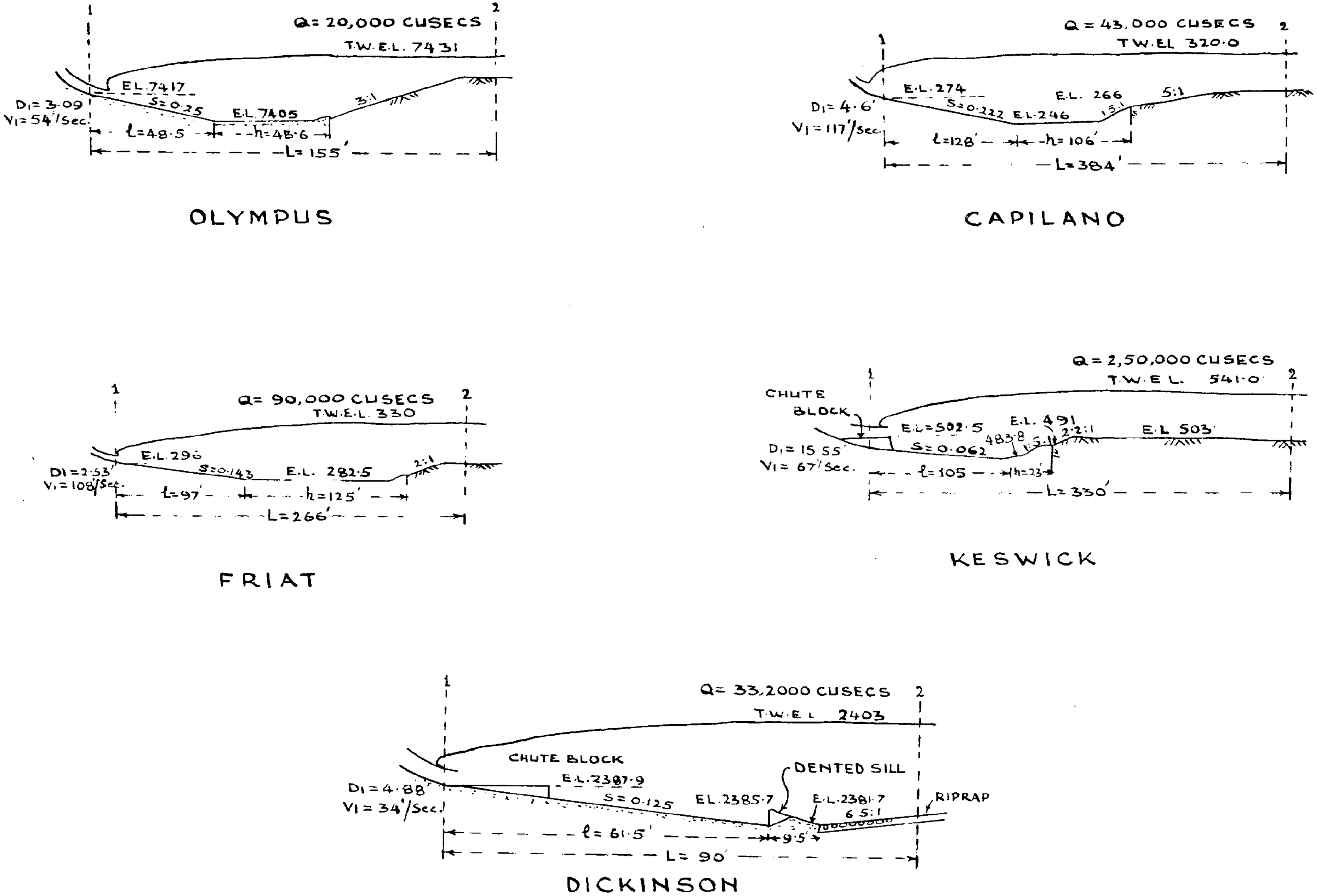
S.No.	Dam	Slope of dam face.	slope of apron tanø	Fall to up end of apron (ft.)	Total maximum discharge Q (Cusecs)	Discha-rge intensity q (cusecs)	Length of sloping apron ℓ	Length of horiz-ontal apron H (ft.)	ℓ+h (ft.)	Actual velocity at entry of basin (ft/sec)	D_1 (ft)	F_1	D_2 (ft)	$\frac{L}{D_2}$	Actual length of jump L (ft.)	$\frac{\ell+h}{L}$
1.	Shastra (U.S.A.)	0.8:1	0.083	491.40	2,25,000	667	256.7	51.9	308.6	141	4.73	11.42	74.5	6.30	469	0.66
2.	Norris (U.S.A.)	0.7:1	0.250	221.00	1,97,000	595	81.5	142.5	224.0	106	5.61	7.88	60.0	7.03	422	0.53
3.	Bhakra (India)	0.8:1	0.100	567.50	2,90,000	1115	224.5	165.0	389.5	156	7.15	10.27	101.5	6.36	646	0.60
4.	Canyon Ferry (U.S.A.)	Varies	0.167	175.60	2,00,000	738	137.0	57.0	194.0	96	7.69	6.10	63.1	6.55	413	0.47
5.	Madden (U.S.A.)	0.75:1	0.250	152.00	2,80,000	625	142.0	8.0	150.0	87	7.18	5.72	55.3	6.90	382	0.39
6.	Folsom (U.S.A.)	0.67:1	0.125	329.00	2,50,000	1033	177.0	147.0	324.0	133	7.76	8.41	88.9	6.50	578	0.56
7.	Olympus (U.S.A.)	Varies	0.250	58.00	20,000	167	48.5	43.6	92.1	54	3.09	5.41	22.6	6.86	155	0.59
8.	Capilano (British Columbia)	0.65:1	0.222	296.0	43,000	539	128.0	106.0	234.0	117	4.60	9.62	60.3	6.85	384	0.61
9.	Friant (U.S.A.)	0.7:1	0.143	282.00	90,000	273	97.0	125.0	222.0	108	2.53	11.97	41.6	6.40	266	0.83
10.	Keshwick (U.S.A.)	Varies	0.062	98.40	2,50,000	1042	105.0	23.0	128.0	68	15.15	3.12	60.6	5.45	330	0.39
11.	Dickinsion (U.S.A.)	0.5:1	0.125	40.90	33,200	166	61.5	9.5	71.0	48	3.50	4.44	20.0	6.00	120	0.59

Apron arrangements are shown in Fig. 6-5 (A) & (B).



EXAMPLES OF EXISTING STILLING BASINS WITH SLOPING APRON

FIG. 6·5 (A)



EXAMPLES OF EXISTING STILLING BASINS WITH SLOPING APRON

FIG. 6-5 (B)

sufficient.

U.S.B.R. PRACTICE FOR THE DESIGN OF BASINS WITH SLOPING APRON.

The U.S.B.R.[1] practice for the design of basin with sloping apron is described briefly as below :-

(i) The apron arrangement should be determined which will give the greatest economy for maximum discharge condition.

(ii) The apron position should be fixed by trial in such a way so that the front of jump will form at the upstream end of slope for maximum discharge and tailwater conditions.

(iii) The length of jump can be obtained from plot showing the relation F_1 and $\frac{L}{D_2}$. The length of basin for a average bed strata may be kept as 60% of jump length.

(iv) After designing the apron for maximum discharge condition, it should be ascertained that the tailwater depth and length of basin available for energy dissipation are enough for $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$ of maximum discharge. If tailwater depth is sufficient or in excess of jump height for intermediate discharges the design is acceptable. If the tailwater is difficient, then apron slope and position may be redesigned.

(v) Only a small solid triangular end sill is sufficient and dimensions are already stated above.

(vi) The stilling basin should be operated under symetrical flow as far as possible. The design of spillway should be such to help to attain this aim.

(vii) Where discharge intensities are greater than 500 cfts where asymetry of flow is involved, and where Froude number is high, the model studies should be conducted for finalising the design of the basin.

In the above, it is seen that for the step (i) and step (iv) a lot of trial work would have been necessary for finalisation of apron slope and position. These two steps can be decided with the help of Fig. 6.2 and Fig. 6.3 without much trial work. The most economical arrangement of apron can be determined with the aid of these two figures.

In this chapter an attempt has been made to present the existing data on the design of fully sloping, and part sloping part horizontal apron is such a form which will be extremely useful for the design of such basins as this will involve less trial work for the designer while selecting a particular apron slope or combination of part sloping and part horizontal apron for a given tailwater condition.

@@££@@

CHAPTER 7.

ROLLER BUCKET TYPE ENERGY DISSIPATORS.

The roller buckets are used for energy dissipation where the tailwater depth is more than required for the formation of jump. The bucket deflects the high velocity flow away from the stream bed. The deflection of flow upwards by bucket lip causes the formation of a surface roller or high boil on the water surface on the bucket and a ground roller downstream of the bucket. The surface roller is effective in dissipating energy while the ground roller causes a reverse flow towards upstream direction along the channel bed. This action tends to deposit the material against the bucket lip.

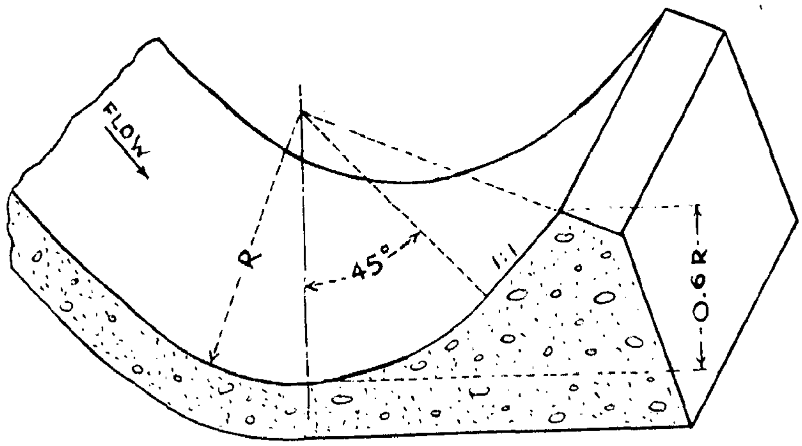
TYPES OF ROLLER BUCKETS.

There are two types of roller buckets.

1. Solid roller buckets.
2. Slotted roller buckets.

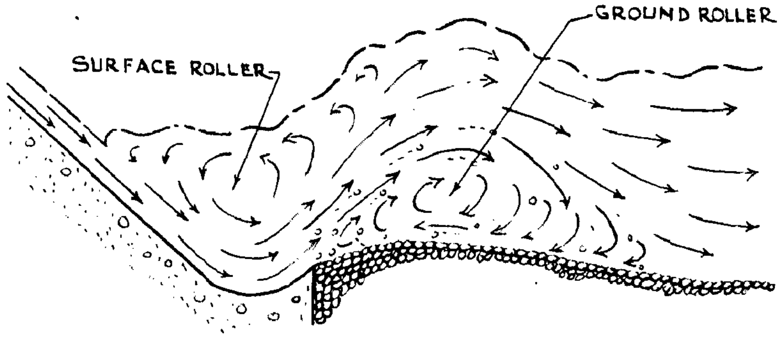
The solid roller bucket consists of a bucket like apron with concave circular profile of considerable radius and a lip (Fig. 7-1). The development of ground roller causes the river bed material to be sucked inside the bucket (Fig. 7-2). The material so brought inside the bucket causes abrasion of bucket surface. This tendency of bringing the material inside the bucket can be checked to some extent by suitably fixing the invert level of bucket. This type of roller bucket has been used below many spillways.

The slotted bucket type of energy dissipators were evolved as an improvement over solid bucket type. It is already stated



A. GRAND COULEE TYPE SOLID BUCKET

FIG.- 7-1



A. SOLID TYPE BUCKET

FIG.- 7-2

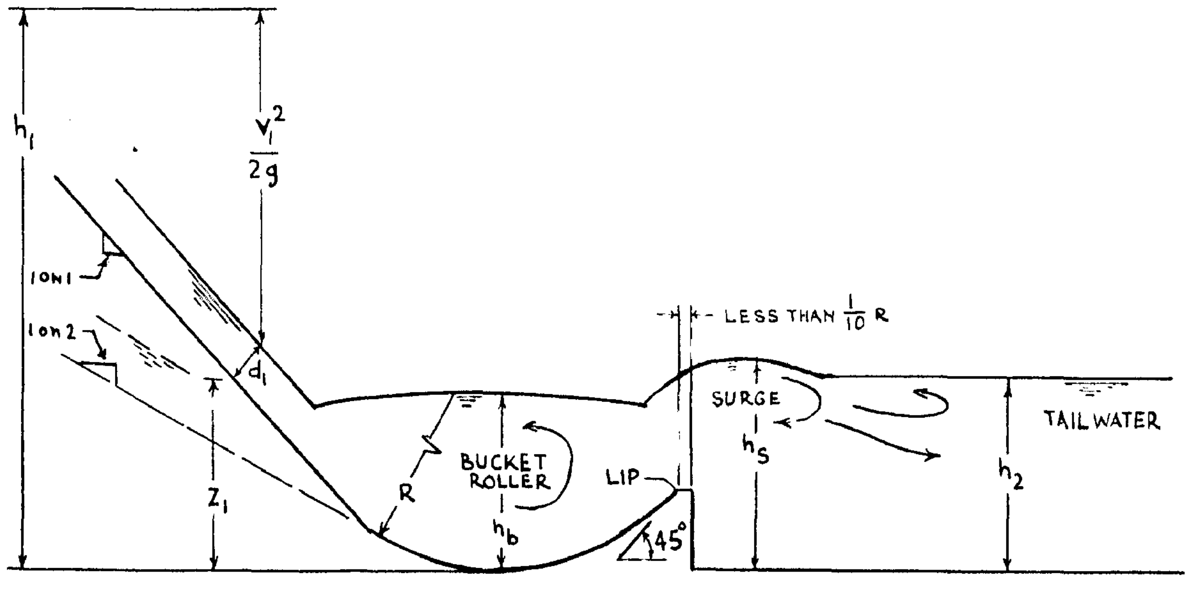


FIG.- 7-3

above that there is tendency of river bed material being sucked inside the solid roller bucket due to action of ground roller. The unsymetrical gate operation also causes eddies to bring in material inside the bucket. The provision of slots in bucket ensures selfcleaning action and the material inside the bucket is sweeped out through slots. Part of flow through the slots spreads laterally and is lifted away from the channel bottom by the apron beyond the slots, causing the flow to be dispersed and distributed over a greater area and providing less concentration than occurs in a solid bucket. In order to maintain the effectiveness of bucket action in dissipating energy, the slots are made just enough to prevent deposition of material at the bucket lip.

SOLID ROLLER BUCKETS.

The solid roller buckets were developed by U.S.B.R. in 1933 for use in Grand Coulee Dam. Later, the experimental studies were also carried out by Mcpherson and Karr[25] and these authors have evolved a method for designing the solid roller buckets. Generally speaking the factors which effect the bucket design are:-

(i) Bucket shape.

(ii) Flow conditions.

The shape of bucket includes the radius of bucket, depth of bucket, lip position and slope. The flow conditions include tailwater elevations and shape of the water surface profile downstream from bucket.

The bucket radius is selected by trial with consideration given to the ability of exit area to withstand high velocities in exit channel. A combination of minimum allowable bucket radius and minimum tailwater depth exists which gives satisfactory performance. Below these limits the performance will not be satisfactory. The buckets having larger radius than the minimum allowable will only slightly improve the bucket performance but the bucket will become costly.

According to Elevatorski[26] the minimum allowable bucket radius can be expressed as a fuction of following variables.

$$R_{min} = f(V_1, D_1, S, g, Y) \quad \text{------------------------(1)}$$

where V_1 = Velocity at entrance to bucket.

D_1 = Depth at entrance to bucket.

S = Slope of spillway face.

g = Acceleration due to gravity.

Y = Vertical distance between bucket lip and channel bed.

By dimensional analysis the equation (1) can be expressed in dimensionless form

$$\frac{R_{min}}{D_1} = f'\left(\frac{V_1^2}{gD_1}, \frac{S}{Y}\right) \quad \text{------------------------(2)}$$

Elavatorski further stresses that studies conducted at U.S.B.R. indicate that both S and Y do not within ordinary limits affect the bucket size but the frictional resistance on the spillway face affects the minimum allowable bucket radius as a result of velocity reduction.

Eliminating $\frac{S}{Y}$ from equation (2)

$$\frac{R_{min}}{D_1} = f''(\lambda) \quad \text{------------------------(3)}$$

where $\lambda = \frac{V_1^2}{gD_1}$

He also stresses that the investigations by Okada and Ishibashi in connection with Sakuma Dam, also substantiate the functional relationship given by equation (3) and the discharge intensity over the spillway also influences the bucket size.

The inference drawn by Elavatorski from U.S.B.R. studies that S and Y do not affect the bucket size within ordinary tailwater limits, has been questioned by Mcpherson and Karr[27] as the U.S.B.R. experiments were conducted with only one approach slope.

The slope of bucket lip and height of bucket lip are important in deflecting the flow upwards and maintaining a sizable ground roller below the bucket. The smaller the roller, the less effective it will be in deflecting the jet away from channel bed. According to Elavatorski[9] it has been established by models that lip slope of 45° gives best performance. In case the lip slope is steeper than 45°, the impact of flow on lip wall disrupts the roller action. When it is flatter than 45° high velocity flow becomes submerged downstream from the lip. He further recommends that the bucket lip should be kept equal to 1/6th of the maximum tailwater depth, and the channel bed elevation should be kept slightly below the bucket lip elevation to avoid bed material being drawn into bucket. Mcpherson and Karr[25] have recommended that the channel bed elevation in general be set at invert level of bucket to avoid bed material being thrown in the bucket.

Tailwater Limits.

The performance of bucket is very much dependent on tailwater conditions. The tailwater requirements for a good bucket performance are higher than required for the hydraulic jump type basin. When the tailwater is insufficient the high velocity flow entering the bucket will break through the supernatant water blanket leaving the bucket in the form of jet which is known as 'Sweepout condition'. The depth of tailwater at this stage is known as sweepout depth. A lot of scouring takes place where this jet strikes the river bed. In order to avoid this undesirable action, the tailwater should be more than sweepout depth. The tailwater so fixed is known as minimum tailwater depth.

According to Elavatorski[26] the minimum tailwater depth can be expressed as

$$T_{min} \doteq f(V_1, D_1, Y, S, R, g) \qquad \text{-------------------(4)}$$

This can be expressed in dimensionless parameters after carrying out the dimensional analysis.

$$\frac{T_{min}}{D_1} \doteq f'\left(\frac{V_1^2}{g\, D_1}, \frac{R}{D_1}, \frac{S}{Y}\right)$$

$$\doteq f'\left(\lambda, \frac{R}{D_1}, \frac{S}{Y}\right) \qquad \text{-------------------(5)}$$

as explained earliar for ordinary limits S/Y may be neglected, then

$$\frac{T_{min}}{D_1} \doteq f''\left(\lambda, \frac{R}{D_1}\right) \qquad \text{-------------------(6)}$$

Thus equation (6) expresses the relation between minimum tailwater depth and minimum allowable radius in a dimensionless form. The relation between these parameters can be established by model tests. The desired minimum tailwater depth sometimes is provided by constructing a low dam just downstream of main dam. This arrangement has been provided in the case of Sakuma Dam in Japan. Where low secondry dam downstream of main dam is to be constructed the necessary energy dissipation arrangement below it will have to be provided. Further it will have to be examined to what extent the creation of permanent pool downstream of main structure affects the stability of the structure. According to Elavatorski[9] the minimum tailwater depth should not be less than 110% of the depth required for hydraulic jump.

When the tailwater depth is much in excess, the conditions for diving flow may be created. According to Elavatorski[26] the model and prototype tests indicate that condition of diving flow is impossible except perhaps in rare cases.

A study of bucket performance with different entrance slopes (1:1 and 1:2), varying discharges, entrance head, tailwater depth and bucket radius was made by Mcpherson and Karr[25]. According to these authors, the energy dissipation downstream from the bucket resulting primarily from the expansion of live stream in the surge, can be regarded as a surge phenomenon for most of the part. The high velocity region downstream from bucket tends to concentrate near the surface.

According to above authors, the surge height downstream hs is a function of tailwater depth h_2, and the depth above bucket invert hb for a given lip angle (Fig. 7-3). If h_2 is very large, the surge will get drowned as will hb. If h_2 is small, surge will form a free trajectory and sweepout will take place. hb and hs can be expressed as functions of following variables considering boundary friction and surface tension are negligible.

$$hb = \phi_1 (q, h_1, h_2, R, g) \qquad \text{--------------------(7)}$$

$$hs = \phi_2 (q, h_1, h_2, R, g) \qquad \text{--------------------(8)}$$

where q = discharge intensity

h_1 = height of maximum head water level above the invert of bucket.

h_2 = tailwater depth

R = radius of bucket

g = acceleration due to gravity.

By dimensional analysis hb and hs can be expressed as

$$\frac{hb}{h_1} = \phi'_1 \left(\frac{q}{\sqrt{g}\, h_1^{3/2}}, \frac{h_2}{h_1}, \frac{R}{h_1} \right) \qquad \text{--------------------(9)}$$

$$\frac{hs}{h_1} = \phi'_2 \left(\frac{q}{\sqrt{g}\, h_1^{3/2}}, \frac{h_2}{h_1}, \frac{R}{h_1} \right) \qquad \text{--------------------(10)}$$

It is seen that, specific service conditions are independent of $\frac{R}{h_1}$ therefore

$$\frac{hb}{h_1} = \phi''_1 \left(\frac{q}{\sqrt{g}\, h_1^{3/2}}, \frac{h_2}{h_1} \right) \qquad \text{--------------------(11)}$$

$$\frac{h_s}{h_1} \doteq \phi_2'' \left(\frac{q}{\sqrt{g}\, h_1^{3/2}} , \frac{h_2}{h_1} \right) \qquad (12)$$

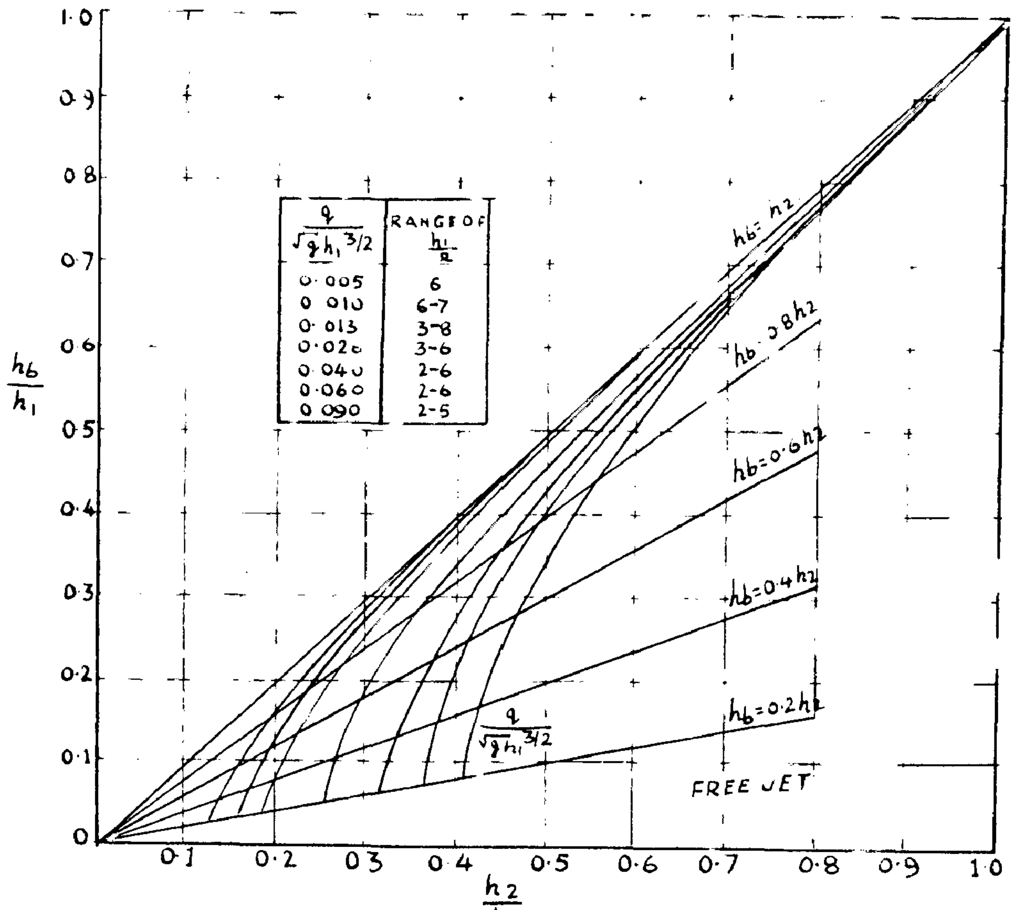
The model tests were conducted on 1:1 and 1:2 entrance slopes and it was found by authors that as hb is progressively reduced, the bucket behaves more and more as flip bucket. The value of $\frac{hb}{h_1}$ for upper limit for a free jet has been found as 0.2 when tailwater is falling and 0.3 when tailwater is rising. The effect of entrance slope to the bucket with reference to paramter $\frac{q}{\sqrt{g}\, h_1^{3/2}}$ and $\frac{h_1}{R}$ was also studied by the authors and the results are shown in Fig. 7-4 for an approach slope on 1 on 1. The authors have also come to the conclusion though with limited investigation of entrance slopes that the curves of Fig. 7-4 also should provide a reasonable anticipation of performance for approach slopes 1 on 2 to 1 on 0.7 provided h_1/R is not too large. With an approach slope on 1 on 1 the bucket roller become crowded at $\frac{h_1}{R}$ values higher than those recommended (shown in Fig. 7-4) and the surge becomes unsteady and pulsates. With smaller slope such as 1 on 2, the relatively longer bucket roller is drowned at higher values of $\frac{h_1}{R}$, the bucket roller and surge merge into a single jump apparantly indifferent to the presence of bucket. In both cases the bucket is too small, relative to depth of entering live stream to govern or control the action of surge.

Energy Dissipation Characteristics.

The value of $\frac{q}{\sqrt{g}\, h_1^{3/2}}$ in terms of Froude number F_1 is

$$\frac{q}{\sqrt{g}\, h_1^{3/2}} = \frac{F_1}{\left(1+\frac{F_1^2}{2}\right)^{3/2}} = \frac{F_1}{\left(\frac{h_1}{D_1}\right)^{3/2}}$$

as $h_1 \doteq D_1 + \frac{V_1^2}{2g}$ measuring h_1 from the same datum for both bucket



CHARACTERISTICS OF BUCKET TYPE ENERGY DESSIPATERS WITH ENTRANCE AND EXIT ANGLE OF 45°

FIG. 7-4

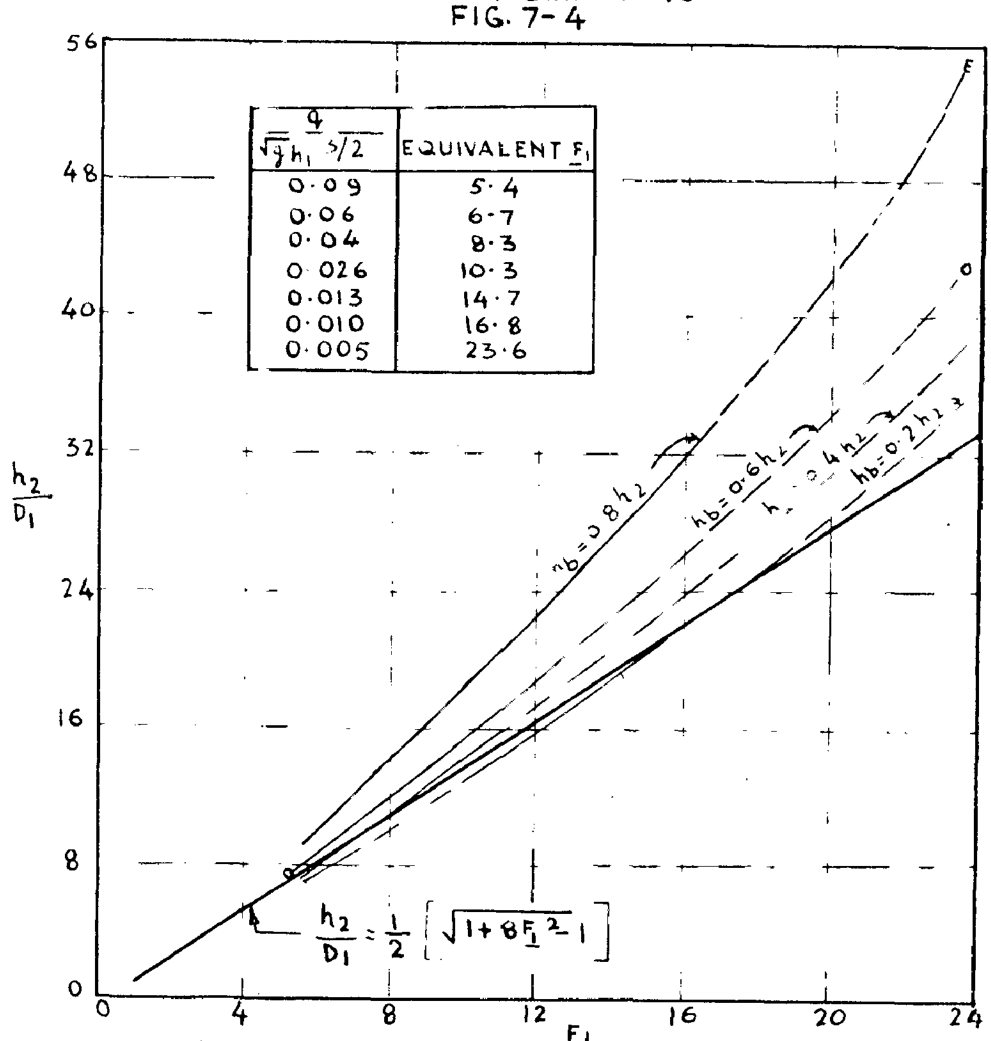


FIG. 7-5 (TAILWATER/EQUIVALENT D_1) VS F_1 FOR 45° EXIT ANGLE

invert and hydraulic jump in horizontal channel. Thus a given value of $\frac{q}{\sqrt{g}\, h_1^{3/2}}$ is equivalent to a specific value of F_1. This relation is shown in Fig. 7-5. For a standing hydraulic jump in rectangluar horizontal channel

$$\frac{D_2}{D_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) = \frac{h_2}{D_1}$$

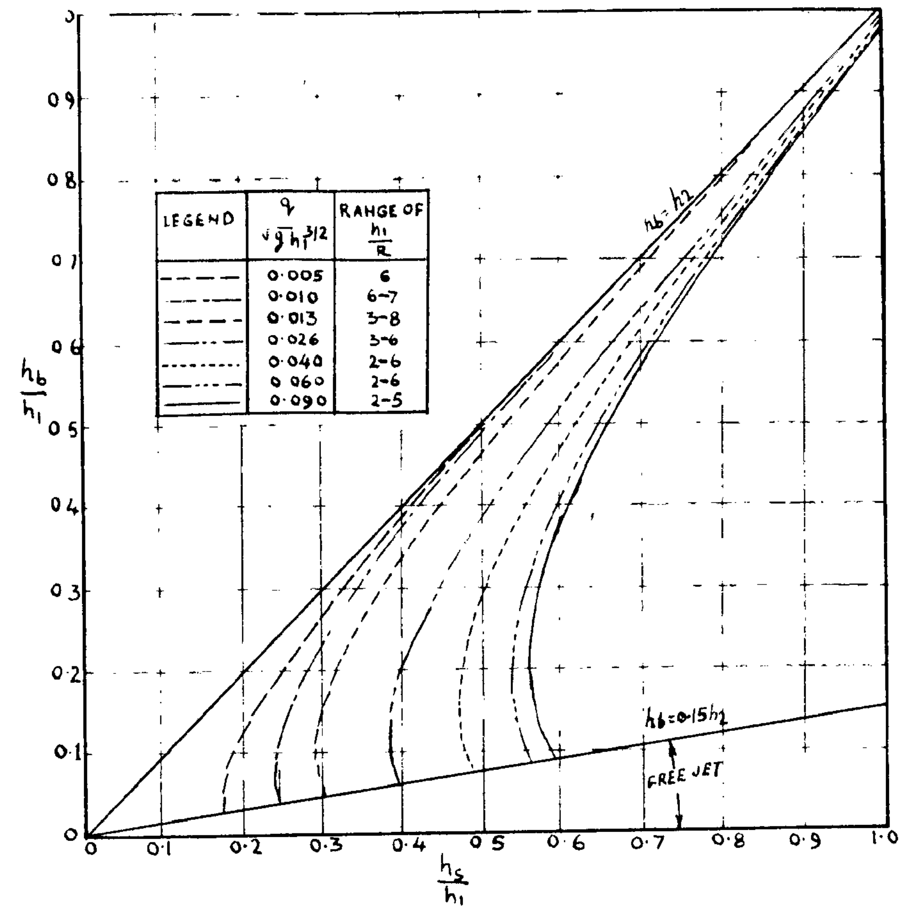
The lines showing a specific value of hb in terms of h_2 are plotted with the help of Fig. 7-4 after converting the value of $\frac{q}{\sqrt{g}\, h_1^{3/2}}$ in terms of F_1 and $\frac{h_2}{h_1}$ into $\frac{h_2}{D_1}$ (by multiplying $\frac{h_2}{h_1}$ with $\frac{h_1}{D_1}$). The relation between F_1 and $\frac{h_2}{D_1}$ shows that for hb equel to $0.2h_2$ the bucket dissipator is in close competition with a jump in horizontal apron when the tailwater is falling. When the tailwater is rising this competition is not so marked. For hb equel to $0.4h_2$ and F_1 as much as 16, the tailwater requirements are fairly close to those for hydraulic jump. Therefore the authors recommend that hb equel to $0.4h_2$ should be considered to avoid free jet condition and to ensure roller action and efficient energy dissipation The authors also indicate that these are only tentative conclusions.

Surge Heights.

Mcpherson and Karr also conducted experiments to study the surge height for entrance slopes 1 on 1 and 1 on 2. The results are presented in the shape of curves shown in Fig. 7-6 and 7-7 respectively. For any particular value of hb, hs can be determined from these curves. The experimental study indicated that entrance slope influences the peak height of surge hs. The surge trajectory is flattened with the decrease in approach slope and vice versa without materially affecting the other variables. The upper limit of free jet was found to be equal to 0.15 hs.

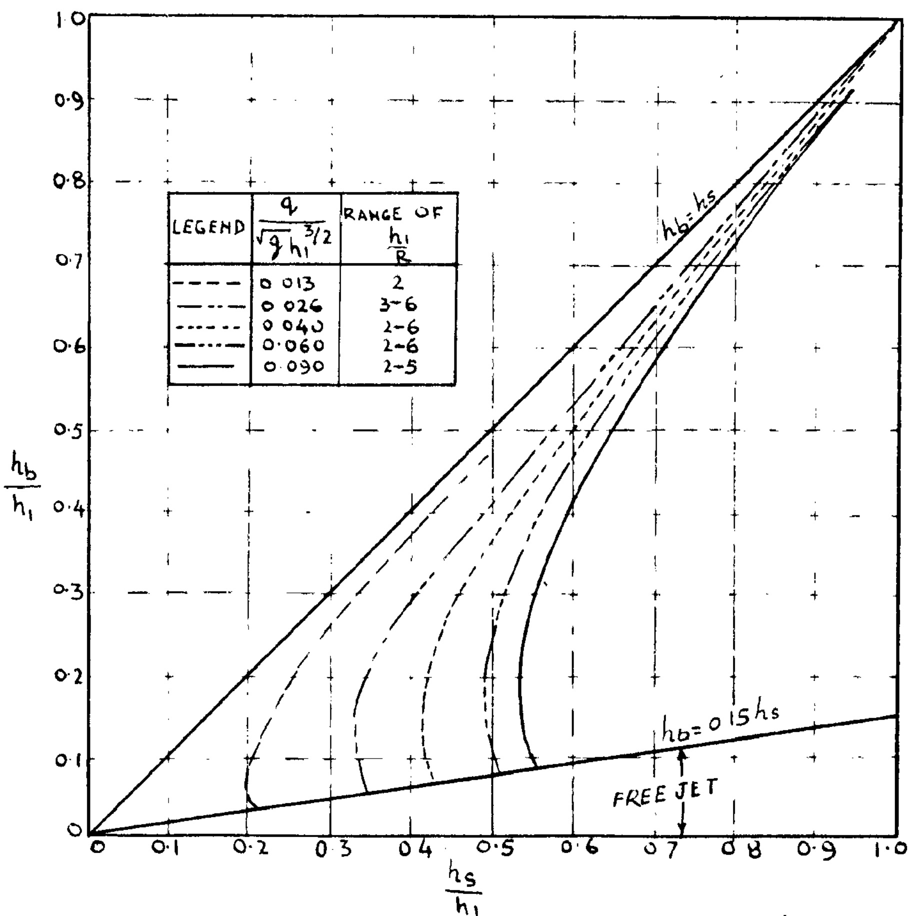
Procedure of Design.

The design procedure is summarised in the following steps:-



ENTRANCE SLOPE 1 ON 1 AND EXIT ANGLE 45°

FIG. 7-6



ENTRANCE SLOPE 1 ON 2 AND EXIT ANGLE 45°

FIG. 7-7

1. First the invert level of bucket is fixed arbitrarily and so the gross head h_1 and the bucket invert is known.

2. The value of q being known, $\frac{q}{\sqrt{g}h_1^{3/2}}$ can be determined for various values of h_1 and q.

3. From the tailwater rating curve, the tailwater i.e. h_2 for respective discharges can be known. The value of $\frac{h_2}{h_1}$ can be determined.

4. Now from Fig. 7-4, the value of $\frac{h_2}{h_1}$ for hb equal to $0.4h_2$ can be obtained which is the safe limit condition.

5. If the value of $\frac{h_2}{h_1}$ calculated from step 3 is more than the value of $\frac{h_2}{h_1}$ computed in step 4, the design is safe. In case it is less, then the design will need modification.

6. The modification can be done by lowering the level of bucket invert and repeating the process again till the above criteria is satisfied.

7. The maximum value of hs can be obtained from Fig. 7-6 and 7-7 as the case may be for maximum value of $\frac{q}{\sqrt{g}h_1^{3/2}}$ and hb equal to 0.15 hs.

8. The value of R i.e. radius of bucket can be obtained from the value $\frac{h_1}{R}$ for maximum value of $\frac{q}{\sqrt{g}\,h_1^{3/2}}$ (Fig. 7-4).

SLOTTED ROLLER BUCKETS.

The slotted roller buckets were developed at U.S.B.R.[1] after extensive experimental studies. The slotted roller bucket has slots which ensure self cleaning action in the bucket. The material sucked inside the bucket is sweeped out through slots. The solid portion between two successive slots is known as "tooth".

Shape of Teeth.

The various shapes of teeth as shown in Fig. 7-8 were tested by U.S.B.R.[1] The type design I was found satisfactory in dissipating energy as well in elimination of piled material along the bucket lip. The tests indicated the formation of small eddies by jets leaving the slots and lifting the bed material to provide abrasive action on the downstream face of teeth. A sloping apron downstream of teeth, therefore, was installed to help in spreading the jets issuing through slots and also to keep loose material away from teeth. The slope of apron is kept upwards and slightly steeper than the slope of slots for better contact with jets and spreading the jet laterally.

The type design II has a profile which confirms to the radius of bucket thus eliminating the discontinuity in the flow passing over the teeth. The pressure measurements have shown the necessity of rounding the teeth.

The tests on type design III indicated improved pressure conditions on the downstream faces of teeth and on sides when the radius was increased. This type of tooth is recommended by U.S.B.R.[1].

Apron Downstream of Teeth.

The model tests by U.S.B.R.[1] indicate that apron with 16° upward slope was found satisfactory. With lesser slopes the flow was unstable, intermittently diving from the end of apron to scour the river bed. When the tests were made with higher slopes it was seen that flow was counteracted to some extent by the directional effects of steep apron . A apron length of 0.5R (R = bucket radius) was found necessary to accomplish lateral spreading of jet and produce uniform flow leaving the apron (fig.7-9).

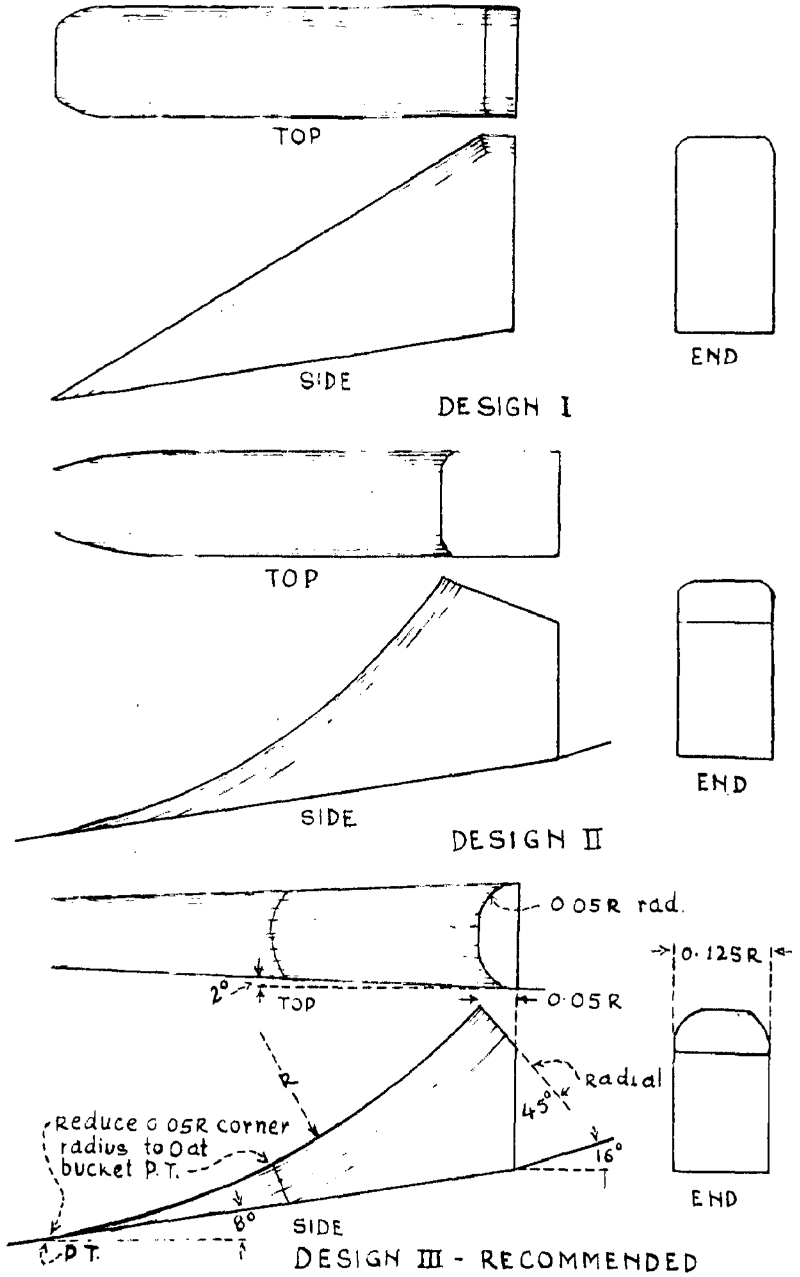
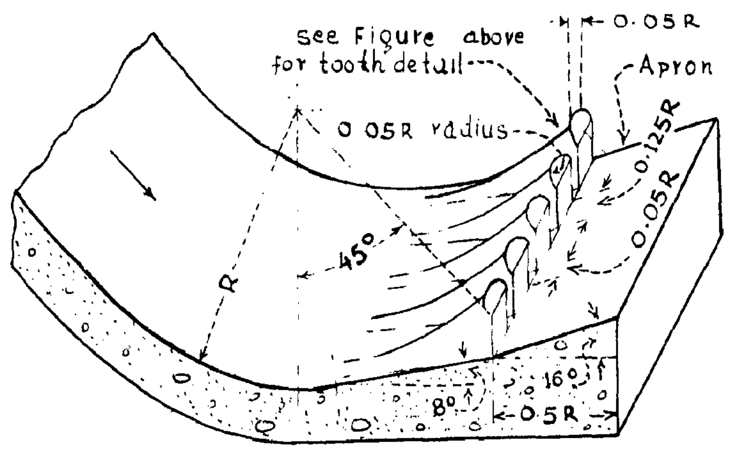


FIG. 7-8



SUBMERGED BUCKETS

FIG. 7-9

Bucket Size And Tailwater Limits.

The model tests were carried out at U.S.B.R.[1] with entry slope of 0.7 and bed of channel set at 0.05 R below apron lip (Fig.7-10). The tests have shown that with lowering of tailwater, a stage is reached when the flow sweeps out of bucket in the form of jet resulting in considerable scour where it strikes the bed. A more undesirable flow pattern occurs just before sweeping out. An unstable condition developes in the bucket causing excessive erosion and water surface roughness. Therefore it is not desirable to design a bucket for both submerged and flip action because of this transition region.

The upper tailwater limit is fixed from diving flow conditions. The tests have shown that as tailwater is raised, a stage comes when flow dives from the apron lip. The diving of flow causes deep scour in the channel near the bucket. The depth at which diving occurs, is affected by shape and elevation of channel bed with respect to apron lip. In order to prevent diving flow condition from occuring at much lower tailwater elevation, the channel bed should be set below the apron lip. The provision of sloping bed reduces the operating range between maximum and minimum tailwater depth limits by lowering the upper tailwater ~~tail~~ limit.

According to Elavatorski[9] the tests conducted at U.S.B.R. have indicated that tailwater requirement of slotted roller bucket at which sweepout takes place, is slightly higher than that of solid roller bucket. The diving takes place when the tailwater depth is excessive in case of slotted roller bucket, while in case of solid roller bucket the chances of flow diving are rare.

Relation Between Various Bucket Elements.

The relationship between various bucket elements was established by U.S.B.R.[1] after extensive studies. For a given height of structure having a particular overfall shape and spillway roughness sweepout depth T_s and minimum tailwater limit T_{min}, are functions of the radius of bucket R and head on crest H (Fig.7-11). The height of structure may be expressed as the height of fall 'h' from spillway crest to tailwater elevation. The overfall shape and 'H' determine the discharge intensity q of spillway. It has been assumed that spillway surface roughness and spillway slope have negligible effect on flow and therefore the same are not considered in the analysis. The experimental tests indicated that the elevation or shape of moveable bed did not affect the minimum tailwater limits.

Thus the T_{min} can be expressed as a function of h, R, and q ,

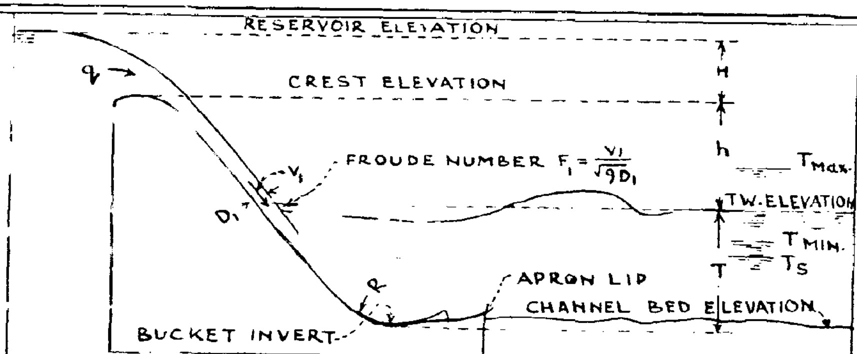
$$T_{min} = f(h, R, q) \quad \text{------------------} (13)$$

The maximum tailwater depth limit is also a function of h, R, q but as the slope and elevation of moveable bed with respect to apron does affect the tailwater at which diving takes place, the channel bed is also a variable.

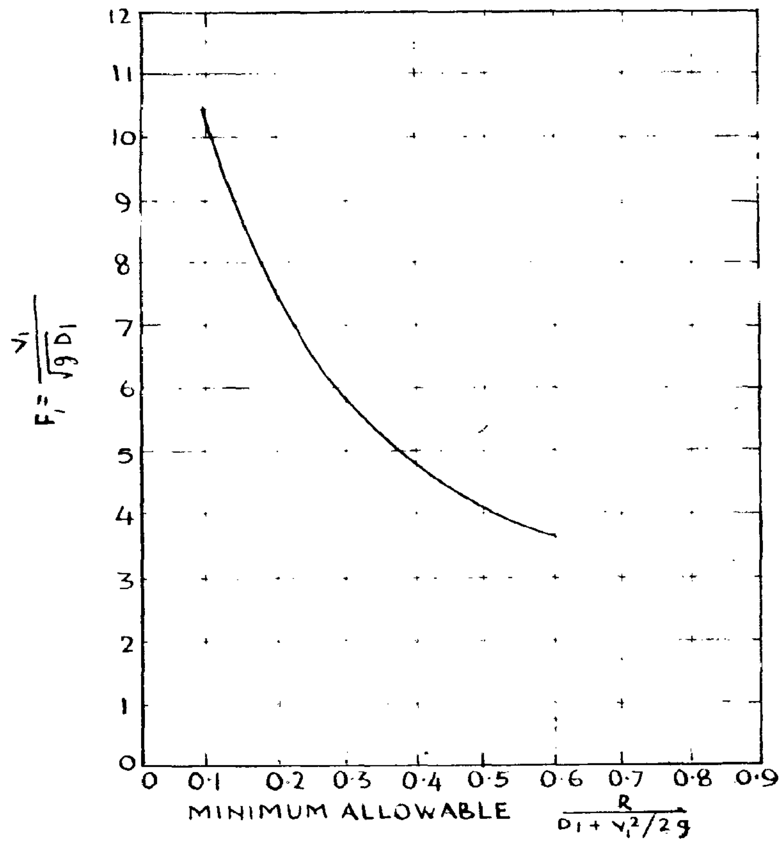
$$\text{Thus , } T_{max} = f(h, R, q, \text{ and channel bed}) \quad \text{---------} (14)$$

The maximum capacity of bucket is slightly greater for intermediate tailwater depth than for the extremes. However the bucket is expected to operate over a range of tailwater depths. The minimum bucket radius, therefore, is function of discharge intensity and height of fall.

$$R_{min} = f(h, q) \quad \text{------------------} (15)$$

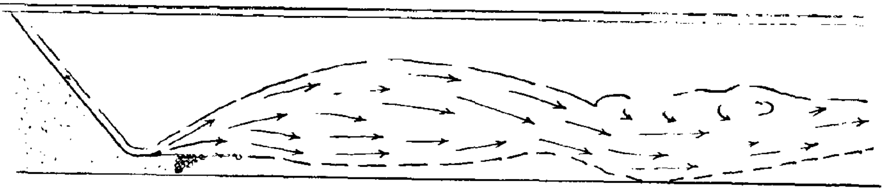


DEFINITION OF SYMBOLS
FIG. 7-11

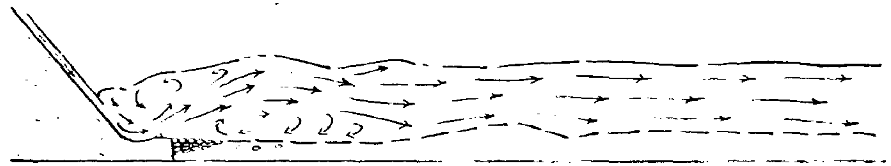


BED LEVEL APPROXIMATELY 0.05R BELOW LIP OF APRON
MINIMUM ALLOWABLE BUCKET RADIUS

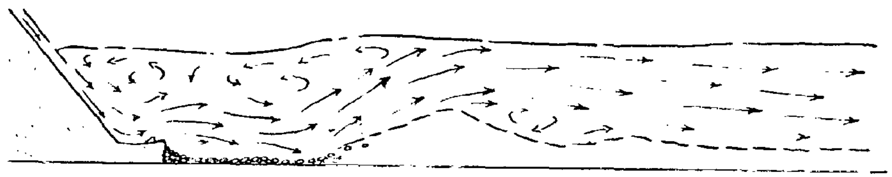
FIG. 7-12



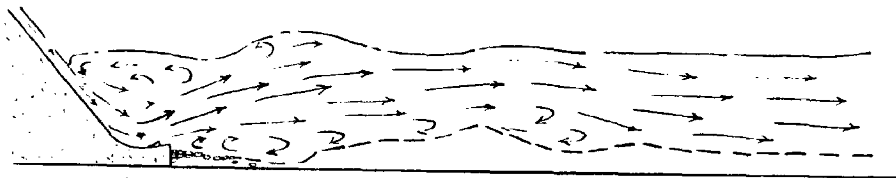
A. TAILWATER BELOW MINIMUM. FLOW SWEEPS OUT.



B. TAILWATER BELOW AVERAGE BUT ABOVE MINIMUM WITHIN NORMAL OPERATING RANGE.



C. TAILWATER ABOVE MAXIMUM. FLOW DIVING FROM APRON SCOURS CHANNEL.



D. TAILWATER SAME AS IN C. DIVING JET IS LIFTED BY GROUND ROLLER. SCOUR HOLE BACKFILLS SIMILAR TO B. CYCLE REPEATS

FIG. 7-10

The Froude number is a function of V_1 and D_1, and since V_1 and D_1 are functions of h, q, the same can be replaced by Froude number F_1.

$$T_{min} \text{ and } T_s = f(R, F_1) \qquad \text{------------------}(16)$$

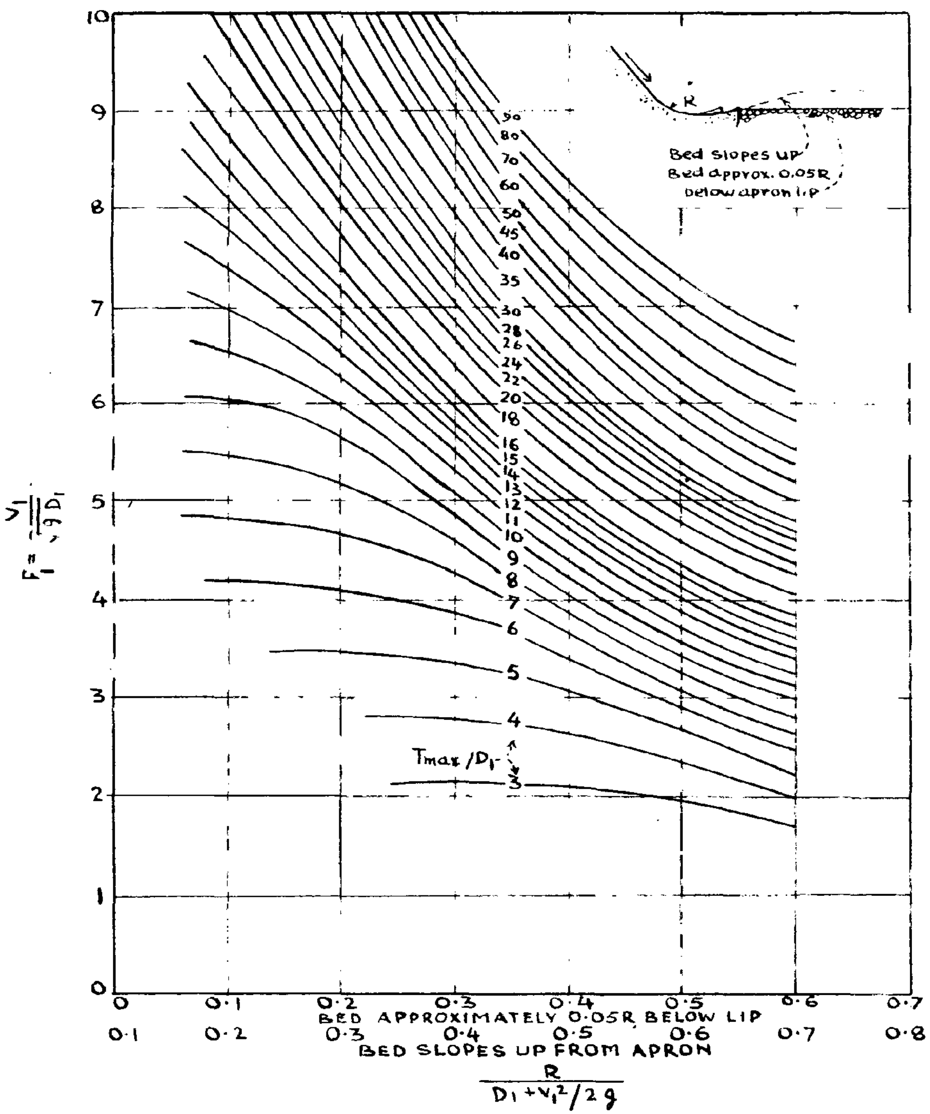
$$T_{max} = f(R, F_1, \text{Channel bed}) \qquad \text{------------------}(17)$$

$$R_{min} = f(F_1) \qquad \text{------------------}(18)$$

In order to express Tmin, Tmax, and Rmin in dimensionless form so that these may be used to predict prototype flow condition, Tmin, Tmax. are divided by D_1 and Rmin is divided by $(D_1 + \frac{V_1^2}{2g})$ i.e. the depth of flow plus the velocity head at tailwater elevation on spillway face. The plot between Rmin and Froude number F_1, is shown in Fig. 7-12. The minimum bucket radius for a given Froude number can be determined with the help of above plot.

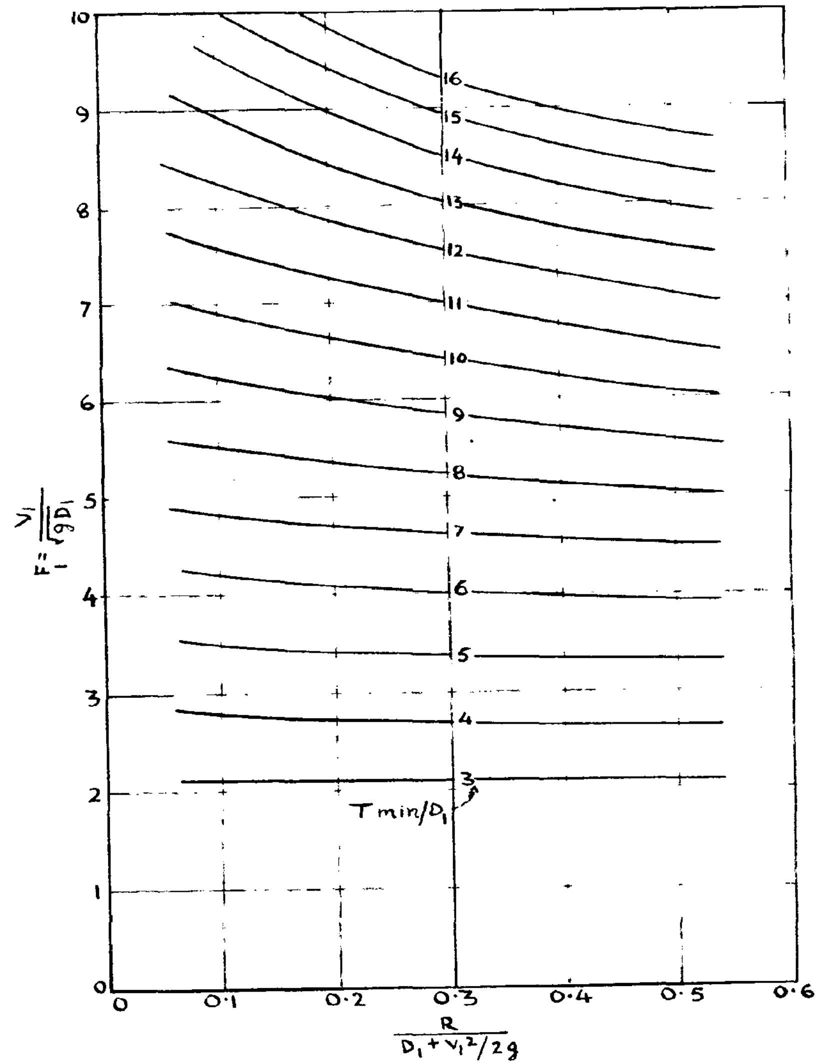
For determining the tailwater depth limit for a given Froude number, the dimensionless ratios of tailwater depth limits $\frac{T_{min}}{D_1}$ and $\frac{T_{max}}{D_1}$ were plotted against F_1 for a computed bucket radius ratio. Then the curves were drawn through both minimum and maximum tailwater depth limits having the same bucket radius ratio values. By cross plotting the data of these curves, separate curves between $\frac{T_{min}}{D_1}$ and F_1 are obtained as shown in Fig. 7-13. Similarly the plot obtained between $\frac{T_{max}}{D_1}$ and F_1, is shown in Fig. 7-14 for sloping bed and level bed of the channel. Similar curves are drawn for $\frac{T_s}{D_1}$ versus F_1 from the available experimental data (Fig. 7-15).

A plot between $\frac{R_{min}}{X}$ (X = height of crest above bucket invert) and F_1 for different values of $\frac{A}{T}$ (where A is surge height above invert and T is tailwater depth) is shown in Fig. 7-16. This can be used for determining the approximate water surface profile within and downstream of bucket.



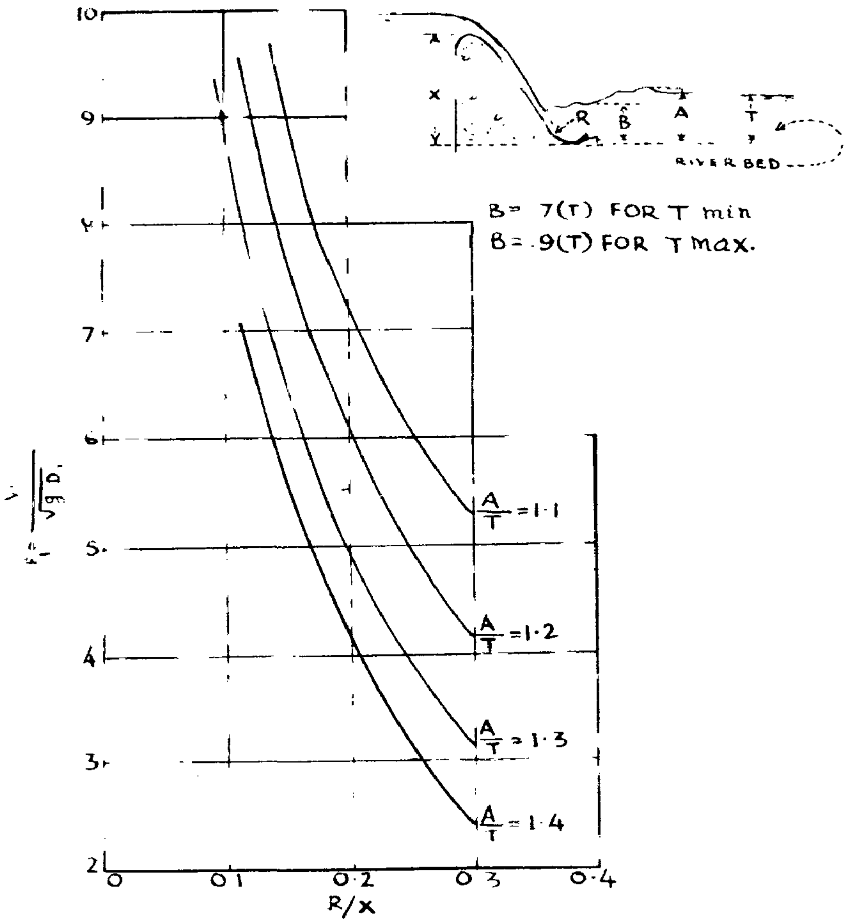
MAXIMUM TAILWATER LIMIT

FIG. 7-14



MINIMUM TAILWATER LIMIT

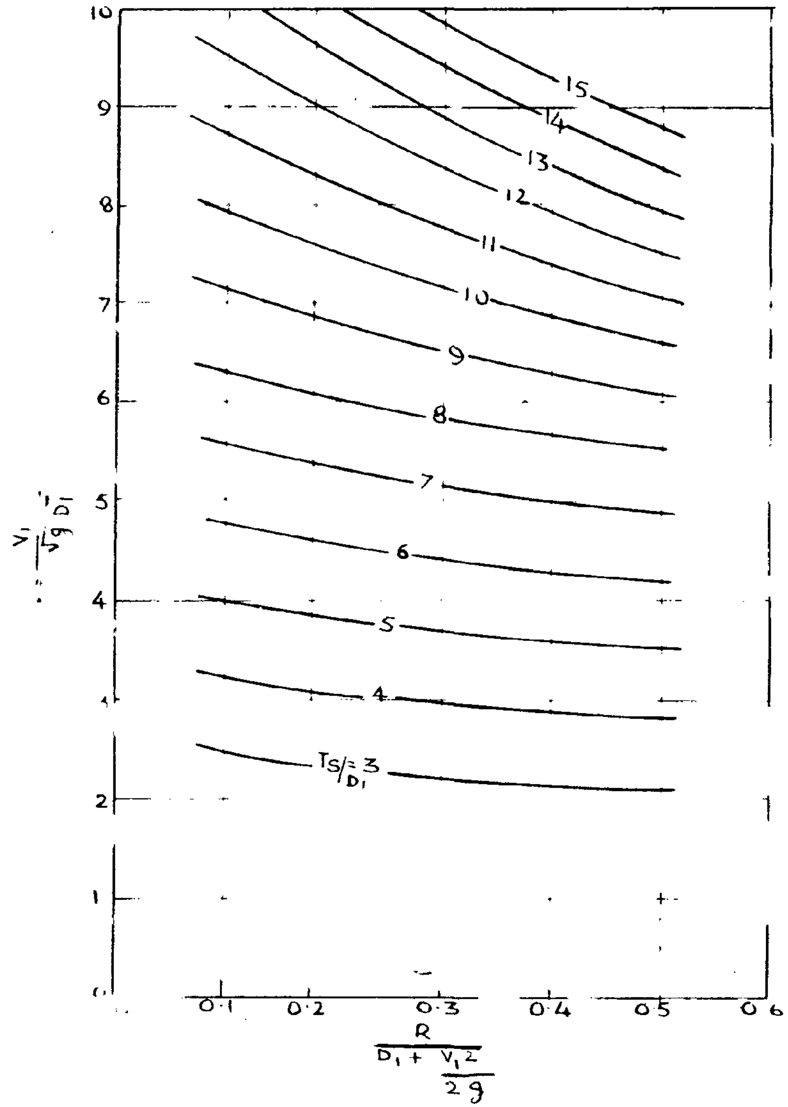
FIG. 7-13



WATER SURFACE PROFILE CHARACTERISTICS

(FOR SLOTTED BUCKETS ONLY)

FIG. 7-16



TAILWATER SWEEPOUT DEPTH

FIG. 7-15

SUMMARY OF SLOTTED BUCKET DESIGN PROCEDURE.

The U.S.B.R.[1] method for the design of slotted roller bucket based on above discussion can be summarised in the following steps.

(1) First q, V_1, D_1, are determined and Froude number is computed for maximum and intermediate flows (Fig. 7-17).

(2) With the help of Fig. 7-12 the minimum bucket radius ratio for a particular Froude number, is computed. The minimum bucket radius R can be computed as V_1 and D_1 are already known. The radius of bucket may be rounded off on the higher side and the value of bucket radius ratio is recalculated for determining $\frac{T_{max}}{D_1}$ and $\frac{T_{min}}{D_1}$.

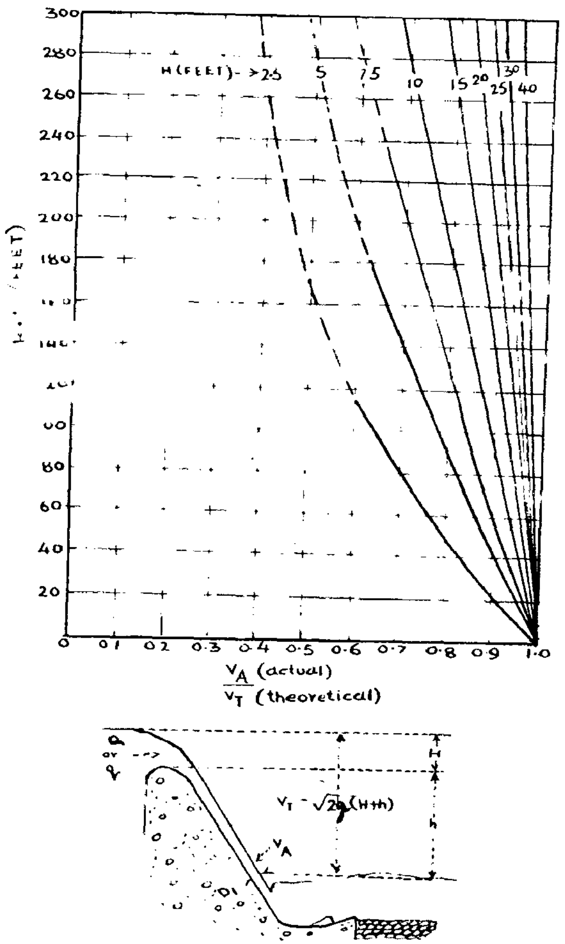
(3) From Fig. 7-13 and 7-14, the value of $\frac{T_{min}}{D_1}$ and $\frac{T_{max}}{D_1}$ are determined for the revised bucket radius ratio. Tmin and Tmax can be calculated as D_1 is known.

(4) The bucket invert elevation is fixed keeping in mind that tailwater elevations lie between Tmax and Tmin. The apron lip and bucket invert should be fixed above the river bed as far as possible. For best bucket performance bucket should be so set that tailwater depth is near Tmin.

(5) The bucket design should be completed by providing recommended tooth size, spacing etc.

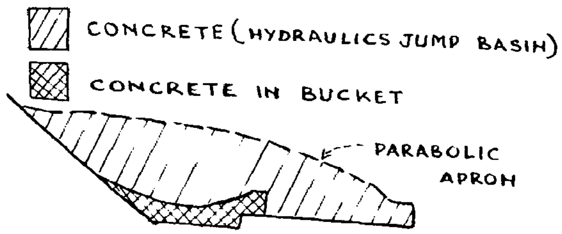
(6) The probable maximum water surface in bucket and downstream can be estimated with the help of Fig. 7-16. This is necessary for fixing the height of training walls.

According to U.S.B.R.[1] the buckets of any size and discharge can be designed with the above procedure and the structure so designed are likely to give good performance and moderate factor of safety. However it recommends that model studies should be carried



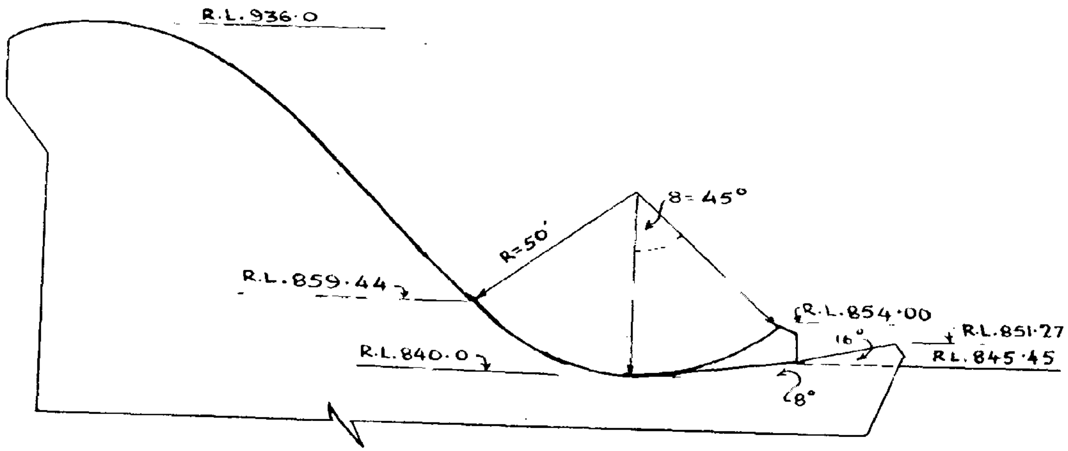
CURVES FOR DETERMINATION OF VELOCITY ENTERING BUCKET FOR STEEP SLOPES 0·8:1 TO 0·6:1

FIG. 7-17



GRAND COULEE DAM

FIG. 7-18



SLOTTED BUCKET DETAILS OF KOTA DAM

FIG. 7-19

out whenever,

(i) Sustained operation near the limiting condition is expected.

(ii) Discharge intensity exceeds 500 to 600 cusecs.

(iii) Velocities entering the bucket are greater than 75 ft/sec.

(iv) Eddies appear to be possible at the end of spillway.

(v) Waves in the downstream channel would be a problem.

EXAMPLES OF ROLLER BUCKETS.

The solid type roller buckets have been used at many places, the details of which are given in table 7-1. It is seen that this type of bucket have been adopted for discharge intensities upto more than 1300 cusecs.

The slotted type rolle-r bucket have been used in U.S.A. at Angostura Dam, Trenton Dam and Missiouri Diversion Dam etc. In our country the slotted type of roller bucket has been adopted at Kota Dam in Rajasthan. The bucket at Kota Dam is designed for a discharge intensity of 1500 cusecs while total discharging capacity is 7.5 lacs cusecs.

ECONOMIC ASPECT OF ROLLER BUCKETS.

The use of roller buckets results in large saving in construction costs. In case of Grand Coulee Dam a hydraulic jump type basin was first proposed. This required a parabolic apron of huge thickness . The fig. 7-18 shows the concrete quantity required for hydraulic jump type basin and roller bucket basin of Grand Coulee dam which gives an idea how economical it is to provide a roller bucket. Similarly in case of Angostura Dam, the slotted type of roller bucket proved very economical as compared to sloping apron. According to Elavatorski[9] the saving in cost of basin amounted to nearly 5,00,000 dollars. In our

Solid Roller-Bucket - Prototype characteristics Table 7-1.

Structure.	Location.	Bucket radius (ft.)	Max. Q (cusecs)	Discharge intensity q (cusecs)	※ Head H (ft.)	Bucket lip slope (degrees)	Bucket lip height (ft)	Tailwater depth at maximum discharge (ft.)	Remarks
Buggs island	Verginia NoCarolina	40	7,70,000	705	167	45	7	70	
Center Hill	,, Tennessee	50	4,57,000	973	226	45	14.6	86.4	Bucket invert varies.
Clark Hill	Georgia-So, Caroline	50	10,58,000	964	185	45	13.6	69.2	
Davis	Arizona California	75	1,92,000	781	177	45	20	70.00++	
Grand Coulee	Washington	50	1,000,000	606	421.6	45	30.1	161.5	
Greensborro	No.Carolina	17	33,000	540	72	45	-	33	
Headgate Rock	Arizona California	40	2,00,000	500	83	45	10.0	55.6	
Murdock	Utah	5	5,000	500	21	37	3.1	-	
Pen Forest	Pennsylvania	17	12,000	300	132.5	45	5.0	-	
Sakuma	Japan	63.1	3,46,000	1315	389 +	45	22.1	90	
Stewarts Ferry	Tennessee	40	1,99,000	607	122 ‡	45	11.7	60.5	Bucket invert varies.
Wolf Creek	Kentucky	50	5,35,400	907	227	45	15.0	102.6	

※ From Maximum pool level to bucket invert.
\+ From spillway crest to bucket invert.
‡ approzimate.
++ Varies.

country the slotted type roller bucket have been adopted at Kota Dam (Fig.7-19) as it was found cheaper in comparision to hydraulic jump basin.

Though the initial cost of roller bucket is less than hydraulic jump stilling basins, the maintenance cost is heavy which may sometimes offset the initial saving in construction cost. During the functioning of bucket of large amount of debris and river bed material is sucked inside the bucket, particularly in case of solid roller buckets. This material causes abrasion and erosion inside the bucket. For instance in case of Grand Coulee Dam a volume of erosion equel to average depth of 2" thoughout the bucket length was observed. Such type of erosion creates difficult and costly maintenance problems.

In the above paragraphs an attempt has been made to discuss the hydraulic design of roller buckets. The factors affecting the design of solid and slotted type roller buckets have been dealt in detail. The design procedure for both type of buckets have been summarised. Some examples of the roller buckets adopted in this country and abroad, have been given. The economic aspect of these types of buckets with reference to hydraulic jump type basins has also been discussed.

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CHAPTER 8.

TRAJECTORY OR SKIJUMP BUCKET TYPE ENERGY DISSIPATORS.

The trajectory or skijump bucket is used for energy dissipation where the tailwater is inadequate for the formation of hydraulic jump and the foundation strata consists of fairly good rock. It is called a skijump bucket when the tailwater is always below the lip of bucket. In case of trajectory bucket the tailwater may be above the lip of bucket at least for certain range of discharges. In case of the skijump bucket the jet is always thrown in the air without any obstruction due to tailwater, while in trajectory bucket the water jet is thrown up pushing the tailwater.

There is hardy any energy dissipation inside the bucket itself. The bucket, however, deflects the high velocity flow at the toe of spillway into a high trajectory jet in the air which finally hits the tailwater and river bed for downstream. The energy dissipation, therefore, takes place due to internal turbulence in the jet, frictional resistance of surrounding air, surface tension, diffusion in tailwater and finally impact against river bed. Before diffusion in tailwater, the jet may partially disintegrate resulting in water drops of various sizes and thus reducing the impact. In the initial stages there is appreciable scour in the zone of impact resulting in formation of deep pool which later acts as cushion for falling water to churn and absorb energy. Since this type of energy dissipation arrangement is restricted to structures where stream bed is composed of fairly sound rock, the scour taking place sufficiently away from structure does not progress towards upstream and thus the safety of structure is not likely to be endangered.

Elavatorski[9] classifies the bucket as high and low, depending upon the location with respect to river bed (Fig.8-1). In

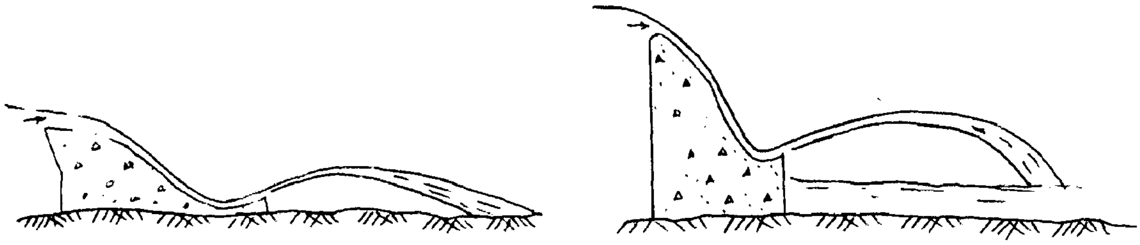
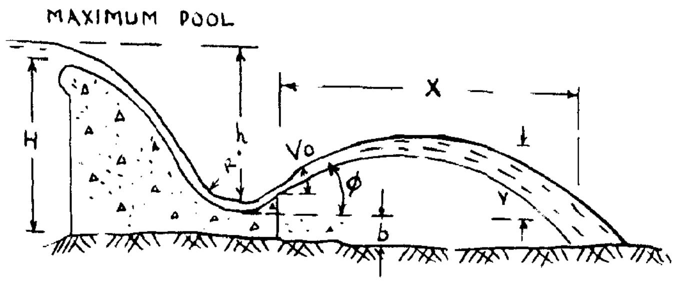


FIG. 8-1



TRAJECTORY BUCKET VARIABLES

FIG 8-2

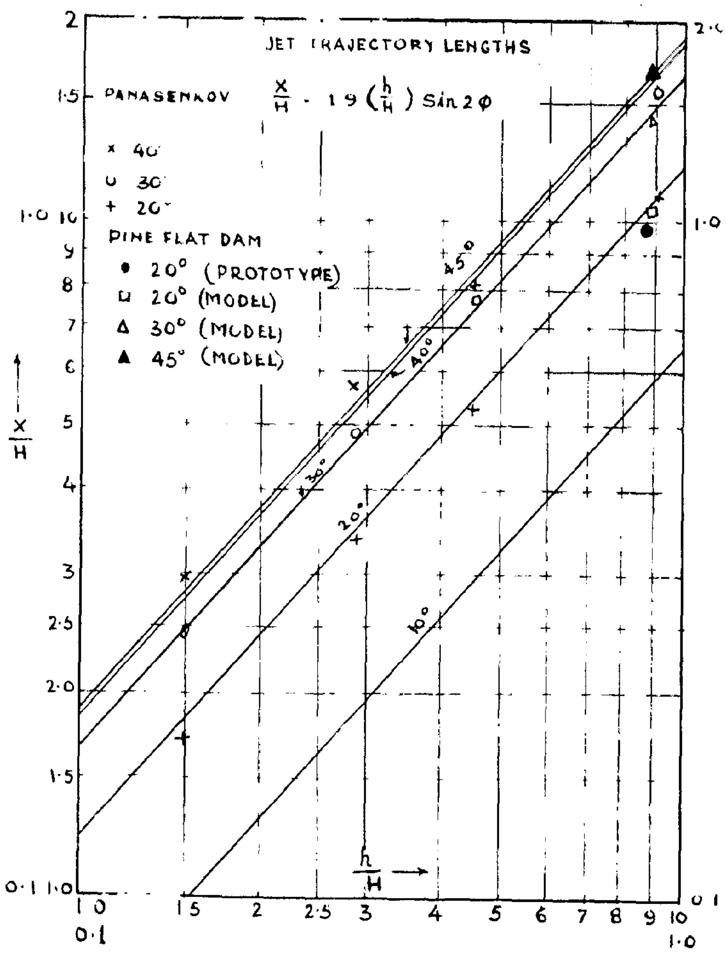


FIG. 8-3

case of high buckets the tailwater is below the lip and the jet is deflected and thrown clear in the air striking the tailwater away in the downstream of bucket. In case of low buckets which are placed quite close to river bed, tailwater may cause resistance to throwing of jet. If the tailwater is quite high at low discharges, the roller formation may occur though it will not result in effective dissipation of energy.

DESIGN CONSIDERATIONS.

The factors which govern the design of trajectory bucket are :-

(1) Bucket shape.

(2) Entrance and exit slopes.

(3) Elevation of bucket lip.

(4) Trajectory length.

(5) Erosion by impact of jet.

(6) Shape of lip.

(7) Pressure on bucket as a result of dynamic effect of curvilinear flow.

Shape of Bucket.

The shape of bucket may be circular or parabolic, but generally the former shape is used. The parabolic shape of bucket has been used in the case of Anchor Dam. According to Elavatorski[9] this shape provides a smooth water surface without increasing the size of the structure. The model studies for this Dam indicated that circular bucket causes more splashing and vibrations. The circular shapes of buckets have been extensively used in India as well as abroad and the performance has been quite satisfactory.

There is no rational formula for the determination of radius of bucket. The radius is generally finalised on the basis of model studies. The minimum limit of bucket radius is governed by the requirement that no excessive splashing or rough water surface occurs. For most of existing structures the radius lies between 30' to 60' with a few exceptions where the radius greater than this range have been adopted. For example, in case of Gandhisagar Dam in Madhya Pradesh, the radius of bucket has been kept as much as 100 feet.

Some emperical formulae have been evolved by various authors for determination of bucket radius. The same are described below :-

(i) According to Vent Te Chow[22] the radius of bucket in feet is given by

$$R = 10^{(V+6.4H+16)/(3.6H+64)}$$

where V = Velocity of flow at toe of spillway in ft/sec.

H = Head in feet excluding approach velocity head on spillway crest.

(ii) Schoklitsche's formulae

$$R = 1.5\ H_d$$

where H_d = designed head over spillway.

(iii) $$R = \sqrt{hH}$$

where h = the depth of overflow.

H = fall from the crest spillway to the invert of bucket.

(iv) Guthree Brown suggests that bucket radius should not be less than three times the maximum jet thickness.

(v) According to U.S.B.R.[1] the radius of flip bucket below tunnel spillways, should be at least four times the maximum depth of flow.

The above formulae for determination of bucket radius are generally used as guides only. The radius is finally adopted as stated earliar, on the basis of model studies.

<u>Entry and Exit Angles.</u>

The entry and exit angles affect the performance of bucket. According to Elevatorski[31] the entrance slope should not be very steep so as to cause erratic uneven flow. Normally the slope should not be steeper than 4 vertical to one horizontal. This can be ascertained from model studies. The exit angle affects the jet throw off. Adequate deflection can be provided with angle of 20° to 30°. The maximum range of values for the angle vary from 10° to 45°. The greater the angle, the greater the deflection of jet further downstream and more spray is produced, but it will require more concrete and therefore the bucket will be more costly. The steeper the angle, more the jet will disintegrate and slow down by air resistance which will cause steep entry of jet in tailwater. With steep entry the vertical component of jet will be greater, resulting in greater tendency of the jet to dig in the river bed and thus dissipating energy. When the entry angle of jet in tailwater is flatter, the magnitude of horizontal component will be greater and high velocity flow may persist downstream of impact area for a good distance if the river bed is not erodible enough to from deep pool. The high velocity flow in river may erode the river banks.

The other factors requiring consideration while fixing the lip angle are the negative pressure at end of lip and impact on the lip. The pressures at the end of lip depend quite a lot on lip angle in addition to head and discharge. According to Joglekar and Damle[23] if the high exit angle is provided, there is possibility of negative pressures just before the edge of the sill due to turning of the jet in the downward direction as it leaves the bucket. This can be minimised by lowering the end sill angle to a slight extent. The extent of actual lowering is decided by model tests. While doing so, it has to be kept in mind that maximum throw is comparatively unaffected even when angle is lowered substantially below 45°. It is clear from table 8-1 that with 35°

angle the loss of throw is only 6% of the maximum throw and with 30° angle the loss of throw is less than 14% . Thus with a little loss in throw the pressures at the end sill are improved. The table No. 8-1 also shows that relative horizontal impact goes on reducing as exit angle decreases. It is also seen from the table that the relative impact on the sill plus the relative loss in throw for a slope of 0.75:1 (37°) is only 4% while the horizontal impact on the endsill get reduced to nearly 2/3rd as compared to 1:1 slope.

Table No. 8-1.

Relative reduction in impact on sill for slopes flatter than 1:1

Angle ϕ (degrees)	slope $\tan\phi$	Relative horizontal impact (compared to vertical sill) $(1-\cos\phi)$	Relative loss in throw (fraction of maximum throw) $(1-\sin 2\phi)$	Sum of (3)&(4)	Remarks
1	2	3	4	5	6
45	1.0	0.293	0.000	0.293	
43	0.93	0.269	0.002	0.271	
41	0.87	0.245	0.010	0.255	
39	0.81	0.225	0.022	0.240	
37	0.75	0.201	0.039	0.240	Minimum
35	0.70	0.181	0.060	0.241	
30	0.58	0.134	0.134	0.268	

Jogleker and Damle are also of the opinion that if the angle of throw is too low and bucket is not high, there is likelyhood of free jet being insuffientły areated which may lead to supression of jet, creating conditions for development of subatmospheric pressures at the end sill. In case the low angle is adopted for any other reason, and with the possibility of supression of jet, the authors are of the opinion that the measures for areating the sill with air ducts and pipes, should be taken.

It is clear from the above discussion that extensive model studies are necessary for finalising the angle of lip.

Elevation of Bucket Lip.

The performance of bucket is greatly affected by the elevation of bucket lip. The length of trajectory is influenced by the elevation of lip. If the lip elevation is not high enough above the river bed, the jet may be unsteady and its action unsatisfactory. The raising of level of bucket lip excessively, may require considerable quantity of concrete and may mean increased erosion at the point of fall. There are no definite rules for fixing the lip elevation. According to Elevatorski[9], the studies on Pine Flat Dam indicated that bucket lip elevation should be same as maximum tailwater elevation. For circular buckets Elevatorski[31] on the basis of existing designs suggests that lip height 'h' above the bucket invert may be adopted as given below:-

For high buckets, $h = 0.10\ R$

For low buckets, $h = 0.125R$

where R is radius of the bucket.

Trajectory Length.

Probably the most important consideration in the design of trajectory bucket is to determine the distance from bucket at which the deflected jet will strike the river bed. If the friction, retardation of air and disruption of jet is neglected then the horizontal and vertical component of jet trajectory is given by (Fig. 8-2).

$$X = \frac{V_0^2 \sin 2\phi}{g} \qquad \text{--}(1)$$

$$Y = \frac{V_o^2 \sin^2\phi}{2g} \qquad \text{------------------------(2)}$$

$V_o = \sqrt{2gh}$ as friction is neglected.

$$X = 2h \sin 2\phi \qquad \text{------------------------(3)}$$

$$Y = h \sin^2\phi \qquad \text{------------------------(4)}$$

Eq (3) can be converted in dimensionless form by dividing with 'H' the vertical distance from maximum pool level to river bed

$$\frac{X}{H} = \frac{2h}{H} \sin 2\phi$$

According to Elevatorski[9], the measurement made by Oblensky and Maitre at St. Etienne-Cantales and Chastang Dam in France, for flows less than half of maximum discharge, indicate that nearly 20% of total energy is dissipated by interaction of jets with air. To compensate the reduction of velocity and loss of energy of jet during flight, the use of following equation is suggested by Elevatorski

$$\frac{X}{H} = 1.9\frac{h}{H} \sin 2\phi \qquad \text{------------------------(6)}$$

The maximum value of trajectory length will occur when ϕ is 45^{o} as $\sin 2\phi$ will then be maximum i.e. unity. The curves for $\frac{X}{H}$ versus $\frac{h}{H}$ based on eq (6) have been plotted for lip angle ranging from 10^{o} to 45^{o} and are shown in fig. 8-3. The experimental results for Pineflat Dam for Model and protptype are also marked in the figure. It is seen that the experimental results of Pine flat Dam and other dams agree quite closely with these curves. The model and protptype studies made at U.S.B.R.[1] have indicated that prototype trajectory length is shorter than the model or theoretical jet and has steeper angle of entry into tailwater. It has been presumed that this difference is caused by greater air resistance encounted by the high velocity prototype jet.

The above equations show that trajectory length is not affected by radius of bucket. Elevatorski[9] indicates that experimental studies conducted for Hartwell dam have shown that a larger bucket radius causes the flow to be deflected slightly higher and further downstream than does a small radius bucket.

It has been stated above that theoratically trajectory length on a horizontal plane passing through the point of projection will be maximum at 45° angle of lip. The model studies have shown that 45° lip angle does not always give the maximum length of trajectory due to air friction and dispersion of jet and the trajectory length observed for exit angles 35° to 40° is found to be maximum. The experimental studies conducted by Rouse, Howe and Matzler[32] while studying the range of fire monitors, indicated that maximum horizontal throw occurs at 30° instead of 45°.

In case of skijump buckets the lip is at much higher elevation than the river bed. In this case the theoretically maximum throw will not occur at exit angle of 45°. The angle at which the maximum theoretical horizontal throw will occur can be derived as below (Fig.8-4).

$$AC = \frac{V_0^2}{2g}\sin 2\phi$$

Level difference between B and D $= h + \frac{V_0^2}{2g}\sin^2\phi$ ---------(8)

Time taken by jet to fall from B to D

$$t = \sqrt{\frac{2}{g}\left(h + \frac{V_0^2}{2g}\sin^2\phi\right)} \qquad \text{-----------(9)}$$

Horizontal distance between B and D = $V_0\cos\phi \times t$

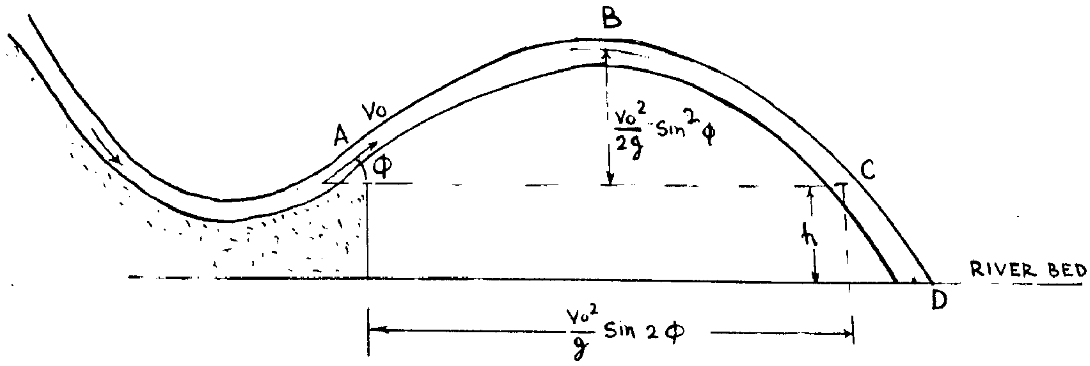
where $V_0\cos\phi$ is the horizontal velocity of jet.

The total horizontal throw from A to D = AB+BD

$$X = \frac{V_0^2}{2g}\sin 2\phi + V_0\cos\phi\sqrt{\frac{2}{g}\left(h + \frac{V_0^2\sin^2\phi}{2g}\right)} \qquad \text{-----------(10)}$$

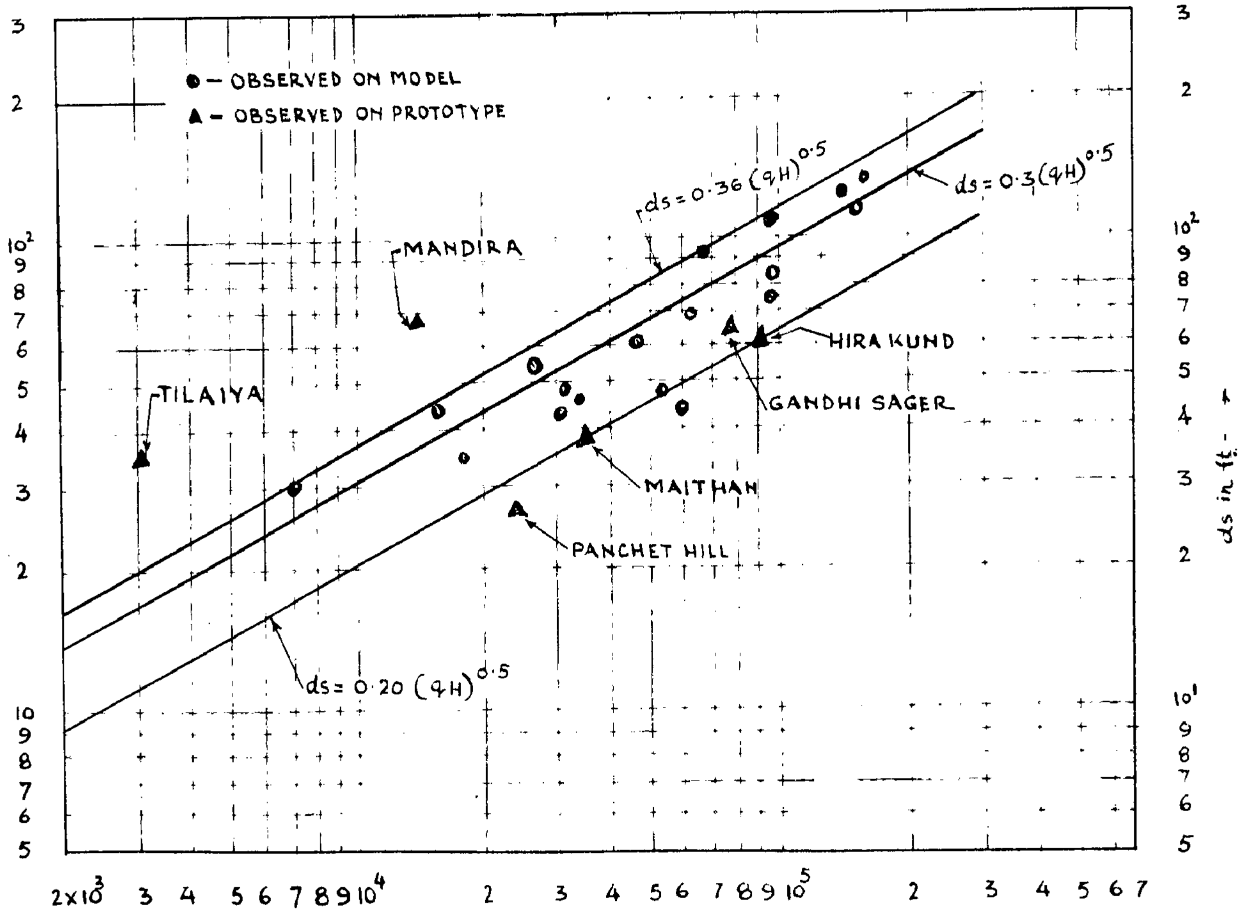
For X to be maximum $\frac{\partial X}{\partial \phi} = 0$,

$$\frac{\partial x}{\partial \phi} = \frac{V_0^2}{2g}\cdot 2\cos 2\phi - V_0\sin\phi\sqrt{\frac{2}{g}\left(h + \frac{V_0^2\sin^2\phi}{2g}\right)} + \frac{\frac{V_0^3}{g^2}\cos^2\phi\sin\phi}{\sqrt{\frac{2}{g}\left(h + \frac{V_0^2}{2g}\sin^2\phi\right)}} = 0$$



B
A
Vo
$\frac{V_o^2}{2g}\sin^2\phi$
ϕ
C
h
RIVER BED
D
$\frac{V_o^2}{g}\sin 2\phi$

FIG. 8-4



● – OBSERVED ON MODEL
▲ – OBSERVED ON PROTOTYPE
$ds = 0.36 (qH)^{0.5}$
$ds = 0.3 (qH)^{0.5}$
$ds = 0.20 (qH)^{0.5}$
MANDIRA
TILAIYA
HIRA KUND
GANDHI SAGER
MAITHAN
PANCHET HILL
ds in ft.
q X H in ft

FIG. 8-5

simplifying we get $\operatorname{cosec}\phi = \sqrt{2\left(1+\frac{gh}{V_o^2}\right)}$ ------------(11)

Knowing Vo, the exit velocity and g, h, the lip angle which will give theoretical maximum horizontal throw, can be computed from equation (11).

The above discussion indicates that actual trajectory length is generally different from theoretical length because of air resistance and dispersion of jet. It is, therefore, desireable to conducted model studies in each case.

Erosion By Impact Of Jet.

The extent of likely erosion of bed material due to impact of jet, is also an important factor to be considered in the design of bucket. If the erosion or scour is more and travels towards upstream side the structure inself will be endangered. According to Elevatorski[31] the factors affecting erosion are:-

(i) Velocity of jet.

(ii) Thickness of jet.

(iii) Angle of inclination of jet entering the pool.

(iv) Depth of tailwater.

(v) Characteristics of river bed strata.

The velocity is dependent on fall of water. The jet thickness is dependent on discharge intensity and velocity. Thus intensity and velocity affect the scour greatly. The effect of bucket lip angle has been already discussed earliar in this chapter. The lip angle fixes the angle of entry of jet into tailwater and so impact on the river bed. The tailwater depth provides the necessary cushion for the entering jet. This results in loss of energy by diffusion of high velocity jet in pool created due to tailwater. Thus the greater tailwater depth reduces the extent of river bed scour.

The most dominant factors which affect the scour is the type of river bed strata i.e. homogeneity of material, presence of cleavages and bedding planes in rock,etc. The scour in the area of impact starts with removal of soft material. The dynamic pressures involved in the impact, act through the crevices and bedding planes and huge chunks of rock are bodily lifted and carried by the flow.

Besides the above factors, frequency of operation of spillway and the duration of operation also affect the scour. The greater the frequency and duration of operation the greater the scour is likely to take place.

According to Damle, Venkatraman and Desai[33] who carried out studies on model and prototype considering only two dominant factors i.e. discharge intensity q and height of fall H, the scour depth is related to product of q and H in following manner.

$$d_s = a\,(q\,H)^b \qquad \text{------------------------(12)}$$

where d_s = scour depth below tailwater level.

a, b = constants.

The value of a, b, for maximum and minimum value of ds were found as 0.36, 0.50 and 0.20, 0.50 respectively. The authors recommend these values for evaluation of scour with the help of eq (12). The lograthmic plot between value of ds and (qH) indicates a general trend and fairly good correlation with actual observed values on model and prototype for certain dams in India e.g. Maithon, Panchet Hill, Hirakud, Gandhi Sagar, Mandira and Tilaya Dam (Fig. 8-5).

In the above study no effect of angle of lip, tailwater depth etc. have been considered as no experimental data was available. Due to these limitations the formula given by the above authors can only provide some rough idea of likely scour in a particular case. The estimation of anticipated scour is usually made with the aid of model studies. In this connection it may be pointed out that the estimation of anticipated scour is more qualitative than quantitative as in a model it is difficult to simulate the protype conditions.

Shape Of Lip.

Elevatorski[31] has suggested that bucket lip should be dentated for better distribution of flow over a larger area of river bed than the flow in a plain curved bucket. The slotted bucket lip will cause alternate ridges and troughs in flow as it spreads longitudinally. According to above author the studies conducted for Clevel-and Dam have indicated that size of corrugat--ions increases with the increase of discharge. The studies have also been carried out by Uppal and Singh on this problem and they called it "double jet deflectors" as part of flow is directed by splitters and shoots into air while the remaining flows through the slots and deflected into air as independent jets. Due to flaring extensions, jets from adjoining sections interact with each other leading to dissipation of substantial portion of energy. As a result of splitting up and interaction brought about by double jet deflectors, it is observed that the rise of jet decreases where as the spread of jet increases considerably. Thus the energy gets distributed over a greater area of the river bed and the scour is reduced. The reduction in scour is stated to be nearly 50% of solid bucket lip type of arrangement. It is also stated by the above authors that cavitation on sharp edges of flow can be controlled by rounding or chamfering the edges.

It is felt that the above type of bucket lip can have limited use. This type of arrangement in the bucket may be useful in trajectory buckets as the tailwater will generally be above the lip elevation and the hydrostatic pressure on the lip will reduce the chancesof cavitation. In case of the skijump buckets, there is more likelyhood of cavitation when velocities are high and there is no hydrostatic pressure above the lip. Thus, this type of lip is not likely to be suitable to skijump buckets. It has also been claimed that with the arrangement of double jet deflectors in the bucket, the area of impact of jet on the riverbed, is enlarged and thus the depth of scour is reduced. This fact may not be much useful in actual practice as the adoption of trajectory or skijump bucket presupposes that the scour will take place where the jet strikes the river bed resulting in the creation of a pool which will later act as cushion for falling jet of water. It may be better to have a deep pool of small area than a shallow pool with large area. The basic consideration however, remains as stated earliar that the scour should not travel upstream so as to endanger the structure itself.

Pressure Distribution On Bucket.

In design of this type of bucket, an important consideration is the force which is exerted on the bucket by water as a result on dynamic effect of curvilinear flow over it. The total force caused by jet deflection can be determined by applying the momentum equation after making the assumptions that the jet deflection is parallel to the slope of bucket lip. However this does not give the distribution of pressure over whole of the bucket. The other method for the determination of pressure is by centrifugal theory. The pressure can also be computed by assuming the flow in the bucket as irrotational vortex flow. The above mentioned methods as discussed here.

(i) Momentum Theory:-

The pressure exerted on the bucket can be evaluated by calculating the impact caused by change of momentum of water deflected in the bucket.

Force = mas x accelaration = rate of change of momentum

or $F = mx\,(V - Vo\cos\emptyset)$

where V = Velocity of water at invert.

Vo = Velocity of water of exit.

$\emptyset$ = angle of projection.

m = mass of flowing water.

This force will be on the bucket between invert and exit of jet. Let the difference in elevation between invert and exit end of bucket is 'h'.

The force/ft of elevation $= m(V - Vo\cos\emptyset) \times \frac{1}{h}$

(ii) Centrifugal Theory:-

The flow of water through the curved bucket will exert centrifugal force in addition to static head of water. The Total force on the bucket will thus be the sum of centrifugal force and static head of water.

The centrifugal force $= \frac{mV^2}{R}$

The static pressure/ft $= \gamma d$, where γ is Unit weight of water and d is the thickness of jet.

Total pressure/ft. of bucket $= \left(\frac{mV^2}{R} + \gamma d\right)$ ---------------(14)

(iii) Pressure by Considering Flow as Irrotational Vortex Flow:-

The determination of pressure distribution by considering the flow in the bucket as irrotational vortex was suggested by Douma[34]. The same has been discussed by Balloffet[35] and he has tried to compare the pressure distribution so arrived with the experimental data obtained from model tests carried out for Hartwell and Pine Flat Dams.

Douma assumed the velocity distribution on the bucket would be that of irrotational vortex. For this flow pattern, all the stream lines near the bucket boundary are circular and concentric with the bucket. The stream lives U and equel potential lines ν

can be determined by the function.

$$u = K \log r = \text{constant.}$$

$$v = K\theta = \text{constant.}$$

The absolute velocity can be expressed as

$$V = \frac{\partial v}{\partial r} = \frac{K}{r} \qquad \text{------------------------------------} (15)$$

Where K is a constant and r is the radius of any stream line. The figure 8.6 shows that the uppermost stream line has radius (R-d) where 'd' is the thickness of jet passing over the bucket and R is the radius of Bucket. The difference in piezometric head between point 1 and 2 located on upper stream line and the bucket is obtained by Bernoulli's equation applied to irrotational flow.

The pressure at point 1 is atmospheric. According to Bernoulli's equation

$$\frac{p_2}{\gamma} + Z_2 + \frac{V_2^2}{2g} = 0 + Z_1 + \frac{V_1^2}{2g}$$

$$\frac{p}{\gamma} = (Z_1 - Z_2) + \frac{V_1^2 - V_2^2}{2g}$$

$$= \Delta Z + \frac{V_1^2}{2g}\left(1 - \frac{V_2^2}{V_1^2}\right) \text{ - - - - - - - - - -} (16)$$

From equation (15) it is known that velocity is inversly proportional to radius.

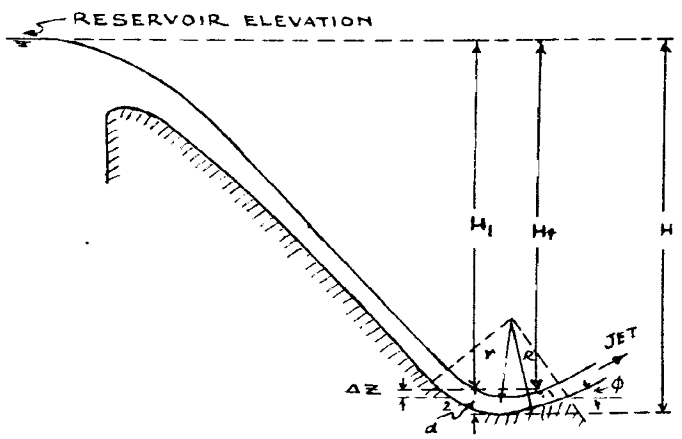
$$(R-d) \times V_1 = R \times V_2$$

$$\text{or} \quad \frac{V_2}{V_1} = \frac{R-d}{R}$$

substituting the value of $\frac{V_2}{V_1}$ in equation (16)

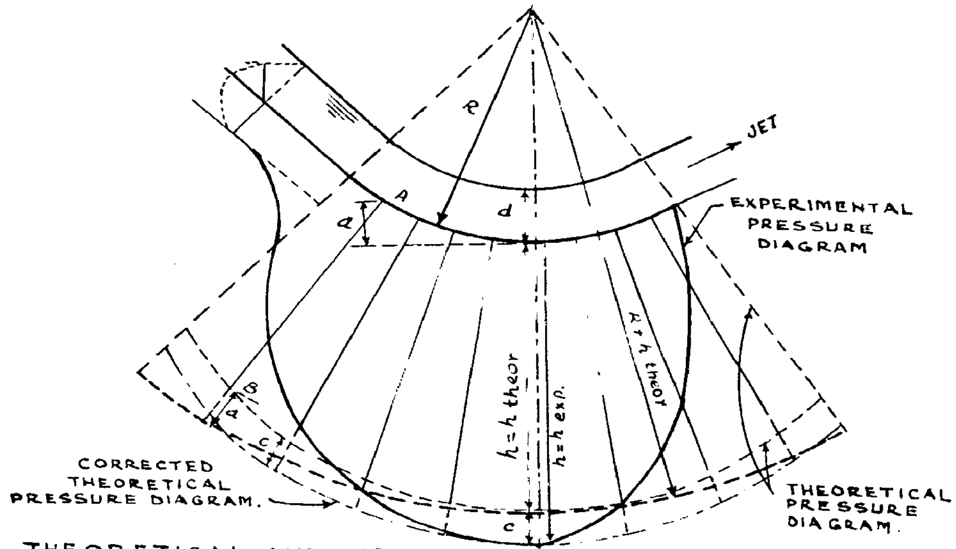
$$\frac{p_2}{\gamma} = \Delta Z + \frac{V_1^2}{2g}\left[1 - \left(\frac{R-d}{R}\right)^2\right] \text{ - - - - - - - - - -} (17)$$

Considering the loss over spillway as negligible V_1 can be taken equal to $\sqrt{2gH_t}$ where H_t is the difference in elevation between maximum water surface and point 1. From this it can be seen that V_1 will also vary according to H_t while an irrotational vortex pattern of flow assumes that V_1 varies along with radius. For



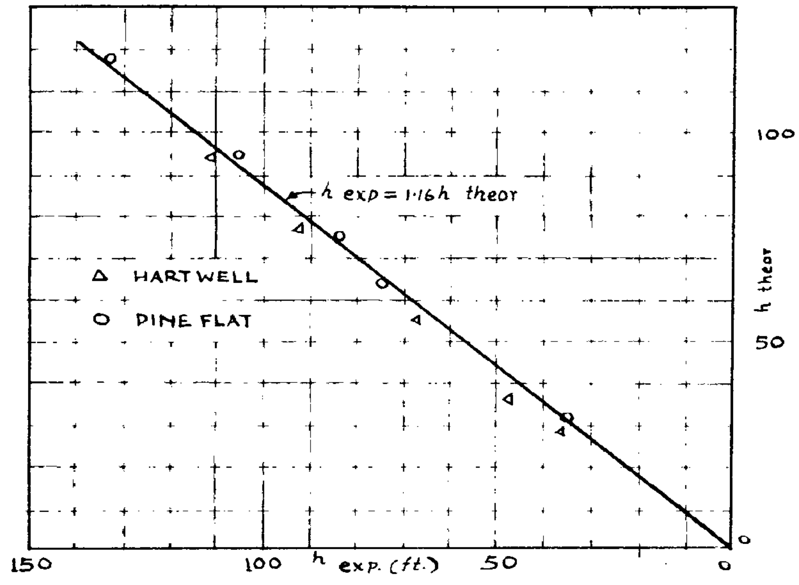
CHUTE FLIP BUCKET

FIG. 8-6



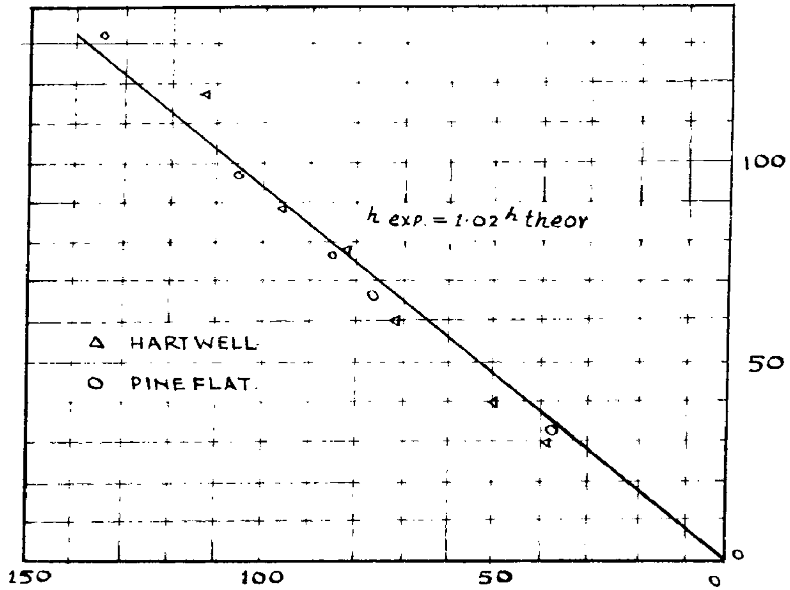
THEORETICAL AND EXPERIMENTAL PRESSURE DIAGRAMS.

FIG. 8-7



OBSERVED VERSUS THEORETICAL PRESSURES USING EQ (24)

FIG. 8-8



OBSERVED VERSUS THEORETICAL PRESSURES USING EQS (20) AND (24)

FIG. 8-9

high dams the variations in Ht at different points in bucket is very small. Therefore point 1 is selected at lowest water surface elevation over the bucket and so

$$Ht \doteq H_1, \text{ and } \frac{V_1^2}{2g} \doteq H_1$$

$$\therefore \frac{P_2}{\gamma} = \Delta z + H_1\left[1-\left(\frac{R-d}{R}\right)^2\right] \text{------------} (18)$$

P_2 can be determined if thickness of jet 'd' is known. The discharge intensity q is known. The d can be determined by integrating the velocity as below-

$$q \doteq \int_{R-d}^{R} V\,dr \text{------------} (19)$$

Substituting the value of V in terms of V_1 with the helop of eq(15)

$$q \doteq \int_{R-d}^{R} \frac{R-d}{r} \cdot V_1\,dr$$

$$\doteq V_1\,(R-d)\int_{R-d}^{R} \frac{dr}{r}$$

$$V_1\,(R-d)\log\frac{R}{R-d} \text{------------} (20)$$

By expanding in log series

$$q = V_1(R-d)\left[\frac{d}{R} + \frac{1}{2}\left(\frac{d}{R}\right)^2 + \frac{1}{3}\left(\frac{d}{R}\right)^3 + \text{------}\right] \text{------} (21)$$

Now from (21) d can be determined as other things are known.

The approximate thickness of jet can be determined by neglecting the variations in velocities across the flow.

$$d \doteq \frac{q}{V_1} \doteq \frac{q}{\sqrt{2g\,H_1}} \text{------------} (22)$$

The maximum pressure will be at lowest point of bucket as is maximum and is equel to d at this point. Thus from eq (18)

$$h = \left(\frac{P_2}{\gamma}\right)_{max.} = d + H_1\left[1-\left(\frac{R-d}{R}\right)^2\right] \text{--------} (23)$$

$$= d + \frac{V_1^2}{2g}\left[1-\left(\frac{R-d}{R}\right)^2\right] \text{--------} (24)$$

Knowing the maximum pressure, theoretical pressure diagram can be determined as shown in Fig. 8-7. A circle is drawn with centre at the bucket centre and radius equal to $(R+h)$. Then the pressure at any point A may be represented by radial segment A B equal to h minus the elevation of point A with respect to lowest point of the bucket surface.

In the above discussion, the curvature of flow is assumed to start at the point of tangency between spillway and bucket. Actually the flow curvature starts from upstream of begining of bucket. For a thin flow however it may be reasonable to assume that flow curvature starts close to tangency point.

Balloffet made a comparision between theoretical and experimental pressure distribution in a bucket. This is shown in Fig. 9-7. The experimental and theoretical values agree fairly. The disagreement at ends of bucket may be due to assumption made in the preceding para. Balloffet also made comparision between maximum theoretical pressure and the experimental data observed on models for Hartwell and Pine Flat Dam and is shown in Fig. 8-8. It is seen that the best fit straight line can be expressed as

$$h_{experimental} \doteq 1.16\ h_{theoretical} \qquad \text{------------------}(25)$$

While computing the theoretical pressures d was calculated from eq (22). When d was computed from eq (20) and pressure was computed from eq (24) with this value of d, the comparision between maximum theoretical pressure with experimentally observed pressure, is shown in Fig. 8-9. The best fit line can be expressed as

$$h_{experimental} \doteq 1.02\ h_{theoretical} \qquad \text{------------------}(26)$$

From the above it is seen that irrotational theory provide better closeness between theortical and experimental values of maximum pressures.

In a converging spillway the distribution of discharge over bucket may not be uniform. The maximum discharge may be at the centre of flow length. In such cases the pressures over the central

TABLE 8-II

DETAILS OF SKIJUMP BUCKETS IN INDIA.

S.No.	Name of Project.	Location (State)	Year of completion.	Designed discharge Q in cusecs.	Discharge intensity q in cusecs.	Radius of bucket R in ft.	Lip angle in degrees.	Type of bed strata.
1.	Tilaya.	Bihar	1953	1,36,000	266	20	30°	Alternate bands of quartizite and Mica schists, highly weathered at certain places running parallel to axis of da
2.	Hirakud.	Orissa.	1956	14,70,000	1120	50	38°-42°	Granite and granitic gneiss partially blocky, few narrow bands of mica schists.
3.	Maithon.	West Bengal.	1957	5,64,000	1035	35	43°	Fresh rock near surface.
4.	Panchat Hill.	Bihar.	1959	6,34,000	840	60	43°	Fresh rock near surface, laminated and fractured.
5.	Mandira.	Orissa.	1959	3,00,000	460	25	-	-
6.	Gandhi Sagar.	Madhya Pradesh.	1960	4,80,000	576	100	30°-42°	Grey whitish quartzites, laminated and fissured.
7.	Rihand.	Uttar Pradesh.	1962	4,71,000	906	60	30°	Good granite rock.
8.	Vaitarna.	Maharastra.	-	2,00,000	300	80	35°	-
9.	Girna.	Maharastra.	Not yet completed.	3,09,000	445	50	35°	-
10.	Nagarjunsagar.	Andhra Pradesh.	Not yet completed.	7,50,000	800	90	30°	Horizontally bedded quartzite sand stone.
11.	Ranapratapsagar.	Rajasthan.	Not yet completed.	7,50,000	658	55	40°	-
12.	Ukai Dam.	Gujarat.	Not yet completed.	13,88,000	1050	90	40°	-
13.	Banas.	Gujarat.	Not yet completed.	2,44,000	460	72	35°	-
14.	Salandi.	Orissa.	Not yet completed.	1,85,000	315	45	30°	-

portion of bucket are greater than contemplated based on the assump-
-tion of uniform discharge intensity. At the sides the pressures are less over the bucket.

Komparision Of Pressures Calculated By Various Methods :-

The pressures on bucket were computed with the help of above three methods in the case of Nagrajun Sagar Dam [36] whose bucket elements are as under :-

Entry slope.	0.7 to 1
Bucket radius .	90 ft.
Angle of lip .	30^{0}
Discharge intensity.	800 cusecs
Velocity at invert of bucket.	138 ft/sec.
Velocity at exit of bucket.	133 ft/sec.

The pressures calculated by three methods are fiven below:-

S.No.	Method	Pressure in lbs.
1.	Momentum theory	2880
2.	Centrifugal theory	3642
3.	Irrotational vortex flow	2890

From the above it is seen that pressure computed by momentum theory and irrotational vortex flow theory are very close while the pressure calculated from centrifugal theory is quite on higher side as compared to the other two methods.

EXAMPLES OF TRAJECTORY AND SKIJUMP BUCKETS.

These type of energy dissipators have been used extensively in India and abroad. The Table 8-II and Fig. 8-10 give the details of projects in India where this type of energy dissipation arrange-
-ments have been provided or proposed to be provided. Besides the

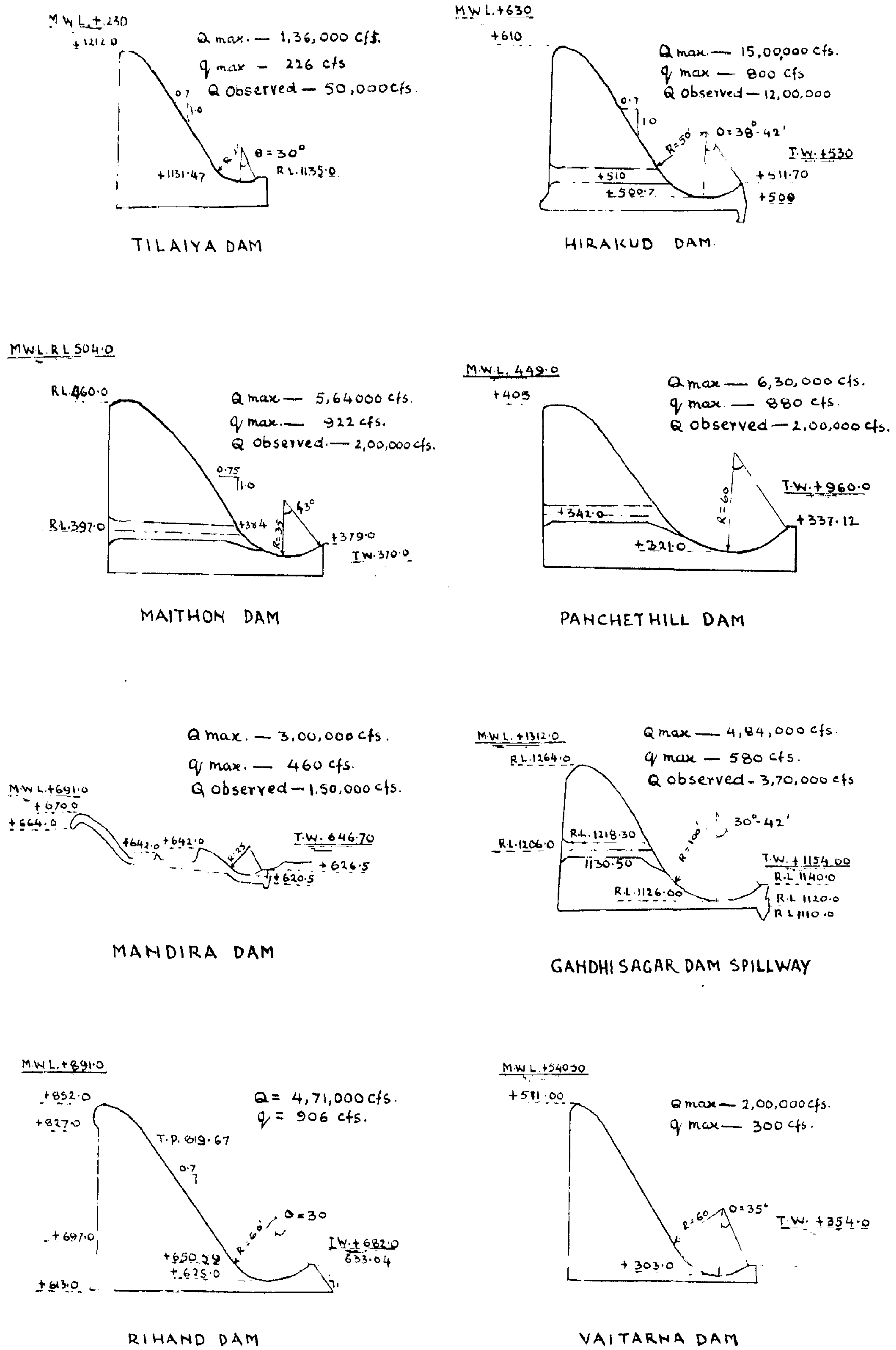

M.W.L.+1230
+1212.0
Q max. — 1,36,000 cfs.
q max — 226 cfs
Q observed — 50,000 cfs.
0.7
1.0
θ = 30°
+1131.47
R.L. 1135.0
TILAIYA DAM
M.W.L.+630
+610
Q max. — 15,00,000 cfs.
q max — 800 cfs
Q observed — 12,00,000
0.7
1.0
R=50
θ=38°-42'
T.W. +530
+510
+511.70
+500.7
+500
HIRAKUD DAM
M.W.L. R.L. 504.0
R.L. 460.0
Q max — 5,64000 cfs.
q max — 922 cfs.
Q observed — 2,00,000 cfs.
0.75
1.0
43°
R.L. 397.0
+384
R=35
+379.0
T.W. 370.0
MAITHON DAM
M.W.L. 449.0
+405
Q max — 6,30,000 cfs.
q max — 880 cfs.
Q observed — 2,00,000 cfs.
T.W. +960.0
+342.0
R=60
+337.12
+321.0
PANCHET HILL DAM
Q max. — 3,00,000 cfs.
q max. — 460 cfs.
Q observed — 1,50,000 cfs.
M.W.L. +691.0
+670.0
+664.0
+642.0
+642.0
R=25
T.W. 646.70
+626.5
+620.5
MANDIRA DAM
M.W.L. +1312.0
R.L. 1264.0
Q max — 4,84,000 cfs.
q max — 580 cfs.
Q observed - 3,70,000 cfs
30°-42'
R.L. 1218.30
R.L. 1206.0
R=100
1130.50
T.W. + 1154.00
R.L. 1140.0
R.L. 1126.00
R.L. 1120.0
R.L. 1110.0
GANDHI SAGAR DAM SPILLWAY
M.W.L. +891.0
+852.0
+827.0
Q = 4,71,000 cfs.
q = 906 cfs.
T.P. 819.67
0.7
1
θ = 30
R=60
+697.0
+650.59
+625.0
T.W. +682.0
633.04
+613.0
RIHAND DAM
M.W.L. +540.50
+511.00
Q max — 2,00,000 cfs.
q max — 300 cfs.
R=60
θ=35°
T.W. +354.0
+303.0
VAITARNA DAM

FIG. 8-10 (a)

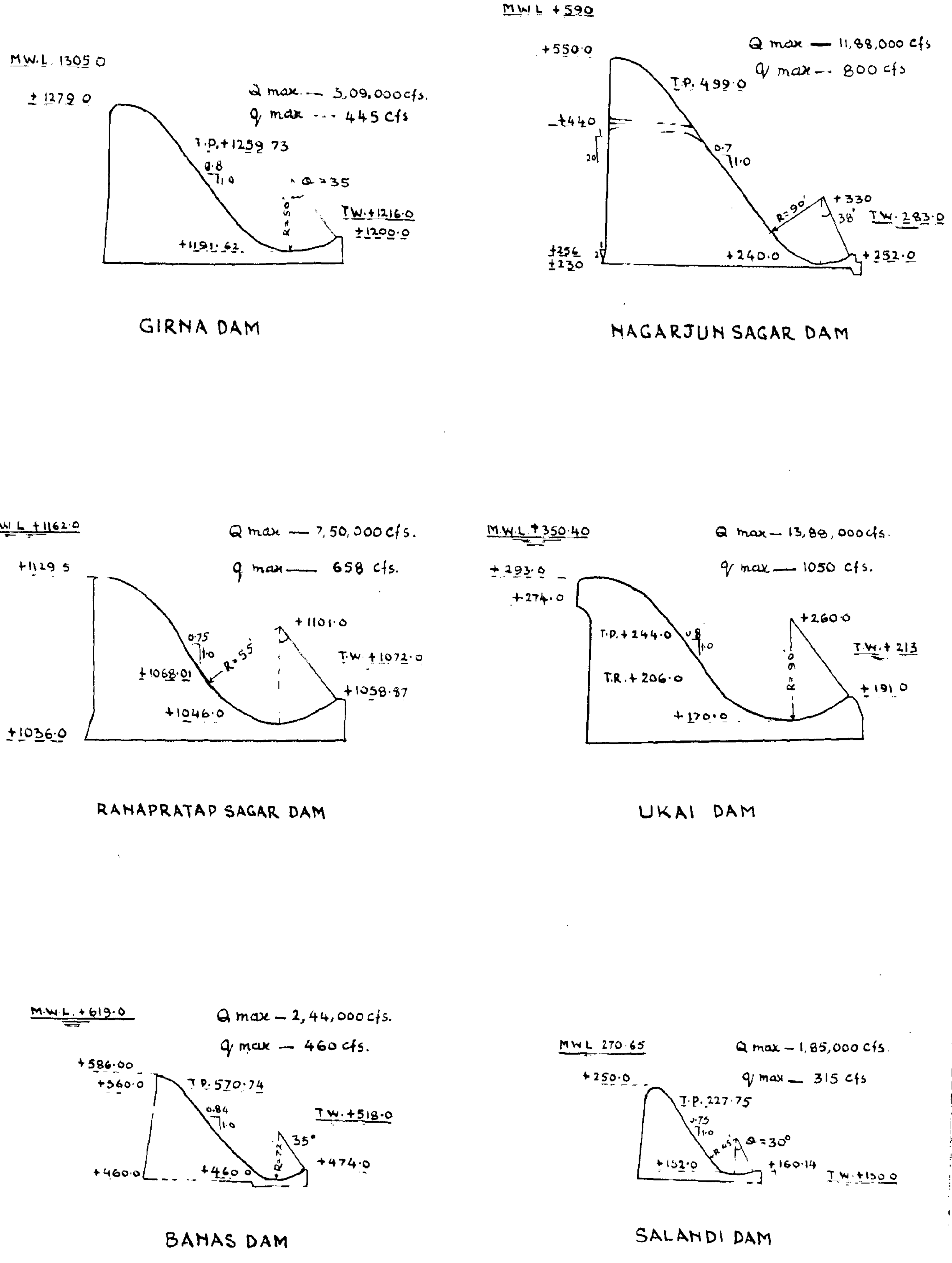

M.W.L. 1305.0
+1279.0
Q max — 5,09,000 cfs.
q max — 445 cfs
T.P. +1259.73
0.8
1.0
Φ = 35
R = 50'
T.W. +1216.0
+1200.0
+1191.62
GIRNA DAM
M.W.L. +590
+550.0
Q max — 11,88,000 cfs
q max — 800 cfs
T.P. 499.0
+440
20
0.7
1.0
R = 90'
+330
38°
T.W. 283.0
+256
+130
+240.0
+252.0
NAGARJUN SAGAR DAM
M.W.L. +1162.0
Q max — 7,50,000 cfs.
q max — 658 cfs.
+1129.5
+1101.0
0.75
1.0
R = 55
T.W. +1072.0
+1068.01
+1058.87
+1046.0
+1036.0
RANAPRATAP SAGAR DAM
M.W.L. +350.40
Q max — 13,88,000 cfs.
q max — 1050 cfs.
+293.0
+274.0
+260.0
T.P. +244.0
0.8
1.0
T.W. +213
R = 90
T.R. +206.0
+191.0
+170.0
UKAI DAM
M.W.L. +619.0
Q max — 2,44,000 cfs.
q max — 460 cfs.
+586.00
+560.0
T.P. 570.74
0.84
1.0
T.W. +518.0
35°
R = 72
+474.0
+460.0
+460.0
BANAS DAM
MWL 270.65
Q max — 1,85,000 cfs.
+250.0
q max — 315 cfs
T.P. 227.75
0.75
1.0
R 45
Φ = 30°
+152.0
+160.14
T.W. +150.0
SALANDI DAM

FIG. 8-10 (b)

bucket elements the type of strata available in river bed has also been indicated. In table 8-III, the details of trajectory type of buckets used in projects located in foreign countries have been given. In table 8-IV the details of skijump buckets used in projects located in different countries have been given.

The above tables clearly indicate that trajectory or skijump type energy dissipators have been used for discharge intensities as big as 2690 cusecs.

PERFORMANCE OF SKIJUMP BUCKETS IN INDIA.

In India, the skijump buckets have been adopted in many dams some of which have been completed and the spillways have operated for some years. Some of the spillways have functioned to its near maximum capacity while the rest have not yet functioned to its full capacity. The details of these skijump buckets are given in table No. 8-II serial No. 1 to 8. A study on the performance of these type of buckets was conducted by Handa and Thomas [37] on six dams i.e. Hirakud, Maithon, Tilaya, Panchet Hill, Gandhisagar and Rihand Dam, and they have indicated that the performance of these buckets have been satisfactory. The scour pattern has more or less followed the pattern anticipated on the basis of earliar model studies and is sufficiently far away so as not to endanger the main structure. The bed strata of the above mentioned six spillways is fairly good.

ECONOMIC ASPECT OF SKIJUMP OR TRAJECTORY BUCKETS.

It has been already stated earliar that this type of energy dissipation arrangement is usually provided where the tail-water depth is inadequate for the formation of hydraulic jump and the bed starta is composed of fairly firm rock. This type of bucket can be adopted provided foundation strata permits, even where the tailwater is adequate for the formation of jump as this will be cheaper than hydraulic jump stilling basin. Where tailwater is inadequate the adoption of hydraulic jump type stilling basin will

TABLE 8.III.

TRAJECTORY BUCKET TYPE ENERGY DISSIPATORS.

Structure.	Location.	Bucket type.	Q cfs.	q cusecs.	H (ft.)	$\frac{h}{H}$	exit slope in degrees.	R (feet)
Anchor.	Wyoming.	high	13,500	193	102	0.44	14	Parabolic.
Arkport.	New York.	low	29,100	341	-	-	45	10.2
Chilhowee.	Tennessee.	low	2,30,000	865	71	1.00	20	32
Conowingo.	Pennsylvania.	low	8,80,000	391	92	0.76	12.5	40
Deneeprostroy.	U.S.S.R.	high	8,35,000	334	-	-	-	37.9
Hartwell.	Georgia.	high	5,65,000	975	161	0.57	30	30
Pine Flat.	California.	low	3,95,000	1350	372	0.89	20	50
Safe Harbur.	Pennsylvania.	low	9,70,000	365	59	0.76	27	42

H = Vertical distance between maximum pool elevation and river bed.

h = Vertical distance between maximum pool elevation and bucket invert.

R = Bucket radius.

q = Discharge intensity.

TABLE 8-IV.

SKIJUMP SPILLWAY CHARACTERISTICS.

Structure.	Location	No. of Spillways.	H (ft.)	$\frac{h}{H}$	Q	q
Castelo de Budo.	Portugal .	two	370	0.82^{x}	70,600	2590
Bort .	France.	one	377	0.64	42,200	977
Chastang.	France.	Two	233	0.59	70,400	1580
Cleveland.	British Columbia.	One	300	0.62	43,000	540
Genissiat.	France.	-	210	0.75	95,000	1935
Kamishiba.	Japan.	Two	361	0.30+	38,000	600
L'Aigle.	France.	Two	295	0.60	70,400	1342
Mareges.	France.	One	255	0.48	24,600	290
Picote.	Portugal.	One	298	0.66	-	-
St. Etienne Cantales.	France.	Two	230	0.53	17,600	753

x = average.

+ = Right spillway only.

q, H = Same as in table 8-III

h = Vertical distance between Maximum pool elevation and spillway exit.

Q = Discharge for single spillway.

require a depressed apron below the river bed level to provide the adequate tailwater depth for the formation of jump. The provision of depressed apron normally requires huge quantity of rock excavation which makes the hydraulic jump stilling basin very costly. The adoption of trajectory or skijump bucket, therefore, is obviously very economical in comparision with hydraulic jump stilling basins. For instance in case of Rihand Dam in Uttar Pradesh the details of which are given in table 8-II, the economic studies conducted by Rehand Design Directorate [38] showed a saving of Rs 30 lakhs over hydraulic jump type stilling basin.

In the above paragraphs, the various aspects pertaining to trajectory or skijump buckets have been discussed. Its suitability and design considerations have been dealt in detail. The examples of these type of buckets in India and abroad have been given. The performance of these buckets in our country have been described. The economic aspect of these buckets vis-a-vis hydraulic jump stilling basins, have also been discussed.

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CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS.

The aim of the present study has been to review the existing available knowledge on energy dissipation arrangements which are commonly adopted below dam spillways. In this initial chapters an attempt has been made to discuss the basic characteristics of hydraulic jump on a horizontal apron, fully sloping and part sloping part horizontal apron. After this, the choice of particular type of energy dissipator with reference to hydraulic conditions have been dealt with. This has been followed by discussion on each type of energy dissipation arrangements viz. (i) hydraulic jump type stilling basins utilising horizontal apron, sloping apron and part sloping part horizontal apron. (ii) roller buckets- solid as well as slotted type. (iii) Skijump or trajectory buckets.

From the discussion on basic characteristics of jump on horizontal apron it is seen that the length of jump has not so far been determined accurately. No rational formulae have been developed for the same. However it can said that the existing knowledge about the jump length is quite adequate for the design of stilling basins utilising hydraulic jump as energy dissipator. This is so, because the stilling basins are seldom designed to contain the full length of jump. As regards energy dissipation is concerned , there is not much controversy about the clear jump being more efficient than the submerged type of jump. Some authorities are of the opinion that the energy dissipation in a submerged jump is not much different than the clear jump as long as submergence is about 10% or so.

The length of jump on a sloping apron is also not beyond controversy but the existing knowledge is quite adequate for the design of stilling basin for reasons pointed out in the preceding para. The energy dissipation in a jump on sloping apron has been studied by very few authors. The existing knowledge indicates that the energy dissipation in a jump reduces with the increase of apron slope. Inspite of this, hydraulic jump type stilling basins with sloping apron are favoured as the jump is most effectively controlled as compared to jump on a horizontal apron.

In chapter 4, the hydraulic design consideration and choice of energy dissipators to suit the various type of site conditions have been discussed. From the discussion it is clear that the choice of energy dissipator is dependent on hydraulic, topographical and geological conditions at site.

The hydraulic jump stilling basins have been extensively used. The stilling basins with horizontal apron generally utilise the various appurtenances such as chute blocks, baffle piers and end sill to reduce the length of jump and to control and ensure the formation of jump even with a little less tailwater. The amount of reduction in basin length due to use of appurtenances still remains a matter of controversy but the general opinion is that the basin length can be reduced by 1/3rd to 2/3rd. The use of baffle piers is not favoured where the velocities are greater than 40-50 ft/sec. to avoid cavitation. As regards shape and placing arrangement of these appurtenances are concerned, different authorities have recommended different arrangements but the general opinion is to place baffle piers as near as permissible without causing cavitation. The purpose is to create as much obstruction as possible without developing cavitation. The use of these stilling basins with appurtenances are recommended for dams of low and medium height. In high dams, high velocities are involved and as the use of some of the appurtenances is not favoured for fear of cavitation, the control of jump becomes extremely difficult. With slight difficiency in tailwater the jump may sweep off the paved apron and scour in the downstream may result which may endanger the structure itself.

The stilling basins with sloping apron utilising hydraulic jump as energy dissipator, are suitable even for high dams involving large discharge intensities and high velocities. The jump movement on a fully sloping apron or part sloping part horizontal apron has a limited range and there is less likelyhood of jump being swept off the paved apron. The tailwater requirement goes on reducing as the slope of apron is increased but the rock excavation increases leading to increase in cost of stilling basin. The rock excavation can be reduced and stilling basin can be made economical with the adoption of part sloping and part horizontal apron though the tailwater requirement is slightly more than the corresponding case of fully sloping apron. The quantitative reduction in tailwater requirement for different sloping aprons and part sloping part horizontal aprons, have been discussed in Chapter 6. The charts have been prepared which will be very useful to the designer in selecting the most economical combination of part sloping and part horizontal apron without much trial, to suit a particular hydraulic condition.

The discussion on hydraulic jump type stilling basins will be incomplete without mentioning the limitations of such type of basins for use below gate controlled spillways. The hydraulic jump stilling basins are designed to function for a particular tailwater and jump height relationship. This presupposes that the discharge over spillway is flowing uniformly over the entire length of spillway which is only possible when the crest is

~~crest is~~ uncontrolled or the gates are so operated that these are lifted equally in all bays. Such operation of gates is hardly possible in actual practice. In such cases the sequent depth required for the formation of hydraulic jump within the basin, will not be available and the basin will not function as anticipated. Therefore this aspect needs to be considered while selecting this type of energy dissipator.

The roller bucket type of energy dissipators are suitable where tailwater is in excess of jump requirement. This type of bucket is definitely cheaper to hydraulic jump stilling basin as the quantity of concrete in the former is much less in comparision to the latter. The solid roller buckets have been used at many dams but these have a tendency to suck river bed material inside the bucket which causes abrasive action resulting in costly maintenance problems The slotted roller buckets have been evolved as an improvement over solid roller buckets. The provision of slots helps to remove the debris sucked inside the bucket and thus ensures self cleaning action. The design procedure of roller buckets have been discussed in chapter 7. The design of these buckets is still in stage of evolution. The studies conducted by various authors and procedure of designs of different bucket elements laid down by them, can only serve as a guide to work out preliminary designs.

The trajectory or skijump bucket type energy dissipators are recommended where bed strata consists of firm rock and tailwater is inadequate for the formation of hydraulic jump. Where the foundation condition permit, this type of bucket is cheaper than hydraulic jump type stilling basin even where tailwater is adequate for the formation of jump . The adoption of this type of bucket is especially very economical where tailwater is inadequate for the formation of jump and huge quantity of rock excavation is involved for providing apron at suitable level to ensure the formation of jump. As regards performance of these type of buckets it can be said that these have functioned satisfactorily in this country. The design considerations of these type of buckets have been discussed in detail in Chapter 8. The fixing of bucket elements for a particular case needs to be based on existing knowledge, experience on similar works and model studies. The formulae for determination of likely scour where jet strikes the ground have not yet been evolved with a degree of certainty as there are many variables affecting scour of which most important is the river bed material. At present the anticipated scour can only be estimated with the aid of model studies which are more qualitative than quantitative.

In the above paragraphs an attempt have been made to summarise the conclusions arrived after conducting the present study. The limitations of existing knowledge on various types of energy dissipators have been stated and it is felt that there is ample scope for further study on the following aspects.

(i) Length of jump on fully sloping apron at higher range of Froude numbers.

(ii) Length of jump on part sloping and part horizontal aprons.

(iii) Energy dissipation characteristics in a submerged jump vis-a-vis clear jump on a horizontal apron.

(iv) Energy dissipation characteristics about jumps on sloping apron.

(v) Reduction in length due to use of appurtenances in a stilling basin and relative location of different appurtenances.

(vi) Design procedure of roller bucket elements.

(vii) Design procedure of Trajectory or skijump bucket elements.

Lastly it is felt that the problem of energy dissipation for each site requires consideration on the basis of particular hydraulic, topographical and geological characteristics. The experience on similar type of existing structures and the present knowledge about designs of energy dissipators can only help in working out the preliminary designs for a particular case. These preliminary designs can be finalised with the aid of model studies keeping due regard to the limitation of such studies.

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BIBLIOGRAPHY.

1. "Hydraulic Design of Stilling Basins and Energy Dissipators" Engineering Monogram No. 25, U.S.Department of Interior, Bureau of Reclamation 1964.

2. Bradley, J.N., and Peterka, A.J., "Hydraulic Design of Stilling Basins" Proceedings A.S.C.E. Vol.83, Journal of Hydraulic Division No. HY 5, October 1957.

3. Silvester, R., "Hydraulic Jump in All Shapes of Horizontal Channels" Proceedings A.S.C.E. Vol.90, Journal of Hydraulic Division No. HY 1, Jan. 1964.

4. Backhmeteff B.A., and Matzke, A.E., "The Hydraulic Jump in Terms of Dynamic Similarity" Transactions A.S.C.E. Vol.101, 1936.

5. Govinda Rao, N.S. and Rajratnam, N., "The Submerged Hydraulic Jump "Proceedings A.S.C.E. Vol. 89, Journal of Hydraulic Division, No. HY 1 , Jan. 1963.

6. Kindsvater, Carl E., "The Hydraulic Jump in Sloping Channels" Transactions A.S .C.E. Vol 109, 1944.

7. Hickox, G.H., Discussion on "The Hydraulic Jump in Sloping Channels" by C.E.Kindsvater, Transactions A.S.C.E. Vol.109, 1944

8. Stevens, J.C., Discussion on "The Hydraulic Jump in Sloping Channels" by C.E.Kindsvater, Transactions A.S.C.E. Vol.109, 1944.

9. Elevatorski, E.A., "Hydraulic Energy Dissipators" McGraw Hill Book Company, 1959.

10. Vischi-Solano Vega, Discussion on "Hydraulic Design of Stilling Basins" by Bradley and Peterka , Proceeding A.S.C.E. Vol. 84, Journal Hydraulic Division No. HY 2 , April 1958.

11. Bradley, J.N. and Peterka, A.J., Closing discussion on "Hydraulic Design of Stilling Basins" Proceedings A.S.C.E. Vol. 84, Journal of Hydraulic Division No. HY 2 , April 1958.

12. Kindsvater, Carl E., Closing discussion on "The Hydraulic Jump in Sloping Channels" Transactions A.S.C.E. Vol.109, 1944.

13. Government of India, Central Board of Irrigation and Power "The Standing Wave or Hydraulic Jump" Publication No.7, 1950.

14. Liliavosky, S., Irrigation and Hydraulic Design. Chapman and Hall Ltd., London, 1955, Vol. 1.

15. Thomas, Dr.A.K.C., Write up on Hydraulic jump based on lectures Delivered at Water Resources Development Training Centre, Roorkee, 1965.

16. Progress Report of The Task Force on "Energy Dissipators for Spillways and Outlet Works Committee of Hydraulic Structures" Proceedings A.S.C.E. Vol. 90, Journal Hydraulic Division No. HY 1 , Jan. 1964.

17. Rouse, H., Siao, T.T., Nagaratnum " Turbulence Characteristics of Hydraulic Jump" Proceedings A.S.C.E. Vol.84, Journal of Hydraulic Division No. HY 1, Feb.1958.

18. Pillai, N.N., and Unni, T.E., "Shapes of Appurtenances in Stilling Basin Design "Proceedings A.S.C.E. Vol.90, Journal of Hydraulic Division, No. HY 3, May 1964.

19. Gumensky D.B., "Air Entrainment in Fast Water Affects the Design of Training Walls of Stilling Basin" Civil Engineering Vol.19, Page 831 to 833 and 889, 1949.

20. Rajratram, N., "Effect of Air Entrainment on Stilling Basin Performance" Central Board of Irrigation, India, Journal of May 1962.

21. Blaisdell, Fred W., "Development of Hydraulic Design, St. Anthony Falls Stilling Basin" Proceedings A.S.C.E.Feb.1947.

22. Ven-Te-Chow "Open Channel Hydraulics" McGraw, Hill Book Company.

23. Central Board of Irrigation and Power "Symposium on Energy Dissipators "Publication No. 70, 1961.

24. Berryhill, R.H., "Stilling Basin Experiences of Corps of Engineers" Proceedings A.S.C.E., Vol.83, Journal of Hydraulic Division, No. HY 3, June 1957.

25. Mcpherson, M.B., and Karr, M.H., " A Study of Bucket Type Energy Dissipator Characteristics" Proceedings A.S.C.E. Vol. 83, Journal of Hydraulic Division No. HY 3, June 1957.

26. Elevatorski, E.A., Discussion on " A Study of Bucket Type Energy Dissipator Characteristics" by Mcpherson and Karr. Proceedings A.S.C.E. Vol. 83, Journal of Hydraulic Division No. HY 5 , Oct. 1957.

27. Mcpherson, M.B., and Karr, M.H., Closing discussion on "A Study of Bucket Type Energy Dissipator Characteristics" Proceedings A.S.C.E. Vol.84, Journal of Hydraulic Division No. HY 5 , Oct. 1958.

28. Dougherty, Gale B., discussion on "A Study of Bucket Type Energy Dissipator Characteristics" by Mcpherson and Karr, Proceedings A.S.C.E. Vol. 84, Journal of Hydraulic Division, Feb. 1958.

29. Peterka, A.J. , and Beichley G.L., "The Hydraulic Design of Slotted Spillway Buckets" Proceeding A.S.C.E. Vol. 85, Journal of Hydraulic Division No. HY 10, Oct. 1959.

30. Mcpherson M.B., Discussion on "The Hydraulic Design of Slotted Spillway Buckets" by Beichley and Peterka, Proceedings A.S.C.E. Vol. 86, Journal of Hydraulic Division, April 1960.

31. Elevatorski, E.A., "Trajectory Bucket Type Energy Dissipators" Proceedings A.S.C.E., Vol. 84, Journal of Power Division, Feb. 1958.

32. Rouse, Hove, and Metzler "Experimental Investigations of Fire Monitors and Nozzles" Transactions A.S.C.E. Vol. 117, Page 1147, 1952.

33. Damle, P.M., Venketraman, C.P., and Desai, S.C., "Evaluation of Scour Below Skijump Buckets of Spillways" Golden Jublee Symposia Vol. 1, Central Water and Power Research Station, Poona, 1966.

34. Douma J.H., Discussion on "Design of Side Walls in Chute and Spillways" by Gumensky, Transactions A.S.C.E. Vol. 119, Page 364-367, 1954.

35. Balloffet A., "Pressures in Spillway Flip Bucket" Proceedings A.S.C.E. Vol. 87, Journal of Hydraulic Division, No. HY 5, September 1961.

36. Central Board of Irrigation and Power Annual Report (Technical) Publication No. 68, 1958.

37. Handa C.L., and Thomas, Dr. K.C., "Performance of Skijump Type of Energy Dissipators" Golden Jublee symposia Vol. 1, Central Water and Power Research, Poona, 1966.

38. Chaturvedi, U.L., Write up on "Overflow Spillway Dams" Lectures delivered at Water Resources Development Training Centre, Roorkee, 1965.

39. Central Board of Irrigation and Power Annual Report (Technical) Publication No. 66, 1957.

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