SPILLWAY CREST DESIGN TO SUIT DIFFERENT OPENINGS OF GATES AND MODES OF OPERATION

A Dissertation
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of
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CERTIFICATE

Certified that the dissertation entitled" SPILLWAY CREST DESIGN TO SUIT DIFFERENT OPENINGS OF GATES AND MODES OF OPERATION" which is being submitted by John Eapen 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 from Ist October,1968 to 15th November,1970 for preparing this dissertation.

Dated: 24. 11. '70

(R. S. VARSHNEY)

A_C_K_N_O_W_L_E_D_G_E_M_E_N_T_S

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Numerous articles on the subject as referred to in the bibliography have been consulted and materials from them have been freely used in the preparation of this dissertation for which, he expresses his gratefulness to the various authors of these articles.

(JOHN EAPEN)

SYNOPSIS

The ogee spillway has been the subject of more research than perhaps any other hydraulic structure. However, there are still certain functions of spillway, which need deeper probe. In the past, tendency had been to fix the co-ordinates of the profile in such a way that they corresponded, more or less , to the under side of a fully ventilated nappe flowing over a sharp crested weir, for the maximum head of water likely to occur over the crest of the spillway. Generally speaking a curve of this nature gives positive pressures practically for all conditions of operations. But when this design head is exceeded, negative pressures occur on the spillway face. Also the present day trend is to fix the design head less than the maximum head that is likely to come over the crest. Therefore, at this time of the maximum flow over the spillway, negative pressures are developed, and this is done to increase the hydraulic efficiency of the spillway i.e. the co-efficient of discharge is increased. All the same, if this negative pressure is allowed to approach vapour pressure of water, cavitation takes place and the spillway is damaged. Even though the flow profile is designed in such a way that negative pressure in the cavitating range is not developed, unless proper care is taken during construction also, cavitation may develop due to surface irregularities. Also obtaining a smooth, well aligned flow surfaces during initial construction is only part of the problem. Retaining the smooth surfaces over a period of years is also necessary which naturally depends on the type and strength of concrete used.

An attempt has been made in this dissertation to analyse the cause of cavitation and remedies to obtain a cavitation free spillway profile.

A comprehensive review is made on various standard crest profiles. Though, a little out of place, a brief mention has also been made of some of the profiles developed earlier.

The occurrence and the mechanism of cavitation in a general way has been dealth in Chapter 3.

This causes of cavitation on spillways such as wrong design, wrong location of the gate seat, wrong type of gate slots, badly finished construction surfaces etc. have been outlined in the dissertation in Chapter 4.

In the fifth chapter some possible remedies to avoid cavitation have been suggesged. Also, a brief review is made on the specifications of construction finishes, types and strength of concrete to be used etc.

CONTENTS

S Y M B O L S

A = Height of up-stream water surface above the river bed in m. or ft.

B = Pressure ~~lead~~ head (depth of any point on the spillway glacis) in m. or ft. of water.

b = Space between successive masonry blocks along the line of the spillway profile in metre or ft.

C = Constant for calculating the air demand for a spillway crest.

C_a = Co-efficient of discharge for flow over a spillway in the equation $Q = C_a . L . \sqrt{2g}\, H^{3/2}$

C_d = Co-efficient of discharge of flow over a spillway in the equation $Q = C_d . L . H^{3/2}$

C_p, min. = Minimum pressure co-efficient.

D = Depth of gate slot in metre or ft.

d = Tolerance allowed between successive masonry blocks of the spillway glacis in metre or ft.

E = Maximum distance lower nappe rises above sharp crest of weir in metre or ft.

ft. = Feet.

g = Acceleration due to gravity in m/sec^2 or ft./sec^2

H = Actual head of water over spillway crest in m or ft.

H_{av} = Average pressure head on a spillway crest in m or ft.

H_d = Design head or head over spillway crest with negligible velocity of approach in m or ft.

H_e = Head over spillway crest including velocity of approach in m or ft.

H_m = Head of water at the centre of opening, when spillway gate is partially open and no flow over the gate in m or ft.

Ho = Head of water over top of gate in m or ft.

Hs = Head of water above the sharp crest of a weir, including head due to velocity of approach in m or ft.

H_u = Head over the centre of opening beneath crest gate when water is flowing over the top of the gate in m or ft.

h_a = Vapour pressure head of water at a given temperature in m or ft.

ho = Ambient pressure head of the fluid in m or ft.

hp = Pressure on spillway crest in m or ft.

h_t = Atmospheric pressure in m or ft. head of water.

hv = Velocity head due to velocity of approach in m or ft.

I = Height of opening beneath the gate in m or ft.

K = Cavitation parameter or number $(ho - ha)/\frac{V^2}{2g}$

K_c = constant in the exponential equation for a spillway profile $\frac{Y}{Hd} = K_c \left(\frac{x}{Hd} \right)^{N_c}$

K_i = Critical or incipient cavitation number.

k = Height of boundary roughness in m or ft.

L = Length of the spillway crest in m or ft.

M = Height of riser on off-set weirs in m or ft.

m = Metre

N = Horizontal or vertical displacement on off-set and overhang weirs in m or ft.

N_c = Value of exponent in the equation for the down stream spillway profile.

P = Depth of approach floor below sharp crest of weir in m or ft.

p_V = Vapour pressure of water in Kgm /cm^2 or pounds / inch^2.

p = Ambient fluid pressure in Kgm /cm^2 or pounds/ft.^{2}

p_g = Partial pressure of gas inside a bubble nuclei in kgm./cm^2 or pounds /inch^2.

p_{min} = Minimum pressure head allowed on a spillway crest in m or ft. of water.

p_r = Reduction of pressure head beneath the nappe in m or ft.

p_v = Vapour pressure of water at a given temperature in kgm/cm^2 or pounds /inch^2.

Q = Total discharge over a spillway in cumecs or cusecs.

q = Discharge over a spillway per unit length in cumecs /m or cusecs /ft.

q_a = Quantity of air required per unit length of spillway crest in cumecs /m or cusecs /ft.

R = Radius for fixing the up-stream quadrant of a spillway in m or ft.

r = Radius of gas bubble in cm or inch.

s = Curved distance of a spillway crest from a reference point in m or ft.

Sec = Second.

T = Height of spillway crest above river bed in m or ft.

U_* = Shear velocity in m/sec. or ft./sec.

$\bar{V}$ = Velocity of the flow of water in m/sec. or ft./sec.

V_h = Horizontal velocity of the jet of water at the spillway crest in m/sec. or ft./sec.

V_v = Vertical velocity of the jet of water at the spillway crest in m/sec. or ft./sec.

W = Width of gate slot in m/ft.

w = Specific weight of water in kgm./m^3 or pounds /$ft.^3$

x_c = Horizontal distance of the nappe crest from the sharp crest of weir in m or ft.

y_c = Vertical distance of the nappe crest from the sharp crest of weir.

Z = Off-set or surface projection into or away from the flow in m or ft.

δ = Thickness of boundary layer.

δ^* = Displacement thickness of boundary layer.

σ = Surface tension of water in kgm /m or pound/ft.

ρ = Mass density of water in Metric slugs /m^3 or slug s /$ft.^3$

CHAPTER : 1

I N T R O D U C T I O N

1.0 GENERAL :

1.1 Spillways are provided for storage and detention dams to release surplus of flood water, which can not be contained in the allotted storage space and at diversion dams to bye-pass flows exceeding these, which are turned into the diversion system. Ordinarily the excess water is drawn from the top of the pool created by the dam and conveyed through an artificial water way (discharge carrier) back into the river or some natural drainage channel. Nearly all spillways fall into one of the six types or are made up of combination of them:

1. Over fall
2. Gates and Orifice
3. Through or Chute
4. Side Channel
5. Shaft or glory hole
6. Syphon.

The over fall type is by far the most common and is adopted for concrete and masonry dams that have sufficient crest length to provide the desired capacity. Over fall spillways provided with crest gates will act as orifices under partial gate openings and as open crest weirs under full gate openings.

1.2 A spillway for a reservoir serves one or more of of the three principal purposes.

1.2.1.0(a) Security against over topping of the dam:

1.2.1.1 The spillway serves as " safety valve" to discharge flood waters in excess of the normal storage capacity of the reservoir and discharge capacities or regulating outlets or power turbines to prevent over-topping of the non-over flow portions of the dam. Performance of the function is the essential requisite of any spillway.

1.2.2.0 (b) Surcharge Control :

1.2.2.1 A spillway may also be designed in some cases to limit the surcharge of a reservoir during floods. This routing of floods through spillway reduces the flood peaks, causing non-damaging releases, in order to avoid excessive damage down-stream of the dam. Because of the dam storage, the eroding tendencies of the fast flowing waters upstream are also curbed to a very great extent.

1.2.3.0 (c) Reservoir Storage Regulations:

1.2.3.1 If suitably located and designed for operational use, the spillway, may be utilised for release of stored waters, either as a subtitute for regulating outlets or to supplement limited outlet capacity under normal operating conditions.

1.3 Hence the importance of a spillway can not be over emphasised. Many failures of dam have been caused by improperly designed spillways of insufficient capacity. Ample capacity is of paramount importance for earth and rockfill dams, which are likely to be destroyed, if they are toppled; where as concrete dams may be able to withstand moderate over topping. " The spillway should be of such a capacity" says Hazra " that the only objection may be that its capacity is too much".

2.0 PURPOSE:

2.1 Having fixed the maximum discharge that has to be passed over a spillway, the next problem is to design a spillway that is structurally safe and hydraulically efficient In order to achieve the latter, the crest of an over flow spillway is usually formed to fit the shape that this overflowing water would take for a selected maximum design head over the spillway crest. The success of any spillway largely depends on its operation in such a way that for most of the time, the design head is not exceeded. But in practice this may not be always possible. For example, during flood times, one or more gates may become inoperative. Under some flood conditions, operational errors may actually increase the discharge at the dam above the natural peak flow of the river. By storing early on the rising side of the flood hydrograph, available storage space may be filled in advance of the peak flow. When the design head is increased the tendency of the over flowing nappe is to move away from the concrete surface. This tendency can though increases the co-efficient of discharge, creates sub-atmospheric pressures, on the spillway face. This is also the case, when the maximum discharge is passing over an under designed spillway. In this type, the spillway design head is fixed lower than the maximum head expected in order to increase the co-efficient of discharge. When this negative approaches the vapour pressure , cavitation takes place. The purpose of this dissertation is to analyse the causes of cavitation, its remedies and to design a suitable profile free from cavitation, at the same time giving a high co-efficient of discharge.

3.0 SCOPE:

3.1 Even though a spillway can be damaged due to many reasons, one of the most harmful effects is due to the action of cavitation. As is well known, cavitation on a spillway surface occurs, when the pressure at the crest falls below atmospheric pressure and approaches the vapour pressure of water. In order to avoid negative pressure on the crest, the shape of a spillway is usually made to fit the shape of a under side of a fully ventilated nappe flowing from a sharp crested weir. But this flow pattern in a case like this differs from that of a flow from a sharp crested weir, the moment a solid boundary is placed below the nappe, as in this case, forces other than gravity also come into play. So the shape of a spillway has to suitably fixed so that no adverse pressure conditions are developed on the crest for all possible conditions of flow and operations. Design of the proper shape of the spillway is only part of the problem. For example, if the other elements of the spillway such as, pier nose, gate slots etc. are also not properly designed, it can lead to sub-stmospheric pressures and consequent. Cavitation on these elements and the adjascent spillway face is only natural. The location of the gate seat play an important role on the pressure variation of a spillway profile at the time of partial gate openings. The inception of cavitation on a spillway face also depends on the surface roughness of the material with which it is constructed and naturally this condition makes it obligatory to specify the standards of construction finishes with a more rational approach, rather than with the help of the rule of thumb. Naturally the best method of avoiding cavitation is the elimination of negative pressure by properly designing a spillway crest and other connected elements of a spillway. However, it may not be always possible to eliminate negative pressure on a spillway for economical considerations. So the problem of selecting a proper construction material also comes in. Even though concrete is the most popular construction material, its cavitation erosion resistance can be considerably improved by increasing its compressive strength especially near the surface by special design and as well as with the help of special construction techniques. In the following pages an attempt has been made to review the various causes as well as some of the possible remedies of cavitation on spillways. However, few pages have been devoted for a review of the cavitation phenomena in general, as it is only imperative that a better understanding of the cavitation phenomena is required in order to evolve suitable remedies to make the structure cavitation free.

4.0. FUTURE SCOPE OF WORK :

4.1 Even though the purpose of this dissertation was to cover all aspects of cavitation on spillways, full justice could not be done on certain aspects of the problem for want of material as little research seems to have been done on these aspects. For example concrete alone was considered as a construction material for spillways, even though some passing mention ~~have~~ has been made about this aspects in Chapter: 5 . It is widely accepted that this cavitation resistance of a material increases in direct proportion to its compressive strength. So the desirability of providing ashlar stones masonry, at least in those areas where stone masonry is popular, has to be explored. But this type of construction can be adopted only after ensuring proper bond between the facing stone and inner layers of the masonry to avoid dislodgement of stones at high velocity flow. Also further work appears to be necessary to specify the surface dressing required. Even though some studies have been made By the U.S.Bureau of Reclamation on the effect of surface roughness on the inception of cavitation, the studies are not elaborate enough to be adopted in all cases and much scope remains for further study in this field. Another field where there is scope for study is in the design of pier noses both on the up-stream and down-stream sides. Even though individual cases are being studied for each and every dam, a general solution to this problem seems to be lacking. Yet another aspect which requires further research is the design of the spillway crest itself after allowing a certain degree of sub-pressures. The usual practice in designing an ogee spillway is to fix various co-ordinates with the help of one of the standard profiles and then check the suitability, expecially from the pressure variations point of view with the help of model studies. As the negative pressure increases the co-efficient of discharge also increases and it seems only logical to allow certain amount of negative pressure on the crest, without really reaching the cavitating range.

Even though many organisations advocate the use of 75% of the maximum head as the design head for fixing the co-ordinates none of them seems to specify the exact value of negative pressure that can be allowed and then to fix the design head and the co-ordinates which produced this desired negative pressure. One or two authors have done studies on this aspect. But their studies were very preliminary in nature and much remains to be done in this field to evolve suitable design curves which could be used for adopting for a wide range of spillways. So a more rational method in the design of spillways can be made if the negative pressure or better still, if the cavitation parameter K_i is also introduced at the design stage to fix the design head for which the co-ordinates are fixed and hence much scope still remains in this field for studies.

CHAPTER : 2

ANTI - VACUUM SPILLWAY PROFILES

1.0 GENERAL

1.1 From the second half of the 19th centuary many investigators, in various countries, have made studies for finding suitable profiles for spillways. Because of the intricacy of the problem, a purely mathematical solution appeared to have no chance. If the profile of a spillway is either angular or too rapidly curved, zones of locally reduced pressures and consequent seperation will prevail. In order to avoid such conditions of instability and possible cavitation, the crest shape of a spillway is usually shaped to conform, at design flow, to lower surface of a ventilated nappe from a sharp crested weir of the same relative height. Spillways with these type of sha pes are termed as " ogee spillways". With the crest so shaped, maximum discharge efficiency, without sub-atmospheric pressures on the crest , will be attained for the free flow with heads equal or less than the head for which the crest is designed. At the designed maximum over flow, there is no sub-atmospheric pressure along the spillway surface, at the same time the positive pressure is also reduced to the minimum possible for the best hydraulic efficiency. In other words, the water just glides over the surface without really touching it and at the same time not seperating from the surface either.

1.2 Even though, the shape of the spillway is exactly made to fit the lower nappe of the ventilated jet issuing from a sharp crested weir, replacing such a free constant pressure surface by a solid boundary imposes additional shear on the under side of the stream. ([33]) It may be possible to evaluate this taking it is a problem of boundary layer formation. However, this may be a more pronounced problem in small scale models (for small values of Reynolds numbers) than in actual structures.

1.3 Some of the spillway profiles developed by many questers and organisations are briefly described herein.

2.0. CREST SHAPE OF OVER FLOW SPILLWAYS:

2.1. Parabolic Profile :

2.1.1 Early crest shapes were usually based on a simple parabola designed to fit the trajectory of a free falling nappe (18) . The shape of the flow of nappe over sharp crested weir can be interpreted by the principle of a projectile shown in figure : 1 . According to this principle, it is assumed that the horizontal velocity component of the flow is constant and that the only force acting on the nappe is gravity. In the time t, a particle of water in the lower surface of the nappe will travel a horizontal distance x from the face of the weir, equal to

$$x = V.\, t \cos \theta \qquad \ldots \quad \ldots \quad \ldots \quad (1)$$

where V is the velocity at the point, where $x = 0$, and θ is the angle of inclination of the velocity V with the horizontal. In the same time t, the particle will travel a vertical distance of equal to:

$$y = -V.\, t \sin \theta + \tfrac{1}{2} g\, t^2 + E \qquad \ldots \quad \ldots \quad (2)$$

where E is the value of y at $x = 0$, apparently E is equal to the vertical distance between the highest point of the nappe and the elevation of the crest. Substituting the value of t from equation (1) , in equation (2), we get ,

$$y = -x \tan\theta + \tfrac{1}{2} g \frac{x^2}{V^2 \cos^2\theta} + E$$

i.e. $$\frac{y}{H_e} = \tfrac{1}{2} g \frac{x^2}{H_e V^2 \cos^2\theta} - \frac{x \tan\theta}{H_e} + \frac{E}{H_e}$$

$$= A\left(\frac{x}{H_e}\right)^2 + B\left(\frac{x}{H_e}\right) + C \quad \ldots \ldots \quad (3)$$

where $A = \dfrac{g H_e}{2 V^2 \cos^2\theta}$, $B = -\tan\theta$ and $C = \dfrac{E}{H}$

The above nappe equation is quadratic and so the nappe surface is theoretically parabolic.Blaidsell (5) has evaluated the values of these constants as $A = -0.425$, $B = 0.055$ and $C = 0.15$ in the case of flow over high spillways, where the velocity of approach is negligible. Experimental data have indicated that this equation is not valid where $\frac{x}{H_e}$ is less than 0.5. For $x/H_e < 0.5$, the pressure within the nappe in the vicinity of the weir crest is actually above atmospheric because of the convergence of stream lines. This analysis of Blaidsell gives the shape of the profile in the form:

$$\left(\frac{x}{H_e}\right)^{1.92} = 2.1\left(\frac{y}{H_e}\right)$$

Though the equation on the basis of simple parabola was used in earlier days, this principle is not correct, as forces other than gravity also act on the nappe.

2.2 Bazin's Profile :

2.2.1 The first and the most extensive studies of nappe, were those by Bazin made between 1886 and 1888 ( ) , in which he reduced his observations to unit head and constructed a basic

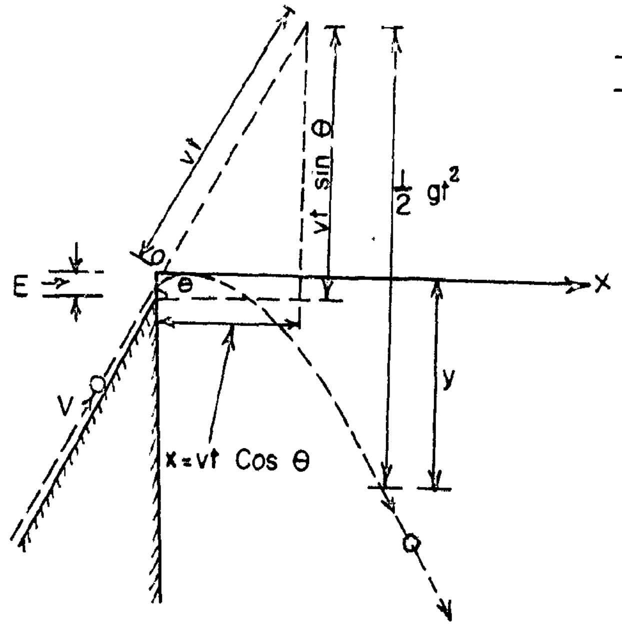


FIG. 1. VELOCITY COMPONENTS FLOW OVER WEIR

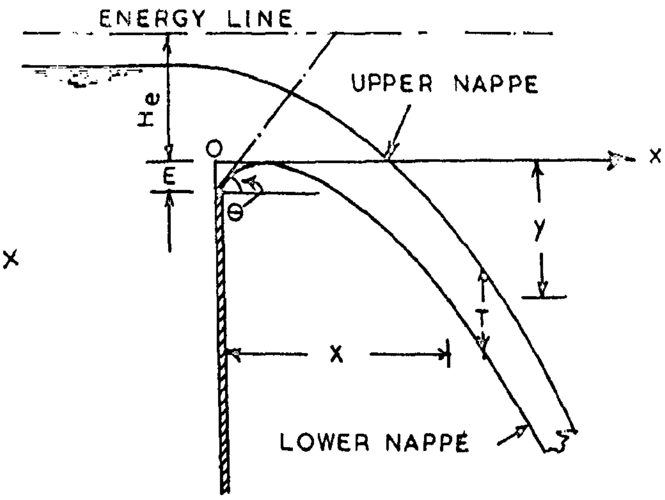


FIG. 2 FLOW OVER SHARP CRESTED WEIR DEFINITION SKETCH

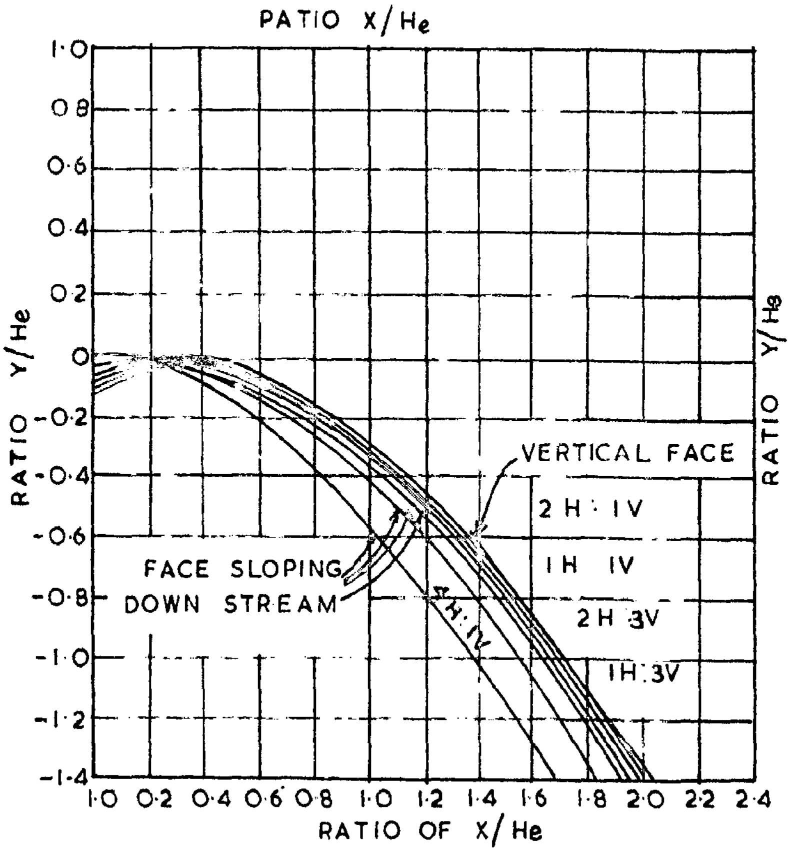
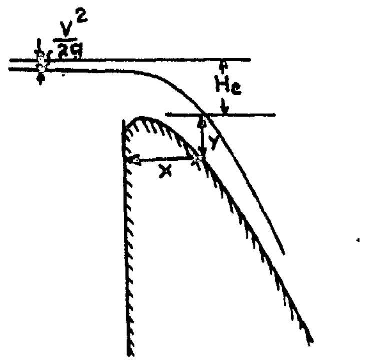


FIG.3. SHAPE OF LOWER NAPPE



DEFINITION SKETCH

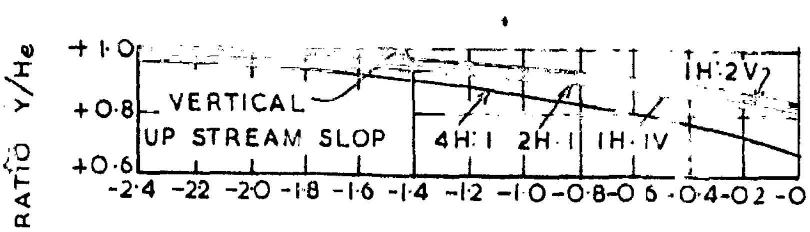


FIG. 4. y/He VERSUS X/He UP STREAM FROM WEIR CREST

curve representing the results of his experiments. Theoretically, the adoption of the Bazin profile, should cause no negative pressure on the crest. However, under actual conditions, there exists friction due to roughness onthe surface of the spillway. Hence, negative pressure on such a spillway can not be ruled out. Consequently, the Bazin profile has been modified and many other profiles for design proposes have been proposed.

2.3 Muller's Profile :

2.3.1 The first attempt in America to develop the shape of an overfall dam to fit the over flowing sheet was that of the Muller in 1908 (26). He attempted to extend a curve from the upper section of the lower nappe through Bazin's data. His expression for the path traced by the mean velocity of the nappe is:

$$x^2 - 2.3\ H_s\ y = 0$$

where, H_s is the head of water over the crest of the sharp weir. He fixed this origin of the co-ordinates at approximately 0.35 H_s above and 0.09 H_s from the theoretical weir crest. He measured downward one third the thickness of the nappe, normal to the curve of the mean velocity to locate the curve of the lower surface. Parker (52) reproduced Muller's curve and demonstrated that it does not fit well with Bazin's curve at the upper section. He attributed this difference to the fact that Bazin's curve were obtained with sharp edged notches, under heads of 0.5 m or less and Muller applied them to thick notches, under heads of 1.5 m or 3.0 m.

2.4 Morrison's and Brodi's Profile :

2.4.1 They have developed a parabolic equation of the form:.

$$x^2 = 1.8\ H_d\ y \qquad \ldots \quad \ldots \quad \ldots. \quad (4)$$

for the lower surface of the nappe, where H_d is the head measured from the highest point over the nappe (48). The origin of is also taken at this point. As a factor of safety they, however, recommend that the equation (4) be modified as:

$$x^2 = 2.55 \ H_d \ y .$$

2.5 <u>Seimeni's Profile</u> :

2.5.1 Seimeni (66) established an equation for the portion of the lower surface beyond x = 0.50 as

$$y = \frac{(x - 0.10)^2}{1.55} + 0.062 x - 0.186 \quad .. \quad .. \qquad (5)$$

in which the origin of the co-ordinates is at the sharp crest of the weir. Unfortunately, this equation does not apply to the most important portion of the overflow section, namely, the portion between the spring point and highest point of the trajectory. Seimeni's profile can also be approximately expressed in the form (30)

$$y = 0.47 \ x^{1.8}$$

2.6 <u>Randolph's Profile</u> :

2.6.1 Randolph of the U.S. War Department (58) derived an emperical equation for the lower surface of the nappe in the form:

$$y = 0.523 \ H_d^{-0.82} \ x^{1.822}$$

Here the origin of the co-ordinates is at the heighest point of the lower nappe surface.

2.7 H.L.Davis' Profile :

2.7.1 A similar equation has been developed by H.L.Davis of U.S.B.R. (82) with the same origin for co-ordinates as:

$$y = 0.485 \; H_d^{-0.882} \; x^{1.822}$$

2.8 Smetana's Profile :

2.8.1 This profile is given by the equation:

$$y = 0.461 \, x^{1.85}$$

2.9 C.V.Davis' Profile:

2.9.1 Davis has developed his own co-ordinates for the profile of the nappe for spillways with vertical face as well as with inclined face (19) . They are given in figure 3 , 4 . 5 and table 1. This table has been derived from an analysis based on studies of the U.S.B.R. and the results obtained from the studies of Bazin and Seiemeni.

2.10. Creager's Profile :

2.10.1 Creager's profile is developed from a mathematical extension of Bazin's data (14) . In the figures 7 & 8 , where the Creager's profile is given, for a value upto x = 0.65, the values are directly taken from the experiments conducted by Bazin. The extension of the experimental data was mathematically done by the following manner. The average velocity in any normal section of the sheet of water was found by Bazin to be very close to one third of the distance from the lower nappe as at point 4 in figure 6. The curve 2 , 3, 4 was drawn through these scaled points

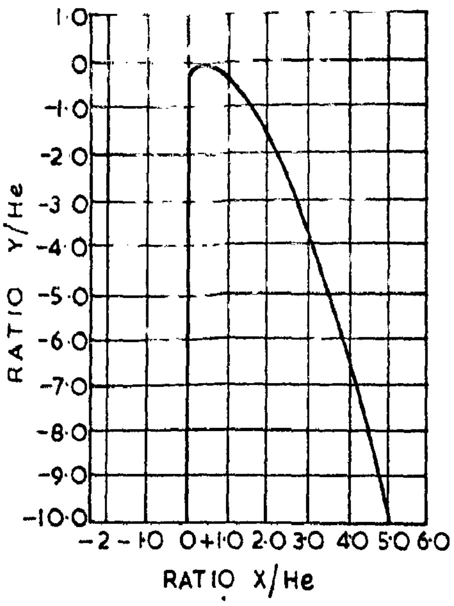


FIG. 5. SHAPE OF LOWER NAPPE

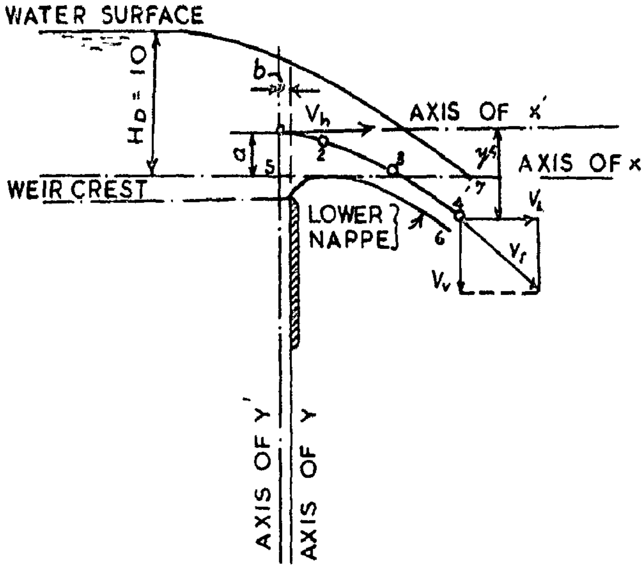


FIG. 6. DEFINITION SKETCH

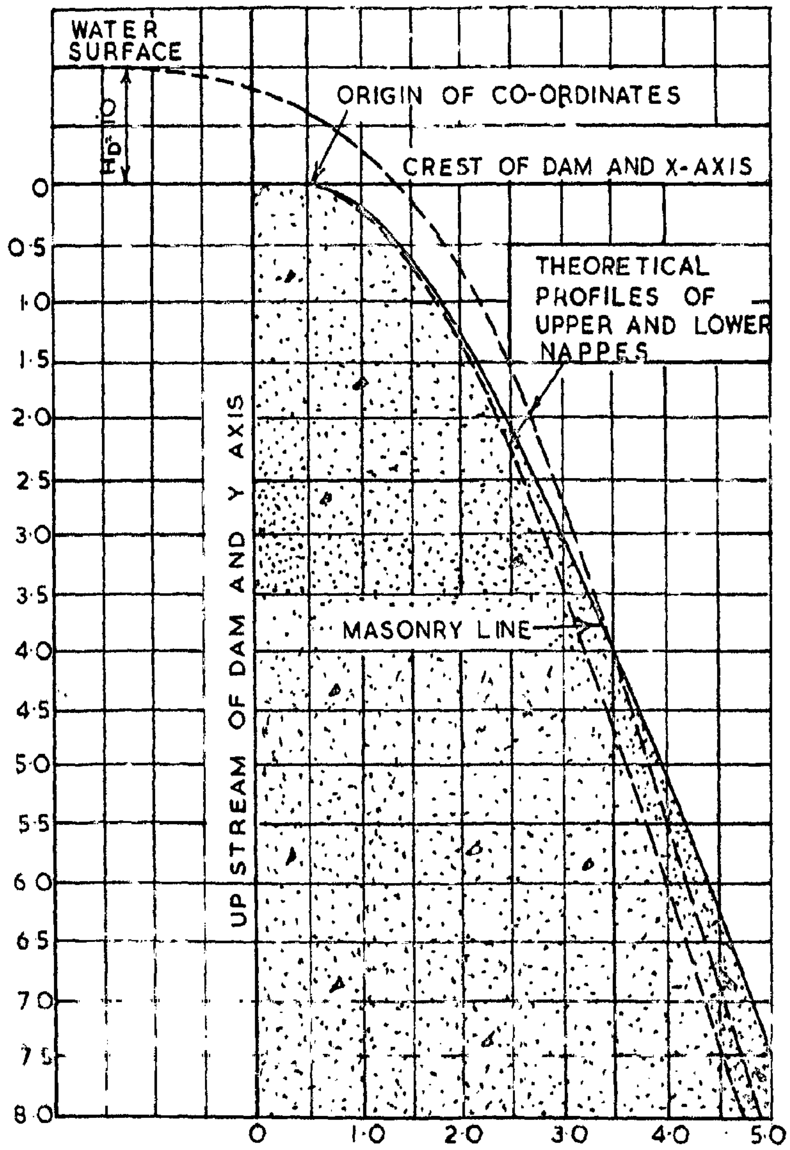


FIG. 7. SHAPE OF SPILLWAY CREST UP STREAM FACE VERTICAL (AFTER CREAGER)

TABLE NO: 1

NAPPE COORDINATES FOR SPILLWAY DESIGN LOWER NAPPE (19)

AFTER C.V.Davis

Horizontal Co-ordinates x/He	VERTICAL	Vertical Co-ordinates y/He							
		Weir face slope							
		Down stream					Up-Stream		
		1H : 3V	2H : 3V	1H : 1V	2H : 1V	4H : 1V	1H : 3V	2H : 3V	1H : 1V
1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
0.0	-0.125	- 0.098	- 0.066	- 0.045	- 0.012	- 0.004	- 0.159	- 0.176	- 0.191
0.10	-0.033	- 0.026	- 0.015	- 0.008	0.000	- 0.003	- 0.044	Same as 1:3 up stream slope	Same as 1:3 up stream slope
0.20	-0.004	- 0.003	- 0.001	0.000	- 0.007	- 0.020	- 0.009		
0.25	0.000	0.000	- 0.001	- 0.003	- 0.014	- 0.033	- 0.002		
0.30	0.000	- 0.001	- 0.004	- 0.009	- 0.023	- 0.049	0.000		
0.50	-0.034	- 0.035	- 0.046	- 0.062	- 0.093	- 0.143	- 0.033		
0.75	-0.129	-0 .138	- 0.158	- 0.182	- 0.228	- 0.315	- 0.128		
1.0	-0.283	- 0.297	- 0.323	- 0.352	- 0.413	- 0.538	- 0.277		
1.5	-0.738	- 0.767	- 0.803	- 0.842	- 0.933	- 1.133	- 0.727		
2.0	-1.395	- 1.437	- 1.483	- 1.532	- 1.653	- 1.928	- 1.377		
3.0	-3.303	- 3.377	- 3.443	- 3.512	- 3.693	- 4.118	- 3.277		
4.0	-6.013	- 6.117	- 6.203	- 6.292	- 6.533	- 7.108	- 5.977		

of average resultant velocities. The curve has the form of a jet spouting with an intial horizontal velocity Vh . The equation therefore is that of a parabola and may be expressed as:

$$x'^2 = \frac{2 \; V_h^2 \; y'}{g} \qquad \dots \quad \dots \quad \dots \qquad (6)$$

which is the equation of the curve 1, 2, 3, 4, with the origin of the curve at point 1, as shown in figure 6. Referring to point 5 the equation (6) becomes,

$$(x + b)^2 = \frac{2 \, V_h^2}{g} \; (y + a) \quad \dots \quad \dots \quad \dots \qquad (7)$$

where x and y are co-ordinates referred to the origin 5 and a , b and V_h unknown constants. Now substituting in equation 7, the measured values of x and y from each of the known points 2, 3, and 4 of the experimental curve, three equations may be formed with 3 unknown terms a, b and V_h and solving them the value of these constants can be evaluated. So having now formed the general equation for mean velocity curve, the next step is to find the thickness of the jet at various points on the curve. The vertical velocity V_v at a point y' below the origin is

$$V_v = \sqrt{2g \; y'}$$

The horizontal velocity, Vh, being known, the resultant velocity

$$V_r = \sqrt{V_h^2 + V_v^2} = \sqrt{V_h^2 + 2gy'}$$

Measuring thickness 6 - 7 , of the sheet from the plot of the experiments, the discharge of per linear meter may be calculated from the following equation:

$$q = \Lambda \, V_r = \Lambda \sqrt{V_h^2 + 2gy'} \qquad \dots \quad \dots \quad \dots \qquad (8)$$

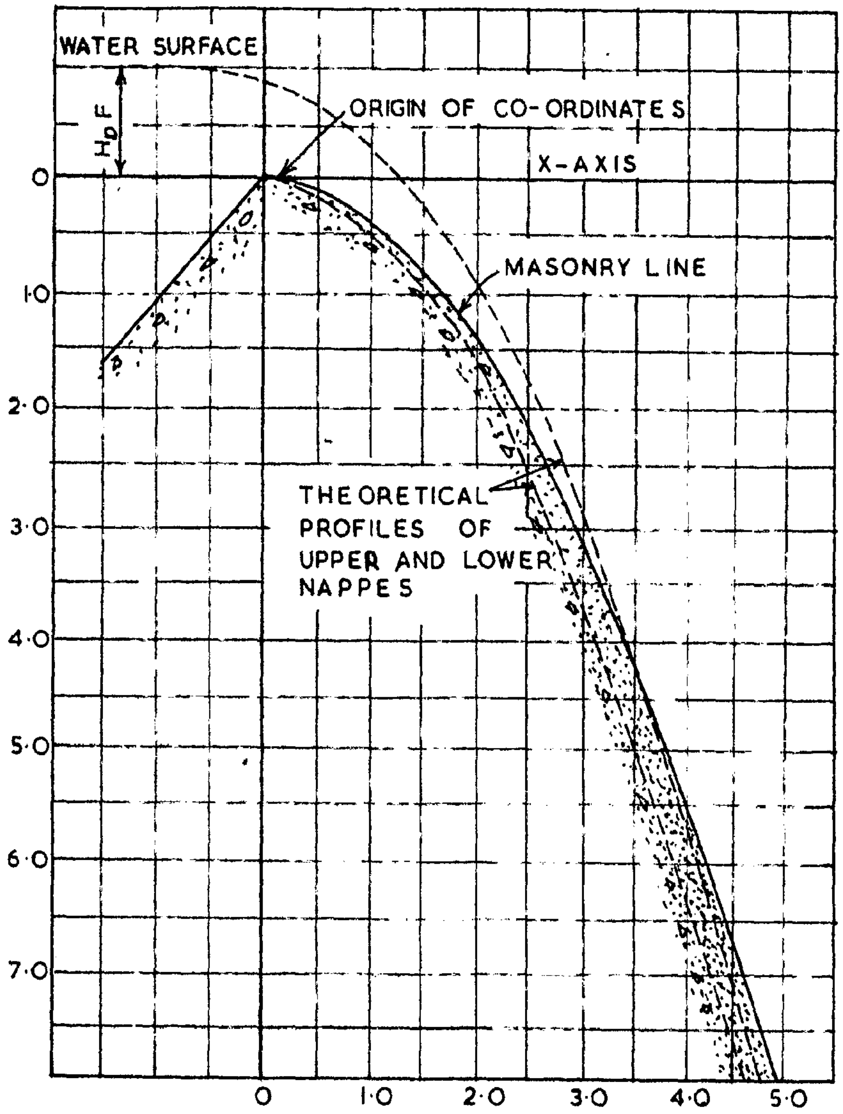


FIG. 8 SHAPE OF SPILLWAY CREST UP-STREAM FACE INCLINED (AFTER CREAGER)

where A is the thickness of the jet at 6 . 7 . The values of q for the two types of crests calculated in this manner were found to agree almost exactly with the discharge found by independent experiments for the same type of weirs.

2.10.2 The thickness of the sheet of water corresponding to any values of y' may now be derived from equation (8), having already evaluated the value of Vh and Vr . In this manner the path of the falling sheet was determined and plotted in figure 7 for unit head. The paths are only approximate as they are extended from experimental points relatively close to the top of the dam and moreover, they may be somewhat affected by the irregularities in the crest. So Creager thought it was advisable to provide a margin of safety by extending the masonry line well into the theoretical sheet as indicated in figure 7 & 8 .

2.11 Modified Creager Profile:

2.11.1 At the time of the first publication of the Creager's data, there were no experimental verification for his mathematical extension and therefore he recommended the use of masonry line, slightly beyond the computed lower nappe line as mentioned in section 2.10 above. However, since then the accuracy of the extensions were substantiated by experiments (69) and (64) and now it is considered permissible to limit the masonry to the computed lower nappe itself (15). The approximate shape of the Creager's profile can be expressed in the form,

$$y = 0.5 \ x^{1.85}$$

origin being taken at the crest of the spillway. The co-ordinates of the Creager's profile is given in Table 2 and figures 9 & 10.

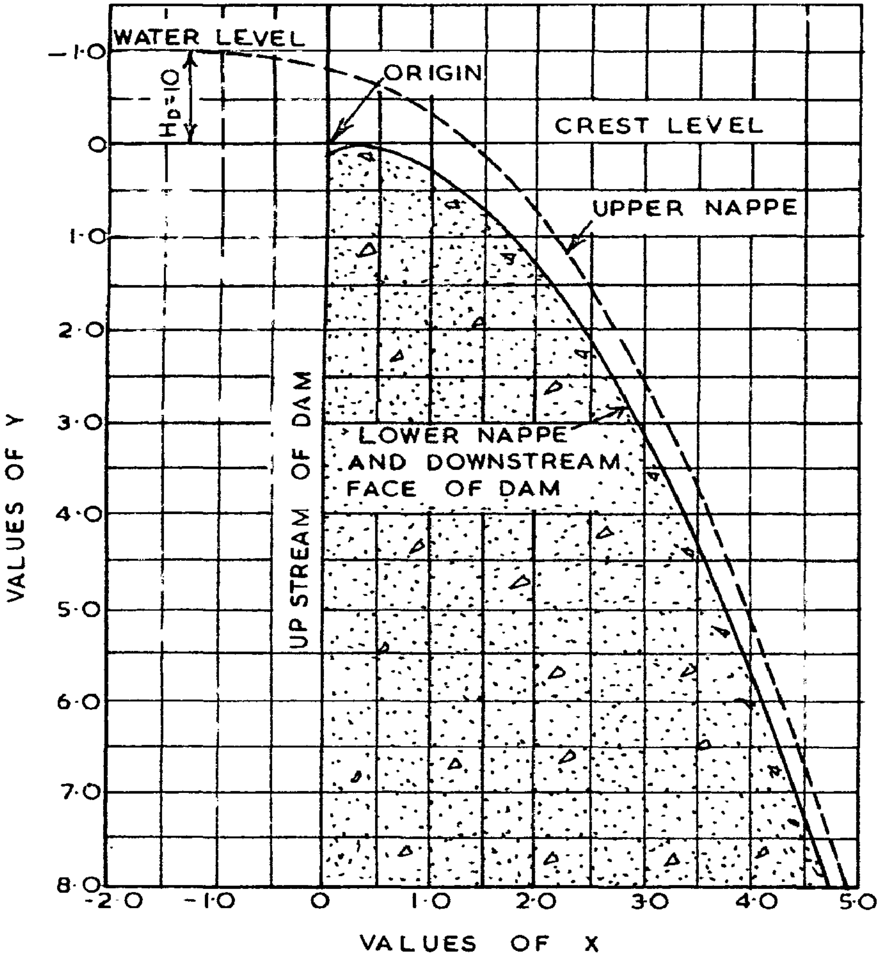


FIG. 9.

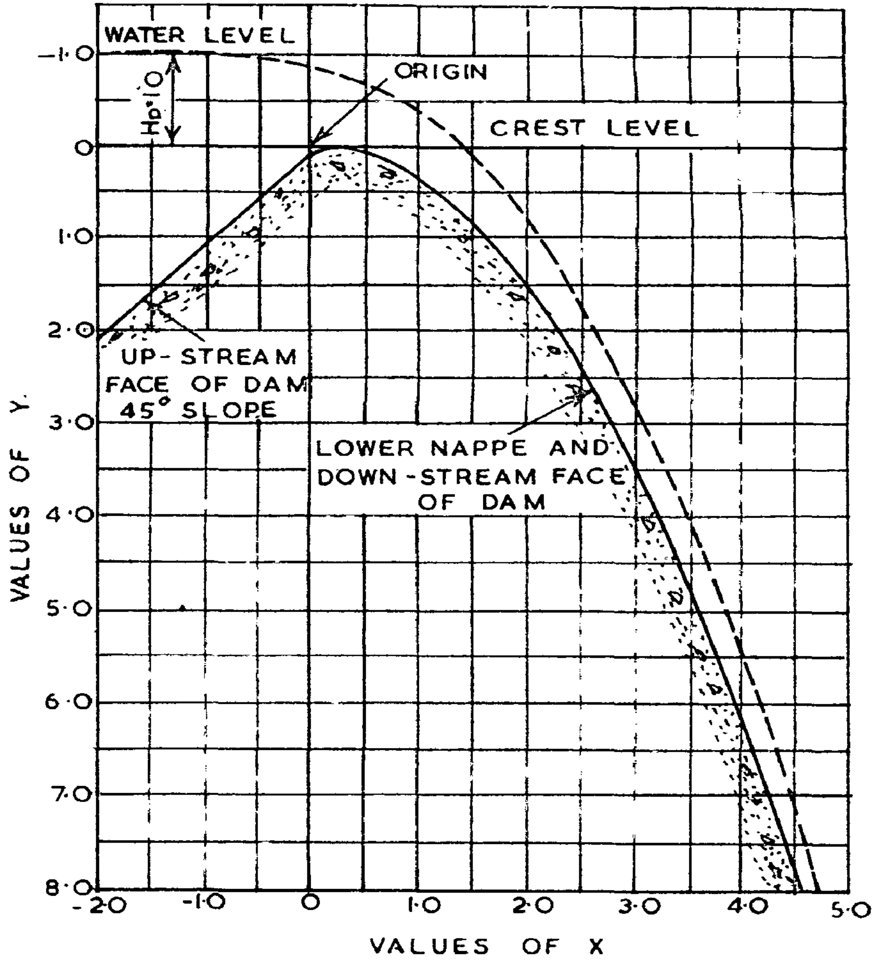


FIG 10

CREAGERS STANDARD CREST PROFILES

TABLE : 2

CREAGER'S CO-ORDINATES FOR LOWER NAPPE

For 1 m HEAD ON CREST

(a) Up-stream face vertical

x	y
0.0	0.126
0.1	0.036
0.2	0.000
0.4	0.007
0.6	0.063
0.8	0.153
1.0	0.267
1.4	0.590
2.0	1.31
3.0	3.11
3.5	4.26
4.0	5.61

(b) Upstream face inclined 45°

x	y
0.0	0.043
0.1	0.010
0.2	0.000
0.4	0.023
0.6	0.09
0.8	0.193
1.0	0.333
1.4	0.700
2.0	1.47
3.0	3.39
3.5	4.61
4.0	6.04

NOTE: The values of the x and y co-ordinates are in metres.

2.12 U.S.Bureau Of Reclamation Profiles :

2.12.1 From 1932 to 1948 extensive experiments on the shape of nappe over a shart crested weir were conducted by the U.S. Bureau of Reclamation (82). These studies were first initiated in connection with the design of spillways for the Hoover dam. On the basis of the experimental data including that of Bazin's the Bureau has developed co-ordinates for the nappe surfaces for various types of spillways. The studies included, spillways, with vertical faces, the upstream faces inclined downstream and spillways with over hangs on the up-stream side. This information is indeed invaluable for accurate analysis and precise design of spillway overflow sections.

2.12.2 Weir With Vertical Up Stream Face :

The principal purpose of the tests conducted by U.S.B.R. was to determine the effect of the velocity of approach on the profile of the lower nappe. The results ob-tained are given in Table No. 3*. It may be seen that the x and y co-ordinates are expressed in the form $\frac{x}{Hs}$ and $\frac{y}{Hs}$, this origin being at the sharp crest of the weir and Hs being the head including velocity of approach at this point. It may be seen that an the approach velocity increases (or as $\frac{hv}{Hs}$ increases, hv is the velocity head due to velocity of approach), the nappe profile for unit head flattens and crosses the profile for smaller values of $\frac{hv}{Hs}$. In other words, the profile in the case of a high dam, where the velocity of approach is negligible, the profile is not as flat as in the case of a low dam, for the same water depth and discharge over the crest. The greatest deviation

* The table given is not the complete table prepared by the US.B.R. Complete reproduction has not been done as it is too lengthy to be included in this dissertation.

TABLE No. 3

U.S.B.R. CO-ORDINATES OF LOWER NAPPE FOR DIFFERENT VALUES OF hv/Hs-VERTICAL WEIR

	Y/Hs.									
hv/Hs x/Hs	0.002	0.010	0.020	0.03	0.04	0.05	0.08	0.100	0.140	0.150
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.04	0.049	0.0477	0.046	0.045	0.044	0.042	0.038	0.034	0.03	0.028
0.10	0.086	0.0838	0.0810	0.0785	0.076	0.0735	0.0655	0.0602	0.052	0.048
0.20	0.1105	0.1072	0.1030	0.0998	0.0965	0.0922	0.0815	0.074	0.0599	0.0540
0.29	0.1110	0.1077	0.1035	0.0995	0.0955	0.0905	0.0775	0.0683	0.052	0.0447
0.30	0.1105	0.1067	0.1020	0.098	0.094	0.0892	0.076	0.0673	0.0503	0.0433
0.350	0.1060	0.1018	0.0965	0.0918	0.087	0.082	0.0685	0.0586	0.042	0.034
0.500	0.07	0.066	0.06	0.055	0.05	0.044	0.03	0.018	-0.001	-0.01
0.700	-0.016	-0.021	-0.028	-0.033	-0.038	-0.044	-0.059	-0.071	-0.091	-0.102
1.00	-0.214	-0.218	-0.224	-0.230	-0.236	-0.242	-0.258	-0.27	-0.29	-0.30
1.50	-0.724	-0.728	-0.733	-0.737	-0.741	-0.746	-0.758	-0.769	-0.784	-0.784
1.75	-1.058	-1.062	-1.066	-1.070	-1.074	-1.078	-1.09	-1.096	-1.102	-1.09
2.00	-1.451	-1.455	-1.46	-1.464	-1.467	-1.472	-1.485	-1.488	-1.48	-1.451
2.25	-1.89	-1.895	-1.90	-1.906	-1.910	-1.914	-1.927	-1.926	-1.906	-1.873
2.50	-2.384	-2.388	-2.394	-2.400	-2.405	-2.408	-2.414	-2.414	-2.371	-2.334

occurs close to the weir crest and hence more care has to be taken in the design of the crest of the weir. It was also observed that the rise of the nappe above the sharp crest was more in the case of a high dam than in the case of a low dam.

2.12.3 Weirs With Upstream Face Sloping Downstream :

Similar experiments were conducted in the case of weir with sloping upstream faces and are shown in figures 12,13 & 14 and the x and y co-ordinates for the various velocities of approach are given in tables 4 , 5 & 6 . From the results obtained, it is interesting to note that the height of the nappe above shart crest of the weir decreases as the angle of the weir increases from the vertical, which means that the contraction of the under side of the jet decreases as the slope of the weir is flattened.

2.12.4 Weirs With Up-stream Over Hang And Weirs With Upstream Off-sets & Risers:

Spillways having this shape have been used in many dams. This type has been evolved as a result of its saving of material. The theoretical form of stable overflow dam is a trapezoid as shown in figure 15. With an overflow having a total head H_e, however, the downstream side of the apezoid does not have sufficient width and the nappe would spring free of the dam as shown in figure 15, and fall along a curve extending a maximum distance or outside the trapezoid. To ensure freedom from undesirable vacuum effects, the space between the nappe and the dam should be filled with concrete. Concrete placed in this loeation, however, is not in position to resist most effeciently over turning of the dam. By moving the point corresponding to the weir crest upstream a distance N, the nappe can be brought

TABLE NO. 4

U.S.B.R. COORDINATES FOR LOWER NAPPE FOR DIFFERENT VALUES OF hu/Hs - 1:3 (H:V)WEIR

TABLE NO. 4

	Y/Hs									
hu/Hs → / x/Hs	0.002	0.01	0.02	0.03	0.04	0.05	0.08	0.10	0.14	0.20
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.04	0.042	0.042	0.04	0.039	0.038	0.038	0.035	0.033	0.029	0.018
0.10	0.069	0.067	0.066	0.065	0.064	0.063	0.058	0.056	0.049	0.038
0.20	0.084	0.083	0.079	0.078	0.076	0.075	0.070	0.065	0.056	0.03
0.25	0.084	0.082	0.08	0.078	0.076	0.075	0.068	0.063	0.053	0.024
0.30	0.082	0.078	0.076	0.074	0.072	0.07	0.063	0.058	0.047	0.014
0.35	0.074	0.071	0.069	0.067	0.065	0.063	0.054	0.049	0.038	0.001
0.40	0.065	0.062	0.059	0.057	0.055	0.052	0.043	0.037	0.026	-0.014
0.50	0.038	0.035	0.031	0.029	0.026	0.023	0.012	0.005	-0.006	-0.048
0.70	-0.048	-0.052	-0.056	-0.059	-0.062	-0.067	-0.08	-0.09	-0.101	-0.138
1.00	-0.248	-0.254	-0.259	-0.262	-0.265	-0.269	-0.278	-0.286	-0.295	-0.319
1.50	-0.767	-0.770	-0.772	-0.772	-0.772	-0.771	-0.773	-0.774	-0.775	-0.759
1.75	-1.108	-1.108	-1.106	-1.104	-1.102	-1.102	-1.097	-1.095	-1.086	-1.051
2.0	-1.499	-1.492	-1.487	-1.484	-1.481	-1.478	-1.465	-1.457	-1.442	-1.401
2.5	-2.426	-2.415	-2.403	-2.398	-2.392	-2.387	-2.371	-2.361	-2.339	-2.272

TABLE NO.5

U.S. BUREAU OF RECLAMATION CO-ORDINATES FOR LOWER NAPPE FOR DIFFERENT VALUES OF hu/Hs

2:3 (H : V) WEIR

TABLE NO. 5

hu/Hs → x/Hs	Y/Hs 0.000	0.01	0.02	0.03	0.04	0.05	0.08	0.10	0.140	0.18
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.04	0.029	0.028	0.027	0.027	0.026	0.025	0.023	0.022	0.018	0.014
0.10	0.053	0.052	0.05	0.05	0.049	0.048	0.044	0.042	0.037	0.03
0.20	0.064	0.063	0.062	0.061	0.059	0.058	0.053	0.05	0.044	0.038
0.25	0.061	0.06	0.059	0.058	0.056	0.054	0.049	0.046	0.04	0.034
0.30	0.055	0.054	0.052	0.05	0.048	0.047	0.042	0.038	0.032	0.024
0.35	0.044	0.043	0.042	0.04	0.038	0.037	0.032	0.028	0.021	0.013
0.40	0.033	0.032	0.03	0.028	0.026	0.025	0.019	0.015	0.007	0.000
0.50	0.001	0.000	-0.001	-0.004	-0.006	-0.008	-0.014	-0.018	-0.026	-0.036
0.70	-0.094	-0.095	-0.096	-0.098	-0.100	-0.103	-0.109	0.144	-0.120	-0.127
1.00	-0.297	-0.299	-0.300	-0.302	-0.304	-0.306	-0.311	-0.314	-0.320	-0.325
1.50	-0.814	-0.814	-0.813	-0.812	-0.811	-0.810	-0.808	-0.805	-0.797	-0.790
1.75	-1.148	-1.146	-1.143	-1.141	-1.138	-1.137	-1.130	-1.124	-1.108	-1.086
2.00	-1.525	-1.522	-1.518	-1.515	-1.511	-1.509	-1.500	-1.491	-1.465	-1.426
2.50	-2.447	-2.437	-2.427	-2.418	-2.409	-2.404	-2.379	-2.362	-2.318	-2.269

TABLE NO. 6

U.S.B.R. CO-ORDINATES FOR LOWER NAPPE FOR DIFFERENT VALUES OF hu/Hs - 3:3 WEIR

TABLE NO. 6

	Y/Hs									
hu/Hs → x/Hs	0.002	0.01	0.02	0.03	0.04	0.05	0.08	0.10	0.14	0.18
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.04	0.016	0.016	0.015	0.015	0.015	0.015	0.015	0.014	0.012	0.011
0.10	0.035	0.035	0.034	0.034	0.033	0.033	0.031	0.03	0.028	0.025
0.20	0.044	0.043	0.042	0.042	0.041	0.041	0.039	0.038	0.035	0.029
0.25	0.041	0.04	0.039	0.039	0.038	0.037	0.035	0.034	0.030	0.024
0.30	0.034	0.033	0.032	0.032	0.031	0.03	0.027	0.025	0.022	0.016
0.35	0.022	0.022	0.021	0.021	0.02	0.019	0.016	0.014	0.01	0.004
0.40	0.009	0.009	0.008	0.007	0.006	0.006	0.003	0.000	-0.004	-0.009
0.50	-0.026	-0.027	-0.027	-0.028	-0.029	0.03	-0.033	-0.035	-0.038	-0.042
0.70	-0.126	-0.127	-0.127	-0.128	.0.128	-0.129	-0.131	-0.132	-0.133	-0.137
1.00	-0.336	-0.336	-0.336	-0.336	-0.336	-0.336	-0.337	-0.337	-0.337	-0.337
1.50	-0.849	-0.846	-0.842	-0.841	-0.839	-0.837	-0.831	-0.826	-0.819	-0.805
1.75	-1.176	-1.173	-1.170	-1.168	-1.165	-1.162	-1.153	-1.147	-1.130	-1.101
2.00	-2.552	-1.549	-1.545	-1.541	-1.537	-1.533	-1.518	-1.508	-1.482	-1.443
2.50	-2.46	-2.454	-2.448	-2.437	-2.426	-2.415	-2.384	-2.364	-2.320	-2.244

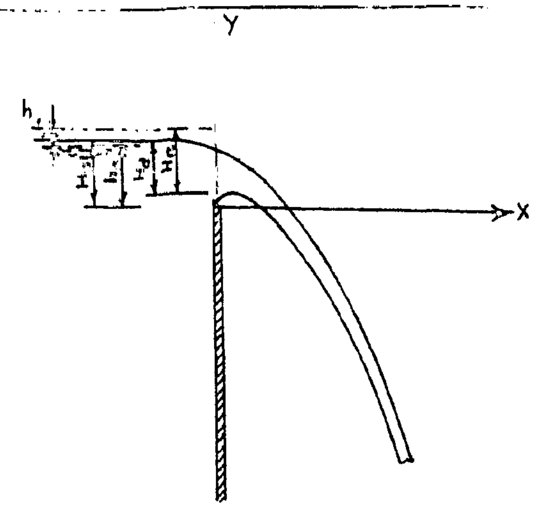


FIG. 11 DEFINITION SKETCH

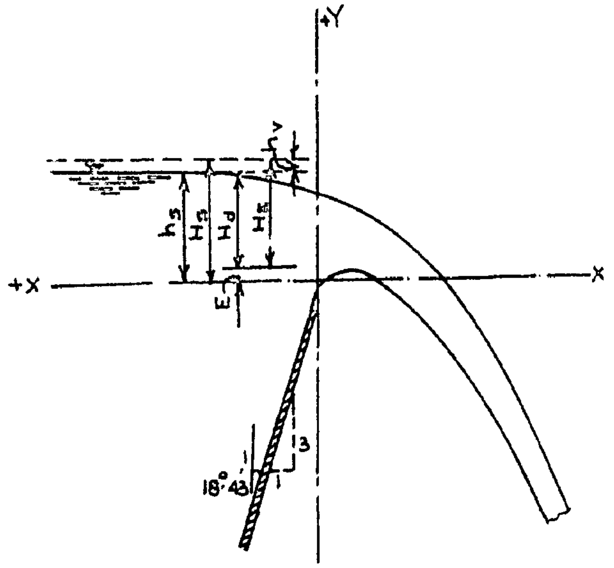


FIG. 12. WEIR INCLINED DOWN STREAM ON 1:3 SLOPE

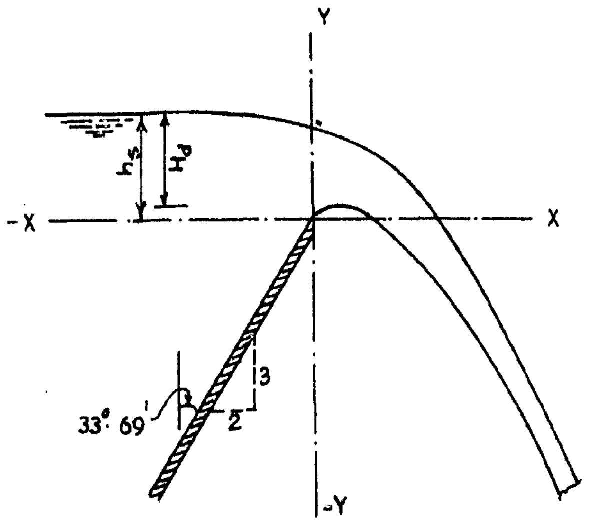


FIG. 13. WEIR INCLINED DOWN STREAM ON 2:3 SLOPE

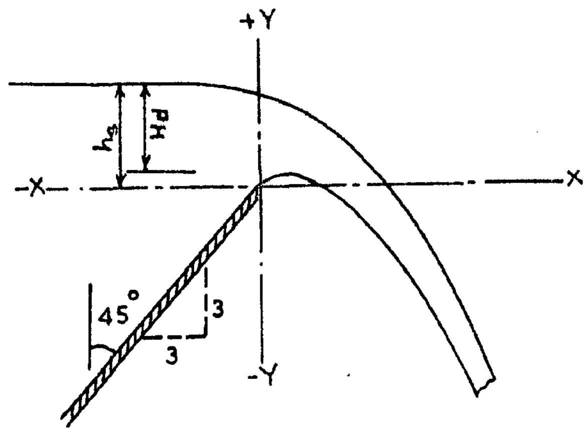


FIG. 14. WEIR INCLINED DOWN STREAM ON 3:3 SLOPE

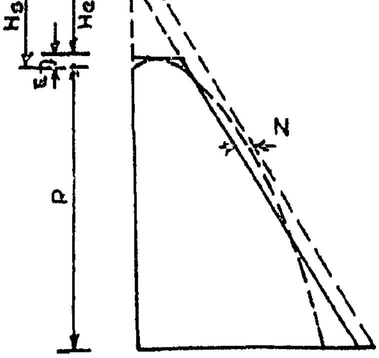


FIG. 15.

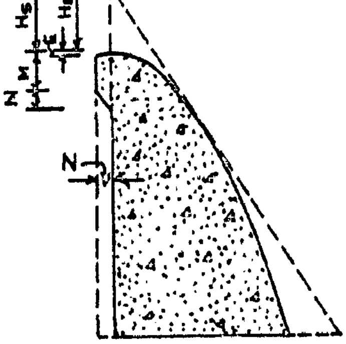


FIG. 16.

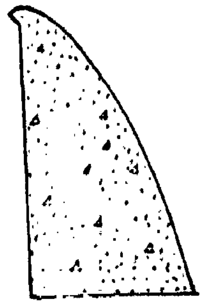


FIG. 17

tangent to the downstream face of the trapezoid. This results in a horizontal off-set in the uppwer portion of the spillway section equal to the distance N. The up-stream end is usually connected to the up-stream face of the dam either by a single inclined surface or by a short section of the vertical face below which is an inclined face as shown in figure 16 & 17.

2.12.4 Tests were conducted on both the types of weirs, namely (1) Up-stream over hang weir as shown in figure 17 and (2) the up-stream offsets weirs with riser as shown in figure 16. In both these cases, the off-set angle was kept at 45°. Even though these weirs resemble each other in shape, their results vary widely. The nappe profiles for over hang weirs obtained after these experiments are given in Table 7. It may also be noted that the values given in table 7, differ considerably from those given for vertical face weir given in table 3. The co-ordinates of the nappe for the off-set weirs are given in table 8. These values are suitable for all values of $\frac{M}{N}$ from 0.5 to 5.0, whefe M is the depth of the off-set.

2.12.5 It was observed during the experiments on the vertical weir and the weirs with the up-stream faces sloping downstream, the lower nappe surfaces were very smooth and glassy in appearance, when head was not excessive nor the depth of approach was shallow. On the other hand nappe surfaces produced by over hang and off-set weirs, was in some cases smooth, but never glassy. Flow conditions for the over hang and off-set weirs were in another respect the opposite of these encountered in vertical weir and weirs sloping downstream. Flow for the vertical and sloping weirs was steadier for deep approach conditions, than for shallow depths. On the

TABLE NO. 7

U.S.B.R. CO-ORDINATES FOR LOWER NAPPE FOR DIFFERENT VALUES OF hu/Hs- OVER HANG WEIRS

TABLE NO. 7

	Y/Hs									
hu/Hs → x/Hs	0.002	0.01	0.02	0.03	0.04	0.05	0.08	0.10	0.12	0.14
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.04	0.084	0.0793	0.0746	0.07	0.0657	0.0614	0.0488	0.0415	0.0353	0.0284
0.10	0.1192	0.1135	0.1078	0.1020	0.0867	0.0914	0.0784	0.0657	0.0570	0.0471
0.20	0.1405	0.1343	0.1281	0.1218	0.1145	0.1072	0.0866	0.0743	0.063	0.0528
0.25	0.1418	0.1351	0.1284	0.1216	0.1132	0.1048	0.0835	0.071	0.059	0.048
0.30	0.1396	0.1317	0.1238	0.1160	0.1077	0.0994	0.077	0.064	0.052	0.040
0.35	0.1325	0.1242	0.1159	0.1075	0.0990	0.0905	0.0684	0.0551	0.0422	0.0289
0.40	0.1225	0.1137	0.1049	0.0962	0.0876	0.079	0.0563	0.0424	0.0296	0.016
0.60	0.049	0.041	0.033	0.024	0.015	0.006	-0.019	-0.033	-0.046	-0.058
0.70	-0.005	-0.013	-0.022	-0.03	-0.039	-0.048	-0.072	-0.086	-0.098	-0.109
1.00	-0.215	-0.224	-0.233	-0.242	-0.25	-0.258	-0.282	-0.293	-0.30	-0.307
1.75	-1.108	-1.108	-1.108	-1.108	-1.106	-1.104	-1.096	-1.089	-1.081	-1.074
2.00	-1.509	-1.505	-1.501	-1.496	-1.492	-1.488	-1.473	-1.468	-1.446	-1.431
2.25	-1.959	-1.953	-1.948	-1.94	-1.931	-1.922	-1.898	-1.879	-1.858	-1.828

TABLE NO. 8

U.S.B.R. CO- ORDINATES OF LOWER NAPPE FOR DIFFERENT VALUES OF hu/Hs - OFF SET WEIRS

TABLE No. 8

	y/Hs.									
hu/Hs → / x/Hs.	0.002	0.01	0.02	0.03	0.04	0.05	0.08	0.10	0.120	0.140
0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.000
0.04	0.05	0.049	0.047	0.045	0.043	0.042	0.039	0.035	0.03	0.025
0.100	0.087	0.085	0.082	0.079	0.077	0.074	0.067	0.06	0.051	0.042
0.200	0.110	0.107	0.103	0.099	0.095	0.091	0.08	0.069	0.058	0.047
0.250	0.114	0.110	0.105	0.100	0.095	0.091	0.077	0.066	0.054	0.042
0.300	0.112	0.107	0.102	0.097	0.092	0.087	0.072	0.06	0.047	0.034
0.350	0.106	0.10	0.095	0.09	0.084	0.078	0.062	0.051	0.038	0.025
0.400	0.096	0.09	0.085	0.08	0.074	0.068	0.051	0.039	0.025	0.011
0.600	0.027	0.022	0.016	0.01	0.003	-0.003	-0.021	-0.033	-0.046	-0.059
0.700	-0.023	-0.029	-0.035	-0.041	-0.048	-0.055	-0.073	-0.085	-0.098	-0.111
1.00	-0.226	-0.233	-0.24	-0.246	-0.253	-0.260	-0.279	-0.291	-0.300	-0.309
1.50	-0.739	-0.743	-0.747	-0.752	-0.757	-0.762	-0.775	-0.782	-0.787	-0.792
1.75	-1.073	-1.075	-1.077	-1.080	-1.083	-1.086	-1.091	-1.09	-1.085	-1.080
2.00	-1.479	-1.477	-1.475	-1.474	-1.470	-1.467	-1.458	-1.452	-1.445	-1.438
2.20	-1.830	-1.826	-1.822	-1.817	-1.812	-1.807	-1,791	-1.780	-1.770	-1.760

contrary, the flow for the over hang and off-set weirs, althou gh never comparing in steadiness with that from its vertical weir or weirs sloping downstream was rougher and more unstable for the deeper approach conditions than with the floor raised in the proximity of the weir crest.

2.12.6 In short, it may be concluded that over flow sections with plane up-stream faces are conducive to stable flow conditions where as those with off-sets and breaks in the up-stream face are not, although many latter suctions are in existence and appear to be giving satisfactory service. However, spillway sections are not recommended with values of $\frac{M}{N}$ less than 0.5 unless the value is zero. For values greater than zero and less than 0.5, flow conditions are extremely unstable.

2.12.7 Even though, for accurate analysis and precise design of spillway overflow sections, the respective tables referred to earlier should be used. But for practical purposes, however, simple equations can be used without essential loss of accuracy. For most conditions, the data can be summarised according to the form shown in figure 18, where profiles is designed as it relates to axes at the apex of the crest (32). That portion upstream from the origin is defined as either a single curve and a tangent or as compound circular curve. The portion downstream is defined by

$$\frac{y}{H_e} = - K_e \left(\frac{x}{H_e} \right)^{N_e} \quad \ldots \; \ldots \; \ldots \quad (9)$$

in which K_e and N_e are constants whose values depend on the upstream inclination and the velocity of approach. Figures 19 &20 gives values of these constants for different conditions.

2.12.8 The approximate profile shapes for a crest with a vertical upstream is shown in figure 21. The profile is constructed in the form of a compounded circular curve with radii expressed in terms of the design head H_d. This definition is simpler than that shown in figure 18, since it avoids the need for solving an exponential equation; further, it is represented in a form easily used by layman for constructing forms or templates For ordinary conditions for the design of small spillways, and where approach height P is equal to or greater than one half the maximum head on the crest, this profile is sufficiently accurate to avoid seriously reduced crest pressures and does not materially affect the efficiency of the crest. When the approach height is less than one half the maximum head on the crest, the profile should be determined from the figures 19 & 20.

2.13. Gate Controlled Ogee Crests :

2.13.1 So far only the shapes of spillway crests without control gates, or when the gates are fully opened in the case of crests with gates have been considered in the profiles described earlier. However, these profiles will not be applicable, when the spillways are in operation under partial gate openings. The jet produced by a partial gate opening may skip over the above profile depending on the type of gate and the conditions of operation. Although the head may be the same for full gate opening as for a partial opening, the lower surfaces of the jets need not correspond, even though no contraction exists at the lower surface in each case. The explanation may be that, the stream lines are horizontal and parallel as they leave the gate for small gate openings, where as for full gate opening the stream lines are neither horizontal nor parallel as they flow over the section, but have components in a downward

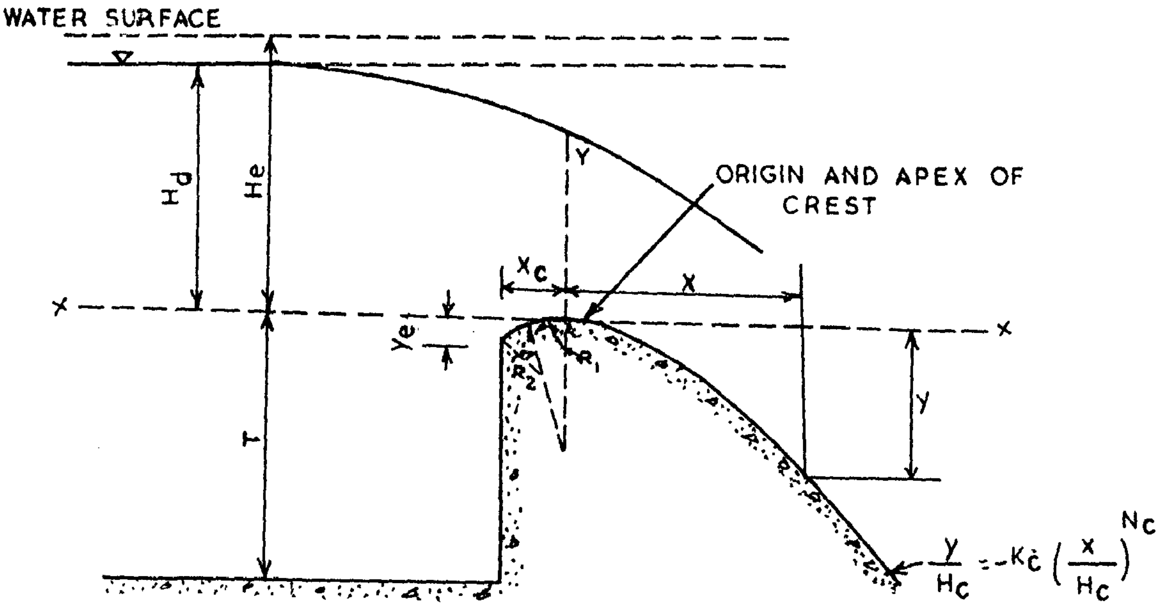
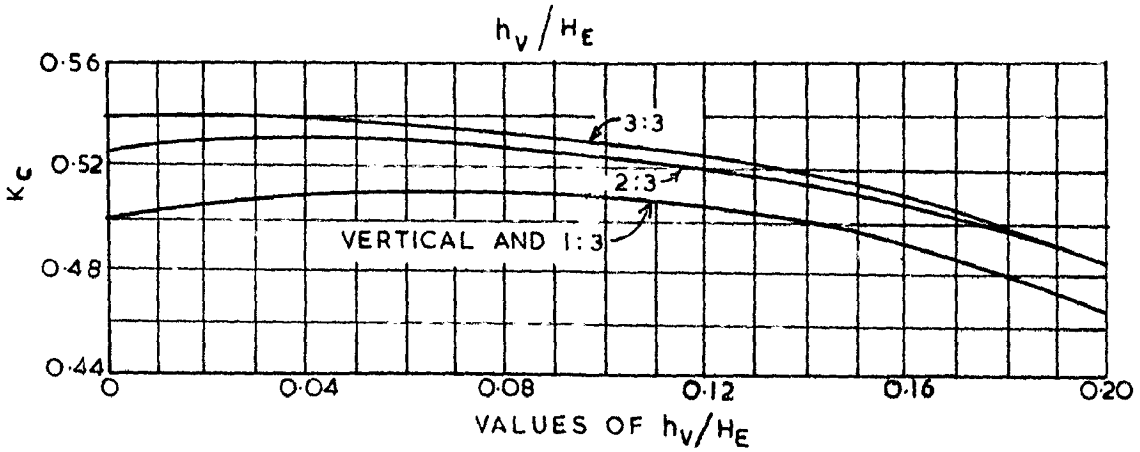


FIG. 18. ELEMENTS OF NAPPE-SHAPE CREST PROFILES



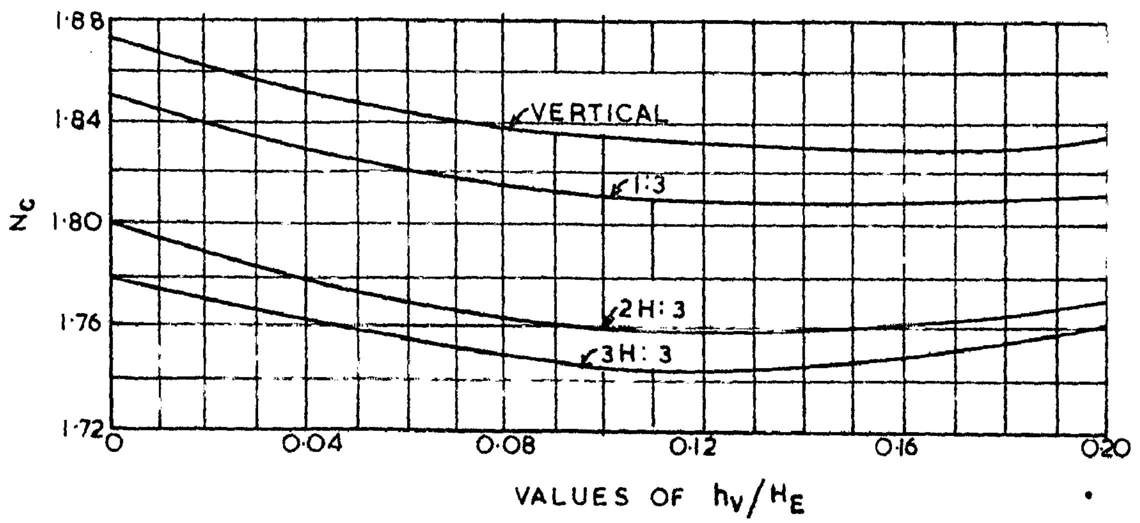


FIG. 19. FACTORS FOR DEFINITION OF NAPPE SHAPED PROFILES — U.S.B.R.

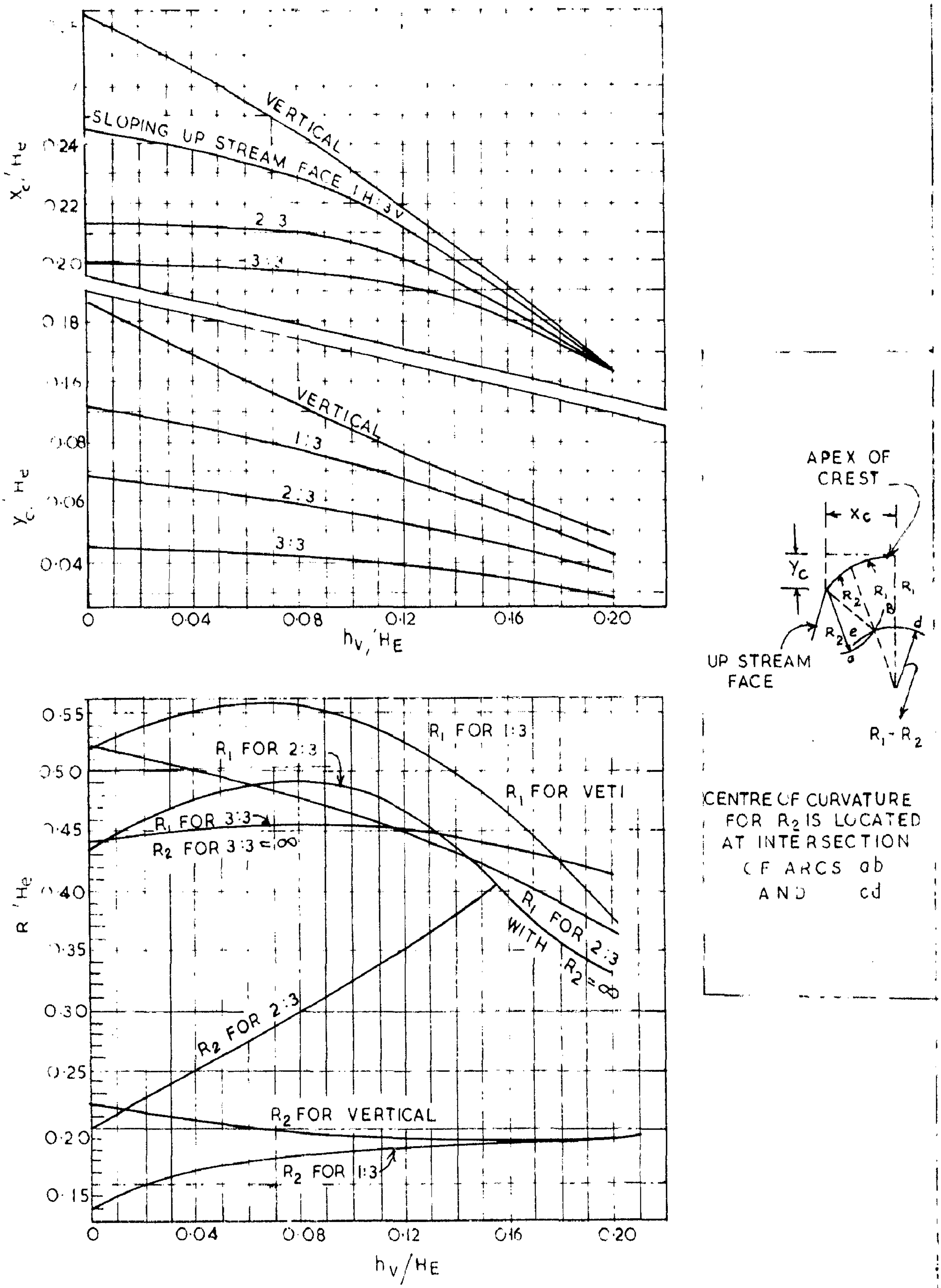


FIG.20 FACTORS FOR DEFINITION OF NAPPE SHAPED CREST PROFILES — U S B R

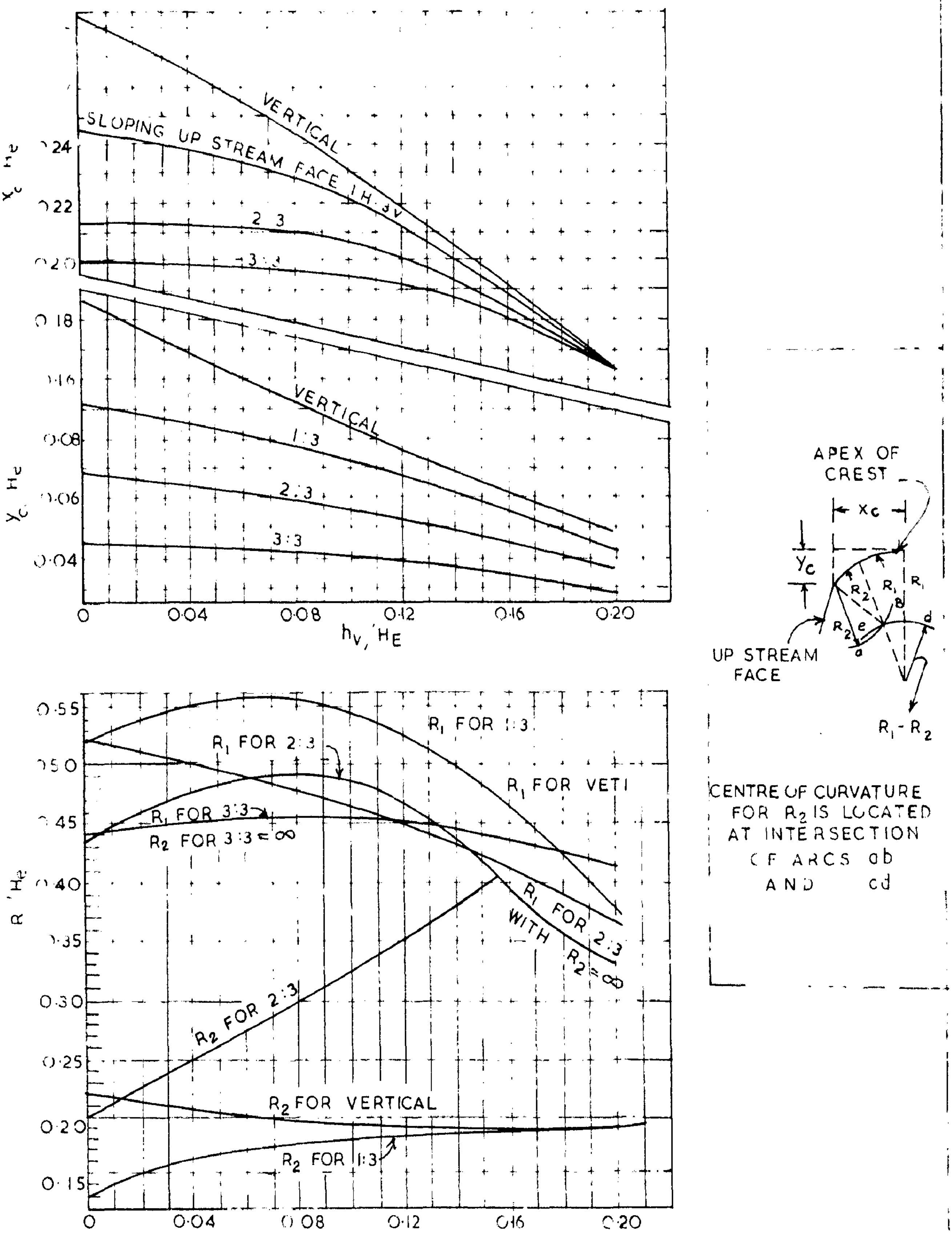


FIG.20 FACTORS FOR DEFINITION OF NAPPE SHAPED CREST PROFILES — USBR

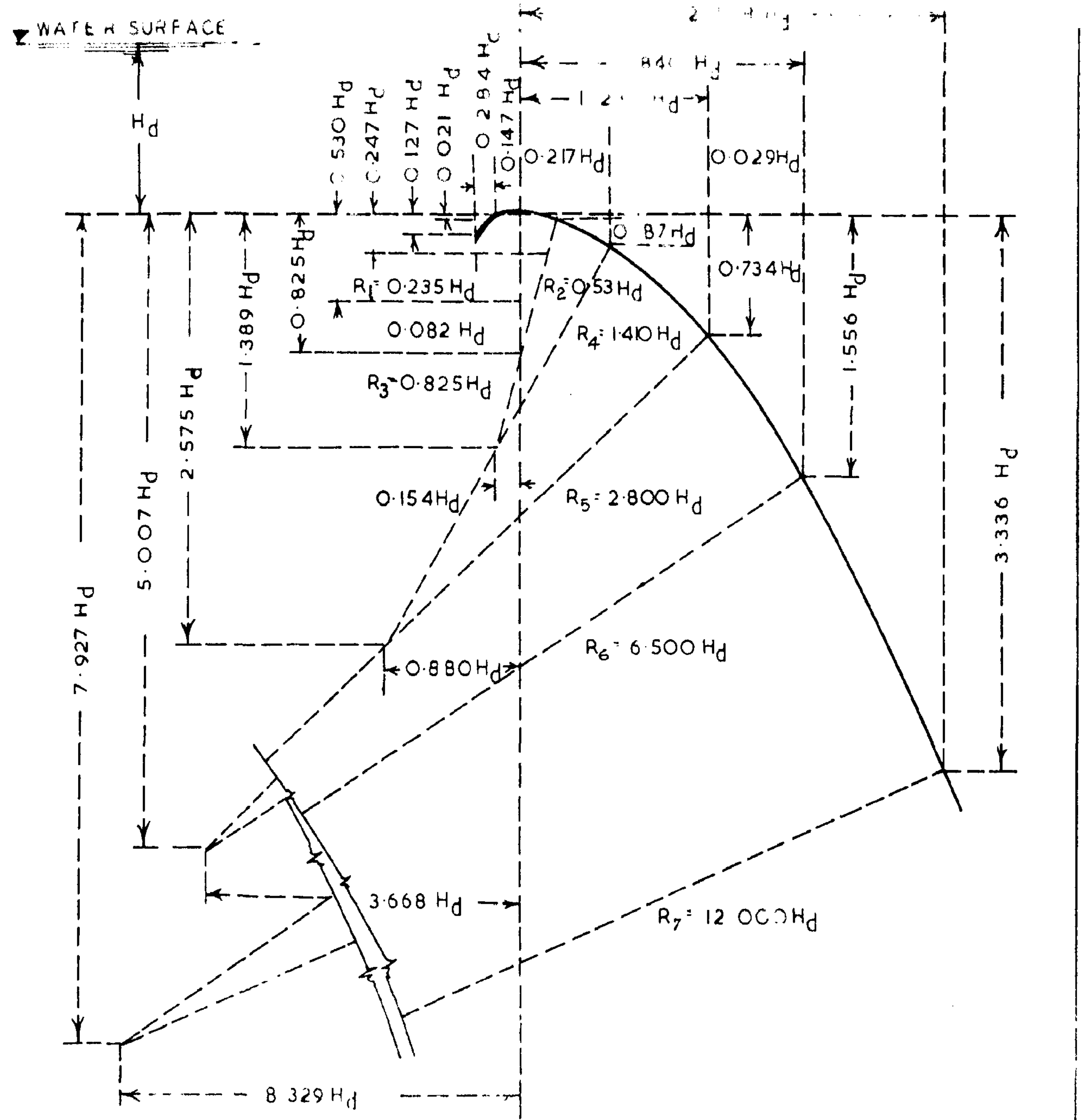


FIG. 21 OGEE CREST SHAPE DEFINED BY COMPOUND CURVES-U.S.B.R.

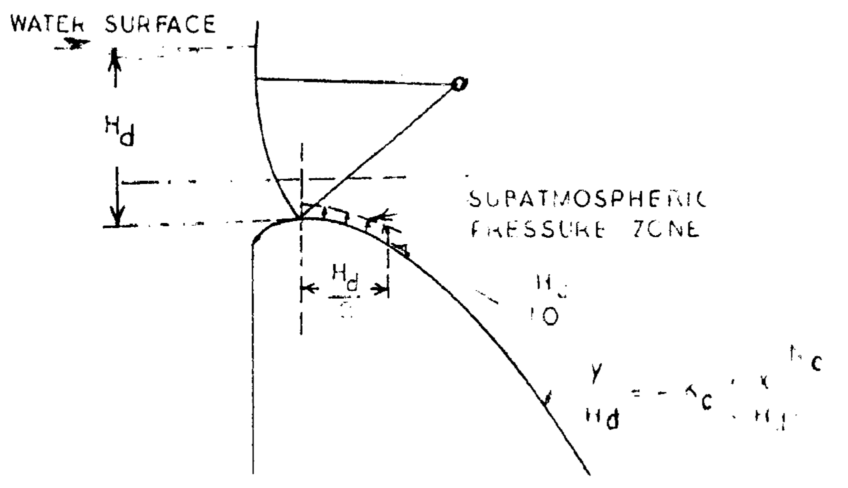


FIG. 22. SUBATMOSPHERIC CREST PRESSURE FOR UNDERSHOT GATE FLOW.

direction [82]. When full head on the gate and with the gate opened, a portion of free discharging trajectory will follow the path of a jet issuing from an orifice. For vertical orifice, the path of the jet can be expressed by the parabolic equation:

$$-y = \frac{x^2}{4\,H_m}$$

where H_m is the head at the centre of the opening. For an orifice inclined at an angle θ from the vertical, the equation will be :

$$-y = x \cdot \tan\theta + \frac{x^2}{4\,H_m \cos\theta}$$

If sub-atmospheric pressures have to be avoided along the crest contact, the shape of the ogee downstream from the gate sill must conform to the trajectory profile. So it has been customary in U.S.Bureau of Reclamation to design the up-stream portion from a gate sill to fit the weir nappe at maximum discharge while the portion of the crest downstream from the gate sill is shaped to fit the theoretical trajectory of the centre line of the jet issuing from a 0.3048 m. (1 foot) gate opening under the maximum head ([81]). The result is a crest flatter than the one designed for maximum discharge alone.

2.13.2 Experiments have shown that when gates are operated with small openings under high heads, negative pressure will occur along the crest in region immediately below the gate, if the ogee shape is thinner than the one described above. Tests showed that this sub-atmospheric pressures would be equal to about one tenth of the design head (figure 22) ([32]) .

2.13.3 The adoption of a trajectory profile rather than a nappe profile downstream from the gate sill will result in wider ogee and reduced discharge efficiency for full gate openings. Where the discharge efficiency is unimportant and where a wider ogee shape is needed for structural stability, the trajectory profile may be adopted to avoid sub-atmospheric zones along the crest. Where the ogee is shaped to the ideal nappe profile for maximum head, U.S.Bureau of Reclamation recommends that the gate sill may be located downstream from the crest. This willprovide an orifice, which is inclined downstream for small gate openings and this will result in a steeper trajectory more nearly conforming to the nappe shaped profile.

2.14 U.S.Army Corps Profile :

2.14.1 On the basis of the U.S.B.R. Data an Boulder Canyon Project (82), U.S.Army Corps of Engineers have also developed spillways for various shapes and approach conditions. Such shapes designated as WES spillways shapes (18) can also be represented in the following form:

$$x^{N_e} = K_e \quad H_d^{N_c - 1} \, y \qquad \ldots \quad \ldots \quad \ldots \qquad (10)$$

as in the case of equation (9)

2.14.2 Standard Crest :

As in the previous case, the slopes of the ogee spillway crest is a function of design head, the height of the dam (which influences the velocity of approach) and the slope of the up-stream face (79) . The design head H_d is usually based on the maximum head resulting from passage of the spillway design flood. However , a lesser design head is sometimes used, in some special cases. Profiles based on equations, for curved surfaces are, in general preferable to compound circular curves to define the shape of the down-stream portion of the crest, since pressure will be more uniform with curves which do not have changes in radii.

2.14.3 The recommended equation for the profile curve down-stream from the crest of a high dam with vertical up-stream face and negligible velocity of approach is :

$$x^{1.85} = 2\ H_d^{0.85}\ y \qquad \ldots \qquad \ldots \qquad \ldots \qquad (11)$$

where x and y are horizontal and vertical distances from the ogee crest as shown in figure (23) and all the dimensions are expressed in the same unit. The shape of the weir up-stream from the crest is fixed with the help of a compound curve as shown in figure 23. But later on the Corps of Engineers made the up-stream profile in the form of an ellipse given by the equation as shown in figure 24.

$$\frac{x^2}{(0.282\ H_d)^2} + \frac{y^2}{(0.15\ H_d)^2} = 1$$

on the basis of analytical and laboratory results, the Corps of Engineers have now recommended to use the following equation for the up-stream profile with the origin at the crest as ,

$$y = 0.724\ \frac{(x + 0.27\ H_d)^{1.85}}{H_d^{0.85}} + 0.126\ H_d$$

$$- 0.4335\ H_d^{0.375}\ (x + 0.27\ H_d)^{0.625}$$

The curve extends (0.27 H_d) up-stream and (0.126 H_d) downstream from the crest point as shown in figure 25. This design is adopted to avoid discontinuity near the upstream face, which may cause an abrupt stimulation of turbulent boundary layer.

2.14.4 Various other experimentors have also given co-ordinates

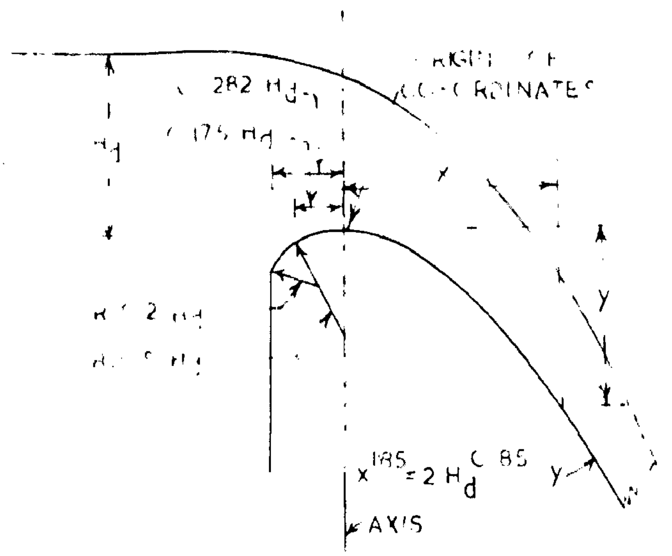


FIG 23 WES SPILLWAY PROFILES

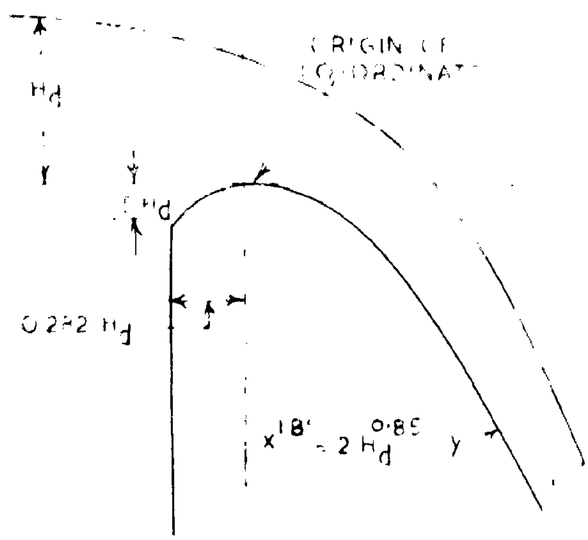


FIG 24 WES SPILLWAY PROFILES

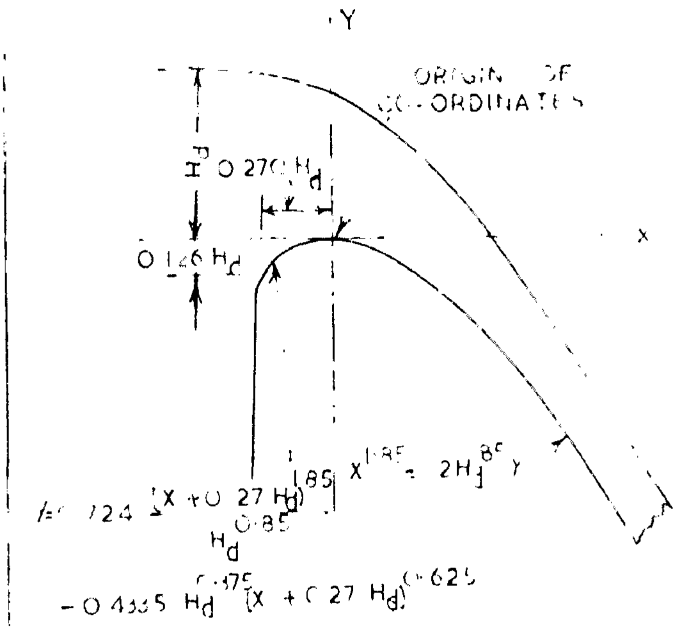


FIG 25 WES SPILLWAY PROFILE

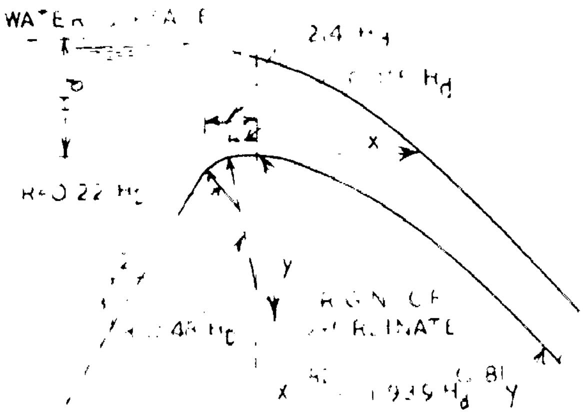


FIG 27 (i)

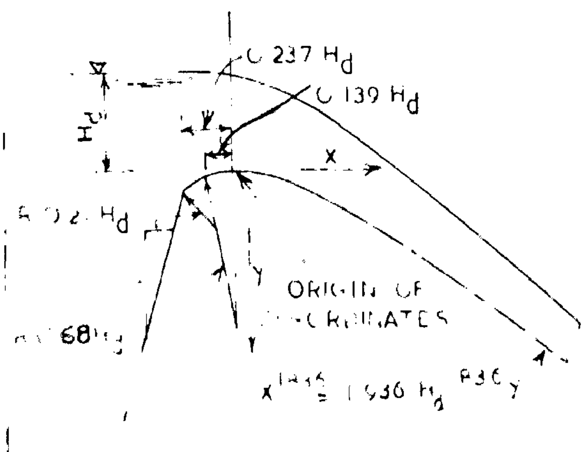


FIG 26

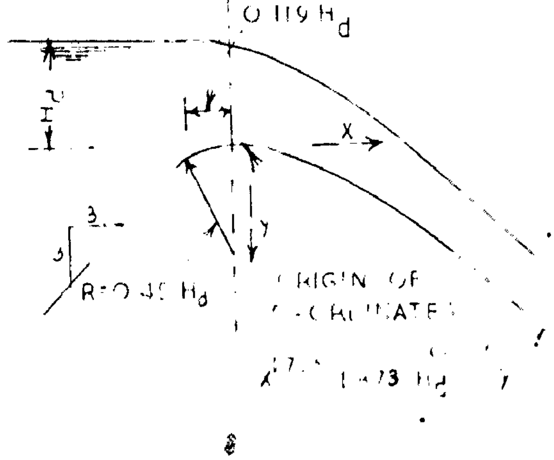


FIG 27 (ii) WES SPILLWAY PROFILE

for the profile up-stream of the crest. Studies conducted at the U.P., I.R.I. indicate that a circular quadrant of 0.282 H_d radius upstream of the crest gives the best results.

2.14.5 In the forgoing, spillway profiles as applicable to vertical upstream faces alone have been mentioned. When the upstream faces are inclined, the values of K_e and N_e for establishing the equation for the downstream side also have to be modified (18). The values of K_e and N_e for various conditions of the up-stream are as follows:

Slope of the Upstream face	K_e	N_e
Vertical	2.0	1.85
3:1	1.936	1.836
3:2	1.939	1.810
3:3	1.873	1.776

The shapes of the up-stream profile etc. in these cases are given in figures, 26,27 & 27b. For intermediate slopes, approximate values of K_e and N_e may be obtained by plotting the above values against corresponding slopes and interpolating from the plot, the required values for any given slope within the plotted range.

2.14.6 The up-stream face of the spillway crest may sometimes be defined to set back, as shown in figure 16. The shape of the crest will not be affected materially by such details, provided the modification begins with at least one half the total head H_e vertically below the origin of the co-ordinates. This is because the vertical velocities are too small below this depth and the corresponding effect in nappe profile is negligible.

2.14.7 The spillway crest shapes shown in figures 25,26,27 & 27b, are not directly applicable to low ogee spillways with an appreciable velocity of approach or with ogee spillways with inclined offsets or recessed up-stream face. For spillways with vertical or $\overset{4}{\cancel{3}}5^{\circ}$ up-stream face slopes and appreciable velocity of approach, the crest shapes may be determined from figures 28 , 29 & 30. Model tests have shown that profiles based on equation (11) are equally suited for to weir heights equal to or greater than one half the maximum head on the crest. Thus the curves on figures 28, 29 & 30 need be used for determining the crest profile of a spillway with vertical up-stream face only, when the weir height is less than one half the maximum head. Weirs with appreciable velocity of approach with up-stream face sloping and vertical, the data available after the studies made by U.S.B.R. (82) have been recommended for use by the U.S. Army Corps of Engineers also. It may also be mentioned here that curves of the U.S.Army Corps of Engineers shown in figure 28,29 , 30 have been developed from the data taken from U.S.B.R. studies (82).

2.14.8 For spillways with vertical or 45° degree up-stream face slopes, figure 28 shows the relative shapes of the lower nappe profile up-stream from the crest centre line for various ratios of hv / H_s . A compound circular curve of two radii reasonably fits the profiles for spillways with vertical upstream faces, while a simple curve satisfactorily fits the profiles for spillways with 45° up-stream faces. Graphs of appropriate radii developed for these conditions are shown in figure 28 in terms of R / H_s for various ratios of hv/Hs. A tabulation of curve data shown in figure 28 has also been developed by U.S.Army Corps of Engineers. For a spillway having

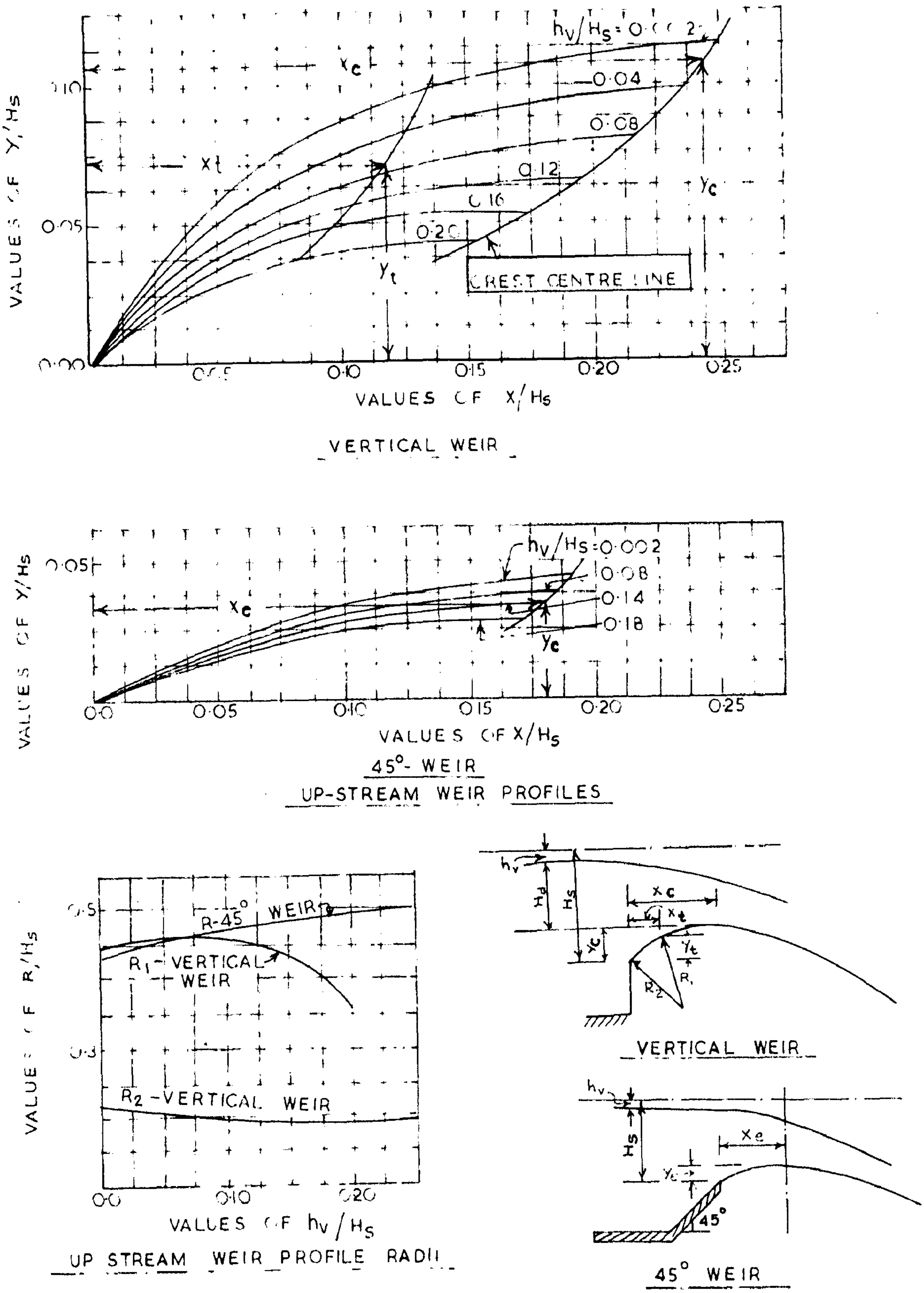


FIG. 28 OGEE SPILLWAY PROFILE DATA-RELATION OF VELOCITY OF APPROACH TO WEIR PROFILE UP-STREAM FROM CREST (WES)

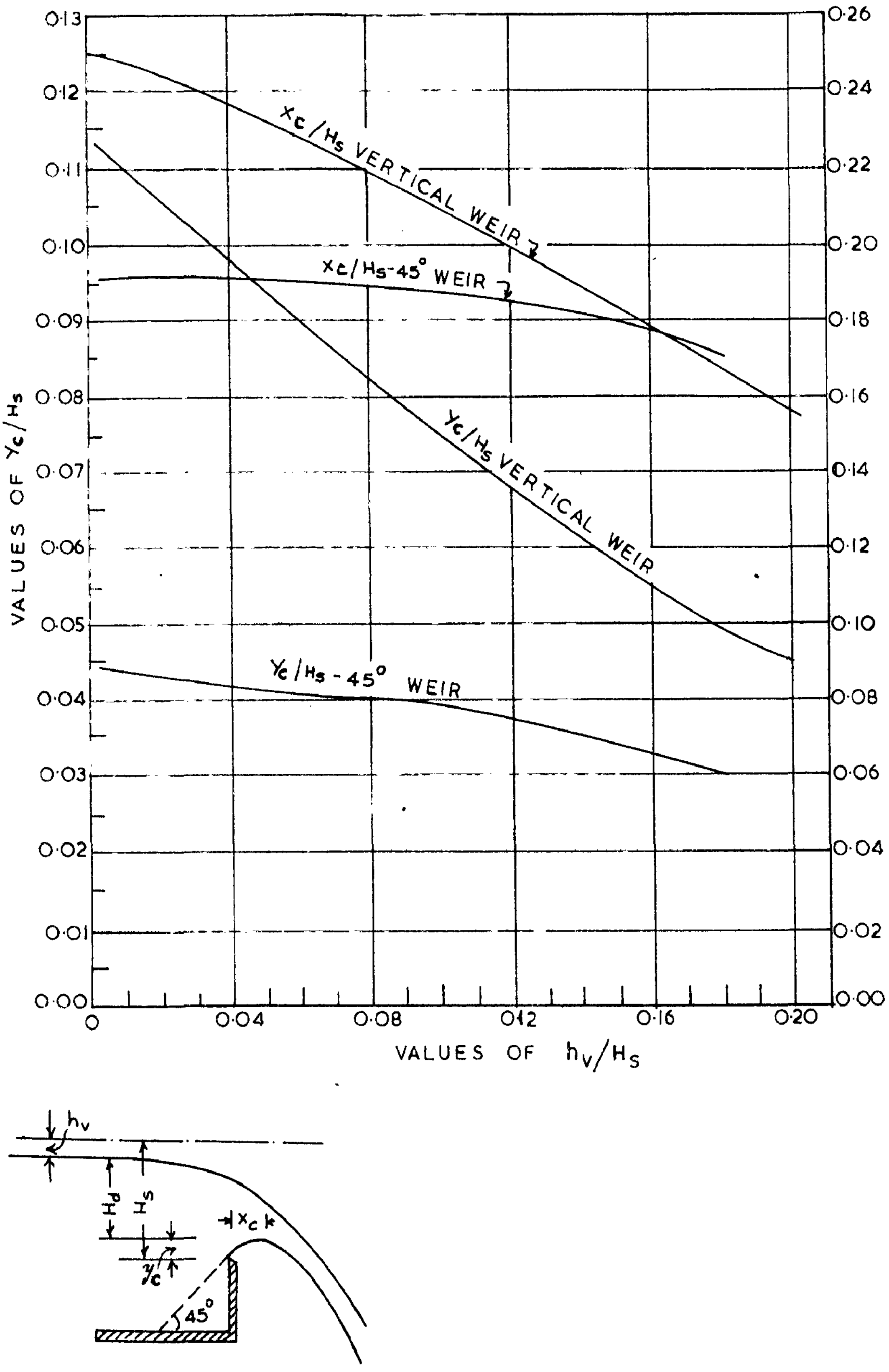


FIG.29. OGEE SPILLWAY PROFILE DATA RELATION OF VELOCITY OF APPROACH TO LOCATION OF WEIR CREST (W.E.S)

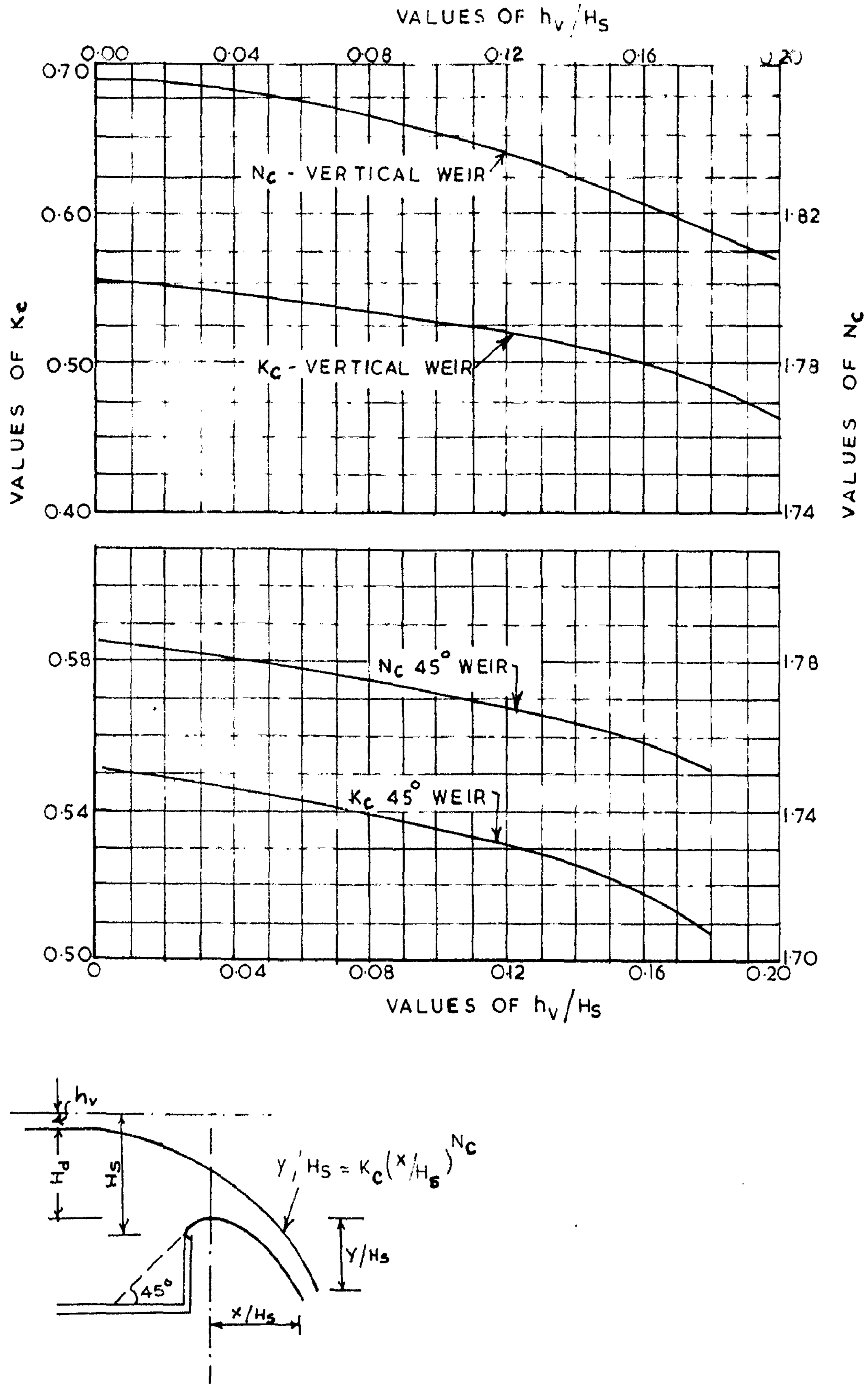


FIG. 30. OGEE SPILLWAY PROFILE DATA RELATION OF VELOCITY OF APPRCACH TO WEIR PROFILE DOWN STREAM FROM CREST

a 45° upstream face and with the values of y_c and x_c obtained from figure 29, the simple circular radius R may be obtained from figure (28). The value of R_D can also be found from the following equation:

$$R = \frac{y_e^2 + x_c^2}{2\, y_e} \qquad \ldots \qquad \ldots \qquad \ldots \qquad (12)$$

For the down stream profile, the equation is similar to equation (10) and the values of the expoments for this portion of the profile can be found out from the curve given in figure 30. Perhaps the method of finding this crest profile can be better amplified by working out a typical problem.

2.14.9 The problem is to find out the crest shape for an ogee spillway with 45° up-stream slope having a head of 9 m. of water over the crest and with an approach velocity of 4 m/sec.

$$h_v = \frac{4^2}{2 \times 9.81} = 0.82 \text{ m.}$$

Assume the height of the nappe crest from the sharp crest of weir y_e = 0.4 m.

$$\therefore \quad Hs = Hd + h_v + y_c = 9.0 + 0.82 + 0.4 = 10.22 \text{ m.}$$

$$\frac{h_v}{Hs} = 0.08$$

from the known value of $\frac{h_v}{Hs}$, the value of $\frac{y_c}{Hs}$ and $\frac{x_e}{Hs}$ can be obtained from figure 29.

$\therefore \quad \frac{y_c}{H_s} = 0.04$, i.e. $y_c = 0.41$ (checks with the value assumed).

Similarily $\frac{x_e}{Hs} = 0.19$ and $x_e = 1.93$

Now making use of the graph given in figure 28, with the value of h_u / H_s = 0.08, R/Hs is found as equal to 0.463.

*values of y_c and x_c. calculated above.

Now checking the value of R obtained above, $\therefore$ R = 4.7 m with the help of equation(12),making use of the $\therefore$ $R = \frac{0.41^2 + 1.93^2}{2 \times 0.41} = 4.85$ m. (which checks the value got earlier).

To determine the value of K_e and N_c applicable to the curve downstream from the crest, referring to the curve shown in figure 30 for h_u / Hs = 0.08 , it will be found that K_e = 0.54 and N_e = 1.775. Now substituting these values the equation of the profile is;

$$y = \frac{0.54}{(10.22)^{0.775}} x^{1.775} = 0.25 \; x^{1.775}$$

3.0 CREST PRESSURES

3.1 The theoretical pressure along the face of an ogee spillway crest approaches atmospheric pressure at the head for which the crest is designed. For heads higher than the design head, sub-atmospheric pressures are obtained along the spillway crest. Experiments have shown that heads upto several times the design head may be passed over an ogee spillway without seperation of the nappe from the crest and that tolerable sub-atmospheric pressure occurs, if the maximum head is limited 4/3 times the design head (64). Experiments have also shown that excessive sub-atmospheric pressures do not occur when crest gates, with sills located 1.5 m to 3 m, downstream from the

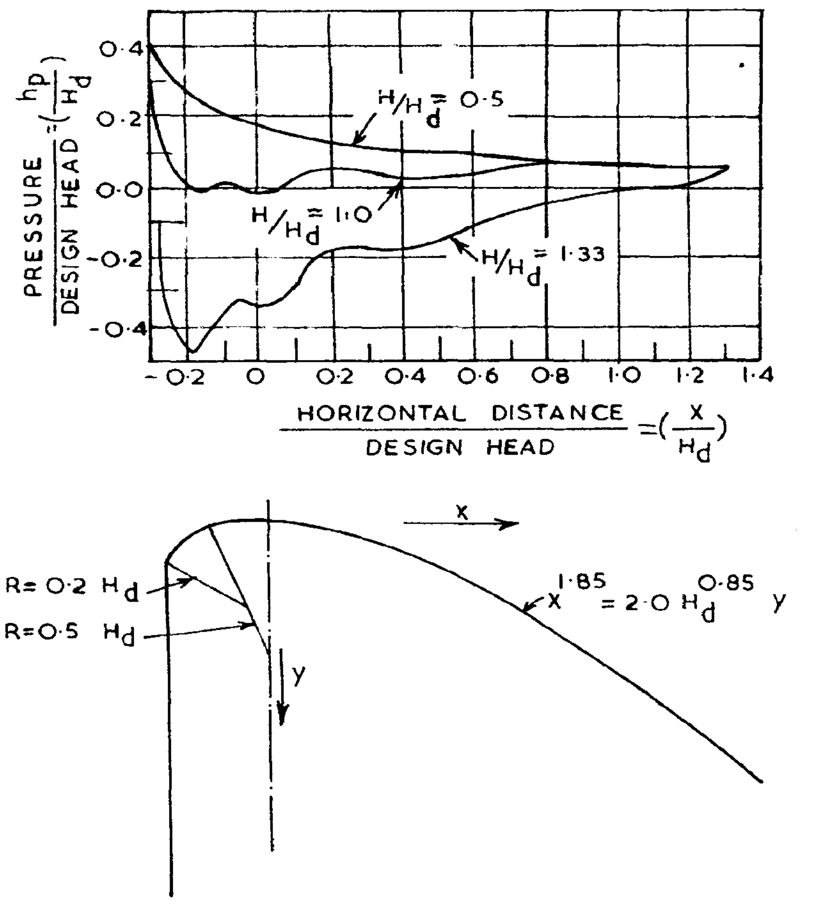


FIG. 31. CREST PRESSURES ON HIGH OVER FLOW SPILLWAYS(NO PIERS)

(AFTER U.S ARMY—WES)

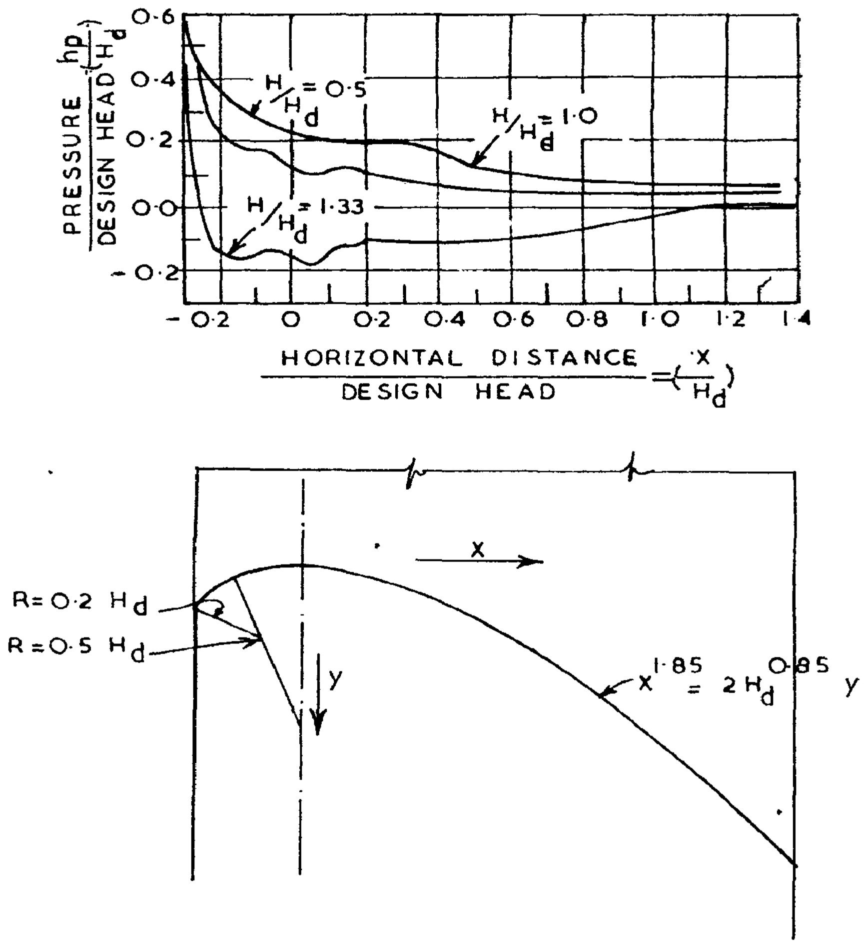
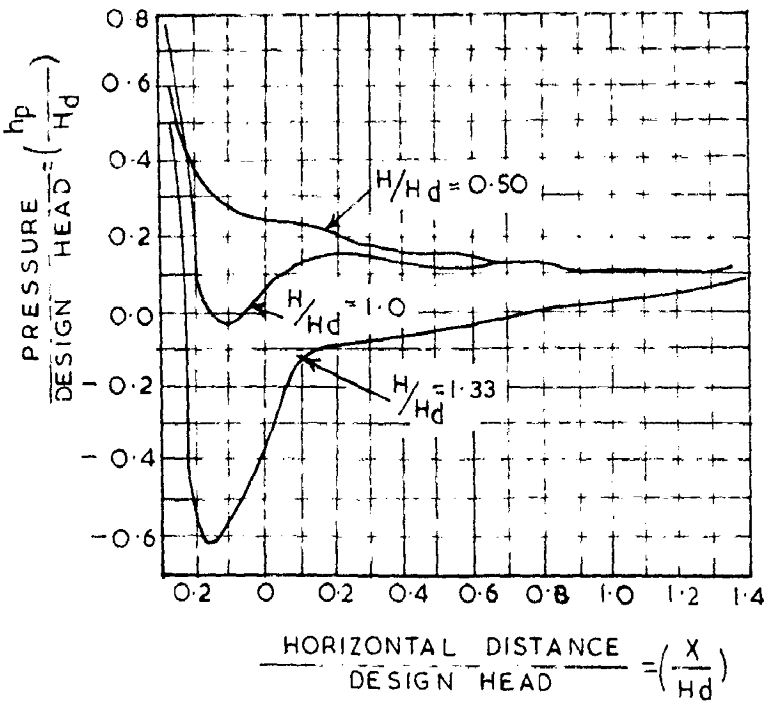


FIG.32. CREST PRESSURES ON HIGH OVER FLOW SPILLWAYS (ALONG CENTRE LINE OF PIER BAY)

(AFTER U.S. ARMY—WES)



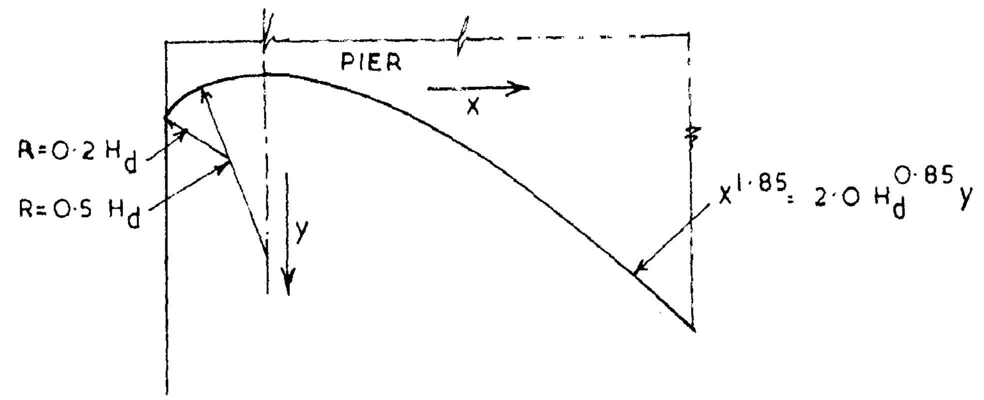


FIG.33 CREST PRESSURES OF HIGH OVER FLOW SPILLWAYS (ALONG PIER)

(AFTER U.S. ARMY-WES)

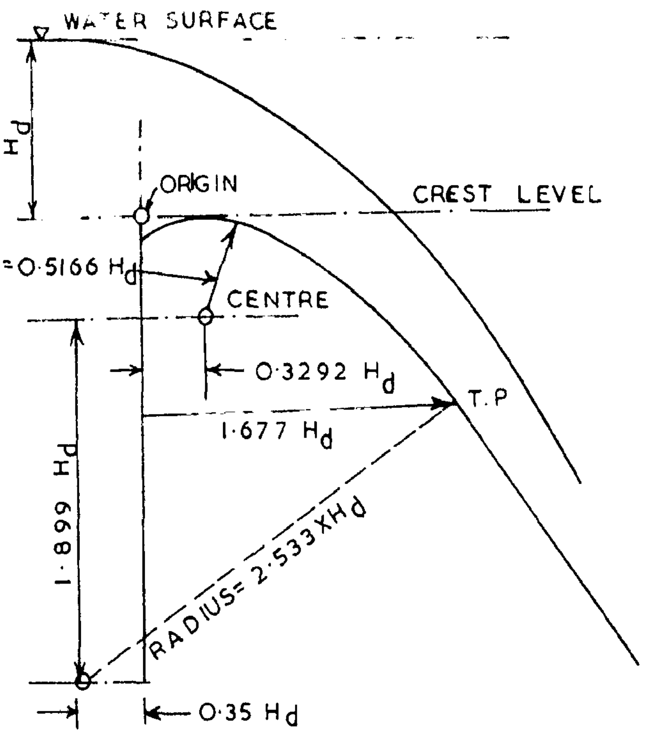


FIG 3·4 TWO CENTRED WEIR CREST

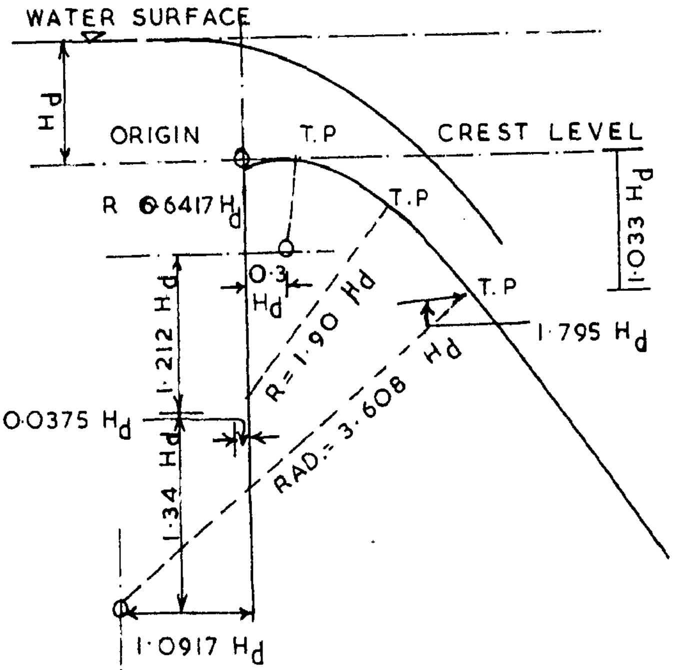


FIG.35. THREE CENTRED WEIR CREST

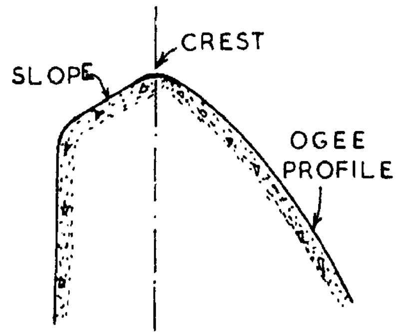


FIG.36. DEFINITION SKETCH

crest, are operated with small openings under high head. Therefore the U.S.Army Corps of Engineers do not recommend the necessity to modity the standard crest for spillways with crest gates. The magnitude of the sub-atmospheric pressure may be determined from the pressure gradients for unit head given in figures 31,32 & 33 which were based on tests conducted at the Waterways Experiment Station.

3.2 Bonnyman and Heefe are of the opinion that when the spillway is to operate under a high head, its profile must be carefully formed to co-ordinates given in any of the standard form(8) Where, however, the head is less than 1.5 m, they suggest alternate geometrical profiles given in figures 34 & 35. All the same, the wisdom of resorting to this emperical profiles, which do not seem to have the backing of, experimental or analytical studies is questionable. Also, a compound curve of this nature does not make the work of the construction engineer easier either.

3.3 In addition to the spillway shapes mentioned above, the type shown in figure: 36, is very commonly used in Europe (64). This type is also used, when drum gates are used at the crest.

3.4 Even though spillway profiles developed by many authors have been briefly discussed above most of them are not being used now a days for some reason or other. Of all these, the spillway curve developed by U.S.B.R. and U.S.Army Corps are being widely used in many countries. In India, the U.S.Army Corps' profiles appear to be the most popular.

CHAPTER - 3

CAVITATION PHENOMENON

1.0 INTRODUCTION

The possibility of cavitation damage was first recognised by Euler in 1754 in connection with the theory of hydraulic machines. But it is only in 1894, that Reynolds first initiated experimental work on this phenomenon of cavitation. R.E.Froude mentioned this phenomenon in 1895, mainly pertaining to ships' propellers. Later on, serious interest on this phenomenon of cavitation arose due to the failure of marine propellers in a single Atlantic crossing and this necessitated the ships to be towed from main sea to the port. In U.S.A. at the very first operation of the spillway tunnel of Hoovar Dam , a huge cavity measuring about 38 m x 14 m deep was formed. Similarly the spillway face of the Gand Coule dam got badly eroded at its first operation and this aroused considerable interest among the American Scientists and Engineers. In India, severe damage to the 5.47 m. dia . volute syphons at Hirbha sagar Jog Power House aroused considerable interest and controversy regarding the damage and whether it was due to erosion or cavitation. After World war II, there is rapid development in the design and construction of very high head hydraulic structures like spillways, gates, values etc., very high speed torpedoes, pumps, turbines etc. with their resulting higher velocities. These have posed serious problems of cavitation.

2.0 CAVITATION THEORIES AND OCCURANCE

The generally accepted word for the destruction and the subsequent erosion of the materials by cavitation action is

" pitting". In brief the cavity occurs in the liquid and pitting in the solid boundary.

2.1 Definition :

2.1.1 Cavitation may be defined as the formation and collapse of cavities in a stream of flowing liquids which results from pressure changes within the stream caused by changes in the velocity of flow. (25)

2.2 Occurrence Of Cavitation :

2.2.1 Cavitation first occurs in a liquid when the equilibrium of small undissolved gas volumes in the fluid becomes unbalanced (36). This happens when the absolute pressure of a fluid flow falls to (or close to) the vapour pressure of the liquid. From Bernoulli's equation it may be seen that pressure in a fluid flow depends on the velocity and position of the flow. Therefore, cavitation is more likely to appear at the top of a conduit than at the bottom. Similarly the higher setting of an hydraulic turbine above tail water will increase the tendency for cavitation.

2.2.2 The production of pressure reduction by increase of the velocity is familiar to engineer and usually results from constriction of a passage as in the case of a venturi - meter or nozzle. Therefore the tendency for cavitation to occur in regions of high velocity is quite obvious.

2.2.3 Apart from high elevation and high velocity there is another factor contributing to pressure reduction and thereby causing cavitation to occur. (4) This is flow curvature as outlined in figure 1, point A denoting the areas of low

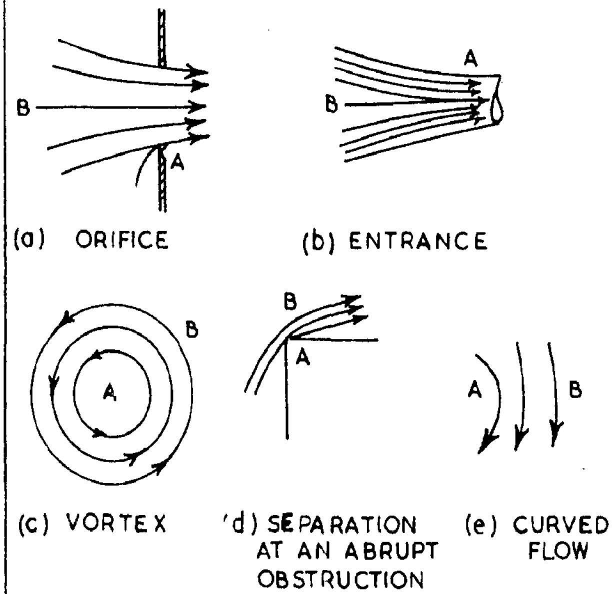


FIG.1 CAVITATION - TYPES

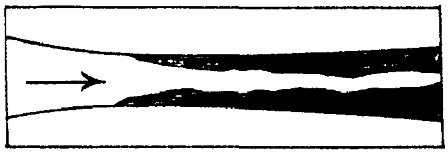


FIG.2. CAVITATION IN A VENTURI SECTION AFTER FOTTINGER, SHEET CAVITATION

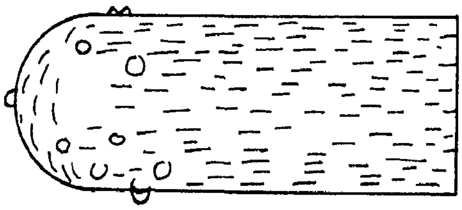


FIG.3. CAVITATION ON BODY OF REVOLUTION

BUBBLE CAVITATION

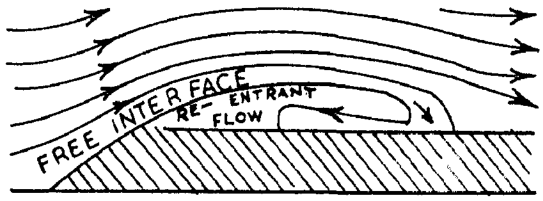


FIG.4. FIXED CAVITY FLOW

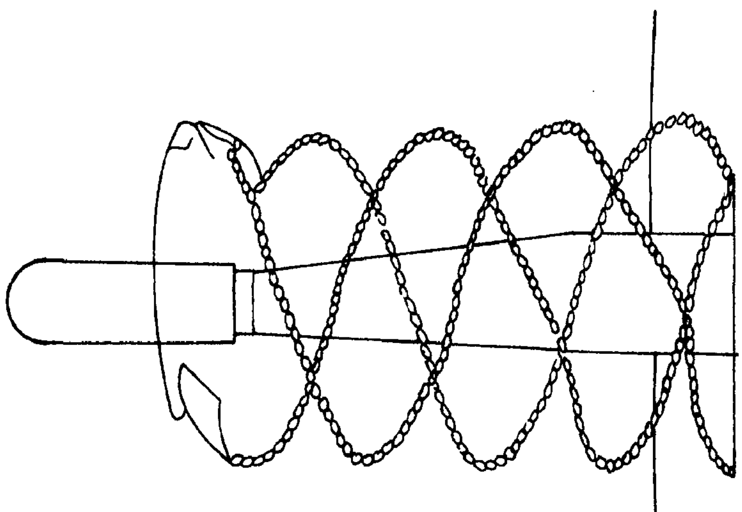


FIG.5. CAVITATION IN THE TIP VORTICES OF A MODEL PROPELLER

pressure and point D, the areas of high pressure. Cavitation caused by the pressure reduction at local flow curvatures is probably more prevalent in engineering practice than in any other field. These flow curvatures may exist on a boundary surface of a relatively easy curvature, such as the bell mouth of a conduit entrance or on the spillway surface of a dam, on the blades of a turbine or propeller or at the abrupt corners which produce flow separation such as gate slots, partly opened valves or offsets in a poor alignment etc. Further more, localised regions of high velocity and low pressure exist in the sharply curved flow at the centres of vortices and eddies, and produce an irregular and unpredictable type of cavitation.

The cavitation mechanism is composed of three parts:

i) Formation of the cavity.

ii) Its travel

iii) Its final collapse.

2.3 Formation Of The Cavity:

2.3.1 After these small undissolved gas volumes in a liquid become unbalanced, these stable nuclei then grow rapidly to form large visible holes or cavities in the fluid. The difference between the growth of cavities in cavitation and boiling is simply the difference between causing cavity growth by decreasing pressure on outside of the cavity (cavitation) and increasing the pressure on the inside of the cavity (boiling).

2.3.2 Although the process of cavity formation is not known quantitatively, a loss of equilibrium may be safely assumed except in the case of cavity at the centre of the fixed vortex.

As a cavity forms and rapidly increases in size vapourisation of the liquid will occur and the gases dissolved in the liquid come out of solution into the cavity. However, due to the turbulence that always accompanies cavitation the speed of the vapourisation and the release of the dissolved gases is quite unpredictable. Little is known of the process except that it occurs with great rapidity.

2.4 Motion Of The Cavity:

2.4.1 It has been observed that the cavity proceeds from its point of formation to its point of collapse with a speed less than that of the liquid. However, in the case of free bubbles and small eddies having cavities at their centres, the speed of the cavity is probably about equal to that of the liquid.

2.5 Collapse Of The Cavity:

2.5.1 Destruction of the cavity begins , when the cavity is in motion and results from the up-stream face of the cavity moving more rapidly than the down - stream face. If the boundary layer prior to the inception of cavitation is unseperated, it is usually observed that at inception, individual cavities enter the low pressure region, become unstable, grow rapidly and collapse, when they move into regions of sufficiently high pressure. The important observation is that each cavity maintains its individual identity through out its process. A preliminary approach to the collapse mechanism was obtained by Rayleigh, by considering the idealised case of collapse of a spherical cavity in a mass of fluid at rest. It was shown experimentally that enormous pressure may be developed locally when water rushes into the void space containing only water vapour. At first this bubble collapse theory was accepted as a complete explanation for the production of intense local pressures and consequent destruction. Later on it was shown by tests that small water jets at high speed produced destruction of the material on solid surfaces.

2.5.2 So it now appears that the intense local pressures developed at the point of collapse are probably due to the combination of these two actions. As the collapse of the cavity occurs at a boundary, ~~draphts~~ droplets may be shot at the solid surface with speeds sufficient to produce deformation and eventual destruction, or irregularities in the cavity walls may produce intense local pressures, when the liquid strikes the solid material at the high speeds produced at collapse. Although the collapse of the cavity is the main cause of impact between liquid masses and solid boundaries, the exact details of the impact mechanism may be quite different in successive collapses.

2.5.3 The principal factors that influence the growth and collapse rate of cavitation bubbles are enumerated below:

1. Surrounding fluid pressure.
2. Surface tension.
3. Initial mass of gas in the bubble.
4. Diffusion of gas into and out of the bubble.
5. The liquid vapour pressure.
6. Heat transfer through bubble walls.

2.5.4 The bubble growth actually continues even after the ambient fluid pressure exceeds the vapour pressure because of the inertia of the fluid. When the pressure tending to retard the bubble expansion is applied sufficiently long to over come the fluid inertia, the expansion ceases, and collapse begins.

The rate of collapse of the bubble depends on the six factors mentioned previously. Higher rates of collapse are associated with (a) high surrounding fluid pressure (b) high surface tension (c) low gas content either initial or due to diffusion (d) low vapour pressure (e) sufficiently high heat transfer to condense the bubble vapour at a rate that does not allow the vapour to act as a permanent gas. It follows that those factors, which increase the collapse rate also increase the intensity of the high pressure produced by the extreme change in momentum, when the fluid moving toward the centre of the bubble is finally brought to zero velocity. It is this intense pressure which produces the noise and boundary damage associated with cavitation. This energy stored in compression of the cavity gas is returned to a kinetic form by re-expansion of the cavity, so that the bubble may cycle through a series of growth and collapse phases before the motion is finally damped. Such rebounding bubbles were observed by Knapp and Hollander of the California Institute of Technology , with the aid of high speed photography (37). Motion pictures taken at 20,000 frames /sec. revealed the following life cycle (Fig. 8). The tunnel velocity was of the order 12.20 m/sec.

1. Formation and growth from first appearance to maximum diameter and time lapse of this is 0.0025 sec.
2. First collapse from maximum dia to first dis-appearance is 0.0075 sec.
3. From rebound from first appearance to second maximum diameter 0.0016 sec.
4. Second collapse from second maximum and second dis-appearance 0.0003 sec.
5. Second rebound to third maximum diameter 0.0004 sec.
6. Third collapse.
7. Final rebound, collapse and dis-appearance.

2.5.6 Actually the bubbles do not completely disappear at each stage ; they shrink in size, grow again and shrink, and finally dis-appear. The life history has been traced upto 5 cycles and after that the cavities become so small, they are not photographed by the high speed camera. Even though the velocity of the water in the tunnel was kept constant, the velocity of the bubble in the tunnel was not constant, but varied with the bubble along the body. The maximum diameter of the bubble was 76 mm. The life of the bubble from instant it was large enough to be detected until the completion of its first collapse was about 0.0025 sec. Formation required about 3/4th of this time and 1/4th time was required for the collapse.

2.5.7 Experiments conducted in venturi set up have revealed damage in two to three regions of reducing intensity on the down - stream side of the cavitating body, showing that bubbles grow, disappear, grow again and disappear a number of times. For some time the rebound phenomenon was thought to be characteristic of all cavitation. But later on it was proved that rebound depends on the air content (28) Cavities formed in water of low air content do not rebound, while those with large amount of gas not only rebound, but oscillate also.

An important fact that has prevented theories of spherical bubble collapse from accurately predicting the history of cavity collapse near its terminal stage is that the cavities become completely non-spherical, as shown in Figure 6. In this figure the collapse history of the cavity, as obtained by Ellis, is shown (21). His experiments indicated collapse pressures of 7×10^3 kg /sq.cm. Recently Naude and Ellis (29) conducted a theoretical analysis for the collapse of a cavity that is initially attached to a boundary as shown in figure 7.

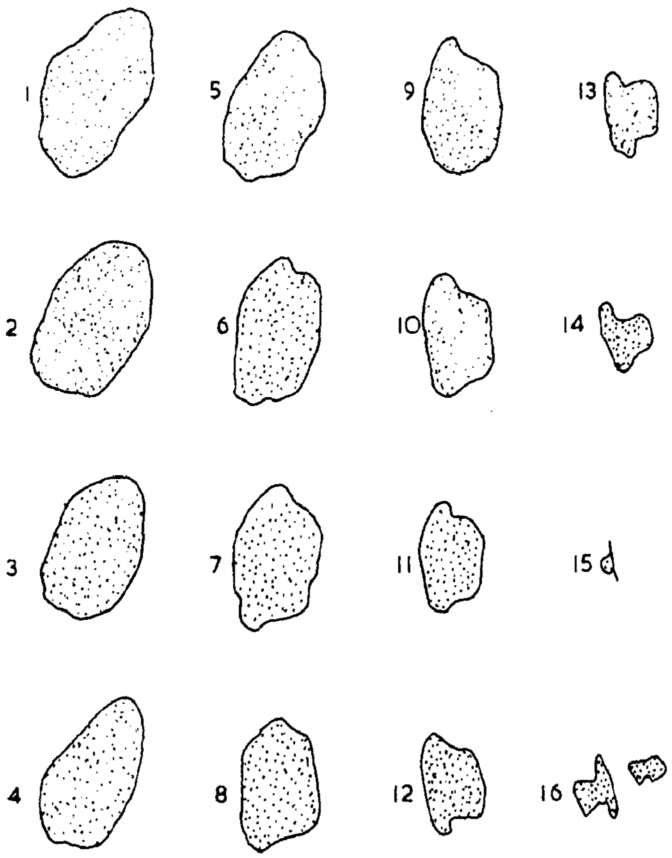


FIG. 6. HISTORY OF A BUBBLE COLLAPSE AWAY FROM THE BOUNDARY

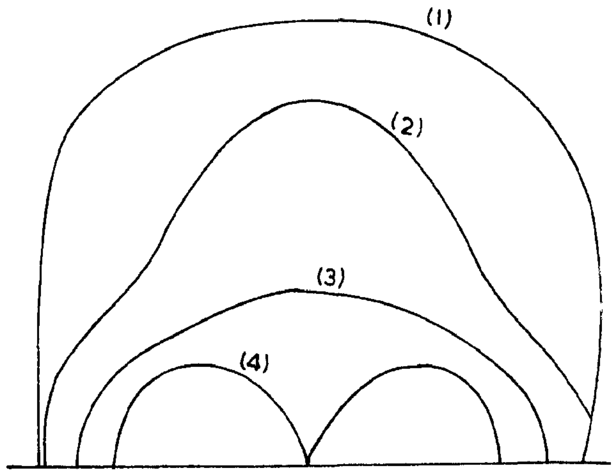


FIG. 7. THEORETICAL SHAPE OF CAVITY COLLAPSING IN CONTACT WITH BOUNDARY

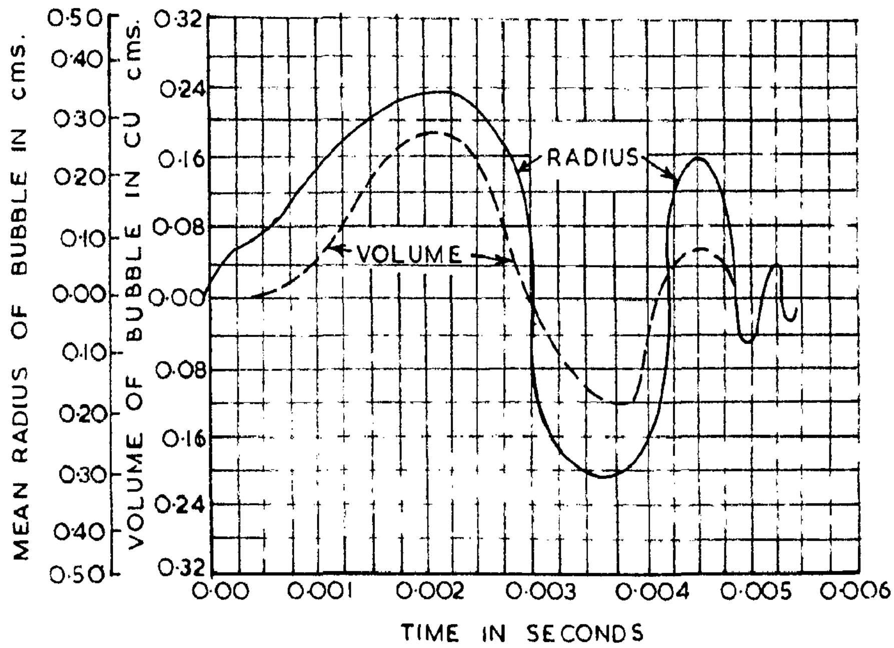


FIG. 8. TIME HISTORY OF LIFE OF BUBBLES

2.5.8 From the random paths that cavities follow before they collapse, it is evident that scattering of the collapse over a sometimes sizable region may be expected. Since this region is three dimensional, however, it appears likely that most cavities will collapse within the free flow and out of contact with the confining boundary surface. The question materially arises whether cavities must contact the boundary surface as they collapse in order to damage this surface. As cavities, completely surrounded by liquid, collapse in this flow, compression waves will be sent out in all directions and will strike the solid boundaries confining the flow. Whether collapse of cavities in the flow can destroy boundary surfaces of the action of the pressure waves is still open to question, ~~but pressure waves is still open to question,~~ but pressure waves may endanger a structure by setting up forced vibration. The pressure wave travelling up-stream into the low pressure region also appears to be instrumental in detaching the succeeding cavity.

2.6 Seperated Flows - Fixed Cavities :

2.6.1 Unseperated flows have their regions of lowest pressure near the boundary and the fluid continuously passes through this zone; that is, individual fluid particles do not remain in fixed position with respect to the body except for the molecular layer immediately adjacent to the boundary. It is for this reason that individual nuclie are subjected to only short transient pressure reductions as they pass by the body ; almost immediately they are returned to high pressure and thus are limited in their growth. These individual bubbles may never be stationary with respect to the boundary. On the other hand, if the boundary layer is seperated prior to the onset of cavitation, or if

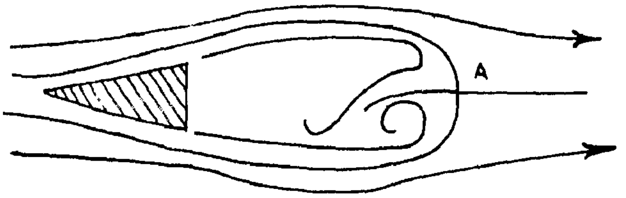


FIG. 9 SUPER CAVITATION

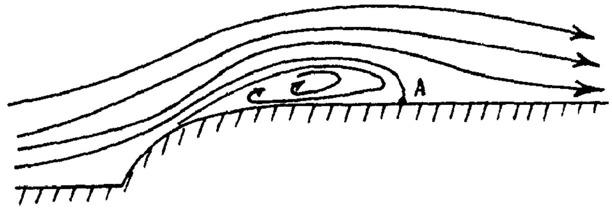


FIG. 10. FIXED CAVITY WITH STAGNATION POINT ON BOUNDARY

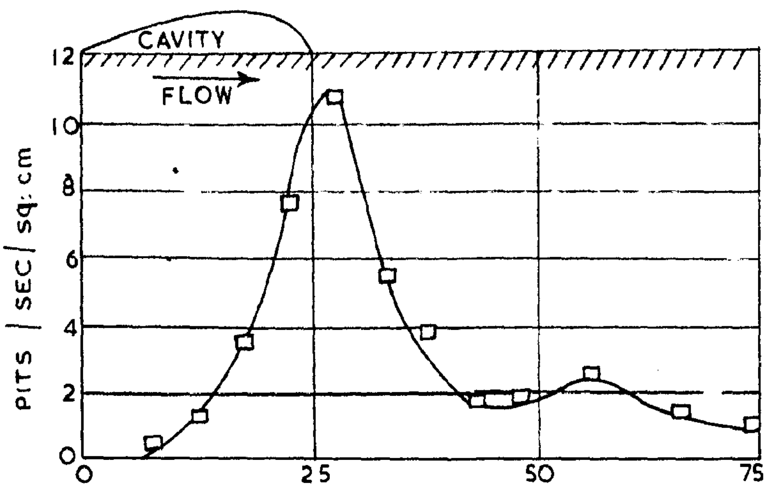
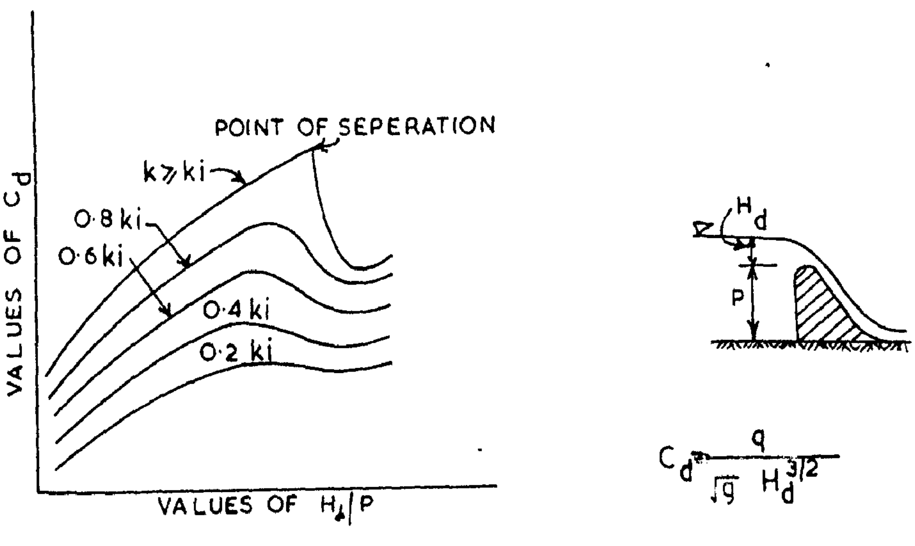


FIG. 11 DISTANCE DOWNSTREAM FROM NOSE TANGENCY-mm
PITTING DISTRIBUTION DIAGRAM



$$C_d = \frac{q}{\sqrt{g}\ H_d^{3/2}}$$

FIG. 12. SCHEMATIC REPRESENTATION OF CAVITATION EFFECT ON THE DISCHARGE CO-EFFICIENT OF A SPILLWAY

Knapp were found to be highly velocity dependent, in fact, a dependence on the sixth power of the velocity was observed. A principal explanation for the great dependence of the velocity is that, higher velocities result in higher stagnation pressure and higher ambient pressure (for a given cavitation number) so that the pressure collapsing the cavities is greatly increased.

2.6.4 In view of the preceeding investigation, it appears, probable that most cavitation damage is associated with fixed cavities rather than bubble or transient cavities. Thus the hydraulic engineer should be particularily careful in his designs to avoid the possibility of fixed cavities in regions in which cavitation is likely to occur.

2.7. Cavitation Erosion And Corrosion : (65)

2.7.1 Erosion is the removal of the material due to water laden with suspended material impringing on the solid surface and the impact of the suspended material results in hammering action and damage. Corrosion may be due to chemical processes or eletrolytic processes or may be both, but cavitation damage is the result of the cavities formed due to hydro dynamic phenomena and their ultimate collapse causing removal of material and damage. In the phenomenon of cavitation it is very difficult to predict or assess the damage due to erosion, due to corrosion and entirely due to cavitation.

2.8. Effects Of Cavitation :

2.8.1 In the design and construction of hydraulic structures, gates, valves and hydraulic machines, it is all aimed at guiding and constraining liquids, to flow along water passages, in specialised directions and in specialised ~~fluids~~ fields. One of the effects of cavitation on such water passages is to alter both the effective size of the passage and the direction of guidance. The flow pattern gets degraded and the resistance to the flow increases. The other effects of cavitation are:

1. Loss of efficiency of the performance of the hydraulic machines like turbines, pumps, propullers etc.

2. Production of noise which is most undesirable in the case of submarines, torpedoes and under water baltistics.

3. Increase in the drag force on the cavitating parts.

4. Undesirable vibrations due to the collapse of the bubbles and due to shock waves set up. The failure the Bhakra diversion tunnel gates and the failure of the Panchet dam near Poona, are mainly due to the vibration of gates and out-let sluice.

5. Material damage due to pitting of the surface.

2.9. <u>Cavitation Damage</u> :

2.9.1 The degree of cavitation damage depends on the number of cavities collapsing on or near the boundary per unit time, the magnitude of the pressure collapsing the bubble and the characteristics of the boundary material. The number of cavities collapsing on the boundary will depend on the nuclei content of the water and cavity flow pattern ; many more cavities collapse on the boundary for fixed cavities than for transient bubble cavitation. Moreover, the individual cavities associated with fixed cavities collapse on the boundary in the vicinity of the stagnation point and thus the damage is usually greater for fixed cavities than for transient cavities.

2.9.2 The damage to the material is a cumulative effect of erosion corrosion and cavitation. Various theories have been put forward to explain the damage due to cavitation and each one of them has its own theories.

3.0 THEORY OF CAVITATION DAMAGE

Various theories have been put forward to explain the cavitation damage and they are:

1. Mechanical action theory
2. Electro - chemical corrosion theory
3. Instantaneous chemical action theory
4. Thermo - dynamic action theory &
5. Strain energy concept.

3.1 Mechanical Action Theory :

3.1.1 This theory was first put forth by Parsons and Cook in 1919 They are of the opinion that the water hammer effect consequent on the rushing of water inside the bubble at its collapse is responsible for the high stresses involved and consequent erosion of the material. In 1935, Boctcher correlated cavitation damage with hardness and supports his surface fatigue or cavitation fatigue theory. He has also noticed that material with high corrosive fatigue strength could resist better cavitation damage. Studies conducted by Prof. Plesset and Fillis have revealed change in the crystalline structure and deformation of the material Plastic deformation also takes place and this leads to fatigue of material and consequent failure. Experiments conducted at the Indian Institute of Sciences, Bangalore(65) on concrete of different compressive strengths have revealed that the cavitation damage is reduced with the increase in compressive strength of the material and this substantiates the fatigue theory. Similar findings on tests made in hard granite also corroborate this theory.

3.2 Impact Theory :

3.2.1 This theory assumes that the water ~~drophts~~ droplets are accelerated by the collapse energy to very high velocities and impinge on this boundary. Haller has shown that repeated drop impacts of moderate magnitude can destroy the material. Raleigh, Knapp and Hollander have concluded that the damage was due to simple impacts rather than due to repetition of stresses. Examination of the material has shown that there is no removal of material, but dents due to continuous hammering will peal off ultimately.

3.3 Material Damage Due To Pitting:

3.3.1 This is a very serious factor in any hydraulic structure or machine. If left unchecked, it may result in serious disaster. The overflow spillway at Grad - Coule dam is one of the outstanding examples showing damage due to pitting. As far as spillways are concerned, this is by far the worst damage cavitation can effect. When the cavity collapses adjascent to a solid surface, or a dropht of liquid strikes the surface at great speed, the mechanical action is similar to striking the surface with a small hammer. In the earlier days, it was widely believed that pitting was primarily a corrosive action. But it was proved otherwise, when Fottinger obtained pitting on glass walls of a vnturi - type cavitation apparatus. In the case of spillways, mechanical action seems to be more predominant.

3.3.2 The pitting of materials by cavitation is primarily an action of fatigue in which the surface skin of the boundary is continuously hammered by the millions of tiny blows, until it cracks and chipps off. It has been generally observed that the surface finish has an effect on the speed of pitting due to cavitation and on the rapidity of destruction of material by water jets. Rough surfaces are invariably destroyed more rapidly than surfaces of smooth finish. Thus as smooth surface of any material is worn away the pitting process accelerates and very rapid destruction, usually, starts. In some cases pitting has stopped of itself, apparantly due to water cushion covering the eroded region and preventing direct contact of collapse point and solid material.

3.3.3 The precise mechanism of accelerated destruction of rough surfaces is not known, but higher stress concentrations in irregular surfaces undoubtedly exist and should contribute decidely to the process. Other effects may be at work in the case of very rough or cracked surfaces as well. For example, it is possible that high pressure nucleus of the collapse cavity (possibly a tiny air bubble) is driven into the fissures in the material and explodes , when the pressure drops in the surrounding region. Another possibility is that the space at the inner ends of the cracks in the material act as cavities themselves and collapse with destructive force, when local pressures at the outer ends are increased by cavities collapsing in the flow. Still another possible explanation is that the high pressure developed at the inner end of the fissure by reflection of a pressure wave entering the fissure (similar to the action of water hammer at the closed end of a pipe line). In short, it may be generally summarised that pitting of the material is due to fitigue essentially under a continuous impact of blows and the chipping of the material accelerates the damage due to stress concentration.

3.4 <u>Electro - Chemical Theory</u> :

3.4.1 Cathodic protection seems to reduce cavitation damage and this indicates that cavitation damage is not only due to mechanical fatigue, but due to electro-chemical effect also.

3.5 Instantaneous Chemical Action Theory :

3.5.1 In 1954, Irwing Taylor, propounded that certain reactive unstable substances are produced at the instance of the final collapse. This reaction is quick in fast flowing liquids. Wheeler after analysing the damage on a number of materials, observed that a part of the material is found as particles in solution.

3.6 Thermo - dynamic Theory :

3.6.1 Wislicenus explains that the damage is due to rise in temperature and impact pressure at the collapse of the bubble.

3.7 Strain Energy Concept :

3.7.1 A.Thiruvengadam has arrived at a relationship between the energy liberated due to the collapse of the bubble and the strain energy of the material producing the damage.

3.7.2 He has further proposed a dimensionless number (27) called " Cavitation Damage Number" based on the concept of dynamic inundations produced by the short range shock waves due to collapse of cavitation bubbles. The number gives the ratio of this energy absorbed by the material in the deformation to the energy of the collapse of the cavitation bubbles.

3.7.3 In conclusion, it may be said that the various theories put forth by several investigators go to show that cavitation damage is a very complicated phenomena attributed to various causes. None of the reasons attributed to damage can occur individually but it is a cumulative effect of all the phenomenon. Also , removal of the material due to corrosion , or erosion can not be ruled out. Though the fatigue theory due to mechanical action is broadly accepted. The other factors which are explained above also can not be ruled out.

3.8. Cavitation Theory :

3.8.1 Several investigators have given different theories on cavitation (7) and factors which govern cavitation are:

1. Tensile strength of liquids.
2. Nuclei content.
3. Turbulence and boundary layer effects.
4. Physical properties of the liquid &
5. Shape of the cavitating body.

3.8.2 Tensile Strength Of Liquids:- In pure water, the only way to create cavitation is to produce tension in it. Tension can only be obtained, if water is free from gas or any solid material. It has been shown that water can with stand tension upto 10,000 atmospheres (61) and the order of pressure necessary to create cavitation is of the order of 30 to 50 atmospheres.

3.8.3 Nuclei Theory:- It has been now generally accepted that no cavitation can take place, if the water is completely free from any type of nuclei. They may be sub-microscopic sized solid particles or gas or air nuclei, whose size may range from 10^{-3} to 10^{-5} cms. and the cavitation process is considered to be the growth of these microscopic cavities to visible size. Experiments conducted indicate that there is not one fixed pressure at which cavitation would take place indicating that variation of this pressure is due to the variation in the size and quantity of the nuclei (67). It has also been shown that (1) water deaerated by boiling and cooling is very difficult to cavitate ;
(2) it is difficult to set up cavitation in water after it is allowed to stand for several hours and this is explained as due

to the dissolution of the bubbles in the water and (3) Application of high pressures to tap water has shown that it is difficult to set up cavitation. This is explained as due to the high pressure imposed on the liquid, the gas and the air bubbles in a free condition would get into solution devoiding the free gas and air bubbles which will facilitate cavitation. When gas is present, because of the low pressure, it expands and these cavities will collapse resulting in cavitation and its consequent effects.

4.0. TURBULENCE AND BOUNDARY LAYER EFFECTS ON CAVITATION

4.1 It has been established analytically that in a laminar boundary layer, minimum pressure exists on the boundary. Accordingly, these are the points at which cavitation would start. Although satisfactory theoretical analysis of pressure gradient across turbulant boundary layer are not available, measurements indicate that even for turbulent boundary layers cavitation will occur only at the surface of the body. In the slow moving fluid in the boundary layer, the nuclei in the low pressure region has got larger time to grow beyond critical size and cavitation process starts. The boundary inequalities change the flow pattern, as the boundary crevices are generally accepted as a source of nuclei. As the bubbles grows from the boundary, they become, subject to the forces of lift and drag and are swept off. The maximum size that these bubbles can reach, before the lift and drag overcome the surface tension force holding the bubble on to the boundary, must depend on the velocity surface tension, and the characteristics of the boundary layer. Thus, it would be seen that irregularities at the boundary increase the threshold for cavitation inception.

4.2 As the boundary layer is developed, if the pressure gradient becomes adverse, the boundary seperates from the surface and they in turn can cause cavitation.

4.3 <u>Physical Properties Of The Liquid</u> :

4.3.1 The physical properties and conditions of the liquid like gas content, the temperature, viscosity, vapour pressure, surface tension and the presence of admixtures will greatly affect cavitation.

4.3.2 It has already been mentioned before that when water is degassed, the susceptibility of cavitation is considerably reduced. The cavitation phenomena being analogous to boiling of the liquid, it is but right to expect that the cavitation should be earlier, if the liquid is at a high temperature. Pitting action is likely to increase with increase in surface tension due to the growth to a large radius, before the bubble breaks off. Similarly, dissolved impurities have a large effect on cavitation as the impurities will only increase the surface tension and hence, the cavitation that could be expected with admixture is greater than in a free liquid. Vapour pressure of the liquid influences the cavitation process very strongly and the growth of bubble depend upon the vapour pressure.

4.3.3 It has been observed by several workers that a large reduction in vapour pressure is accompanied by a pronouced increase in pitting action. Viscosity of the liquid also plays an important role in cavitation. The bubble can move more freely in a liquid of lower viscosity. Cavitation threshold will be earlier than in a liquid of lower viscosity than in a more viscous fluid.

4.4 Effects On Foreign Substance :

4.4.1 It has been shown by various experimentors that presence of solid materials brings about a discontinuity in the flow and causes cavitation. The interface between the solid particles and the liquid provides a discontinuity and this facilitates cavitation.

4.4.2 The actual conditions, are much more complex and cavitation inception pressure do not havea fixed value. Depending upon the conditions of the liquid and solid boundaries, the inception may occur at average pressures higher than vapour pressure. The inception of cavitation in a liquid medium is dependent on a variety of hydro-dynamic and other effects.

4.5. Types Of Cavitation Bubbles:

(a) Transient Cavities:- They are small individual bubbles, which grow oscillate, collapse and disappear. These are also termed as " Bubble Cavitation" (Figure 3)

(b) Free Individual Travelling Cavities:= These bubbles are fairly spherical and submicroscopic in size, but they gradually grow in size and are carried in the stream to the regions of higher pressure. The bubbles are not generally connected to one another , but they come off in quick succession.

(c) Steady State Or Fixed Cavity:- These are observed behind blunt bodies and sharp leading edge of hydrogoils (Figure 4). This cavity is a fixed one in the statistical sense. This sort of cavity is formed by the breaking away of the flow from the guiding surface at a low pressure point, usually assumed to be the point at which the pressure on the surface has fallen to vapour pressure of the liquid. From this point the stream follows a trajectory determined by the pressure field and usually guiding surface at some distance down stream from the break away point. The fixed cavity is the space existing between the guiding surface and the free surface of the flow as shown in the figure (4) .

(d) Single Cylindrical Cavities:- They are usually very big in size and are often created by magnetostriction oscillations of a bar. Such single cavities can also be formed in the wake of certain body shapes.

(e) Non-Stationary And Periodic Cavities:- These cavities vary with time (65). These cavities form in the flow periodically and are carried away by the flow. It may also be a cavity which changes its form in a stationary fluid under the influence of variable pressure.

4.6 Classification Of Cavitation:

The phenomenon of cavitation has been classified in several ways:

(a) Sheet Cavitation:- These cavities are trains of free travelling cavities, which cling as a sheet to the entire area of the section. Trains of individual free travelling cavities or portions of the fixed cavity type come in quick succession like a sheet studded with perals of various sizes. (Figure 2).

(b) Tip Or Vortex Cavitation:- If the fall of pressure is due to low strength of a vortex, cavitation bubbles issue out of the tip of a propeller blade as helical strings one following the other. This type is called ' Tip ' or 'Vortex' cavitation. Its path is helical in shape (Figure 5).

(c) Steady Stationary Vortex Cavities:- When blunt bodies like disc are exposed to high velocities, cavities are formed behind them. These are also called ' Wake Cavitation'.

(d) Cavitation can also be produced by creating tension in unsteady flow, such as magnetostriction oscillator. These are usually single cylindrical long bubbles, which grow shrink and collapse.

(e) When a torpedo enters water at high velocities, cavities are produced at the interface of water and torpedo. These cavities are unsteady and non-stationary. They cover portions of the body and in the wake developed by them.

(f) Acoustic Cavities:- Sonic vibrations of high frequences produce cavities in the fluid and such cavities are termed as 'Sonic Cavities'. They are very small single bubbles.

(g) Pseudo Cavitation:- If large gas filled bubbles are present in the liquid, it is possible for these bubbles to increase their ambient size many times even at pressures exceeding the vapour pressure. This stable bubble growth is not cavitation by the accepted definition, and has been termed pseudo cavitation(96) Pseudo cavitation is not likely to cause cavitation damage because high collapse velocities are not associated with such stable air cushioned bubbles. However, the existance of pseudo cavitation in a structure, can influence the hydro-dynamic performance of the structure.

Practical observations on the behaviour of hydraulic works on certain rivers, in particular on the Nile, have shown the existance of a particular phenomenon, which Lelivasky (43) calls as pseudo- cavitation. Although more smooth, the erosion produced is in this case somewhat similar to cavitation erosion, but it is in fact the effect of very sharp sand particles projected against metal or stone surfaces by local vortexes. This type of erosion is normally encountered in India also and the action of sand on the turbine blades at Khalima power house in U.P. India is one such case (84).

From the above it could be seen that a study on cavitation should essentially include the following:

1. Inception of cavitation.
2. Cavitation damage
3. Prevention of cavitation.

5.0 CAVITATION INCEPTION

5.1 Nuclei Theory :

5.1.1 In order to understand the inception of cavitation, it is very helpful to examine the conditions for static stability for a spherical gas volume surrounded by a liquid. Although cavitation is a dynamic phenomena, the basic principles of inception will be revealed by such static analysis.

5.1.2 As seen from Figure 13, the forces acting on the inside of the bubble are those due to partial pressure of the gas p_g and the partial pressure of the liquid vapour p_v. At the inter face (the surface of the bubble) is the surface tension force $2\pi r\sigma$, in which is the bubble radius and σ represents the surface tension per unit length. The surface tension per unit cross sectional area of the bubble is therefore $\frac{2\pi r\sigma}{\pi r^2}$ or $\frac{2\sigma}{r}$. This term $\frac{2\sigma}{r}$ is often referred to as the ' surface tension pressure'. This surface tension pressure tends to collapse the spherical bubble. Outside the bubble is the ambient fluid pressure p . For static equilibrium the following equation

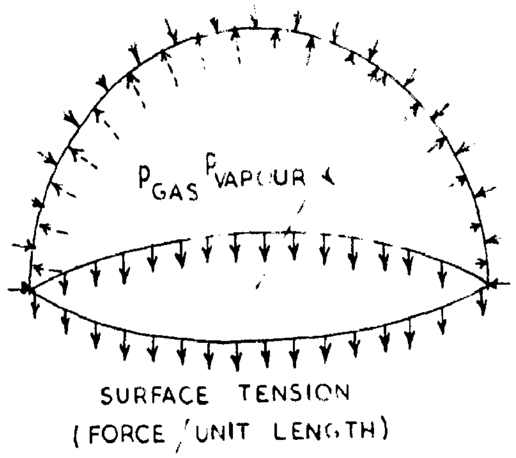


FIG 13. STATIC FORCES ON A SPHERICAL BUBBLE

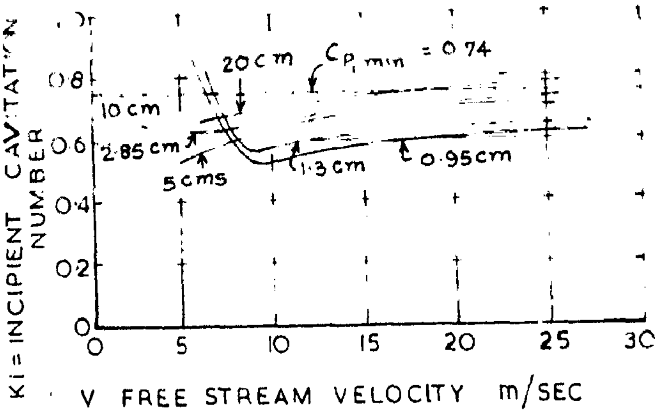


FIG. 15 INCIPIENT CAVITATION NUMBER VERSUS FREESTREAM VELOCITY FOR HEMISPHERICAL NOSED BODIES

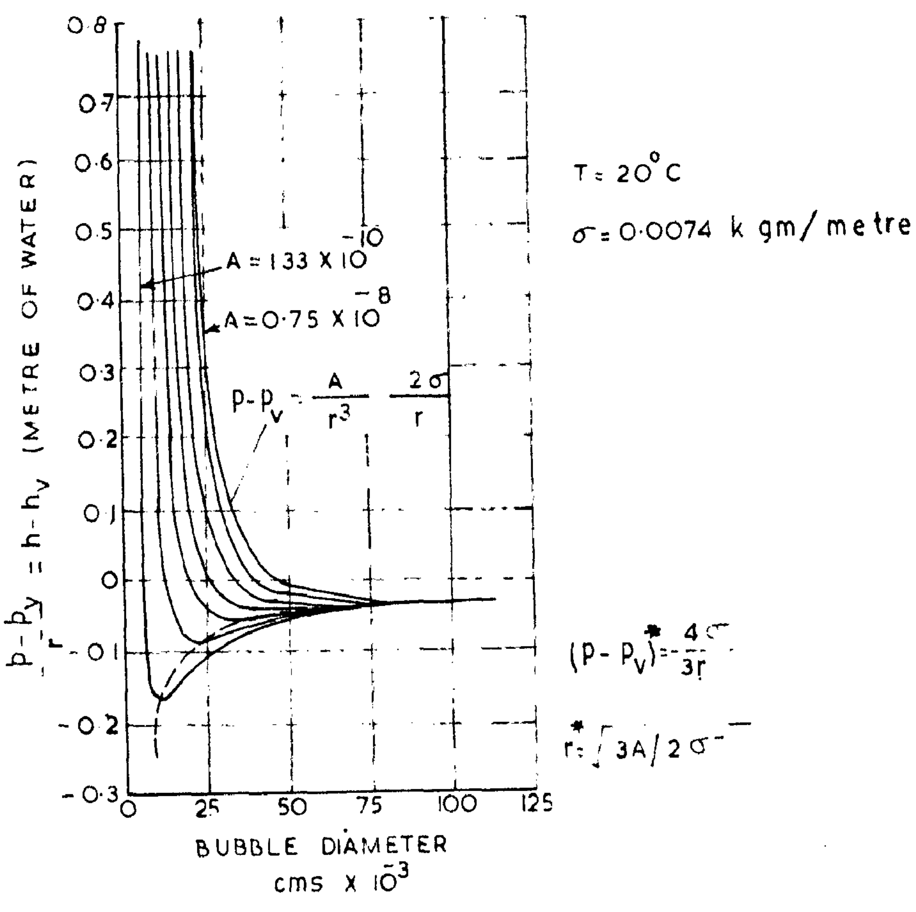


FIG 14 PRESSURE VERSUS BUBBLE SIZE

must be satisfied:

$$p_v + p_g = p + \frac{2\sigma}{r} \qquad \dots \dots \qquad (1)$$

Now, if it is assum-ed that the temperature and weight of the gas in the bubble remains constant, as the surrounding fluid pressure is reduced, then the pressure p_g, for a given weight of gas will vary inversely with the volume of the gas bubble i.e. $p_g = \frac{A}{r^3}$, where A is proportional to the number of molecules or the weight of the gas and r refers to the radius of the sphere. Therefore the equation (1) takes the form:

$$p - p_v = \frac{A}{r^3} - \frac{2\sigma}{r^3} \qquad \dots \dots \qquad (2)$$

In the figure 14, $(p - p_v)$ is plotted versus r for a constant value of surface tension corresponding to 20°C water and various values of the constant A. The most significant and interesting point to observe in figure 14, is that the curves have a minimum. This minimum can be found mathematically by differentiating equation 2, with respect to r and equaling to zero.

i.e. $$\frac{d(p - p_v)}{d_r} = -\frac{3A}{r^4} + \frac{2\sigma}{r^2}$$

$\therefore$ For $(p - p_v)_{min}$, $$\frac{3A}{r^4} = \frac{2\sigma}{r^2}$$

Or $$\frac{3A}{r^2} = 2\sigma,$$

$$\therefore \quad r = \sqrt{\frac{3A}{2\sigma}}$$

This minimum value of r is designated as r^*. Substituting the value of r^* in equation 2 ,

$$(p - p_v)_{min} = \frac{A}{(3A/2\sigma)r^*} - \frac{2}{r^*}$$

$$= -\frac{4}{3r^*}$$

i.e. $p^* = p_v = -\frac{4}{3r^*}$, where p^* corresponds to the values of p at minimum or critical points. The existence of the minimum value is the relationship between (p - pv) and r explains, at least, the beginning of the cavitation process because it reveals that (p - pv) reaches its critical or minimum value, then without further reduction in the ambient pressure, the bubble will continue to grow, that is, the bubble becomes unstable.

5.1.3 It may be noted from Figure 14, that the nuclei instability can not occur unless the fluid pressure is reduced below vapour pressure and that the magnitude of this reduction below vapour pressure depends on the original nuclei size. It may also be seen from equation 3 that the critical radius for a bubble containing only vapour (i.e. $A = 0$) , is zero and, consequently the fluid pressure must be infinitely negative in order to cavitate such a bubble. Of course, this theory can not be applicable, when the bubble radius approaches molecular size.

5.114 Some experimental data in support of the nuclei theory is also available (04) , (41). For example samples of pure water were subjected to several thousand kg/sq.cm. pressure for several hours in order to dissolve all gas present in the fluid and thus purge it of practically of all nuclei. After releasing the pressure, the boiling point of several samples, was found to be 140° C - 200°C. This result is comparable to the value of $p - p_v$ in Figure 14 approximately 210 m. of water, or from equation 3, the largest nuclei present in the sample must has been less than approximately 15 x 10^{-6} cm. in diameter.

5.1.5 The previous investigation has considered only the static stability of the cavitation nuclei. It appears reasonable to expect that, if nuclei are subjected to transient pressure reductions that critical pressure for instability might be considered less than the value given in equation 3. However, it has been shown that the vast majority of cases, the critical pressure predicted by the static analysis is not significantly altered by the duration of the transient. It was found that the pressure for a time slightly greater than the natural period of oscillation of the bubble. For a bubble of dia. 0.0025 cm , the pressure need remain at the critical value for 10 micro seconds or less. Thus for nuclei diameters of the order of 0.0025 cm. which are subjected to the low pressures that are hydro-dynamically produced in large hydraulic structures, the results obtained from the static analysis of the stability of the spherical gas bubble may be expected to closely approximate the dynamic stability of such a bubble. On the other hand, cavitation experiments conducted at high velocities on small scale models with short low pressure rejoins can be misleading, if it is assumed that dynamic effects do not influence the critical nuclei size and pressure.

5.1.6 Equation 3, and the previous investigations suggest that the almost universal assumption of vapour pressure as the critical pressure for cavitation may not be justified. However, it may be seen from Figure 14, that the vapour pressure is a good approximation to this critical pressure, if spherical nuclei of appreciable size (larger than 0.0025 cm) are present in the fluid. Free spherical bubble of this large size probably do exist near the surface of river and sea water, and generally speaking the assumption of the vapour pressure as the critical pressure for large scale hydraulic structures is adequate. On the other hand, when cavitation investigation are made in the laboratory with water that is treated to remove undissolved gas nuclei, the assumption that the critical pressure is the vapour pressure can be greatly in error. Similarly water discharged from the lower strata of high head dams may be void of large nuclei and the assumption that cavitation begins at vapour pressure may be incorrec

5.2. Origin of Nuclei :

5.2.1 The concept of spherical gas bubble as the nuclei for cavitation, although convenient as a model for analysis, is difficult to justify in the practical case. Some nuclei source other than free gas bubbles must be postulated in order to explain the cavitation that is observed in fluids in which free gas bubbles of the required size for instability are not observed.

5.2.2 However, there may be three distinct sources of nuclei, each capable of causing the phenomena of cavitation:

1. The free undissolved bubble, usually a microscopic in size. These on reaching the critical pressure, become unstable and grow rapidly.
2. The nuclei that exist in the crevices of foreign particles.
3. The nuclei that exist in the crevices of the boundary material, apparently small gas volumes trapped in the boundary material grow to imcriscopic size either because of dynamic instability within the crevice, or because of diffusion of gas from the sup-r saturated liquid into the initially microscopic " Crevice gas" or both. The size to which such a " boundary bubble" may grow depends on the relationship between the hydro-dynamic drag on the bubble and the surface forces holding the bubble to the wall. Once the bubble is swept away, it may be of stable size, but if the

pressure continues to decrease along its path, it may become unstable and cavitate. The degree to which this source of microscopic nuclei is responsible for cavitation inception depends on the presence or absence of nuclei sources (1) and (2) and their relative critical pressures.

5.3. Prediction Of Cavitation Inception :

5.3.1 It has been indicated that if the pressure in the fluid is reduced below vapour pressure and if nuclei of sufficient size are present, then the nuclei will become unstable and grow rapidly and this phenomenon is defined as cavitation. It is now important to review how such low pressures may be produced hydro-dynamically and thus to predict the conditions of cavitation inception.

5.3.2 Unseperated Flow:- On stream lined bodies, if the curvature is sufficiently mild, it produces nearly ideal flow without boundary layer seperation. For a given body shape, the magnitude of the pressure reduction at the location of the minimum pressure can be written as centrain percentage of the total dynamic pressure $\frac{1}{2} \rho V^2$.

i.e. $p_{min} - p = C_{p_1 \, min} \frac{1}{2} \rho V^2$ ... ... (4)

in which, $C_{p_{1min}}$ is known as the minimum pressure co-efficient. The value of $C_{p_1 \, min}$ is not appreciably influenced by P or V, if the boundary layer does not seperate from the boundary.

For a given size nuclei in the fluid, there is a pressure p* (From equation 3) that will cause nuclei instability and thus cavitation. By establishing p_{min} in equation 4, as equal to p^* , the hydro-dynamic conditions for cavitation inception are established as:

$$\frac{p^* - p}{\frac{1}{2} \rho V^2} = C\ pi,\ min \qquad \dots \quad \dots \quad (5)$$

5.3.3 Thus, if the nucleus size is known for the ambient conditions, p^* may be found from equation (3), and because cp, min is known or can be found either experimentally or theoretically, then equation 5 gives the relationship, between the ambient pressure and stream velocity for which cavitation will commence. Equation 5 may be written as:

$$\frac{p^* - p_v + p_v - p}{\frac{1}{2} \rho V^2} = \frac{p^* - pv}{\frac{1}{2} \rho V^2} + \frac{p_v - p}{\frac{1}{2} \rho V^2}$$

$$= Cp_i\ min \qquad \dots \quad \dots (6)$$

As previously indicated, it is a reasonable assumption that relatively large nuclei will present in river or sea water and thus the value of p^* is nearly equal to the vapour pressure. Also, for large hydraulic structures, in which cavitation may be expected, the value of V is usually high. Thus the term $\frac{p^* - p_v}{\frac{1}{2} \rho V^2}$ can often be neglected and the equation (6) is replaced by:

$$\frac{p_v - p}{\frac{1}{2} \rho V^2} = C_{p_i}\ min \qquad \dots \quad \dots \quad (7)$$

The negative of the left side of equation 7, is known as the insipient cavitation index K_i. This parameter is now universally used for correlating the characteristics of cavitation flows. The value of the parameter $\frac{p - p_v}{\frac{1}{2} \rho V^2}$ at any condition other than inception is simply denoted without the subscript as K. Thus

$$K = \frac{p - p_v}{\frac{1}{2} e V^2} \quad \text{and}$$

$$k_i \; \cancel{ki} = \frac{p - p_v}{\frac{1}{2} e V^2} \quad (\text{ at inception})$$

$$= - C_{p_i min}.$$

Thus, if the value of K for a given structure and operation condition is such that $K > ki$, the structure should operate cavitation force. However, if $K < ki$, cavitation should be expected with the degree of cavitation, depending on the magnitude of ($\cancel{ki} - \cancel{k}$) $k_i - K$.

5.3.4 It may be noted that equation 9 , may not be adequate for small scale models tested in water that has been ~~e~~specially treated for the removal of nuclei. For examples several geometrically similar axisymmetric bodies have been investigated for cavitation inception at various speeds and the results are presented in figure 15 ($\frac{17}{39}$). It may be seen from the figure, that the measured incipient cavitation indices were less than $CP'_{i\,min}$ and depended on both the model size and test velocity. However, the data indicates that the incipient cavitation index does approach $C_{p'i\,min}$ for large size and high speeds. It is quite probable that the scale effects shown in the figure 15 are primarily caused by the low concentration of nuclei and the small nuclei sizes present in the test water.

5.3.5 Seperated Flows:- If the flow about a body is decelerated too rapidly, the boundary layer seperates and the pressure distribution along the boundary is no longer a true indication of the minimum pressure in the field. Two examples of seperated flow are shown in Figure 16 (a) & (b). As is shown in the figure, a

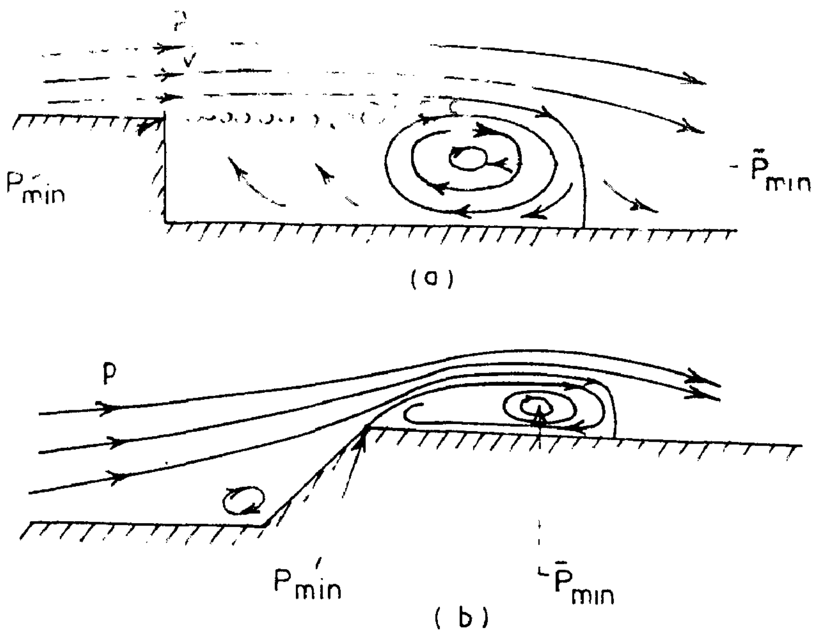
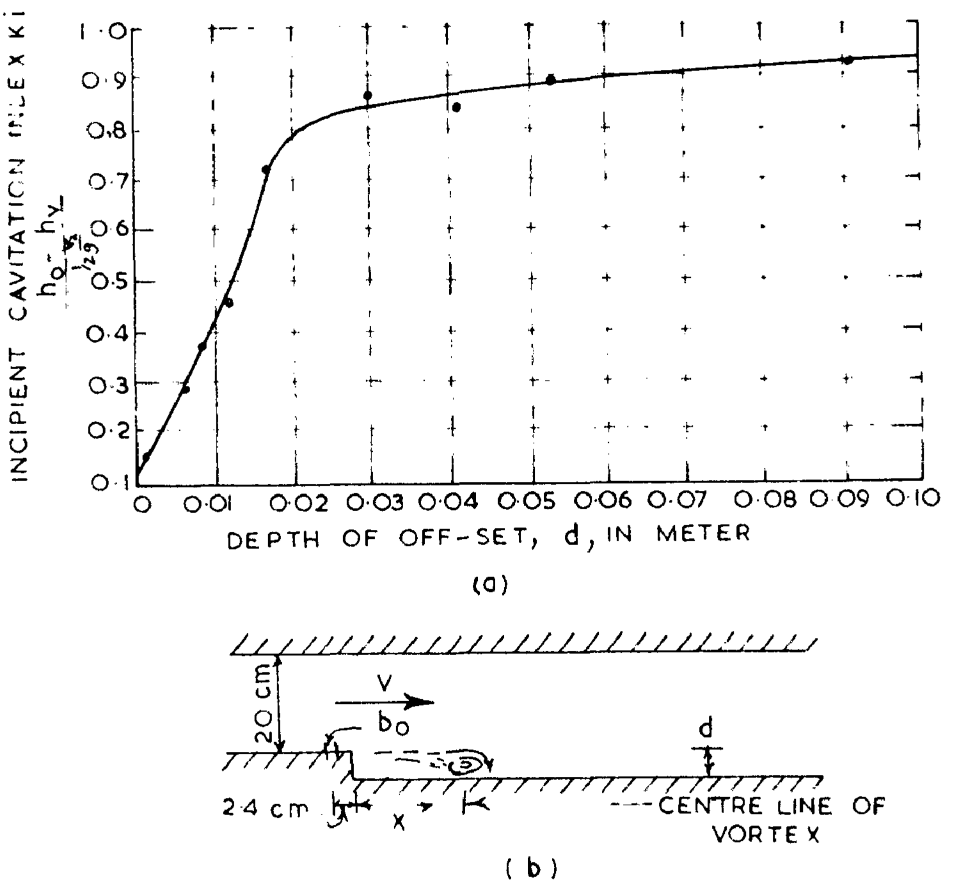


FIG. 16 EXAMPLES OF SEPERATED FLOW



NOTE -
CAVITATION FIRST OCCURS WITHIN AT $x \simeq 3d$

FIG. 16(a) CAVITATION CHARACTERISTICS OF VERTICAL OFF-SETS AWAY FROM FLOW vortex is formed and cavitation will be incipient when ki is the negative of the minimum pressure co-efficient in the flow field $\bar{C}_{pi}$min, the barred symbon being used to distinguish from the minimum pre co-efficient at the boundary $c_{pi_{min}}$. No exact method of obtaining $\bar{C}_{pi}$,min in terms of the measured boundary pressure is known. For the case of a two dimensional offset in uniform flow, the results of the experiments conducted at the U.S.Waterways Experiment Station are shown in Fig. 16. In these experiments on off-sets, incipient cavitation always occurred in the centre of the stable vortex formed downstream from the offset.

5.4 Inception Of Cavitation In Isolated Surface Irregularities:

5.4.1 If the body is sufficiently smooth the on-set of cavitation can be determined from the theoretical value of the minimum pressure. But, generally the surface is rough. The roughness may be evenly distributed like a machined surface or i may be isolated in mature such as an aggregate protruding from an otherwise evenly finished concrete surface. If the surface irregularity protrudes a significant distance into the stream the minimum pressure will be lowered below that for a smooth surface. Hence cavitation will result and the body will not perform as designed.

5.4.2 Calehuff, Wislicemus, and Walker showed that cavitation characteristics were related to the relative roughness height $\frac{k}{\delta}$, where k is the height of the roughness and δ is the thickness of the boundary layer.

5.4.3 Hall (89) conducted experiments on isolated surface irregularities in the turbulent boundary layer of a flat plate. The relative height of roughness ranged from 0.014 to 3.4 and the roughness Reynolds number from 370to300000. He concluded:

1. The incipient cavitation number of an isolated surface irregularity in a turbulent boundary layer is dependent upon the relative height of the roughness, the boundary layer shape parameter, the velocity and other variables yet unknown.

2. For given values of velocity and boundary layer shape parameter, the incipient cavitation number increases with relative height of roughness, which is rather the most significant variable to describe the incipient cavitation number.

3. For given values of velocity and relative height of roughness the incipient cavitation number increases with a decrease in the boundary layer shape parameter (Boundary layer shape parameter is $\frac{\delta^*}{\theta}$, where * is the displacement thickness and θ the momentum thickness

4. In most cases, the incipient cavitation increases with velocity.

5.5 <u>Cavitation Damage</u>:

5.5.1 This has been already covered in section (3.7.5).

5.6 <u>Prevention of Cavitation Damage</u>:

5.6.1 Naturally the most effective means of prevention of cavitation damage in new structures is the elimination of cause of cavitation. Therefore the designer should have in mind, the facts that sharp curvatures, abrupt corners or any combination of circumstances which produce flow curvature, vortices, eddies, seperation or high local velocities are all conduice to cavitation.

5.6.2 In completed structures, where cavitation and pitting occur and drastic modifications are not possible, various methods have been successfully employed to arrest the damage. Replacement of eroded parts by a tougher, or more resistant material has been proved. This method has been particularly useful in conduits in dams, where steel liners has been used, where pitting was occured in concrete. In some structures pitting process has been allowed to continue until it arrests itself, presumably, by the creation of water cushion. However, this method has little appeal as a good solution.

5.6.3 In the case of hydraulic machines the solution of proper cavitation resistant material may reduce the damage. A short representative summary of the relative resistance of metals used in hydraulic machinery has been compiled by Rheingaus (60) and is given below:

Alloy	Wt. loss after 2 hrs . m:gms.
Rolled stellite*	0.6
Welded Aluminimum Bronze	3.2
Welded stainless steel (two layers 17 cr , 7% Ni)	6.0
Hot rolled stainless steel (26 Cr , 13% Ni)	8.00
Tempered, rolled stainless steel (12% ~~Ni~~ Cr)	9.0
Welded Mild steel	97.00
Cast Steel	105.00
Alluminium	124.00
Brass	156.00
Cast Iron	224.00

5.6.4 Admission of air to the low pressure regions have been proved effective in both hydraulic structures and hydraulic machines. Higher air content of water has been seen to cushion the cavity collapse and reduce its destructive effects, and air

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* This material is not suitable for ordinary use, in spite of its high resistance, because its high cost and difficulty in machining.

introduced will upstream from the cavitation region is to " break the vacuum" there, thus raising the pressure and tending to eliminate the true cavitation. The air introduced in this manner presumably forms large bubbles, which are compressed down stream in the high pressure region but are prevented from collapsing by the cushioning effect of large quantity of air, which can not be absorbed by the water. ~~Examples of air injection by Rasmussen~~ (25).

5.6.5 Of very recent interest are the possibilities for cathodic protection against cavitation damage. Although the use of cathodic protection was first suggested on the basis of electro-chemical postulates of cavitation damage, it is now quite conclusively established that the protection action is associated with the evolution of hydrogen and the associated cushioning effect provided by the gas (26), in just the same way as air injection systems.

CHAPTER: 4

CAUSES OF CAVITATION ON SPILLWAYS

1.0 GENERAL:

1.1 There have been cases of cavitation erosion on different elements of spillway structures such as baffle piers, apron slab beyond baffle piers, overflow surfaces of spillways, surfaces of spillway pier down stream of gate slots etc. The cavitation erosion on various spillway elements can not be tolerated, because of the repairs of the damaged portions are costly and difficult. Many often the normal operation of the structure may also be disturbed by cavitation erosion. Only with spillways operating rather seldom, one can allow its elements to the action of cavitation and that too for a short period. Many factors contribute to the creation of cavitation on spillways. Fore most among them is the reduction of pressure on the spillway crest due to wrong design. The crest pressure of a gated spillway depends to a large extent on the position of the gate sill. Among other factors, which add to the lowering of spillway pressures, are improperly designed gate slots, surface irregularities, mode of operation of spillway etc. Some of these aspects which may create cavitation on spillways are discussed in this chapter.

2.0 CAVITATION DUE TO WRONG DESIGN:

2.1 From the discussion in Chapter 3, it is quite evident that a reduction in pressure is the root cause for the inception of cavitation. Cavitation will occur in a liquid when dynamic conditions cause the pressure to fall below vapour pressure of the liquid. So naturally a study of the cause of cavitation leads us to a study of pressure variations due to wrong design.

2.2 As a general rule, the base curve of the spillway crest is the profile of a fully ventilated nappe of water flowing from a sharp crested weir. Such a method has been founded on the belief that the presence of a solid structure below the nappe and in immediate contact with it, will not appreciably affect the course of the freely falling particles of water, that is, neither will the nappe tend to spring loose from the spillway face, nor will it exert pressure on it. But the condition of the flow changes with the introduction of a solid wall beneath the force falling nappe, even though it exactly fits the shape of the nappe. A flow of this nature is not only governed by ~~far~~ gravitational forces, but by the viscosity and surface tension of the liquid also. These fluid properties exert a tangential force on the spillway face. If the fluid were frictionless, it would not have been possible to exert any tangential force and the flow on an ideal spillway face should be exactly as the flow from a sharp crested weir. If this were the case, the pressure along the profile would have been exactly atmospheric at all points. The pressure distribution in an ideal case like this is shown in figure 1.

2.3 A hydro dynamical approach to the problem of curving flow, such that over spillway crests, shows that conditions at any point in the flow are dependent upon those directly upstream. It has been demonstrated by Bazin and many other investigators that any change in the crest of the spillway will result in a change of the nappe profile (64). Logical as this reasoning may seem, designers are prone to determine carefully the down stream curve of the spillway face and then proceed to round off, bevel or prolong the short up stream curve to suit their fancy despite the fact that the down stream curve most emphatically depends on this curve which they modify.

2.4 In order to study a spillway properly designed, according to the profile of the nappe over a sharp crested weir of the same proportions and to illustrate the affect of a change in the curvature in the immediate vicinity of the crests, the results of the experiments done by Rouse and Reid (64) are quite revealing. Even though these experiments were done about 35 year ago, the results are still valid. Two spillway profiles of practically of the same shape were tested known as profile No. 1 and profile No. 2 shown in figure (2). It is seen from the pressure distribution shown in the figure, (2) that positive pressures occur at the top of the spillway crest preceded by a negative pressure for some portion of the up-stream curve in the case of profile 1. Now for profile No. 2 the downstream portion was kept same as in profile No. 1, and the upstream was rubbed a little. It may be noted that even though the difference in curvature was very slight, the pressure variations were considerable. In addition to this ideal form of crest, another type tested was with a short horizontal section at the top, such as frequently encountered in practice, preceded

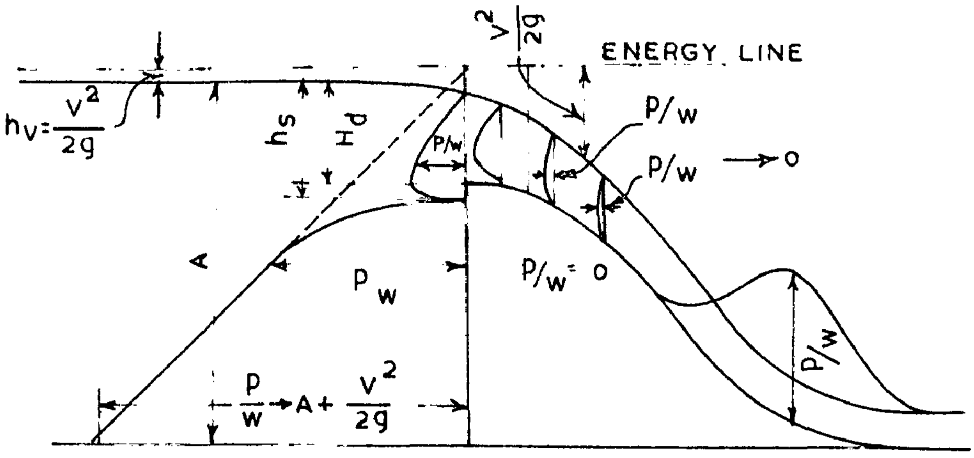


FIG. 1. PRESSURE DISTRIBUTION ON AN IDEAL SPILLWAY FACE

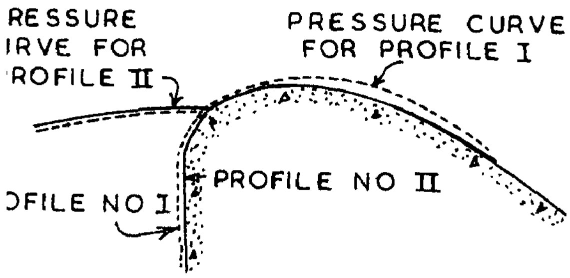


FIG. 2.

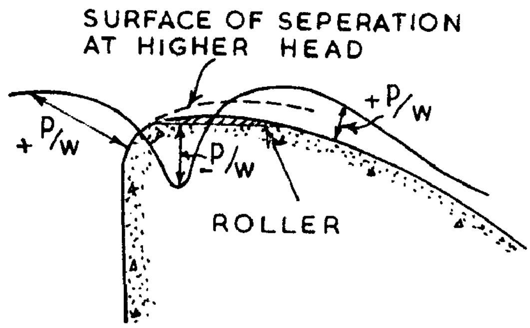


FIG. 3

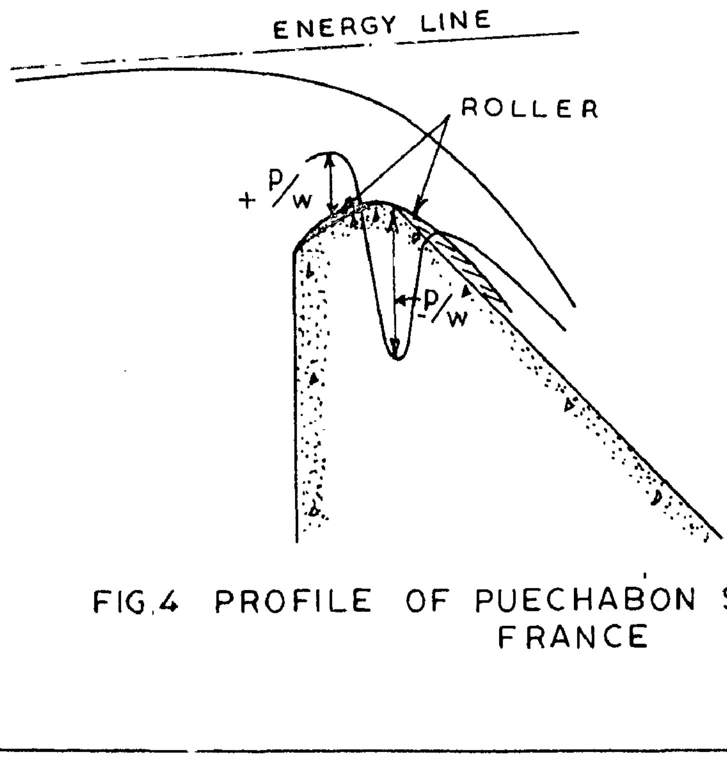


FIG. 4 PROFILE OF PUECHABON SPILLWAY FRANCE

by a short curve of constant radius as shown in figure (3). As may be seen from the figure (3), the departure of the pressure curve from the ideal curve is very marked, since the change in the form lies more nearly in the regions having the greatest influence of the flow.

2.5 Another important point to be kept in mind is the avoidance of sharp corners on the spillway crest. For example in the case of the profile shown in figure (3), that even under the design head the stream had left the crest over a short distance, beginning with the abrupt change in the curvature, the space below being filled with a small roller. The obvious conclusion is that sudden changes in curvatures are to be avoided, for although a roller may at first fill the region of seperation, negative pressure at this point will create partial vacuum and if it goes below a certain limit, cavitation will be the result. As a further example of the formation of cavities at this point of abrupt change in curvature, the case of the Pucchabon spillway in France is cited, as this type of design is commonly used in Europe [47]. Results of the studies conducted by Escande are reproduced in figure (4). The illustration shows definitely two places of seperation at places of too rapidly change in surface profile. Another point to be noted is that, at no point along the continuous curve of types shown in profile I or II, will it be possible for such a cavity to exist. For the same reason, if air introduced at any point it will be immediately carried down stream, because the steady change in curvature and is sufficient to ensure against seperation even under relatively high heads

Obvious, however, even for the ideal crest there is a limiting head under which the curvature becomes excessive and for which a clinging sheet is impossible. From the forgoing it is quite evident that proper care has to be taken in the shape of a spillway crest to avoid cavitation, even though reports of actual cases of cavitation damage occurred due to wrong design are few.

3.0 CAVITATION DUE TO THE WRONG LOCATION OF GATE SEAT:

3.1 The common type of crest gates used for spillways are vertical lift gates and radial or tainter gates. However, the radial gates are the most widely used crest control gates. One of the main draw backs of vertical lift gate is the cavitation potential it creates on account of the gate slots in the piers. Also under certain conditions of operation vertical lift gates may vibrate with possible damage to the gate itself. Similar damage occurred to the bottom portion of the vertical lift gates of Bouneville dam in U.S.A. [25]. It was found out that the cause of vibration was cavitation, when there was partial opening of the gate with submergence. So providing radial gates, the cavitation potential of the spillway crest is greatly reduced. But the installation of radial gates is only a partial solution of the problem. For, the pressure variation of a spillway crest is affected by the location of the gate seat also.

3.2 A standard spillway crest is usually designed to conform to the shape of the lower nappe down stream from a sharp crested weir. This profile is called datum shape. When gates are to be used on a spillway, the same crest shape is used from the upstream face to the gate sheats. Down stream from the gate seats, the crest profiles are sometimes made to correspond to the theoretical trajectory of a jet flowing from a unit high rectangular orifice at the design head. However, several factors other than hydraulic phenomena are usually considered when determining the location of the gate seat. If the gate, seats on the crest, the theoretical curve will be comparatively flat and the spillway will contain considerably more concrete than is usually needed for stability. If the gate seats below the crest, the profile will be cheaper, but a higher gate is necessary to retain the same storage pool. Structural consideration also sometimes dictates the gate seat location. If a bridgespans a spillway, it might be necessary to place the gate down stream from the crest to provide space for gate lighting mechanism or to provide clearance, when the gate is raised. The gate might have to be located down stream some times to insure that the trunnion will be above the water surface.

3.3 Many hydraulic engineers believe that when the gate is located on the crest there is a tendency for the jet to spring free from the crest at higher reservoir elevations and small openings, this may result in sub-atmospheric pressures and possible cavitation damage to the spillway. The designers also believe that placing the gate downstream from the crest results in flow being directed down ward; the tendency of the jet to spring from the crest is reduced. It is also widely believed that lower the gate seat, higher will be the pressure along the spillway, and also that cavitation damage occurs immediately down stream of the gate seat, if it is not properly located. However, observations of some prototype spillways indicate that cavitation damage to spillway surfaces occurs down stream from the region where the control gates have any effect on the flow [59]. No damage to the spillway crest in the vicinity of the gates has been found which can be attributed to the crest shape and gate seat location. Hence, in order to have a better knowledge of crest pressures due to various location of the gate seat, model studies were conducted by U.S. Bureau of Reclamation on five spillway profiles [80]. For the test one piezometer was placed at each tested gate seat and others were in equally spaced downstream along the spillway profile. The five crests, used in the investigations, provided a wide range of steepness and gate seat location. The location The results of these ~~results~~ experiments are very briefly discussed below:

3.4 <u>Ross Dam (Figure 5)</u> :

3.4.1 The design gate seat is 10.16 cm. below the crest, the scale ratio being 1:24.14. The datum shape is slightly steeper than the design shape. A comparison of the design shapes and datum shapes indicate that the overfall section from the gate seat downstream was designed from the theoretical trajectory of a jet from an orifice. From the reasoning previously out lined, pressure along the profile should increase, when the gate seat is down stream from the design location and reduce when the gate seat is up-stream.

3.4.2 The pressure profiles in figure (5) show that for the same gate openings and heads, the pressure for all gate seat locations downstream from the design gate seat are practically same. However, when the gate is seated upstream from the crest, peizometers from the crest down to the second piezometer below

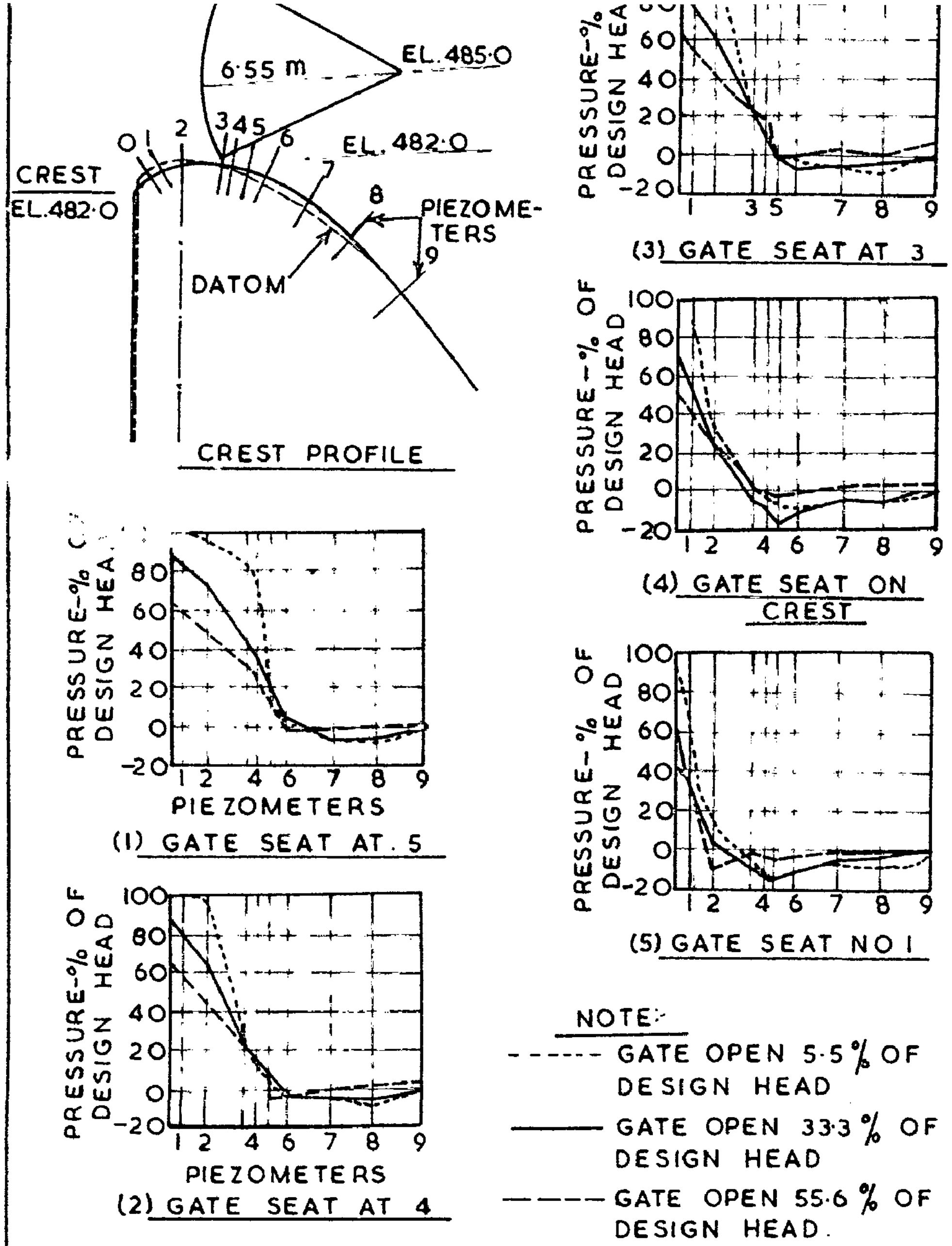


FIG. 5. ROSS DAM CREST-PRESSURES

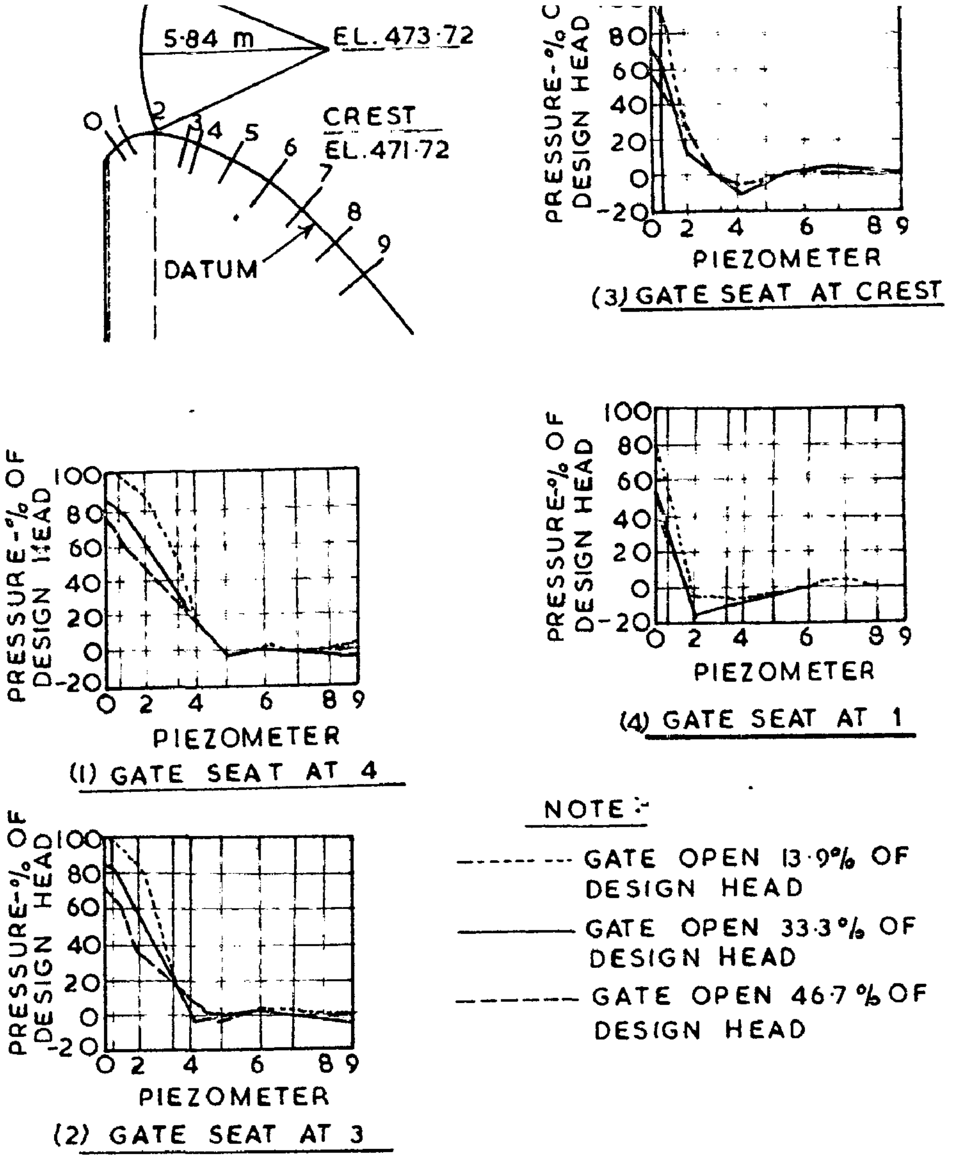


FIG. 6. ATLUS DAM-PRESSURES OBSERVATION

EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION the design gate seat showed pressures lower than were obtained for the other seat locations. And their trend, which may be observed from the pressure profiles is that zone of sub-atmospheric pressures moved up-stream as the gate seat moved up-stream. However, for all gate positions the down stream peizometer (No.9) showed pressures more than atmospheric.

3.5.0 <u>Atlas Dam (Figure: 6)</u> :

3.5.1 The design gate seat is on the crest. The down stream shape is the same as the design shape, indicating that the profile was obtained from the lower nappe shape of a sheet of water passing over a sharp crested weir. According to reasoning, the pressure along the profile should have the highest values when the gate seat is 30.5 cm. below the crest and should become smaller as the crest moved up-stream. The pressure profiles in figure: 6 , show this to be true in that for both down stream gate seat locations, the peizometers down stream from the gate seat indicate nearly atmospheric pressures for all gate openings. However, it may be noted that in both cases, the lowest pressure obtained was practically the same. Also, when the gate seals up-stream from the crest, the peizometers from the gate seat to peizometer:6 indicate sub-atmospheric pressures. Generally speaking peizometers No. 6 to 9showed nearly atmospheric pressures for all gate seat location at all gate openings. Peizometers No. 2 to 5, were affected by the gate seat location and the pressure was reduced at each peizometer as the gate seat was moved up-stream.

3.6.0 <u>American Fall Dam (Figure :7)</u> :

3.6.1 The design gate seat is on the crest. The datum shape is considerably steeper than the design shape, indicating that

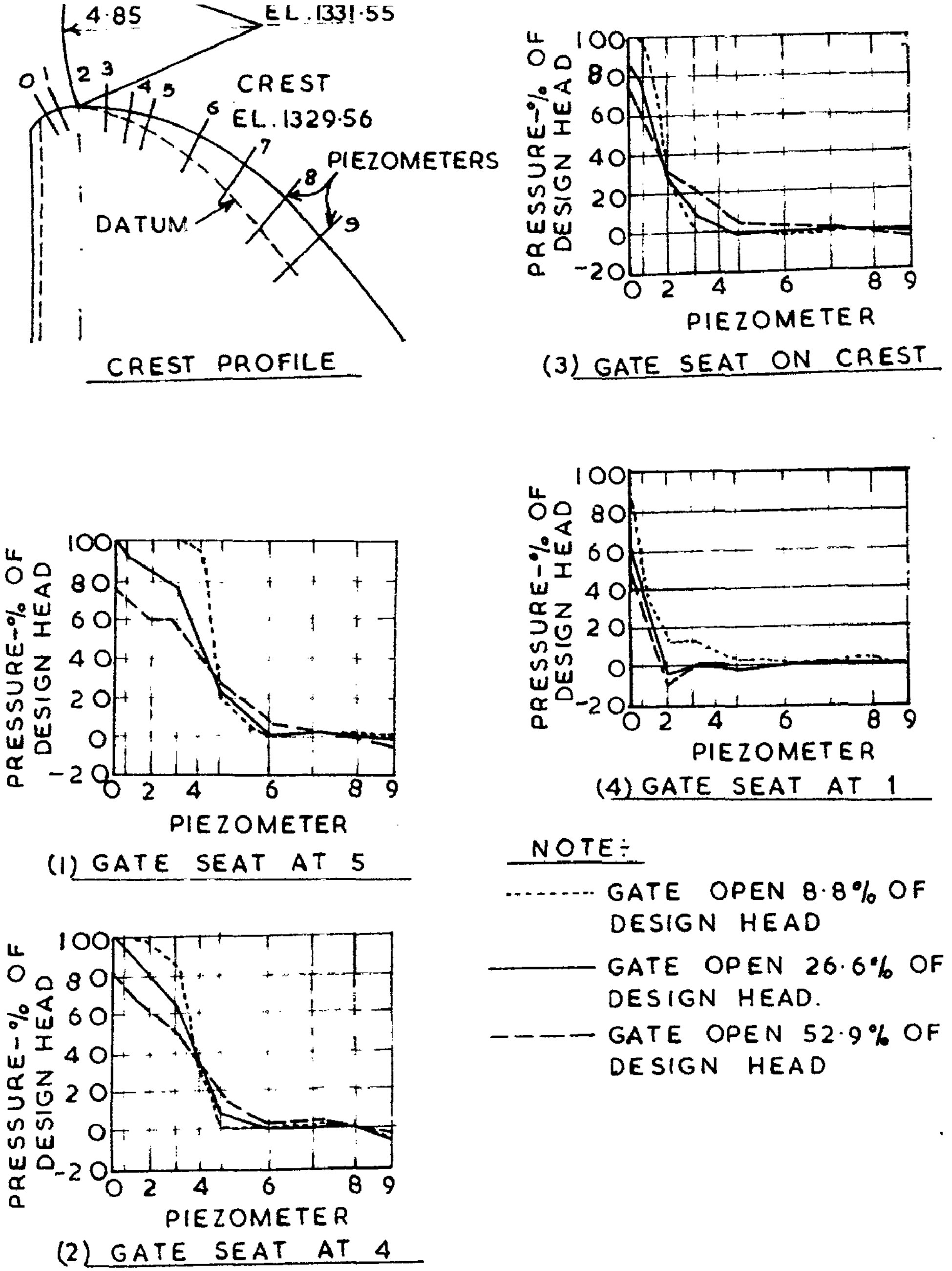


FIG.7. AMERICAN FALLS DAM CREST-PRESSURES

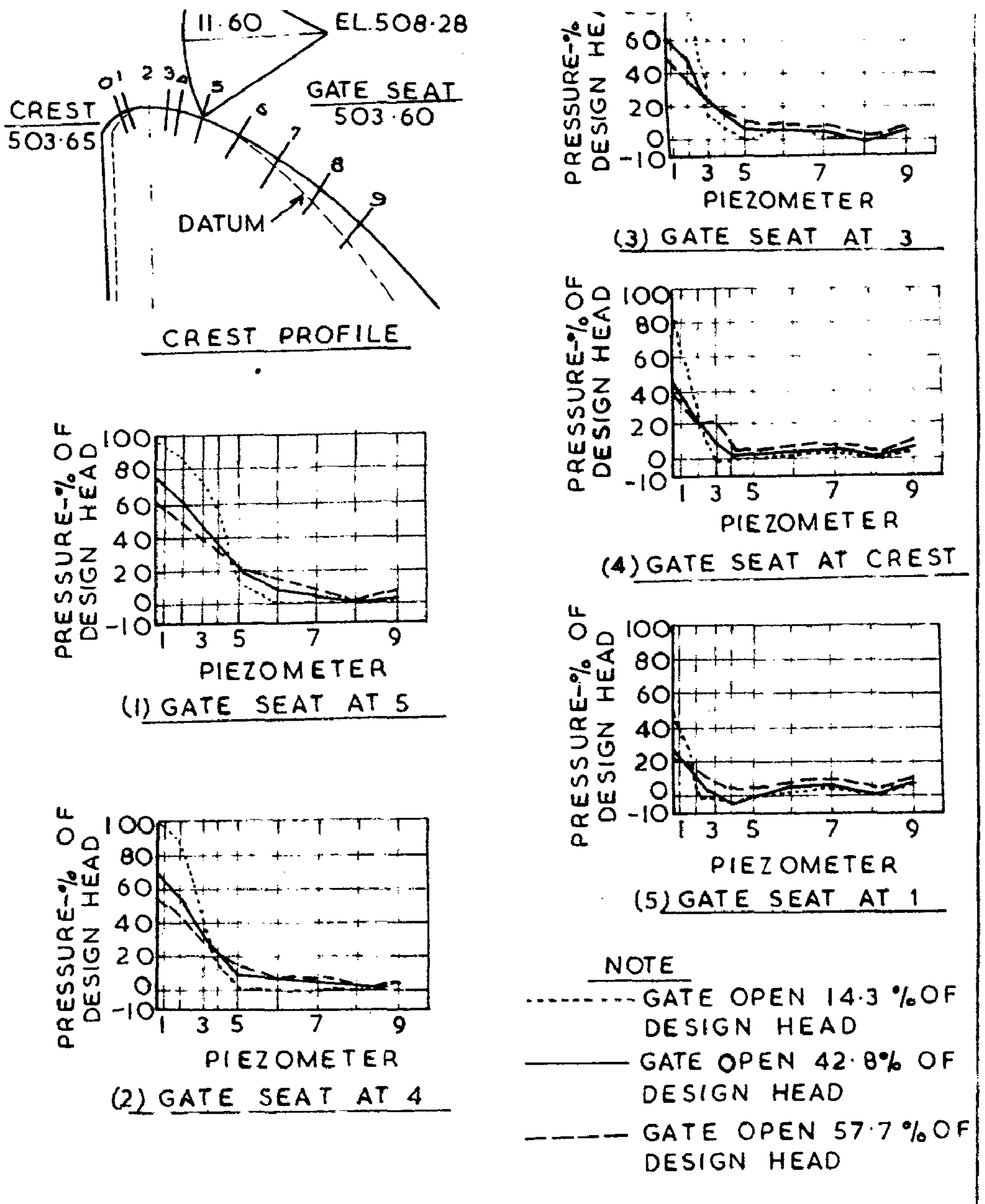


FIG. 8. BHAKRA DAM CREST PRESSURES

the profile from the crest down stream was obtained from the theoretical trajectory of a jet flowing from an orifice. The pressure should be near atmospheric for small gate openings; when the gate is at the design location , for down-stream gate seat locations, the pressure should be higher; and up-stream locations, the pressure should be lower. The pressure profiles in figure; 7, indicate that performance agrees with the reasoning. The only pressures significantly sub-atmospheric were those at the crest peizometer when the gate seat was up-stream from the crest.

3.7.0 Bhakra Dam (Figure; 8) :

3.7.1 The design gate is 0.75 m. below the crest on the down stream side. The datum shape is steeper than the design shape. For this crest, the highest pressures should occur with the gate at the design location and they should become successively lower as the seat location moved up-stream. Also, the peizometers should indicate near atmospheric pressures, when the gate seats on the crest. The peizometers down-stream from the gate show slight reduction in pressure as the seat is moved up-stream and when the gate seats on the crest and upstream from the crest, the pressures are a little above and below atmospheric.

3.8.0 Canyon Ferry Dam (Figure; 9) :

3.8.1 The deisgn gate seat is 15 cm. below the crest. The datum shape is steeper than design shape. The relative shapes of the two profiles indicate that the profile down stream from the design gate seat was derived from the theoretical trajectory of a jet flowing from an orifice. Therefore, the pressures

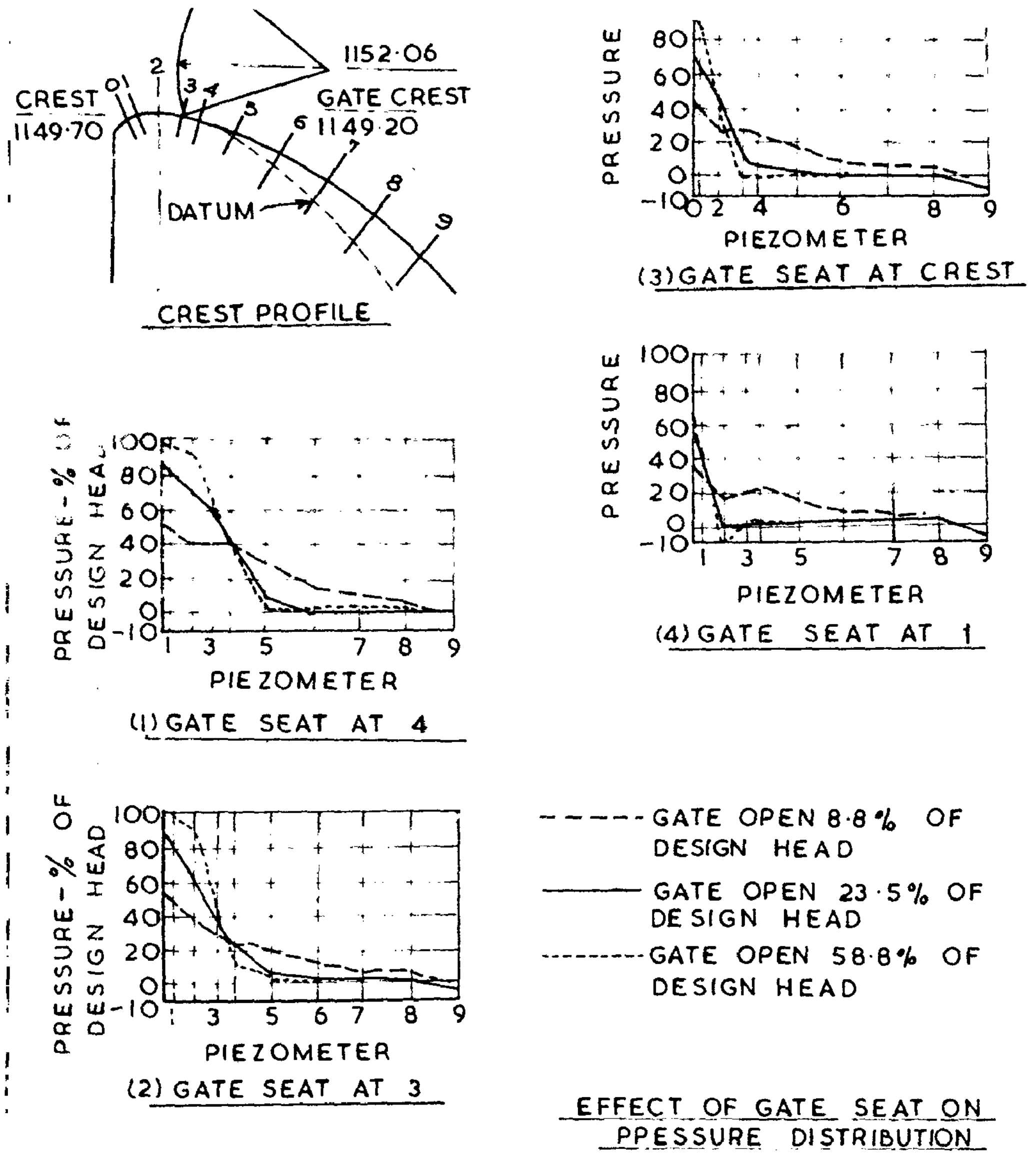


FIG.9. CANYON FERRY DAM CREST-PRESSURE

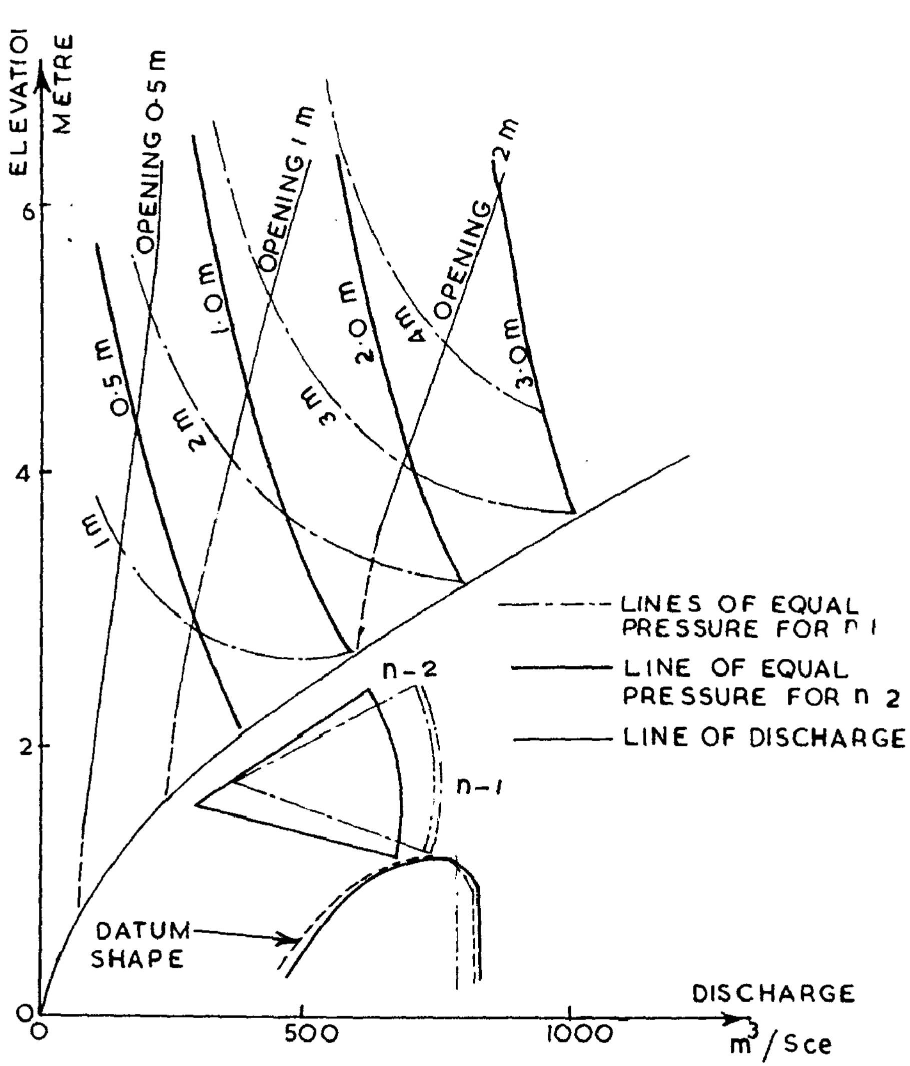


FIG.10 DISTRIBUTION OF MINIMUM PRESSURE AT PARTIAL GATE OPENING (EDGARD DESOUZA DAM BRAZIL)

down stream from the gate should be near atmospheric, when the seat is at the design location, higher when it is at the down stream location and lower as the gate seat is move up-stream. The pressure profiles on figure: 9 indicates that the pressure increase, when the gate seat is down stream from the design seat and decrease when the gate seat up-stream, even though the difference was little. However, for gate openings of 8.8 and 23.5% of the design head, the pressures down-stream from the gate seat are practically the same for any gate seat location.

3.9.0 Conclusions:

3.9.1 The pressure tests show that the gate seat location has a minor effect on the profile pressures, when the gate is on or down stream from the crest. When the gate seats up-stream from the crest there is a sharp reduction in the pressure at the crest, particularly at small gate openings. The theory that a gate seat down stream from the crest will give a down ward direction to the flow passing beneath the gate seems to be confirmed by these tests. Although the increase in pressure was small it was noticeable for all crests. It may also be noted that the lowest pressure for partial gate openings occur some distance downstream from the gate seat and not immediately down stream. This also confirm with the observation made on certain proto type spillways(59) The lowest pressures occurred on two crests that most nearly corresponded with datum shapes. Fig. (5) and (6), and generally occurred for a gate opening of about 33% of the design head. For the other three crests figures (7),(8),&(9 the lowest pressures occurred at the smallest gate openings. The lowest pressure recorded was about 20% of the design head below atmospheric.

3.10 Studies on the aspect was also done in Canada [57] in connection with the design of the spillway for Edgard de Sonza dam in Brazil. These tests also have confirmed that, if the gate is located just on the crest, the pressure down-stream will be low at small gate openings than at full gate openings. Usually even a slight shifting of the gate seat on the down-stream side of the crest is sufficient to keep the pressure for all gate openings higher or equal to the minimum pressure occurring at full gate openings. Figure (10) shows very clearly the reduction of minimum pressure obtained by moving down stream the gate seat of Edgard de Senza dam.

3.11 The problem is exactly the same if a vertical lift gate is used instead of a radial one. This was also experimentally proved in Canada in connection with the design of Hart Janne dam spillway [57].

4.0 CAVITATION DUE TO IMPROPER GATE SLOTS :

4.1 One of the many factors that has to be borne in mind in designing a vertical lift gate is the design of the grooves or slots in which the ends of gates are supported. Flow turbulences within the gate slots and cavitation damage just down stream from the gate slots occur, when vertical lift gates are operated at small openings under heads in excess of about 10 metres. High velocity flow passing over gate slots, create vortices in the slot. At partial gate opening, the vortices travel down the slot and on to the spillway face at the bottom of the slot. Unless gate slots are properly designed turbulence and cavitation will occur to cause the destruction of the roller chains, and wheels as well as costly damage to flow surfaces. Turbulent surging within the slots can be controlled by baffle plates attached to the bottom of the gates and to some degree by reshaping of the slots.

4.2 One of the striking examples of damage occurring on spillway piers and face due to a wrongly designed gate slot is that of the damaged spillway of Bonneville dam in U.S.A.(16) It has a concrete ogee spillway. The crest gates consist of vertical lift gates with fixed wheels. The maximum head of water over the spillway is 17.8 m. The spillway gate slots are 2.14 m. wide and 0.61 m. deep. The pier walls down stream from the slot is in line with the wall up-stream from the slot and no set back, taper or bevel exists. The 10 cm. rounded corners of the slot are armoured with steel. Soon after the spillway was put into operation, it was noticed that the concrete at the base of the piers and just down-stream from the gate slot was eroded. It appears that the jet of water issuing from beneath the gate impinged on the down-stream side of the gate slot and was diverted round the armoured dornor, but failed to adhere to the concrete pier wall down-stream there from. This probably created cavitation in this area resulting in the erosion of the concrete. After each high water, these eroded areas were patched with various types of concrete and admixtures but none were successful in with-standing the attack. Accordingly in 1941, steel armour plates of 12 mm. thick and 1.5 m. wide were installed on the pier walls and welded to the corner armour. A 61 cm. wide plate was also installed on the horizontal ogee surface adjacent to the wall. However, these 12 mm. plates were also found to be defective and in 1945, it was replaced with 30 mm. thick armour plate. Generally speaking these 30 mm. plates have withstood the cavitation attack during the successive years.

4.3 Another typical example of this type of damage to spillway is the case of parker dam spillway (16). The spillway of Parker dam has 5 nos. of 15.2 m x 15.2 m stoney gates to pass flood waters. The spillway was in use for a long period during the early stages when power house was not in operation and during this time the head over the spillway varied from 12m to 15 m. As a result of this continued operation, an eroded condition similar to that on the faces of the Bonoville dam began to develop on the faces of the spillway piers and on spillway crest immediately down stream from the gate slot. It first appeared below the gate which had the longest record of operation, but there was evidence of it down stream ~~flow~~ from the other gates as well. Subsequent operation of other gates had gradually developed the same pattern on all ten of pier faces.

4.4.0 Cavitation erosion of this nature is not peculiar to American dams alone, Similar cases have been reported from Europe also (22).

4.5.0 In recent years model studies, have been conducted by the ~~4.5~~ U.S. Bureau of Reclamation on most of the types of gate slots commonly employed. The tests concerned slide gate slots in particular, but the result are applicable to wheel gates also since the main difference in the two types is size rather than shape (10). Based on these experimental results, some type of gate slots, which are more likely to cause cavitation damage are briefly discussed here.

4.5.1 <u>Slot in line with the up-stream and downstream walls:</u>

4.5.1.1 One of the most common types of gate slots used in many spillways is shown in figure (11) (The dimensions of the gate slot are those used for testing). This type of slot does not have any offset, either up-stream or downstream of the slot and the upstream and downstream faces are in the same line. The tests showed the pressure of separation zone along the wall immediately downstream of the slot and the presence of complex currents inside the slot. A plot of the pressure distribution on the wall downstream from the slot showed severe sub-atmospheric pressures in the separation zone as shown in figure (12). The pressure at the down stream face of the leaf decreases as W/D increases, where W is the width of the slot and D the depth of the slot. However, this type of slots serves adequately for heads less than about 11 metres.

4.5.2 Slots with deflector on up-stream corner and parallel <u>down stream wall (Figure: 13):</u>

4.5.2.1 Deflectors at the upstream edges of the slots, produce an ejector action, which lowers the pressures at the slot for below the reference pressure at the down stream face of the gate leaf and this will induce cavitation. Similarly, the pressures, within the slot at the down-stream corner, were severely sub-atmospheric, as were the pressures below the down stream corner for large deflectors and small W/D ratios. There may be a small deflector design which would perhaps give satisfactory pressure conditions, but its size will be critical. A very large deflector, which would cause a heavy contraction can be used successfully at a slot, when aeration is provided.

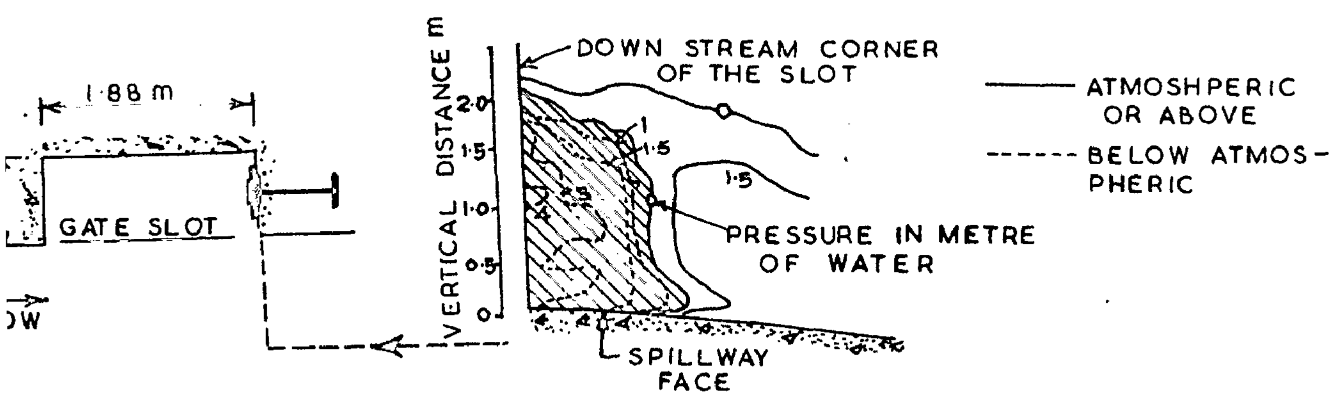
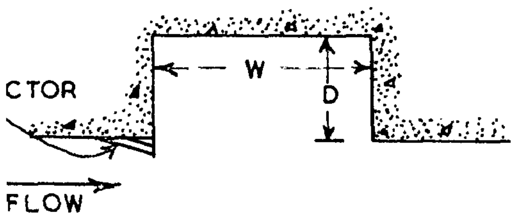


FIG.11.

FIG.12. PRESSURE ON PIER DOWN-STREAM OF SLOT



13. SLOT WITH DEFLECTOR UP STREAM CORNER WITH PARALLEL DOWN STREAM WALL

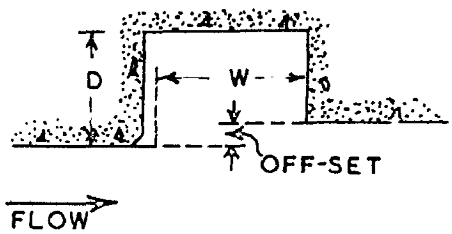
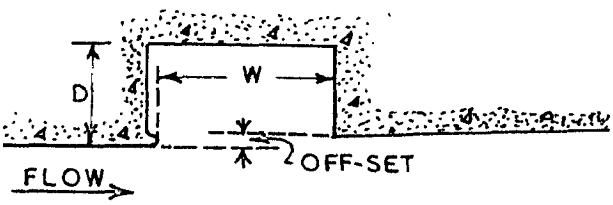


FIG 14. SLOT WITH OFF SET DOWN STREAM CORNER AND PARALLEL DOWN STREAM WALLS



15. SLOT WITH OFF-SET AT DOWN STREAM CORNER AND DIVERGENT DOWN STREAM WALL

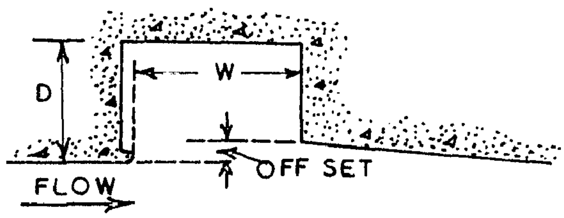


FIG.16. SLOT WITH OFF-SET AT DOWNSTREAM CORNER AND CONVERGENT DOWN STREAM WALL

4.5.3 Slots with off-set at downstream corner and parallel down-stream walls (Figure: 14) :

4.5.3.1 Slots with offset corners and parallel downstream walls will have severe negative pressures, within the slot and on the flow surfaces immediately down-stream from the slots. The pressure within the slot will be lower for large offsets than smallones, and the pressure down-stream from the downstream corner will be lower for small off-sets, than for large ones. This design appears to be adequate for small off-sets at moderate heads. For small W/D ratios the pressure within the slot at the down stream corner was severely sub-atmospheric. The pressure increased gradually as W/D increased.

4.5.4 Slots with offset at down stream corner and divergent down - stream wall (Figure : 15) :

4.5.4.1 Slots with off-sets at down stream corners and divergent walls, have low pressures within the slot ~~on~~ or immediately down stream from the down stream corner, depending on the amount of off-set. The design is only satisfactory with large off-sets at a limited range of slot widths, and small operating heads. The pressure within the slot increased slightly as W/D ratio increased. Pressure immediately below *
*downstream corner may be above or greatly below atmospheric depending on the offset and W/D ratios. Generally pressure decreased as W/D ratio increased.

4.5.5 Slots with off-set down stream corners and convergent down - stream wall (Figure : 16) :

4.5.5.1 Slots with offsets at down stream corners and constant rate of convergence of the down stream walls, have low pressures just down stream from the line of intersection of the converging and parallel walls. The pressures decrease as the rate of convergence increases and intersection of the walls may become cavitation potentials.

4.6 Similar experiments to find the cavitation potentialities of gate slots were conducted in France also, in the laboratories of Societe Grenobloise d' Etudes et d' Application Hydrauliques, Grenoble[2] and certain results agree with those obtained by U.S.Bureau of Reclamation studies. From the results of these studies it was found that the classical vertical lift gates with upstream seating and normal slots without off-sets and without any submergence at the down stream side, the cavitation no: K for inception was around 0.25 at difficult openings (i.e. for gate openings less than 0.2 and greater than 0.7 of design head) and K = 0.1 for favourable openings. With the down stream of the gate drawned, the value of K was rarely below 1. From these findings it may be possible to conclude that it would be possible to operate gates having normal slots without off-sets, without cavitation risks for higher heads, if the operation of the gate can be limited to the safe ranges i.e. between 1/3 and 2/3 and full gate openings.

5.0 CAVITATION DUE TO SURFACE IRREGULARITIES:

5.1 Experience has shown that surface irregularities such as voids, roughness, change in alignment, protrusions, undulations and abrupt off-sets can cause cavitation damage in regions of high velocity flows over spillway faces. Cavitation damage has also been known to occur at relatively small surface irregularities, where the velocities have been high. It is quite likely that cavitation damage of roughened surfaces continues its destruction far down stream from the area affected by the intial cavitation pocket. Jet action also contributes its share to the damage by washing away loosened particles. The relative destructiveness of a jet and of cavitation has been demonstrated on many occasions. Fresh mortar has been splashed on a concrete surface and allowed to remain and set up; later the surface has been subjected to high velocity flow. The mortar being merely sticking to the surface will remain even though exposed to the full force of the jet, but the cavitation caused by its presence was seen to erode away the concrete down-stream.

5.2 Cavitation and damage to the concrete surface are related to the relative height of the particles forming the boundary roughness and the thickness of the boundary layer(12) For example, if a spillway is operated at small gate openings, the average rate of flow up-stream from the gate is at a low velocity and the turbulence at the boundary is small. At the down stream end of the gate, it is another matter. A turbulent boundary layer begins to form in the vicinity of the gate. Velocities are greatest near the spillway face. Here cavitation could be easily caused by small intrusions of roughness into the flow.

5.3 Cavitation damage due to surface roughness on the face of a spillway has been reported from U.S.S.R. (74). This happened on over flow dam of the V.I. Lenin Volga hydro-electric station, which has an installed capacity of 2.3 million kw, is one of the largest hydro-electric stations in the World, and was fully commissioned in 1967. After the passing of the highest flood discharge, which occurred during

1958, cavitation damage to the spillway face was observed in the form of caverns at places of pronounded roughness in concrete. The damaged surfaces were restored by guniting and the surfaces were given a smooth finish. Inspection during the subsequent years showed no sign of cavitation damage, which itself proves that surfaces roughness on the face of the concrete was the only cause of the damage observed earlier.

5.4 The problem concerning the severity of surface roughness, which might be tolerated remains to be solved. However, the studies made by the U.S.Bureau of Reclamation, though exploratory in nature, throw some light on the subject (1: Studies were conducted with two typical specimens used in Davis dam, which had some erosion in spillway buckets due to cavitation. Two specimens were used, and the roughened surfaces used in the study have been referred to as specimen No. 1 and No. 2. The average exposed aggregate of specimen No. 1 extended about 18 mm above the lowest point of the roughened surface and that of specimen No. 2 was 6 mm. The studies were conducted in a closed conduit of rectangular cross section. However, these results can be extended to open channel flow also, as the stream velocity near the boundary are affected by a given surface in the same manner for either closed conduit or open channel flow.

5.4.1 On the basis of these studies, curves relating the pressure and shear velocity at the inception of cavitation are given in figure: (17). The various values of shear velocities were obtained by using the ~~Kevmen~~ Karmaw - Prondth equation for velocity distribution in a turbulent flow which states that:

$$\frac{V}{U_*} = 5.75 \log_{10} \frac{y}{k} + 8.5$$

where y is the distance above the bottom, k the average value of the surfaces protrusion and U_* the shear velocity. If the values of velocities at two places V_1 and V_2 are known for two values of y_1 and y_2, the term k can be eliminated in the following manner.

$$V_1 = U_* \left(5.75 \log_{10} \frac{y_1}{k} + 8.5 \right) \quad \text{and}$$

$$V_2 = U_* \left(5.75 \log_{10} \frac{y_2}{k} + 8.5 \right)$$

$$\therefore \quad V_1 - V_2 = U_* \left(5.75 \log \frac{y_1}{k} - 5.75 \log \frac{y_2}{k} \right)$$

$$= 5.75 \; U_* \log \frac{y_1}{y_2}$$

$$\therefore \quad U_* = \frac{V_1 - V_2}{5.75 \log_{10} \frac{y_1}{y_2}} \quad \ldots \quad \ldots \quad (1)$$

So, with the values of V_1 and V_2 obtained from the experiment for two values of y_1 and y_2 respectively, U_* for various pressures are evaluated.

5.4.2 It may be noted in this connection that the average velocity in a flow required for the inception of cavitation the boundary surface is dependent on the approach conditions of the channel also. A study of the velocity profile reveals that for the same average velocity, the velocity near the boundary is appreciably greater for a smooth surface than for a rough one. If the approach to the roughened area were a reasonably smooth surface sufficiently long to establish uniform flow, the average velocity of the approaching stream necessary to cause cavitation on the rough surface would be less than that computed from the shear velocity curve in figure (17). This probably explains the reason why cavitation erosion is very often seem in concrete, when preceded by a steel lining.

5.4.3. Since the velocity near the boundary is the one which would attack the roughened concrete surface, it is the one which must be calculated to ascertain whether cavitation would exist on the damage or roughened area.

5.4.4. The effect of having a smoother approach channel on the inception of cavitation on the rough concrete surface downstream can be better amplified with the calculation of average velocities for a typical case.

5.4.5. In the first case the channel is assumed to be of uniform roughness of this type in specimen No.1, let the depth of flow is 1.52 m. Then the shear velocity for the inception of cavitation is found from the graph in figures (17) to be equal to 1.28 m/sec.

Let the critical velocity at 7.5 mm above the bottom be 18 m/sec. The average velocity in the channel corresponding to this velocity of 18 m/sec. at 7.5 m:m can be obtained by making use of the equation (1) and assuming that average velocity occurs at 0.367 depth above the channel depth after Vamoni(85).

i.e.

$$1.28 = \frac{V_{avg} - 18}{5.75 \log_{10} \frac{.367 \times 1.52}{0.0075}}$$

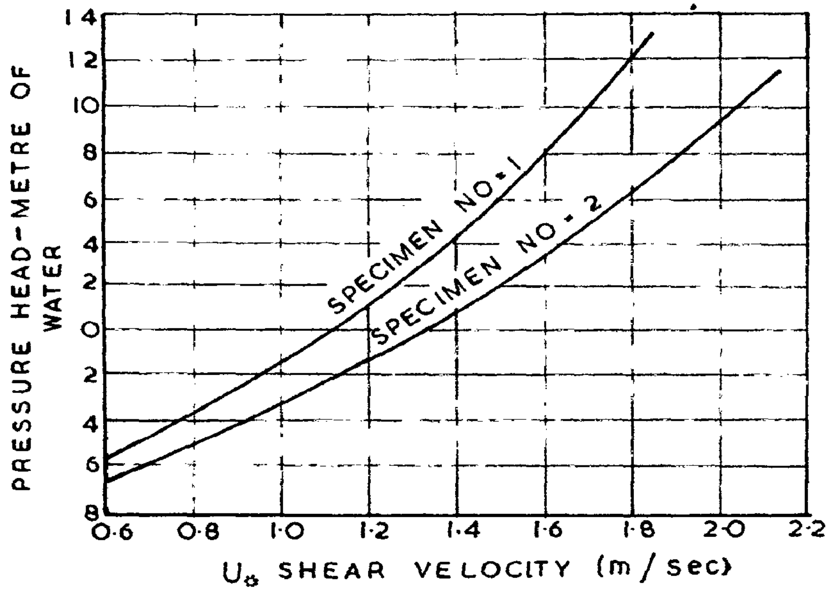


FIG.17. SHEAR VELOCITY– PRESSURE HEAD RELTIONSHIP FOR INCIPIENT CAVITATION

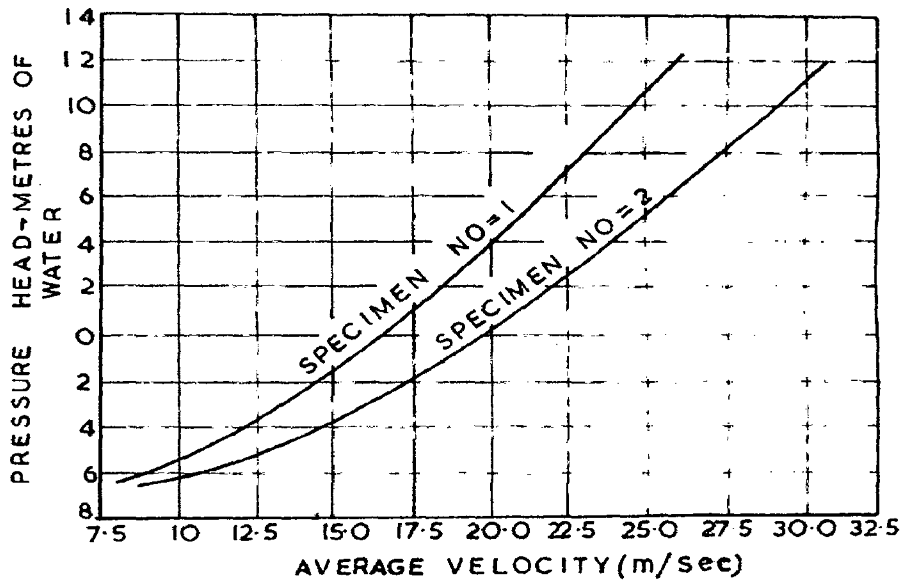


FIG.18. AVERAGE VELOCITY PRESSURE HEAD RELATION FOR CAVITATION INCEPTION

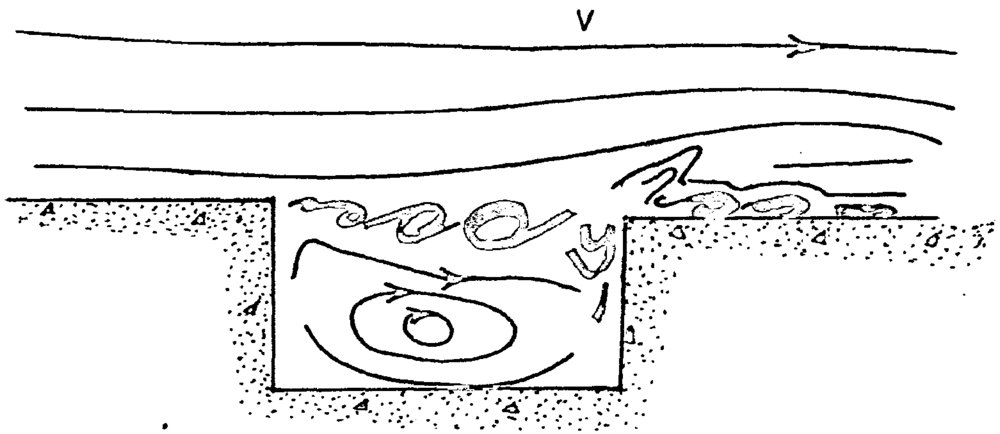


FIG.19. APPROXIMATE PATTERN OF FLOW IN A SLOT ON SPILLWAY

$\therefore$ V_{avg} = 31.7 m/sec (Average stream velocity at which cavitation will occur with specimen No. 1, with rough approach surface).

Now assuming that the approach channel to the roughened surface is smoother and has about one quarter the boundary shear of the roughened area, then

$$0.64 = \frac{V_{avg} - 18}{5.75 \log_{10} \frac{.367 \times 1.52}{0.0075}}$$

$\therefore$ V_{avg}. = 24.8 m/sec. (Average stream velocity at which cavitation will occur on specimen No. 1 with a relatively smooth approach surface).

5.4.6 In the case of flow with a blunt velocity profile, the average velocity can be considered to exist near enough to the boundary to effect the roughened area. Therefore, if the average velocity is critical, then cavitation will occur on specimen No. 1 at V_{avg}. = 18 m/sec.

Comparing the approach velocities and the average stream velocities for incipient cavitation to exist on the roughened surface in the illustration.

Approach condition	Average stream velocity
Specimen No. 1 texture	31.7 m/sec.
Relatively smooth approach	24.8 m/sec.
Blunt velocity profile	18 m/sec.

From the data obtained in the laboratory, a chart was made plotting the average velocity near the roughened surface versus depth of flow (or pressure head) for the condition of incipient cavitation for the two tested surfaces and shown in figure (18). It may be noted, however, that figure (18) shows the most rigorous conditions i.e. that of a stream flowing with a blunt velocity profile. If a stream has partially or fully developed boundary layer, and the average stream velocity is used, this chart will show pressures which are well into the safe range.

6.0 CAVITATION DUE TO SLOTS ON SPILLWAY FACE :

6.1 Some times slots are given on spillway faces especially near the crest to accommodate a gate seat or for some other purpose. Slots of this nature can also cause cavitation. Detailed studies on the cavitation aspects of these slots were carried out at the Kuybishev Civil Engineering Institute, Moscow [62]. Figure (19) shows the pattern of flow in the slot. Cavitation arises first of all inside the vortices running off the down-stream corner of the slot and then inside of the slot running off the up-stream corner. So a slot of this type has two cavitation co-efficients, one for the up-stream edge and the other for the down stream edge. The critical cavitation parameters for various types of this kind of slots have been evaluated on the basis of these studies.

7.0 CAVITATION DUE TO IRREGULARITIES ON ACCOUNT OF DEFECTIVE CONSTRUCTION :

7.1 Irregularities due to defect in construction finishes create cavitation [11]. Typical types of irregularities include abrupt off-sets into the flow, abrupt off-sets away from the flow, abrupt curvatures and slopes away from the flow, voids, roughened surfaces and protrusions. Some of the typical irregularities en-countered during construction of hydraulic structures are shown in figure[20]. Off-sets occur frequently and represent one of the most troublesome of many types of irregularities. Off-sets may be at an angle with the flow, but are usually either parallel or perpendicular to it. The perpendicular or transverse into the flow off-sets are usually the most objectionable (Figure 20 a). Off-sets away from the flow, and protruding joints (Figure 20 b & 20 f) are also objectionable. Off-sets in joints parallel to the direction of flow are not objectionable, but irregularities in the joints may be troublesome. Surfaces that curve too abruptly away from the flow (Figure 20 c) induce low pressures that can cause cavitation. Appreciable changes in slope or alignment away from the flow (figures 20 d) voids in surfaces (Bug holes), surface roughness etc will cause cavitation under certain conditions. The formation of vapour cavities in low pressure zones and cavitation damage caused by the collapse of the cavities are shown in figure (21). Air can not enter the underside of the jet just downstream from the irregularity, and thus a low pressure zone must form to cause the flow to stay in contact with the surface. The higher the flow velocity, the more the tendency of the jet to leave the surface and lower the pressures become. As a rule, the larger irregularity the lower the velocity will be to give incipient cavitation. The shape of the irregularity will also influence the velocity at which incipient cavitation will occur. Two general types of irregularities will be discussed here (i) Those that protrude into the flowing stream and (ii) those that recede away from the stream.

7.2.0 Protruding Irregularity:

7.2.1 Sharp cornered off-sets are frequently found in construction at joints in forms or joints in the surfaces (figure 20 a). Voids in concrete surfaces, transverse grooves or protruding joint are in some respects similar to direct off-sets (figure 20 f). Sharp off-sets into the flow at joints subjected to high velocity flow have high cavitation potentials. In addition, they may have the undesirable feature of subjecting the joints to extremely high impact pressures. A protruding off-set of this type shown in figure (20 f) can occur on a spillway face at joints between two successive horizontal lifts. A typical case of this is the cavitation erosion occurred on the spillway face of Grand Coube dam (50). There were bulges in the concrete face of the spillway between successive lifts. The maximum bulge was about 7.5 cm. for a horizontal lift of 1.52 metres. Deepest pitting occurred just down stream from the bulge and wilder pitting occurred further downstream. The area where the pitting occurred was about 90 metres below the spillway crest, where the velocities might have been as high as about 35 metres /sec.

7.2.2 A round cornered off-set as shown in figure (22) into the flow, i.e. an off-set corner with a rounded edge is perhaps the most common type encountered in the field because most corners are or become rounded during construction and operation. Research on these types of off-sets have shown that with a constant radius of rounding, the cavitation potential of an off-set increases as the height of the off-set increases and that with a constant off-set the cavitation potential increases as the radius of the

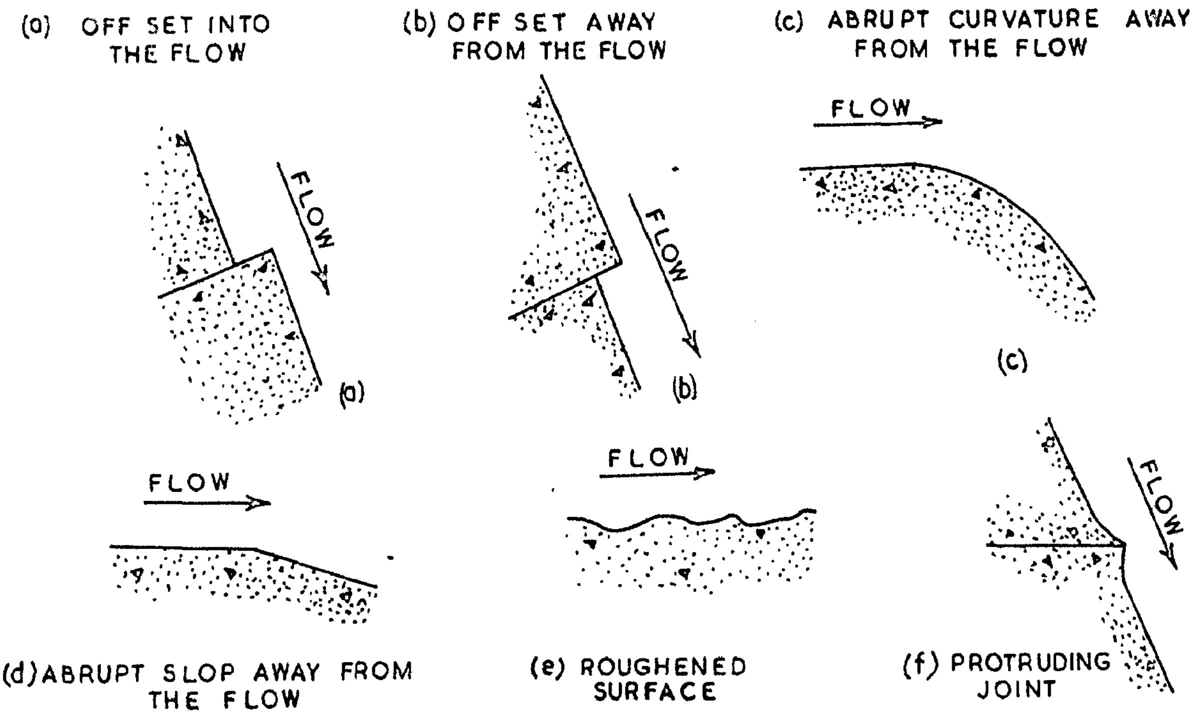


FIG. 20. POSSIBLE CONSTRUCTION IRREGULARITIES IN FLOW SURFACES

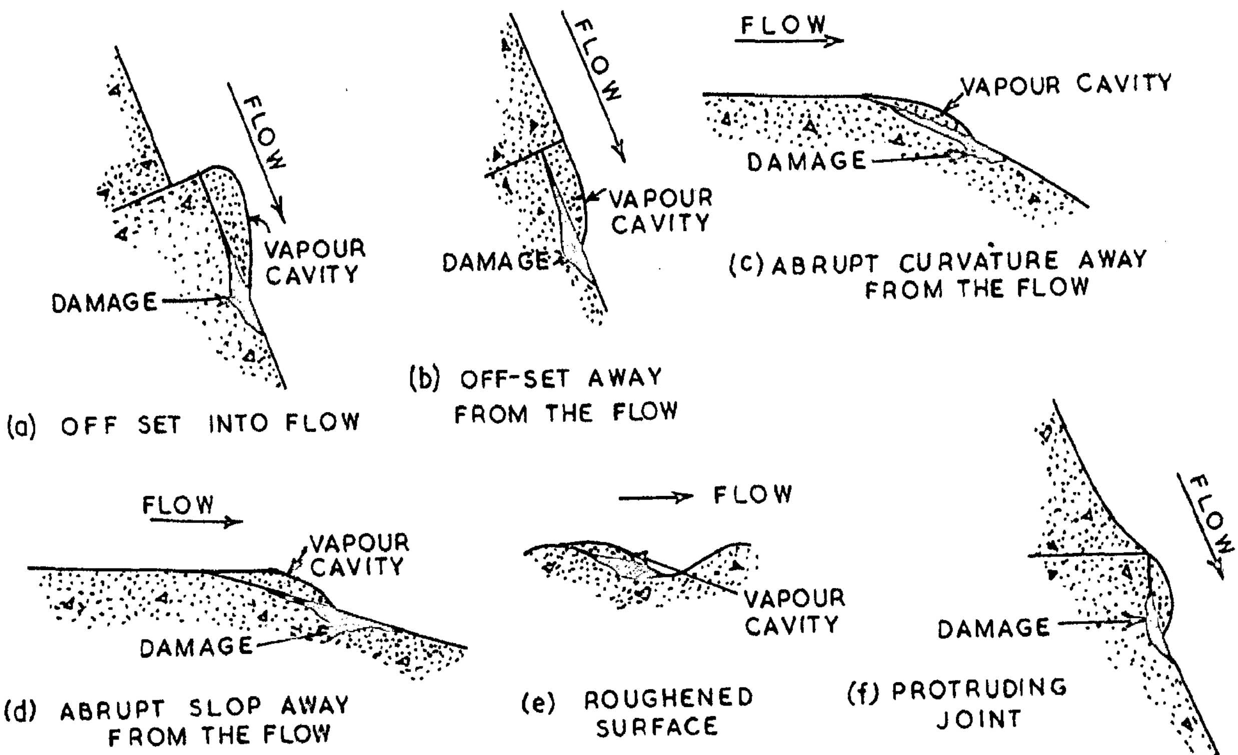


FIG. 21 POSSIBLE OCCURRENCE OF CAVITATION AT FLOW SURFACE IRREGULARITIES.

rounding decreases.

7.3.0 Receding Irregularities:

7.3.1 Sharp or squared cornered off-sets away from the flow are encountered frequently (figures 20 b & 21 b). Although serious, these type of off-sets probably do not have quite the cavitation potential as those off-set into the flow. Round cornered off-sets away from the flow are not unusual in concrete construction. The rounding in this case may worsen conditions rather than improve them as it does in the case of off-set into the flow. The off-set of rounding on the cavitation potential is not known. Off-sets that slope away from the flow are encountered occassionally on spillway surfaces (figures 20 c , 20 d 21 c & 21 d). The surface receding from the flow in this manner may have a high cavitation potential due to abrupt change in the alignment of the surface. Unless the angle is small or so large that the main flow definitely seperates from the surface, cavitation is probable. The critical angle will depend on the velocity, water passage shape, ambient pressure etc. Higher the velocity smaller the angle must be to prevent cavitation pressures.

8.0 CAVITATION DUE TO WRONG OPERATION OF SPILLWAYS:

8.1 It must be realised that for one head known as the design head, for which the spillway crest has been designed, will the pressure distribution around the spillway crest will be near ideal condition. If the design head is exceeded, the natural trajectory of this will be outside the spillway profile and would follow the established profile through a reduction in pressure along the crest. So during the operation of any spillway, it has to be ensured that this design head is not exceeded. But instances are many, where the design head has been exceeded due to wrong operation. The defect in operation may be due to human error or mechanical error on operation mechanism of the spillway gates.

8.2.0 <u>Mechanical Errors</u>:

8.2.1 It is quite probale that at the time of the peak discharge over the spillway, the operation mechanism of one or more gates may fail to function. A condition like this will naturally raise the water level over the crest above the design level and the tendency of the nappe will be to move away from the spillway face creating a partial vacuum beneath it. Because of this raising of the water level, water will flow over the crest gates, which happened to be inoperative, as shown in figure (23). A free falling nappe of this type over a spillway, immediately lowers the pressure behind the crest gates. A reduction in pressure like this may cause cavitation.

8.3.0 <u>Human Error</u>:

8.3.1 Under some flood conditions operational errors may actually increase the discharge at the dam above the design peak and consequently the head over the crest goes beyond that of the design head. By storing too early on the rising side of the hydro graph, available storage space may be filled in advance of the peak inflow. A situation like this may cause cavitation.

8.4.0 <u>Simultaneous Operation Of The Spillway And ~~Studies~~ Sluices</u>:

8.4.1 Cavitation damage had occurred to the Folsom Dam in U.S.A., when its spillway and sluices were simultaneously operated[12]. Folsom dam has a spillway of ogee type with crest elevation of + 127.5 m. The drop from the crest to the stilling basin is 92.0 m. maximum. Normal flow regulation for irrigation and flood control is accomplished by eight 1.52 m. wide by 2.74 m. high curved sluices through the spillway section. These sluices are in two tiers four each at elevation + 64.00 m and + 85.00 m. respectively as shown in figure (24).

8.4.2 The passage of flow through the sluices alone caused no problems. Flow passed smoothly through the sluice and down the spillway face. However, during the flood of December,1955, the flow was passed over the spillway at a head of as much as 11.25 m. at the same time when the flow was passing through the sluices. The condition existed for approximately 7 days. When the flow over the spillway ceased, damage to the exist portals of the top tier was immediately noticed. Closer examination revealed a definite line of damage on each side wall beginning at the block-out section and extending to the spillway face. It appears that the abrupt change in the alignment at the block out section caused low pressure areas to extend along the line of the damage. In the case of Folsom dam, the height of the block out section is 1.22 m. It is believed that had this height been less, damage would not have been as severe. Damage was also experienced on the spillway face immediately down stream from the sluice exit.

8.4.3 A similar situation occurred at Table Rock in Missouri during construction. From May 24,1957 to July 15,1957, flood caused flow to over top one of the low monoliths at the same time that one of the sluices was discharging. Observations

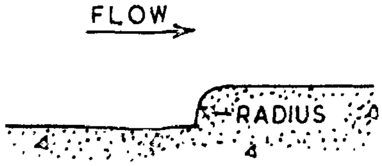


FIG. 22. ROUND CORNERED OFF SETS INTO THE FLOW

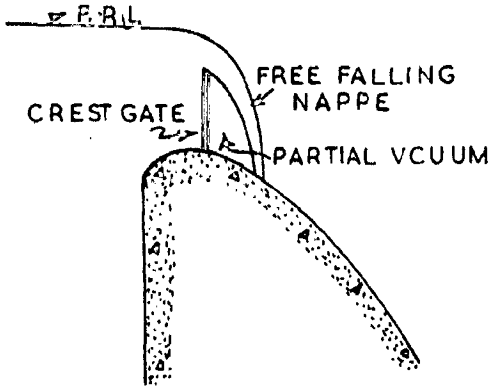


FIG.23 FLOW OVER A SPILLWAY WITH THE GATE INOPERATIVE

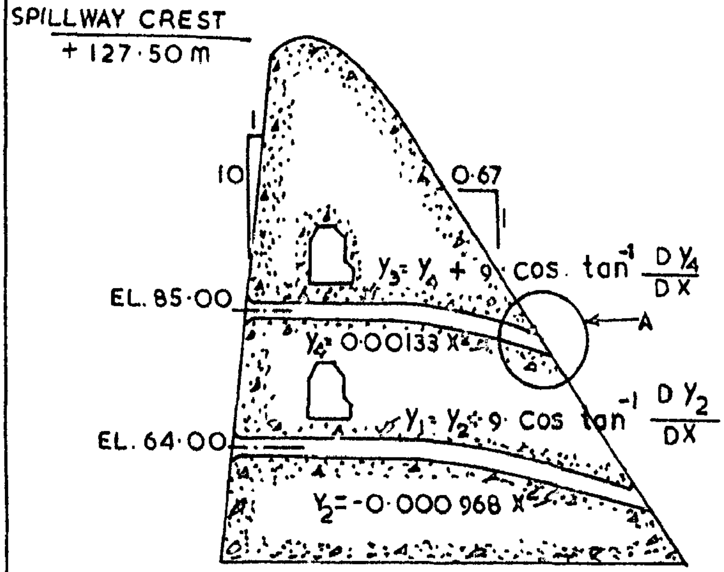
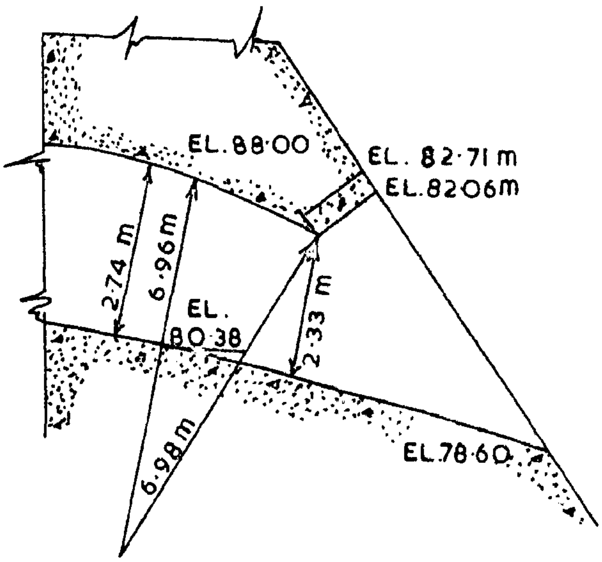


FIG.24. FOLSON DAM SPILLWAY



DETAILS AT A

indicated damage to the concrete at exit portal, resulting from simultaneous flow through the sluice and over the low monoliths. The eroded areas are approximately 0.60 m. high and 1.35 m. long. They have maximum width of approximately 10 cm. and depth of 4 cm. No erodued areas were noted at the exit portals of conduits discharging along; thus indicating that damage from cavitation action was caused by the over topping of the low monolith.

CHAPTER: 5

ANTI - VACUUM REMEDIES

1.0 INTRODUCTION:

1.1 The development of sub-atmospheric pressures or partial vacuum between down stream face and under side of a nappe can create undesirable conditions.

1.1.1 The resultant force on the spillway section may be increased due to this reduction of the back pressure.

1.1.2 An increase is the lifting forces in the case of gates.

1.1.3 The sub-atmospheric pressures can develop cavitation. The result will be the erosion of the spillway face, due to pitting. This can also create vibration. While the amptitude of the vibration may be small, the accumulation of the forces within the dam can cause secondary forces, particularily if the natural frequency of the structure bears a particular relation to that of the vibration of the nappe. These vibrations are also destructive to the mortar and therefore can cause cracks that necessitate special anchoring bolts to stone linings in the case of masonry spillways.

1.2 Among the three possible harmful effects that may cause damage to the spillways, the damage due to cavitation is by far the most harmful. Having discussed briefly in Chapter:4, about some of the causes that may create cavitation, the purpose of this chapter is to suggest some possible remedies to the prevention of cavitation. The best method, however, of profecting a spillway from the forces of cavitation is by the proper design to eliminate these forces followed by suitable construction methods so as to obtain a smooth and uniform surface for the flow of water. The sloping faces of over flow spillways have not been damaged by cavitation, where proper care has been taken to produce surfaces of sound concrete without irregularities.

2.0 ANTI-VACUUM DESIGN :

2.1 As is known, a spillway profile is usually designed for the best hydraulic efficiency i.e. to have the co-efficient of discharge as large as possible without really creating sub-atmospheric pressures. Some times, some other factors are also taken into consideration in shaping the spillway crest. The condition normally demands that for the maximum possible of over flow, the pressure in each vertical be distributed in such a way that no sub-atmospheric pressures can arise at the surface. At the same time, no considerable positive pressures are also desirable at critical points, as other wise the solution will be too un-economical. If the curvature is too slight in relation to the height of the overflow, the pressure at the crest rises as shown in figure (1.a). If the curvature is of the correct type, the pressure at the crest becomes zero as shown in figure (1.b) and if the curvature is too sharp, negative pressure results as shown in figure (1.c). If we can allow some sub-pressures, which do not cause cavitation we can get an efficient hydraulic profile, which would be economical and give

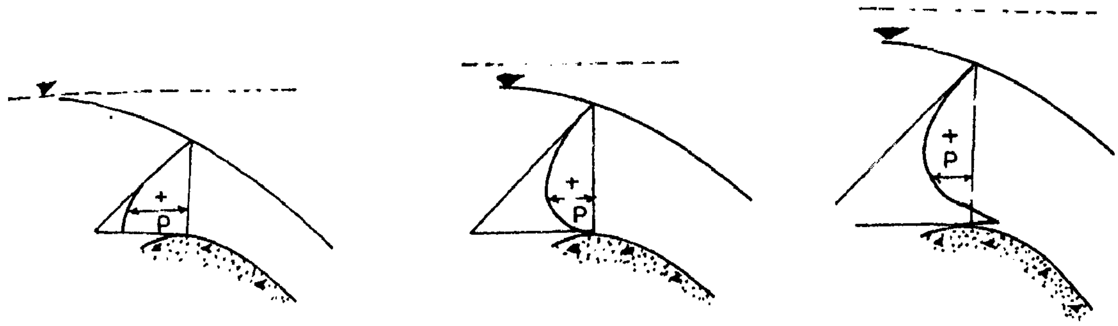


FIG.1 (a) FIG.1 (b) FIG.1 (c)

PRESSURES ON FLAT IDEAL ACTUAL- PROFILES

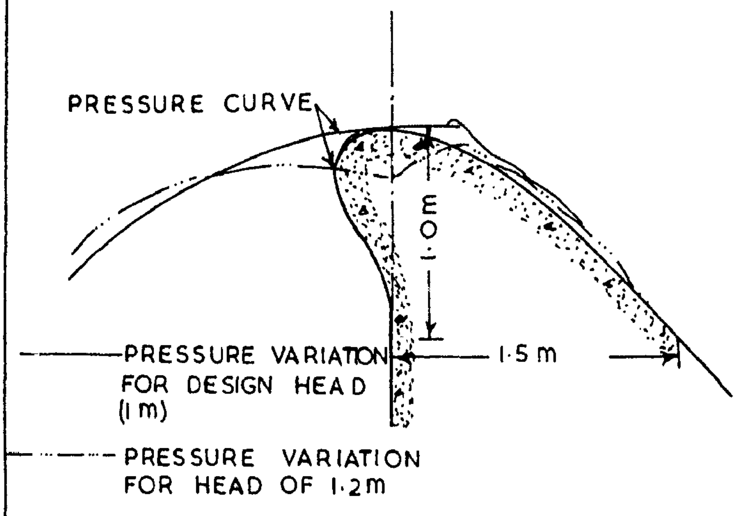


FIG.2. PRESSURE CURVE FOR NORM PROFILE III

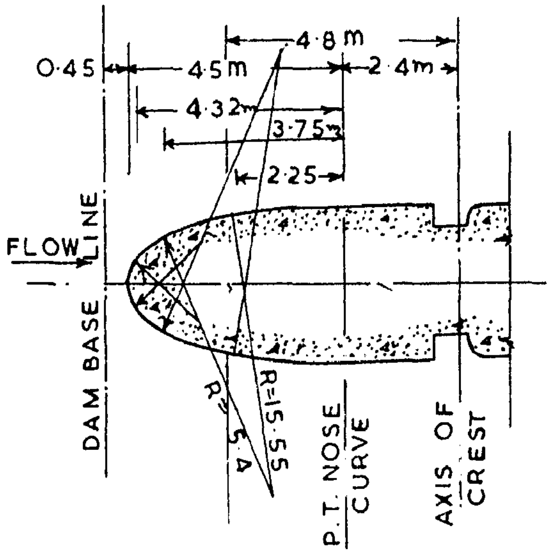


FIG.3. PIER NOSE SHAPE OF MCNARY DAM

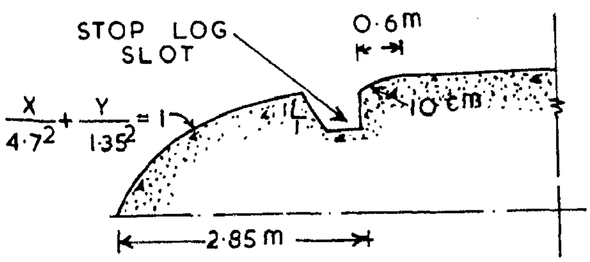


FIG.4 SPILLWAY PIER NOSE-CHIEF JOSEPH DAM

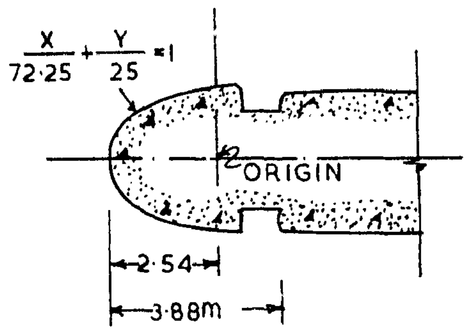


FIG.5 PIER NOSE-DALLES DAM SPILLWAY

high coefficient of discharge.

2.2.0 Of late, Prof. Antoni Grzyweinski of the University of Vinna has made lot of studies on the aspect of designing suitable spillway profiles for various conditions. He has suggested few profiles known as " Anti - Vacuum profiles"(30) He has classified the general profiles used in practice into 3 groups (depending on the height) as follows and calls them as Norm Profiles I,II and III.

2.2.1 Norm Profile : I

2.2.1.1 This profile is obtained, if the air filled space under the nappe of a sharp crested over fall is replaced by a solid wall. In this, the curve of the overfall starts discontinously at an angle. He recommends this for use for very low dams.

2.2.2 Norm Profile: II

2.2.2.1 This profile is developed by means of tangential transition from the vertical to the overfall curve. This is to be used in those cases in which for some reason a cantilever crest as provided for in Norm Profile III can not be used as for instance in the case of the overfall sheets of movable weir crests.

2.2.3 Norm Profile: III

2.2.3.1 This is developed through an uniform continuation of the crest curve beyond the vertical tangent as shown in figure (2). This is also recommended for raising the existing capacity of a spillway without adversely affecting the pressure distribution on the crest. The pressure distribution for this profile designed for a head of 1 metre is shown in figure (2). All the same, advantage in resorting to this type of profile over the other widely used practical profiles such as U.S.B.R. , W.E.S. etc. are not very convincing. For, it may be noted from the figure (2) that for an actual head of 1.2 m, i.e. when $\frac{H}{Hd} > 1$, negative pressures of about 0.25 m is seen at the crest.

3.0 DESIGN OF THE UP-STREAM QUADRANT:

3.1 As has been amply explained in Chapter: 4, any abrupt change in curvature is to be avoided to have a cavitation free profile. However, it may be seen that almost all the profiles, which are widely being used, the tendency is to provide a compound curve for the up-stream quadrant, probably with the only exception of the design followed by the U.S.Army Corps of Engineers. Even the U.S.Army Corps of Engineers were making use of a compound curve, using the radii equal to 0.2 H_d, and 0.5 H_d where H_d is the design head over the crest. The revised equation now being used, was developed by the U.S.Waterways Expériment station, making use of the co-ordinates computed by McNoun, Hsu and Yih (46), which were found by employing and relaxation techniques to determine the path of the jet in a region close to the up-stream face of the dam, where precise experimental measurements would have been difficult to obtain. By eliminating the surface of discontinuity at the vertical spillway face inherent in the older crest shape, the new shape is believed to provide more favourable pressure conditions, especially for the heads in excess of the design head. It was also reasoned that with the older criteria, some seperation may occur at the discontinuity resulting in relatively rapid development of the turbulent boundary layer, an increase in the amount of air entrainment and probably a need for higher side walls in some designs.(47)

3.2 The universal practice of designing a spillway profile is to fix the various co-ordinates, for a particular design head making use of one of the standard profiles given in Chapter: 2, After having fixed the co-ordinates like this, the next step will be to verify whether there is any adverse pressure variation on crest for any condition or opeaation, with the half of hydraulic models. But any altogether different approach to the problem has been suggested by R.Lemonie (42). His method is to allow certain amount of sub-atmospheric pressure on the crest to increase the co-efficient of discharge and at the same time keeping the pressure above the cavitating range by introducing the cavitation parameter K also in the design.

3.2.1 For evaluating the cavitation parameter on the crest he considered the shape of the crest also, by introducing a dimensionless term π, which is a function of s/H, where s is the curved distance of the crest from a reference point, and H, the head of water over the crest, such that at any point on the crest the average pressure $H_{av} = \pi H + h_t$. h_t is the atmospheric pressure. The corresponding value of velocity is in the form of:

$$V = \sqrt{2g \vartheta H}$$, where ϑ is also a function of the same nature as λ. Hence, using Bernoulli's equation the value of H_{av} can be calculated, if A is the elevation of the point below the up-stream water level;

i.e. $$V^2/2g = A + h_{at} - H_{av}.$$

$$\therefore \quad V = \sqrt{2g\ (A + h_{at} - H_{av})}$$

$$= \sqrt{2g\ (A - \lambda H)}$$

$$\therefore \quad \vartheta = \frac{A}{H} - \lambda$$ and hence the cavitation parameter K at any point on the spillway.

$$= \frac{\lambda H + h_{at} - h_a}{H}$$

$$= \frac{\lambda}{\vartheta} + \frac{h_{at} - h_a}{\vartheta H}$$

Experimental studies were made to find the relation between the co-efficient of discharge:

$$Ca = \frac{Q}{L \cdot \sqrt{2g}\ H^{3/2}}$$, K , and head of water of the crest H,

On the spillways of Castillon and Cosaque dams in France, where the co-efficient of discharge was 0.53* for a head of 4.5 m in the case of Castillon and 0.53 for a head of 3.54 m and 0.54 for a head of 3.75 m in the case of cosuque dam spillway. The results of these experiments done for

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* The co-efficient discharge in these studies correspond to Q

different water heads over the crest is shown in figure (25.a) It shows the value of K for different heads along the surface of the profile. Making use of the curve available in figure (25.a), curves relating K_{min} and head of water over crest are traced for the two given discharges of 0.53 abd 5.4 as shown in the figure (25 b). If studies of this nature are carried out on different types of profiles, a family of curves of different values of co-efficient of discharges can be plotted on this figure (25.b). From these curves the most suitable profile which gives the best co-efficient of discharge for a particular maximum head and K_{min} can be easily found out.

3.2.2 Studies on a typical Creager profile were also done and similar curves, but different in nature were obtained relating cavitation parameter K, co-efficient of discharge and the head of water over the crest, as shown in figure (26). According to Lemonie, a spillway profile with K_{min} = 0.5 will be hydraulically efficient and at the same time free from cavitation risks. Even though no elaborate design curves have been developed by Lemonie for generalising the design, the idea does give a new approach to the design of spillway crest incorporating some cavitation risks.

4.0 DESIGN OF THE PIER NOSE:

4.1 The shape of the pier nose is equally important in eliminating sub-atmospheric pressures along the pier and adjacent spillwayfaces. It is all the more important due to the fact that the lowest pressure on a spillway

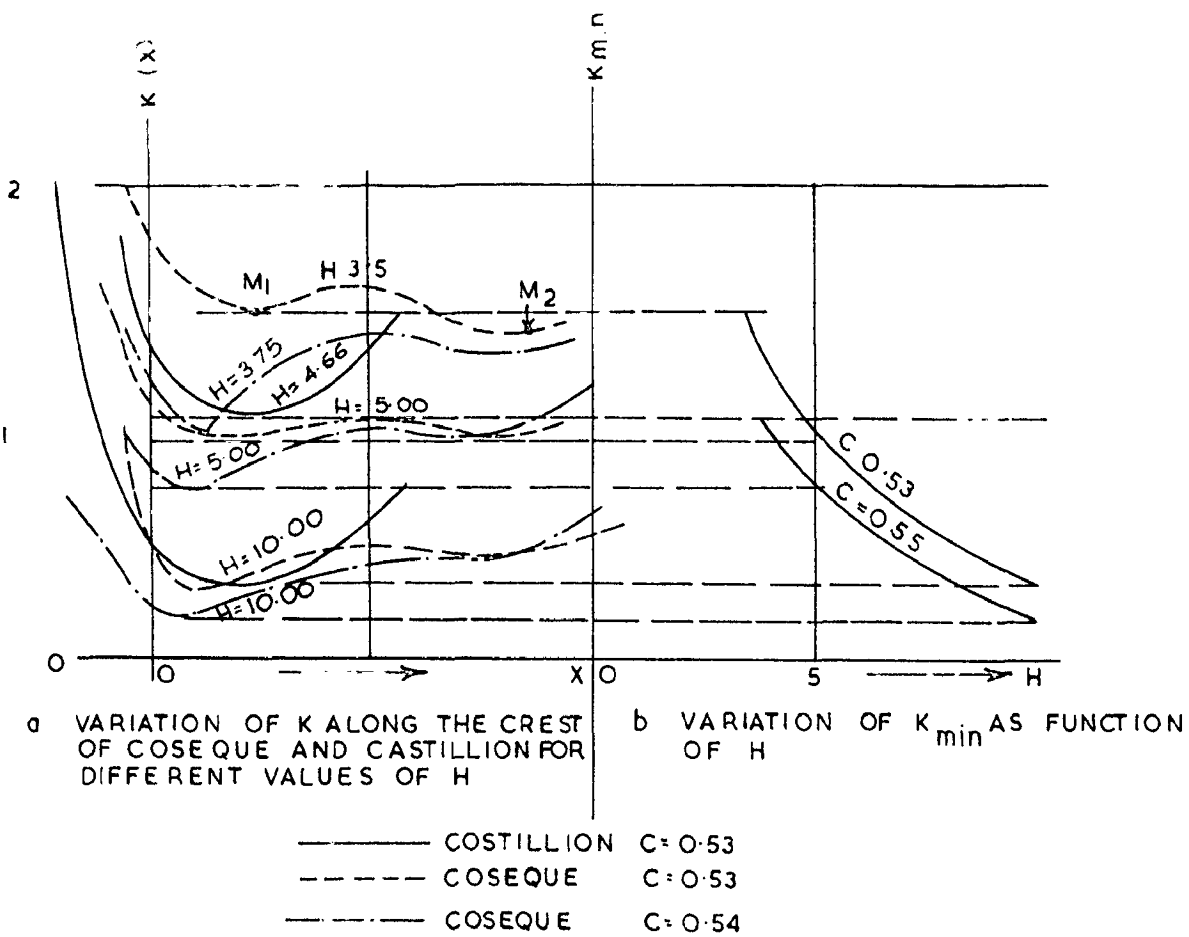
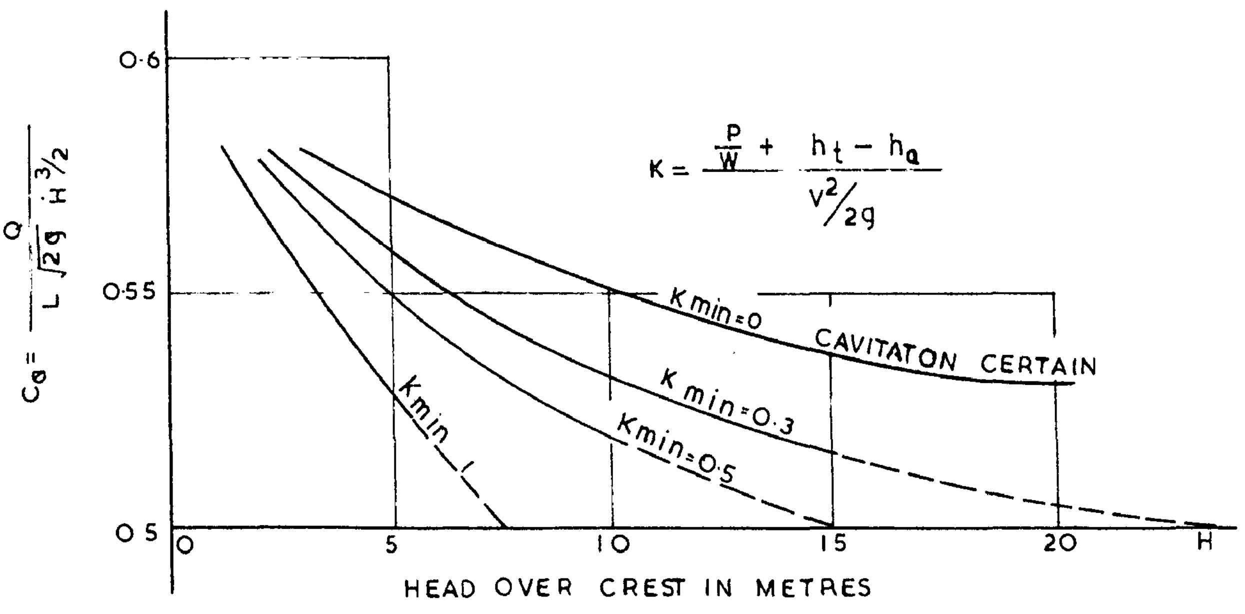


FIG. 25.



h_t – ATMOSP. PRESSURE
h_a – WATER VAPOUR PRESSURE
IN TERMS OF HEAD OF WATER

V, P – VELOCITY AND PRESSURE ABOVE THE ATMOS PRESSURE AT A POINT ON THE PROFILE

FIG 26 APPROXIMATE CURVES GIVING THE MAX CO-EFF OF DISCHARGE REALISABLE AS A FUNCTION OF WATER HEAD occurs at the corner of the pier and ogee face as seen from figures 31,32 & 33 in Chapter: 2. However, a thorough general study on the shape of the pier noses seems to be locating, at least from the cavitation point of view, even though the practice seems to be to analyse its influence on pressure variations for individual cases. Perhaps this appears to be reason that various shapes are seen adopted for different projects, even though they were constructed by the same organisation. Obviously the reason being that, though practically all of them are streamlined in shape, they show different characteristics for different spillways. To illustrate this point better, some typical cases from the dams constructed by the U.S. Army Corps of Engineers are cited here.

4.2 By and large, U.S.Army Corps of Engineers used to adopt semi-circular pier noses for the projects under their control[88]. However in the case of McNary Dam, an aerafoil shaped pier nose as shown in figure(3) was adopted as this was found to cause the minimum wave formation and vortex action. The proto-type test on this spillway showed the minimum pressure on the spillway as 0.42 m of water when the spillway was operated for a head of 13.6 m i.e. 68% of the maximum head. In the case of Chief Joseph dam tests in the model with several variation of pier noses and gate slots were tried in an efforts to eliminate the need of armour plating down-stream from the gate slots to protect the piers from cavitational damage. However, efforts to eliminate cavitational pressures, When the maximum discharge was 36,000 m^3/sec.

were found futile and an elliptical shaped pier nose as shown in figure (4) was provided without armour plate down-stream from the gate slot, as it was considered that it would be more economical to repair the piers in the case of damage from such high flows of infrequent occurrence. In the case of Dalles dam spillway, the pier nose is in the shape of 1.7:1 ellipse as shown in figure (5). However, in the case of Detroit and Cougar spillways, semi-circular pier noses were provided, which showed no adverse pressure variations.

4.3 Even though the earlier practice of the U.S.Corps of Engineers has been to provide semi-circular pier noses, the latest trend seems to be in favour of elliptical shape or some adoption of the elliptical shape for pier nose and abutment curves (88). Ratios of semi major to semi minor axes have been as high as 3.0 or more. Even though most of the spillway pier noses are stream lined, situations had actually arisen, where blunt faces had to be provided for piers. For example in the case of Dworshark dam, which is being constructed now, the width of the spillway pier is fixed as 6.71 m, as it had to accommodate the emergency gate slot for the sluice. Because of its unusual width it was not possible to have an elongated pier nose. So circular pier and abutment were proposed and tested. For the maximum flow the minimum abutment pressure was-7.62 m. So an elliptical abutment curve, having a semi-major axis of 7.62 m. and semi-minor axis of 2.54 m. was provided and the pier face left as blunt faced with elliptical curves similar to the abutment provided on either side. This arrangement produced a minimum pressure of - 3.35 m. of water for the abutment and -1.22 m of water for the piers which were considered safe.

4.4 Perhaps one of the most extensive studies for the design of pier nose has been done in India in connection with experiments in Rihand dam spillway. In these studies as many as 12 different shapes were tested in hydraulic models. It is interesting to note that a pier nose in the form of a reverse parabola, as shown in figure (27) gave the best results from the hydraulic point of view. But it could not be adopted as it was found structurally not feasible. The final design adopted is shown in figure (28). In the case of abutment curves an elliptical shape conforming to the equation $x^2/50^2 + y^2/10^2 = 1$ was adopted for the left abutment. However, on the right side it was not possible to give this type of transition, because of the structure for supporting the transformer and hence a circular quadrant of radius of 3.84 m was adopted.

5.0 GATE SEAT LOCATION :

5.1 As has been discussed in Chapter : 4, the position of the gate seat does play an important role in the pressure variations of the spillway crest. It was also seen that different locations of the gate down stream of the crest did not make much difference between them, as far as the pressure on crest were concerned. So it is absolutely essential that the gate seat is located downstream of the crest. The Indian standard specifications also stipulate that the gate seat has to be located in such a way that the vertical tangent, touching the skin plate of the tainter gate, should not go beyond the spillway crest towards the up-stream side[90].

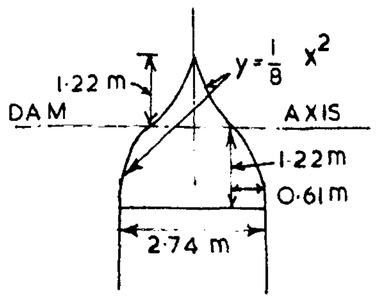


FIG. 27

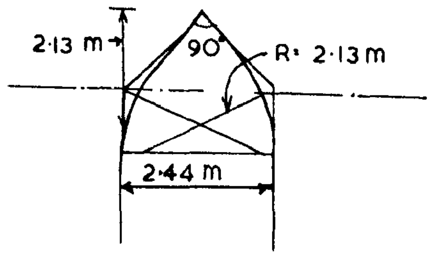


FIG. 28 RECOMMENDED PIER NOSE FOR RIHAND DAM

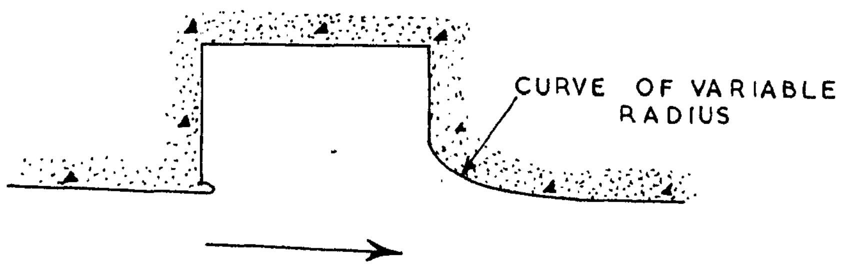


FIG. 29. TYPICAL DESIGN FOR A GATE SLOT

Even though a gate seat location down stream of the crest will suffice for all practical purposes, a more rational method has been suggested by Parisut and Michel[57] in fixing the location of the gate seat. They recommend the use of cavitation parameter K, having already evaluated the pressure variations and velocities for different gate seat location from experiments conducted for the particular profile. Hence the cavitation co-efficient $K = \dfrac{p/w + h_t - h_a}{V^2/2g}$

where,

p/w	=	minimum pressure head on spillway due to any position of the gate seat location.
h_t	=	atmospheric pressure
h_a	=	Vapour pressure of water
$V^2/2g$	=	Velocity head at the point of minimum pressure.

The cavitation factor has an advantage of stating not only the sub-atmospheric pressures that could induce cavitation, but also the velocity head that could be transferred locally into additional negative pressure behind an irregularity of the surface. For spillway, they recommend the same criteria put forward by Lemonie [42], which is as follows:

K	= 0	- cavitation is certain
K	= 0.5	cavitation damage exists, but can be tolerated if spillway face is constructed very smoothly so that no irregularity will be present and if the spillway will operate conditions giving these low value of K only for a small percentage of time.
K	= 1.0	- No danger of cavitation.

6.0 DESIGN OF GATE SLOTS:

6.1 Having already discussed about the types of gate slots on spillway piers that may cause cavitation and possible danger to the piers and spillway surfaces, a review is made here on the types of slots which will not give rise to cavitation. Based on the studies of U.S.Bureau of Reclamation the following types of shapes can be used without the risks of cavitation (10).

6.2 Slots with off-sets at the downstream corners and constant rate of converging walls with a rounded in tersection will be free of cavitation at rather high heads. However, the exact heads upto which this type is suitable have not been found out for which further studies appear to be necessary. It is best to use fairly rapid rate of convergence in so far as the pressure within and just down stream from the slots are concerned, but the rate of convergence should not be rapid enough to cause difficulty in adequately rounding the line of intersection. A 24 : 1 rate of convergence with a 30 cm. rounding is considered adequate for moderate heads, but a larger radius would be more desirable.

6.3 A slot design using off-set corners and a variable rate of convergence is the most desirable from the hydraulic stand point. A typical gate slot of this type is shown in figure (29). Arcs used in this design should have radii in the range of about 250 times the off-set of downstream corner. Ellipses can also be used with excellent results. The up-stream corners of the gate slot should not be rounded or notched, both are detrimental to the pressure distribution. The widening of the slots permits more expansion of the jet into the slot, tending to increase the concentration at the down-stream corner. However, pressure conditions are acceptable ~~to~~ for a wide range of W/D ratios in design using off-set corners and with converging walls. This is particularly true for the 24:1 convergence and long radius of curved convergence. Sharp down-stream corners of the gate slots should always be off-set away from the flow. The off-set of the down-stream corners of a gate slot should be small and related to the slot width within reasonable limits, this off-set is not critical.

6.4 Another method of avoiding cavitation due to gate slots is to provide slot fillers[87]. By this method the end beams of the gate are extended down into recesses within the spillway crest, when the gate is closed and rises with the leaf to fill the slot section to form a smooth, continuous flow boundaries on which no sub-atmospheric pressures or cavitation will occur. Evidently this kind of extension of the end beam is possible only where the spillway crest is far above the tail water to allow for drainage of water. The ideal design of this type would be the one which filled the slots at all openings. But in many cases this may not be possible from structural point of view. However, for a gate height of 16 m , 2 m long slot filler has been structurally found feasible. Even though these type of fillers have been successfully used to eliminate flow disturbances created by slots, fillers are expensive and complicate the design of the gates. Moreover, wells to receive the followers may become plugged with debris and may present serious drainage problems. In cases, where these limitations can be overcome, and where slot fillers produce truely flush smooth flow surfaces, they can entirely eliminate the danger of cavitation at slots.

6.5 Though suitable gate slots on the lines described above may be adopted for any future projects, situations may actually arise, when it will be required to modify an existing slot, which was already badly designed. So the problem will be to make some modification, which will not be difficult to construct with the structure already in existence. In a situation like this, the U.S.Bureau of Reclamation suggests a modification similar to that shown in figure (6). Pressure from the slot is also shown in figure (6). For the piers which are already under construction the Bureau recommends the type shown in figure (7). For future projects, the recommendation of the Bureau is to have a slot of the type shown in figure (8).

6.6 In the case of dams constructed by the U.S.Army Corps of Engineers, it has been the more or less accepted practice to provide a convergent wall with a slope of 12:1 and an arc connecting the off-set to the down-stream corner of the slot and the sloping converging wall. They have also provided gradual convergence of the down stream wall with off-set by a single arc joining the off-set and the down stream parallel wall.

7.0 AVOIDING CAVITATION DUE TO ROUGHENED CONCRETE SURFACES:

7.1 Design of a suitable ogee profile, free from negative pressures, is only part of the problem and this suitable design must be followed by smooth construction finishes. It is a well known fact that if a concrete surface is subjected to too much of surface roughness in a high velocity flow, it will not be free from cavitation potential. However, the problem concerning the severity of surface roughness, which might be

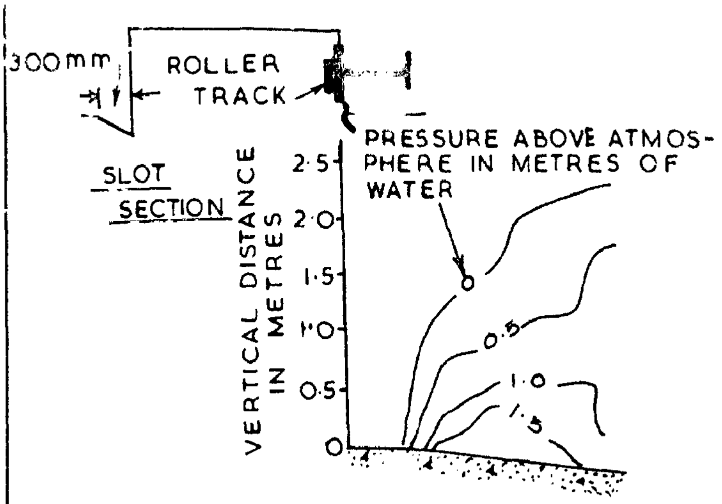


FIG. 6. CORRECTIVE MEASURES FOR EXISTING GATE SLOTS

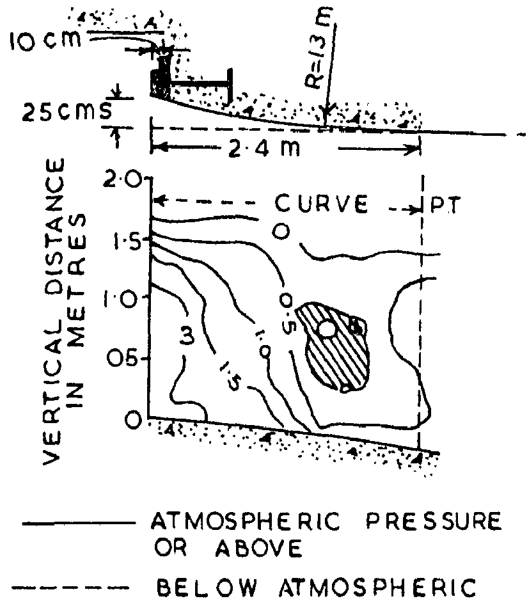


FIG. 8. TYPICAL DESIGN FOR A GATE SLOT

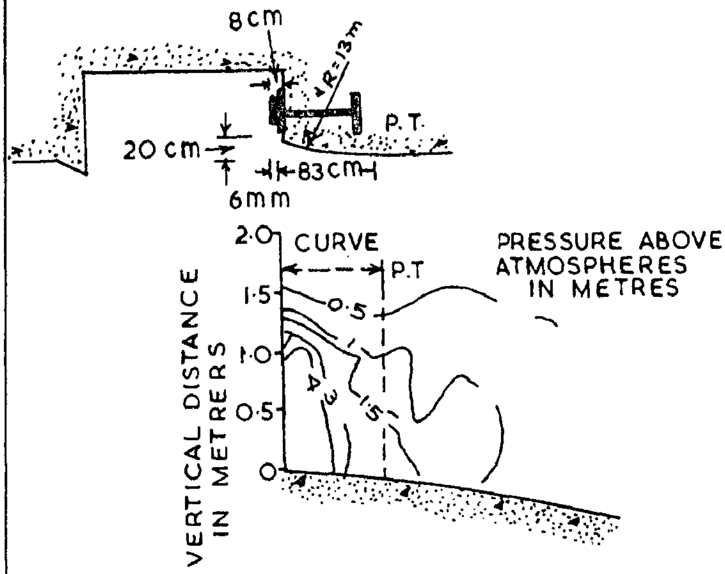


FIG.7. DESIGN OF GATE SLOTS FOR STRUCTURES UNDER CONSTRUCTION

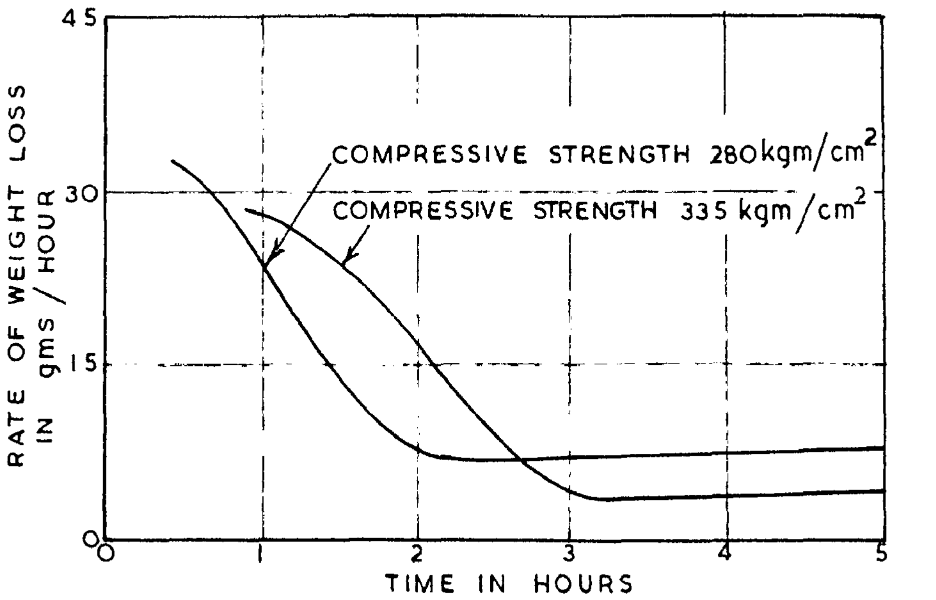


FIG. 9. VARIATION OF RATE OF DAMAGE OF CONCRETE WITH RESPECT TO TIME

tolerated still remains unsolved. If more curves are developed between pressure head, and velocity for incipient cavitation for various kinds of surface roughnesses similar to the figure (18) shown in Chapter:4, specifications for construction finishes can be made more exact than now being made based on the rule of thumb.

7.2 In this connection it may be worth mentioning the studies made by Thiruvengadam at the Indian Institute of Science, Bangalore on the effect of surface roughness on cavitation damage (76). Figure(9) shows that the intensity of damage (rate of weight loss per hour of test) is maximum at the beginning of the test on a natural concrete surface (compressive strength of concrete is shown in figure itself). The rate of damage falls down and reaches a constant value after a particular time depending on the compressive strength. A granite block, having a compressive strength of 1150 kgm/cm^2, was polished till the surface was polished till it becomes glassy in appearance was tested. Figure(10) shows that how the damage develops and falls down after prolonged testing. Then different surface roughnesses were produced varying from 0 to 6 mm on the same granite and each specimen was tested for 3 hours and figure (11) shows the variation of the rate of damage with the surface roughness. From these experiments two points are clear. One that the intensity of cavitation damage on polished surfaces increases with time and then decreases. The initial increase in the damage is due to the progressive cracking on account of repeated collapse pressures of the bubbles. The decreasing trend may be due to the cushioning out of the shocks by water filling the pits already formed. Therefore it is clear that polishing the surface

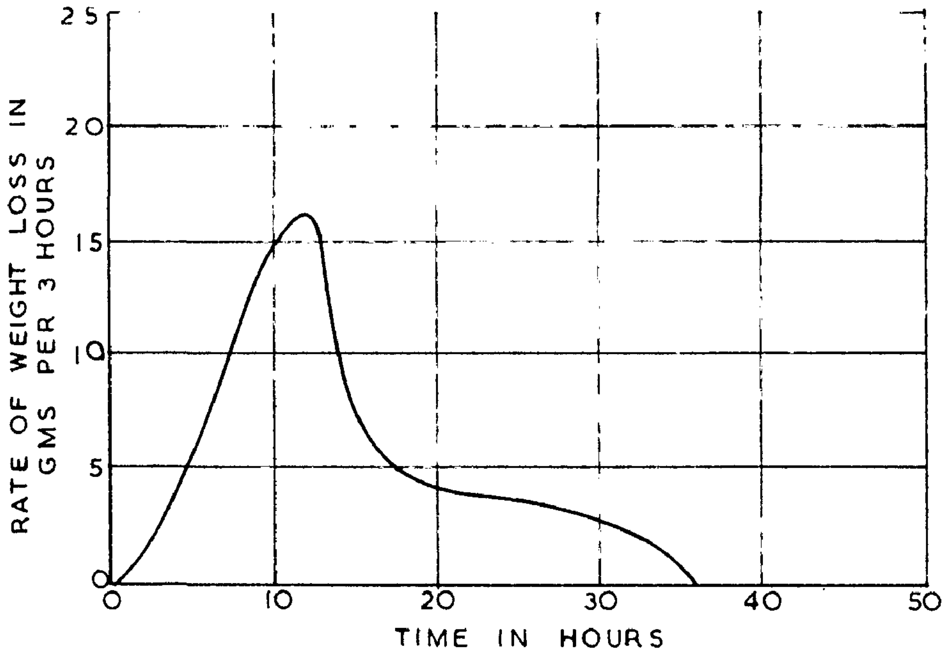


FIG.10. RATE DAMAGE OF HIGHLY POLISHED STONE WITH RESPECT TO TIME

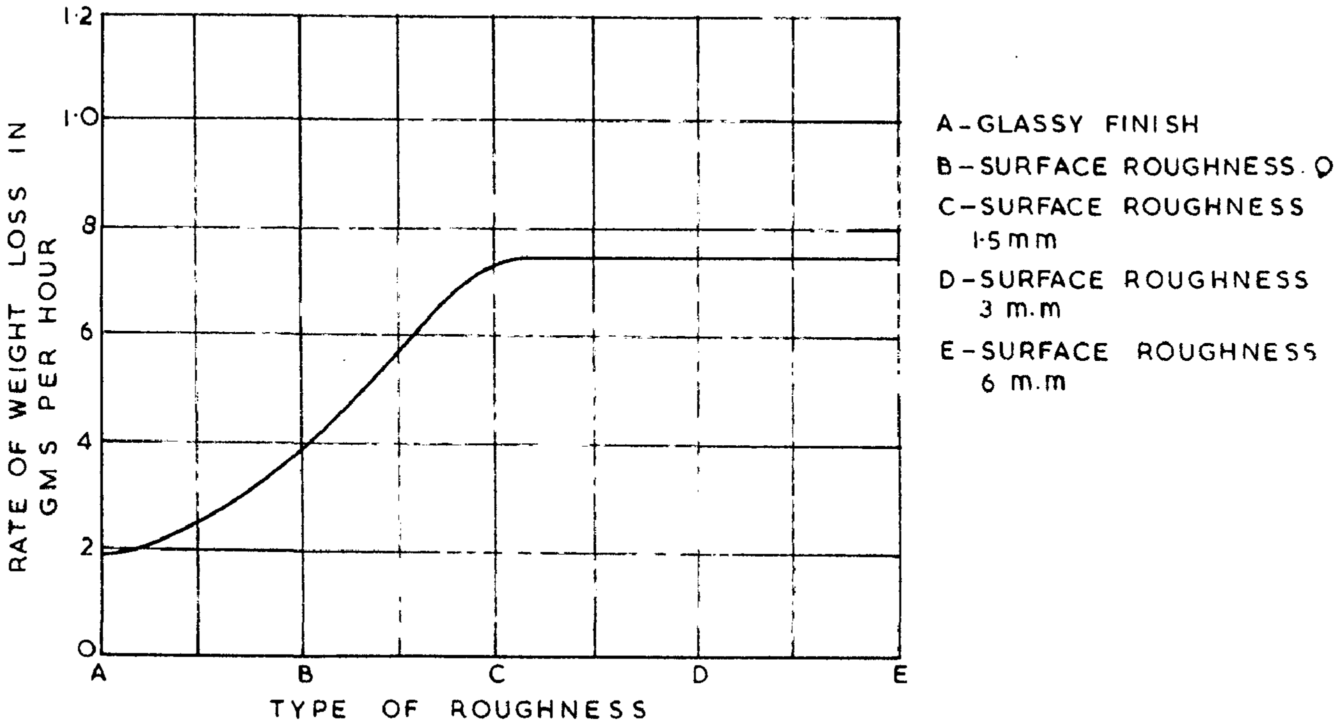


FIG.11. EFFECT OF SURFACE ROUGHNESS ON DAMAGE FOR GRANITE STONE

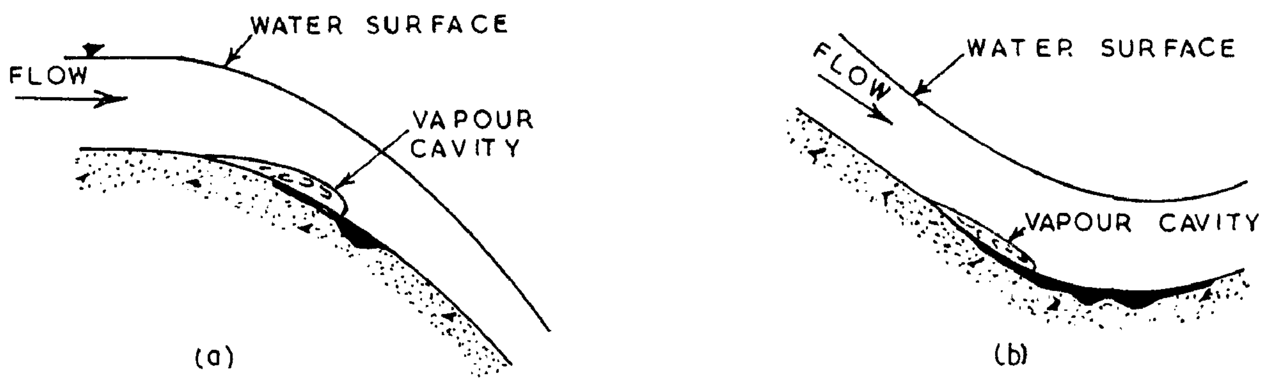


FIG.12. CAVITATION ON CURVED SURFACES

is only a temporary remedy. Second point is that roughness as such has no effect on cavitation damage beyond a certain limit.

7.3 But the fact remains that for the same pressure and velocity, the inception of cavitation potential is more in the case of roughened surface than in the case of polished one and once cavitation starts, resistance to cavitation erosion depends more on compressive strength than on the roughness. So the surface roughness is still a matter of concern for the hydraulic engineer in so far as the inception of cavitation is concerned. Hence it appears that the conclusion arrived by Thruvenga dam is applicable only after cavitation has actually started and not for the inception of cavitation.

8.0 CAVITATION FREE SURFACE FINISHES:

8.1 Good alignment of surface is essential, if cavitation erosion is to be avoided, where there is fast flowing water. Even the best concrete can not be made that will resist the destruction of cavitational forces in the long run. But, at the same time, it is comparatively easy during construction to design the surfaces, over which the fast flowing water will move so that there is no budge or off-sets. The idea of insisting strict dimensional finishes for flow surfaces of high velocity flow is to prevent irregularities that will trigger cavitation, which in turn will inflict extensi~~on~~ve damage on the structure and require costly repair and maintenance. The nature of irregularity and its location on the structure will determine what finish limits must be adhered to and if the offset is too great, what corrective treatment must be applied. Actually cavitation damage may reach a stable condition or it may progress to reach extensive proportions as shown in figure (12). The cases in which surface curvature are away from the flow (figure : 12 a), cavitation damage will, where the curvature is just right, reach a point where recirculation takes place. This raises the pressure above the vapour pressure and cavitation ceases. The cases in which the surfaces encroach on the flowing stream (figure 12 b), the cavitation will be progressive causing extensive damage.

8.2.0 Treatment Of Irregularities:

8.2.1 U.S.Bureau of Reclamation has made extensive laboratory studies to establish the velocity pressure relationship for incipient cavitation for different sizes of square and round off-sets into the flow [11] as shown in figure (13). With a known velocity and a known absolute pressure, it is possible to determine the size of the off-set that can be tolerated. A better form of the graph shown in figure (13) has been evolved by Thiruvengadam in the form of a dimensionless graph (77). Here the off-set is considered as a single isolated roughness protruding into the flow, then the roughness Reyonolds number will be $\frac{VZ\rho}{\mu}$, where V is the velocity and Z the height of the off-set. The relative roughness would become Z/H , where H is the depth of flow beyond the off-set. Such a graph is shown in figure (14)

8.2.2 Rounding the corners of abrupt surface irregularities will decrease their cavitation potential. An indication of the effectiveness of the rounding of sharp corners into the flow can be determined from the chart in figure (15).

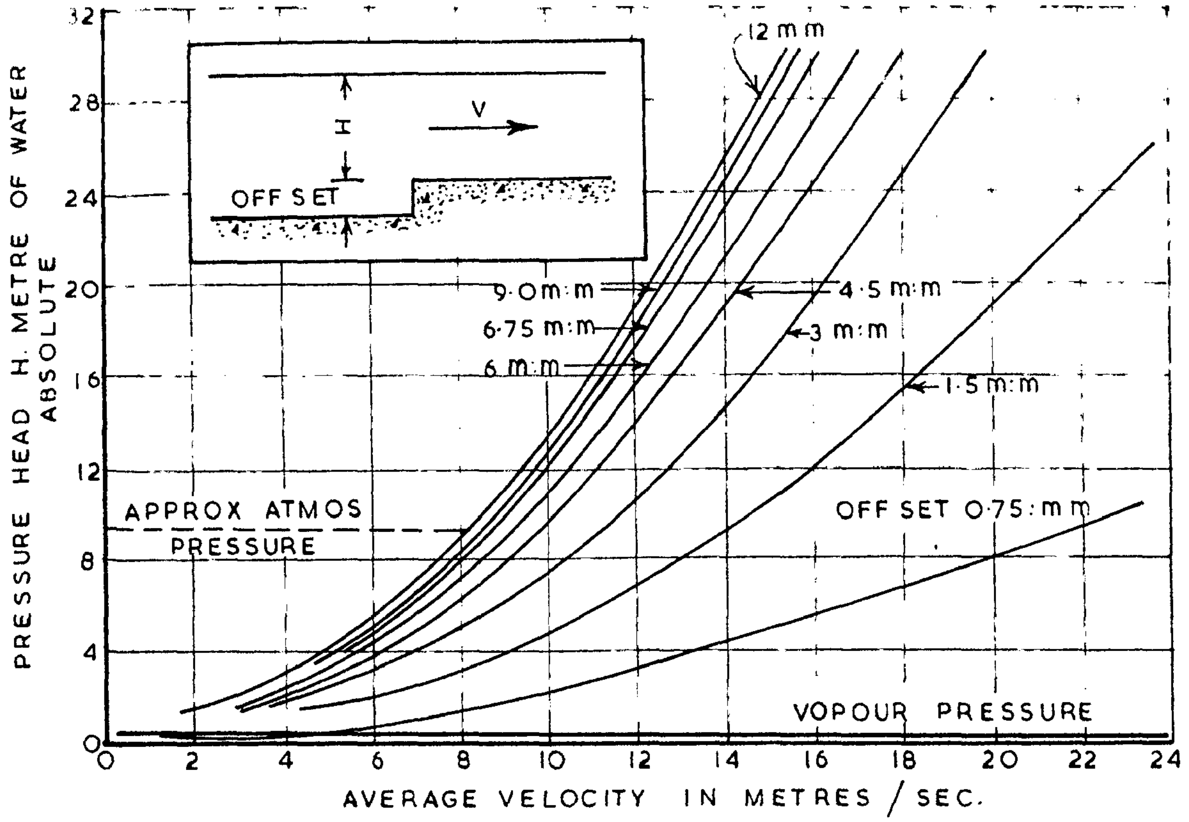


FIG. 13. CRITICAL PRESSURE AND VELOCITIES FOR SQUARE EDGED OFF SETS INTO FLOW

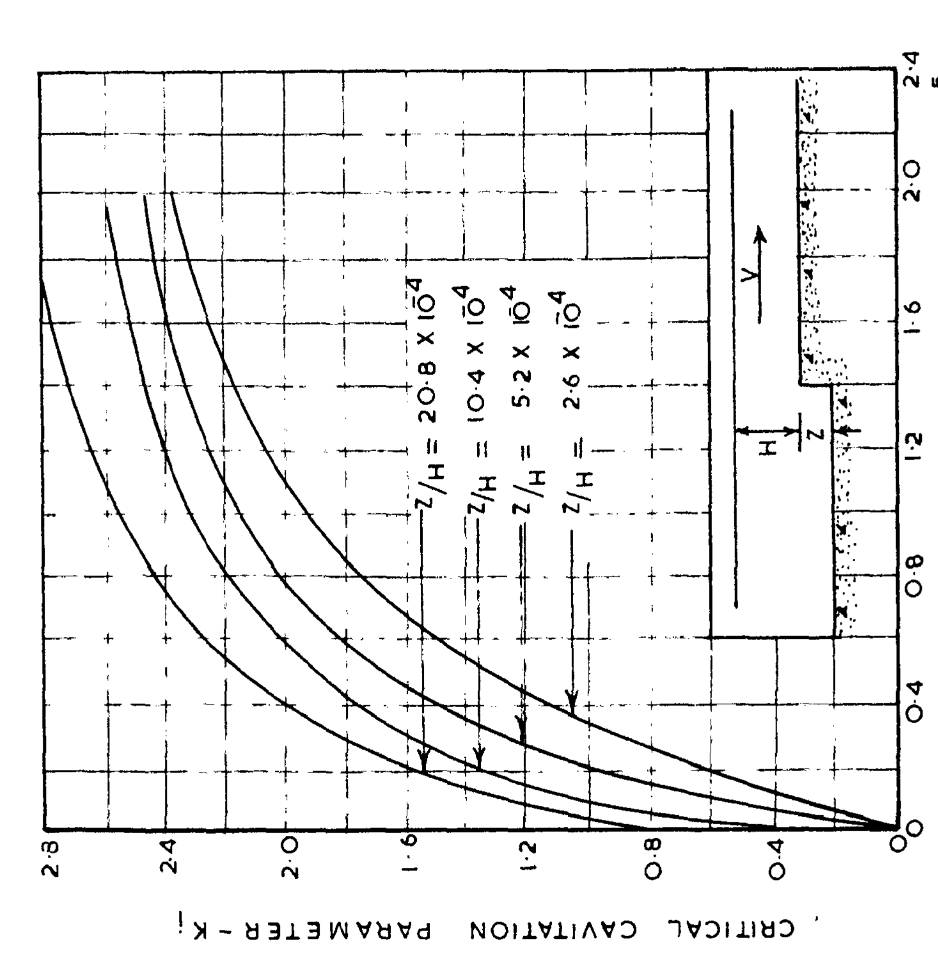


FIG. 14. RELATION BETWEEN OFF-SET REYNOLDS NUMBER AND CAVITATION PARAMETER

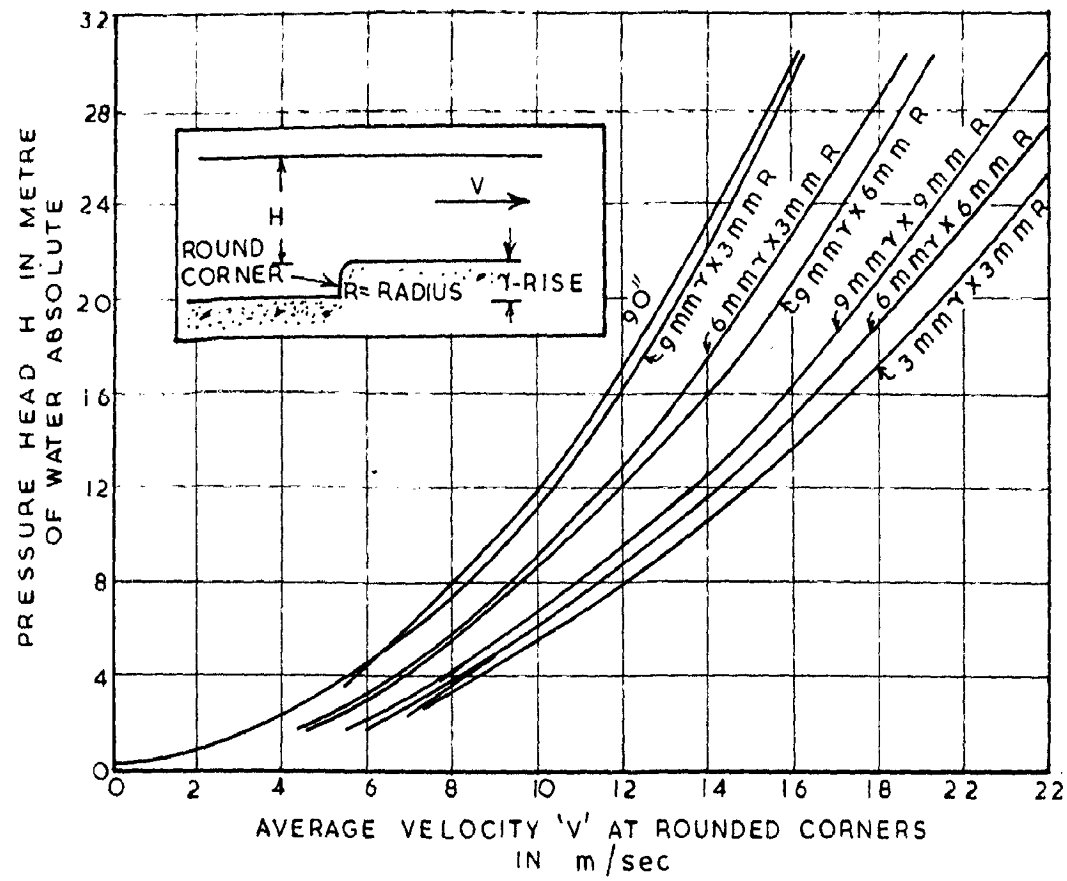


FIG.15. AVERAGE VELOCITY RELATIONSHIP FOR INCIPIENT CAVITATION

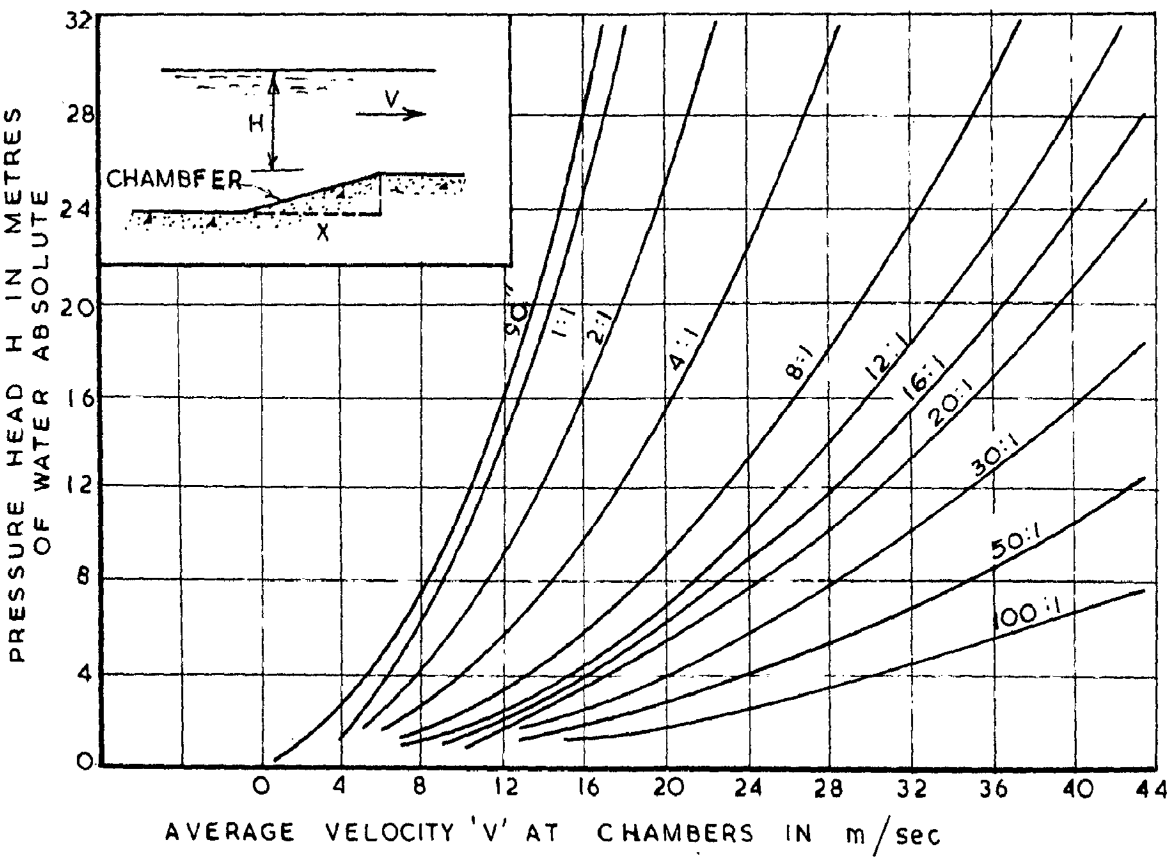
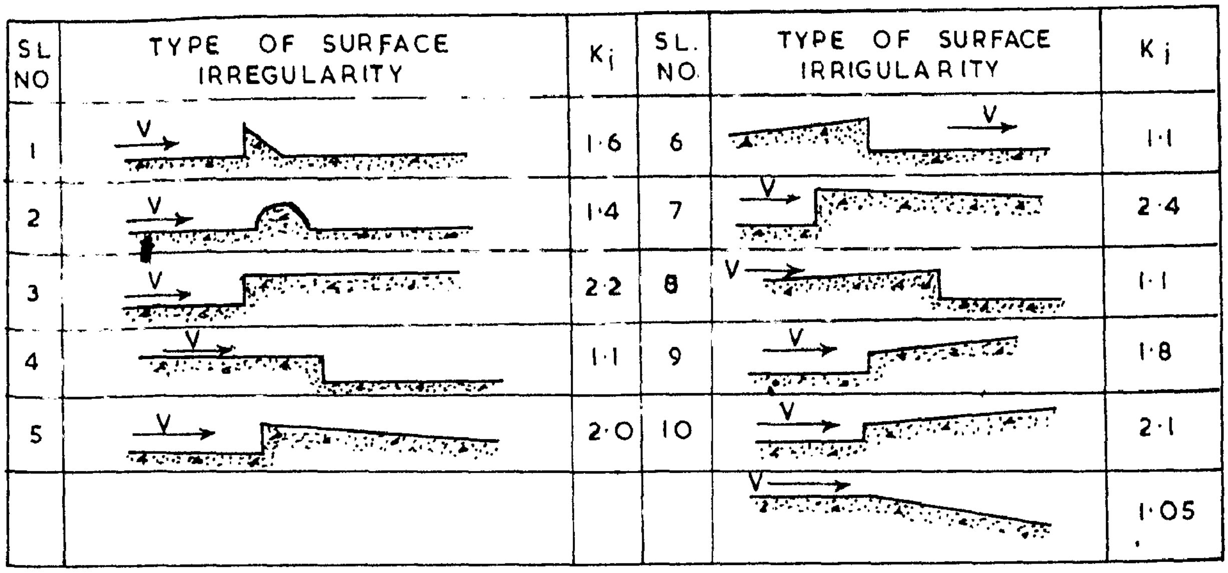


FIG. 16. AVERAGE VELOCITY RELATIONSHIP FOR INCIPIENT CAVITATION INTO THE FLOW CHAMFERS

In general, larger the radius of rounding, lesser will be the cavitation potential. Smaller off-sets into or away from the flow may be ground or flat slopes to decrease or eliminate the cavitation potential. In the case of slope of the grinding or the change in the alignment of the surface resulting from the grinding is important. The chart in figure (16) will assist in delineating the critical slope change for various velocities and pressures. The U.S.Bureau of Reclamation usually require that objectionable irregularities be removed on slopes of 1:20 , 1:50 or 1:100, where ambient pressures are low and velocity range is 12 m/sec to 27 m/sec, 27 m/sec to 36 m/sec, and greater than 46 m/sec. respectively[11] On spillway faces, abrupt off-sets that are not parallel to the direction of flow and which are offset in to the flow should not exceed 3 mm [84]. Limits of off-sets shown in figure (16) also agree with this specification. In the case of pier faces also, the specifications require that off-sets into or away from the flow be limited to 3 mm. The irregularities or off-sets are always treated by grinding in accordance with the slope velocity relationship shown in figure (16). Boundary layer has an important bearing on the finish limits allowed. Complete irregularities on specified slopes is required, where the boundary layer of the flow has been destroyed or disturbed and the velocity near the flow surface is high as down stream of gates of outlet works. When the boundary layer is completely developed as in a spillway channel, the specified finish are permitted and only the excess offset is removed from the specified slopes.

8.2.3 Studies on these aspects have been done in U.S.S.R [62]



SL NO	TYPE OF SURFACE IRREGULARITY	K_i	SL. NO.	TYPE OF SURFACE IRRIGULARITY	K_i
1	V	1·6	6	V	1·1
2	V	1·4	7	V	2·4
3	V	2·2	8	V	1·1
4	V	1·1	9	V	1·8
5	V	2·0	10	V	2·1
				V	1·05

FIG.17. CRITICAL CAVITATION PARAMETER FOR DIFFERENT TYPES OF SURFACE IRREGULARITY (HEAD OF WATER 10 TIMES > SUFACE PROJECTION)

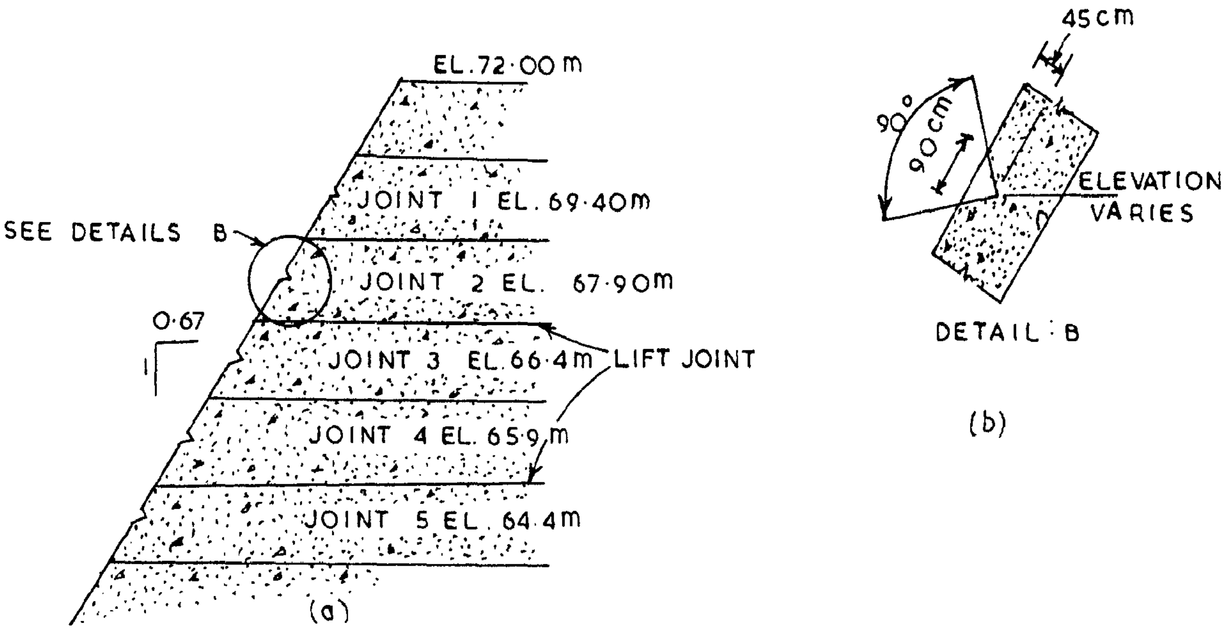


FIG. 18. DETAILS OF V. JOINTS ON SPILLWAY FACE OF CLARK HILL DAM

also and the critical cavitation number for various types of surface irregularities, obtained after these studies are given in figure (17).

9.0 EFFECT OF V- NOTCH CONSTRUCTION JOINTS ON THE SPILLWAY FACE:

9.1.0 A brief mention will be made here on the effect of having V-notch construction joints, as on the spillway face, as is being done in the case of up-stream faces. The usual construction practice is to provide flush horizontal lift joints on spillway faces. However, in one of the Corps of Engineers Projects, Chamfered V-joints similar to those often formed in the non-over flow section of concrete dams were proposed for the overflow section[56]. The resulting discontinuities at the V-joints were considered as possible sources of cavitation. So model studies were conducted in the laboratory. These results were supplemented by prototype tests made of cutting several V-Notches in an existing spillway at Clark Hill dam. The location of the V-Notches etc. are shown in figure (18). From the tests it was concluded that it may be possible that little cavitational damage would occur down stream from the V-joints of spillways having moderate heights.

10.0 SELECTION OF CAVITATION RESISTANT CONCRETE:

10.1 The best method of providing a cavitation free structure is to avoid the formation of the cavities by suitable hydraulic design. The selection of a proper type of concrete to resist cavitation is not a solution in itself. It is only a cure rather than prevention, for even the best form of concrete will not be able to resist the attack of cavitation over a long period. Even then, selection of a proper type of concrete will reduce the harmful effects from cavitation to a great extent. Studies on the resistance of cavitation erosion on concrete has been done by many organisations all over the world.

10.2 In the case of concrete, properties such as strength, durability and uniformity depend on the water cement ratio, cement aggregate ratio, grading, mixing of the admixtures etc. At the Indian Institute of Science, two different mixes were taken, one was hand mixed and the other was vibrated. The results showed that irrespective of the cement content in the mix, cavitation damage was constant as long as the compressive strength remained constant(72). The findings can be summarised as follows.

10.2.1 The cavitation damage in concrete is inversely proportional to the compressive strength.

10.2.2 With the increase in curing of concrete, the cavitation damage decreases.

10.2.3 So long as the compressive strength remains the same, immaterial of the cement aggregate ratio, the cavitation damage remains the same.

10.2.4 Addition of admixtures like surkhi, air entraining agents do not materially change so long as the compressive strength remains the same.

10.2.5 Surface fineness and surface hardening improves the initial resistance of cavitation damage considerably.

10.2.6 Tests conducted on hard rocks and bricks indicate that the cavitation damage decreases with the increase of compressive strength.

10.2.7 Polishing of the surface decreases the initial damage considerably, but with the increase in time the surface roughness increases and the cavitation damage increases.

10.3 Results of the studies done at the S.You.Zhuk Hydro Planning Research Station of the U.S.S.R. have produced the following results (3).

10.3.1 Reduction of water- cement ratio from 0.6 to 0.5-0.4 increase of the cement mortar strength from M 400*- M600* (with the same water cement ratio). Vacuum treatment of the specimen surfaces and increase the age of concrete from 28 to 90 and 180 days increase the cavitation resistance of the concrete by a factor of 2 to 4 or more.

10.3.2 The use of gravel instead of grit, dense and strong granite instead of lime stone and curing under normal conditions than using heat moisture treatment increase the cavitation resistance by a factor of 1.5 - 1.6. The resistance of the concrete is favourably affected by using artificial sands.

10.3.3 No affect on the erosion of concrete exerted by variations in the cement content, other conditions being equal or by change in the maximum filler grain size between 10 and 20 mm.

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*M 400 - corresponds to a compressive strength of mortar specimen of composition 1:3 to 400 kgm /cm2 (average) and that of M600 to 600 Kgm./cm2 (average).

10.4 Hence, it may be seen that the greater the effect of the parameters on the strength and density of concrete, the more obvious is their value for increasing the resistance of the concrete to cavitation. Figure (19) plots the relative erosion of the concrete versus the compressive and tensile strengths. In figure (19) the relation between erosion and tensile strength is seen to be less clearly marked. Thus the compressive strength is a better index of the cavitation resistance. It may be noted from figure (19) that when the strength of concrete varies from 440 - 520 kg./cm^2 there is practically no change in the relative erosion. From this it can be concluded that further increase in the strength does not lead to any appreciable reduction in the relative erosion of the concrete. Figure (20) plots tests results on good and poor concretes in comparision with the test results on other materials, which are most resistant to cavitation. It may be seen that the relative erosion of even the best concrete is one or two orders higher than the relative erosion of granite and several times higher than the relative erosion of polyester plastic concrete.

10.5 From the various experiments conducted in U.S.S.R, it is now believed that resistance to cavitation erosion of concrete surface depends also on the strength of the binder between the aggregates also[34]. The erosion begins with the disintigration of the morter enveloping the coarse filler grains. After the exposure of the grains, disintegration develops at the contacts between the mortar and the filler grains. As the cement is washed away, the adhesion between

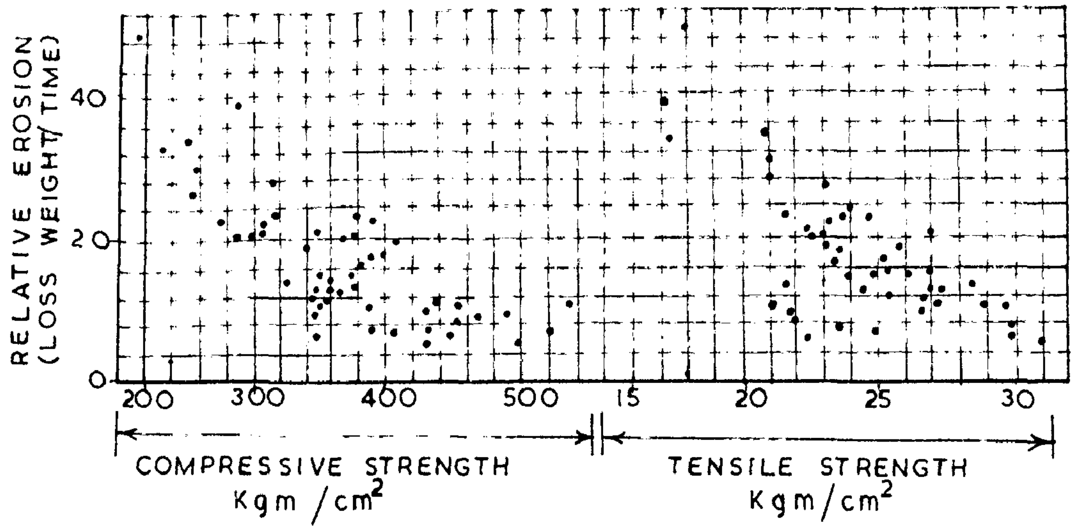

FIG 19. CAVITATION RESISTANCE OF CONCRETE

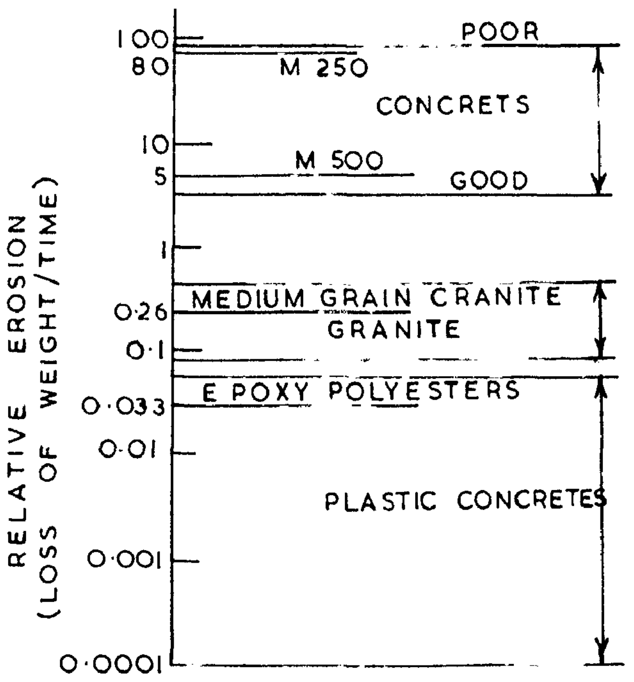

FIG.20. RANGES OF RELATIVE EROSION
OF SPECIMENS OF CONCRETE
GRANITES AND POLYMER MATERIALS

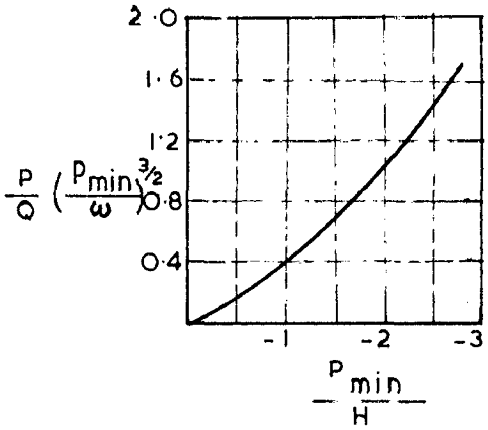

FIG.22. DISCHARGE–MINIMUM PRESSURE CHARACTERISTIC OF A SPILLWAY

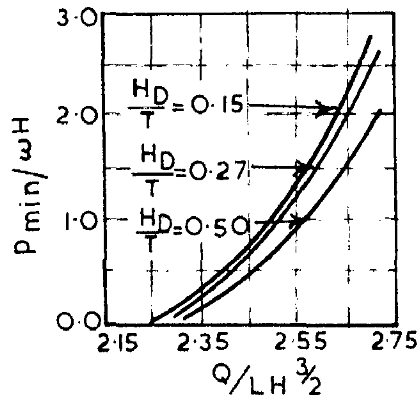

FIG.23. MINIMUM PRESSURE AS A
FUNCTION OF DISCHARGE
CO-EFFICENT

the filler grains and the mortar is so weak that due to the velocity of the flow breakes off filler grains and carries them away exposing deeper layers of concrete. The relative erosion of these materials decreases in direct proportions of the strength of these structural links. In concrete these links are between the cement bond and the filler; in finely crystalline granite, they are due to the inter-crystalline forces of interaction ; and in the polymer materials, they are intermolecular forces of physio chemical nature, which are many orders of magnitude stronger than the links in the concrete between the cement bond and the filler, so that the polymer has a much higher resistance to cavitation erosion. For example the cavitation resistance of plastic concrete on a base of PVC resin was found to be 1.15 times higher than mild steel.

10.6 Some preliminary studies to find the effect of surface hardnening chemicals against cavitation damage were done at the Indian Institute of Science, Bangalore(73). Several chemicals and water rupellent agents like Impermo Nubindex-A.C.C. water repelling agent and Ironite were added as admixtures to a concrete of 1:2:4 proportion by volume and a water cement ratio of 0.55 (Ironite is pure iron filings crushed fine) and subjected to cavitation damage. The chemicals were added to cement mortar and plastered to the concrete surface tested. The results were erratic, but the trend is that any chemical which increases the compressive strength of concrete will stand cavitation better. Application as mortar is butter than adding to concrete as admixtures.

11.0. MIX PROPORTIONS AND MATERIALS:

STRENGTH:

11.1 It has been conclusively arrived at from the discussions above that the resistance of concrete to the forces of cavitation increases with the increased compressive strength of concrete. So the American Concrete Institute recommends that when a structure is subjected to forces of cavitation, the mix is to be proportioned for a minimum strength of 420 kgm /cm^2 [91].

11.1.2 Aggregates:

11.1.2.1 It has been demonstrated that the larger particles of aggregates are plucked out or passed out of concrete by the forces of cavitation more easily than smaller particles. So the American Concrete Institute recommends a maximum size of 20 mm[91]. All the same recommendations on the aggregate dimensions are contradictory. Grimm of Germany suggests the maximum size as 5 mm. and Is.G.Ginzburg and A.M.Tehistya Kov of U.S.S.R recommend the maximum size as 60 mm.[3]. Granite crushed stone is considered the most suitable aggregate for the prepration of anti-cavitational concrete. The bond of the aggregate is more important than hardness in the case of cavitation.

12.0 REPAIRS TO ERODED AREAS:

12.1 Where concrete has been damaged by erosion, it is almost certain that the repaired section will be damaged also, unless the cause of the erosion is removed. Sometimes it may be economical to replace the concrete periodically than to shape the structure to produce streamlined flow.

In repairing the concrete it is essential that necessary attention be paid to many details, because experience has shown that repairs that were carelessly made, had become defective later on and had to be replaced. American concrete Institute lays down the following procedure (91). All damaged concrete and loose and broken particles should be removed. The holes to be repaired should be enlarged to a minimum depth of 15 cm. with a minimum area of about 930 sq.cm. It is extremely important that the concrete to be repaired be thoroughly cleaned by wet sand blasting followed by working with an air-water jet. The holes should be kept continuously wet for not less than 12 hours prior to the placing of new concrete. Cavities should be free of any water at the time of placing and preferably should be surface dry. It is also necessary to grind the surface properly after it has been hardened.

12.2.0 Because of its high resistance to erosion, plastic concretes of polymer materials are now being widely used in many countries as protective castings. Plastic concrete were used for the flow structures of Preist Rapids and Rocky Beach dams in U.S.A. and their successful operation over a period of 4 years permitted the metallic lining covering on the submerged spillway of Milford dam to be replaced by expoxy plastic concrete (23) & (24). Similar protective doatings were also given to an area of about 200 m^2 during construction of Charvak Hydro-electric station in U.S.S.R (71)

13.0 THE USE OF STONE MASONRY FOR SPILLWAYS:

13.1 Some studies have been done at the Central Water & Power Research Station, Poona on use of stone masonry for spillways especially from the point of view of resistance to cavitation damage (35 A). It is stated that for a normally designed spillway crest for heads upto 10 m to 12 m, a fairly rough dressing of the surface will be suitable and free from negative pressure conditions for both the crest and downstream glacis for low dams of the order of 25 m or so. However, if the crest is under designed, pressures are well affected by the surface roughness. Based on the model studies recommendations have been made for the allowable tolerences for the masonry facing for the negative pressures which are considered safe:

B - m.	P_r. m.	b - cm.	Allowable tolerance d - cms.
9	7.5	45	6.0
24	6.0	45	2.3
30	3.0	45	0.70
45	3.0	45	0.45

where,

B = Depth of any point on glacis below the up-stream water level.

P_r = Allowable negative pressure in metres of water.

b = Distance between successive masonry blocks along the line of the profile.

d = Allowable tolerance between successive blocks.

This criteria also agrees more or less with the observed conditions. However, in the case of transitions from the spillway face to the horizontal apron, where the velocity is high, difficulty has been observed in a number of cases and stone masonry has not been found satisfactory and it has been the opinion of C.W.& P.R.S to recommend the use of concrete in all such transitions The main draw back of the stone masonry spillway is that of the possible dislodgement of the face stones from the spillway face, when subjected to high velocity flow. In general the recommendation of C.W.& P.R.S. can be summarised as follows:

13.2 For over flow crest high degree of dressing is not required. Coursed rubble masonry with stone size (say about 60 cm. x 30 cm x 60 cm embedding) would have no detrimental effect and in some cases randon rubble masonry is preferable to cut stone masonry due to the difficulties in handling, placing etc. and in view of the possibility of better strength and bond than the former.

13.3 For the down-stream glacis, concrete lining and cut stone masonry are equally satisfactory for heads upto 25 m. or so. However, more studies are required to specify the exact type of surface dressing required.

14.0 INCREASING CAVITATION RESISTANCE BY SPECIAL SURFACE TREATMENTS:

14.1 In general, erosion resistance of concrete increases as the strength of the concrete is increased. Material and methods which tend to increase the strength of the concrete at the surface or through out the mass also increases the erosion resistance. For improving the strength of the surface of the concrete vacuum process and absorption form lining are usually adopted. Both these methods allow the escape of water and air voids from concrete. Surfaces treated by these processes are dense, hard, free from voids, and are more resistant to the action of cavitation.

14.2 Vacuum Process:

14.2.1 This process essentially consists of applying a vacuum to the surface of concrete through a mat which permits the escape of air and water to the vacuum pipes. The vacuum process can remove as much as 20% of the water to a depth of 15 cm., but the penetration of vacuum is not effective beyond 30 cm. of depth. The vacuum process improves the concrete surface through the removal of portions of the mixing water required in providing satisfactory workability for placing concrete. It apparantely does not remove any of the entrained air from the interior of concrete, but does remove the objectionable surface voids caused by air collecting against the form. By this process the strength of concrete is increased as much as 50%. Because of the slight imperfection in alignment which usually occurs where the vacuum process is employed and because of the irregularities of the mat employed in the process, grinding of the concrete surface is recommended to smooth out imperfections, where high velocity flows are expected. The vacuum process was used for improving 32,550 sq. m. of Shasta dam spillway face(7). Vacuum process was also adopted for finishing the top portion of spillway crest of Rihand dam(70).

14.3.0 Apsorptive Form Lining:

14.3.1 Absorptive form liner can also be used in finishing the concrete surface. This kind of liner serves two purposes. Firstly it produces a case hardening of the surface by withdrawing the excess water from the concrete. Secondly it eliminates the surface irregularity produced by ' bug holes ' . In the case of Brazea River project in Canada concrete having a strength of 380 - 400 kgm./cm^2 could be raised to 500 - 730 kgm./cm^2 by resorting to this process(53). Spillway surfaces of Norfolk and Friant dams were also finished with the help of absorptive form liners.

15.0 AERATION OF SPILLWAYS:

15.1 One of the most widely adopted methods in eliminating vacuum in a flow is introducing air into those areas. All natural waters contain air and impurities in sufficient amount to allow cavitation to occur above vapour pressure It is therefore important to distinguish between air normally contained in water, which aids forming of cavities and air added to the water for the purpose of reducing cavitation pitting. The exact part that entrained air play in reducing pitting is not known, but it is widely believed that the air introduced produces cushioning effect and this helps to reduce the pitting action. The method of introducing air into the water would probably have some effects on the characteristics of the mixture. Air introduced into the flow system in small bubbles well distributed across the flow section would be more beneficial than air introduced in larger bubbles and concentrated at one local area. The air introduced follows a flow stream line and its effects extended over a relatively narrow bond. Studies to know the effect of entrained air on cavitation pitting were done at the U.S.Bureau of Reclamation(50) and the results

The resulting curve showed that the entrained air reduced the amount of cavitation pitting. Small amounts of air (as low as 2%) have definitely beneficial effect, while 7.4% of air is necessary to remove theharmful effect entirely from this 2 hour test. It is also clear from the results that entrained air also increased the cavitation number by increasing the pressure in the cavity producing region.

15.2 When the lower nappe of the falling sheet of water does not adhere to the spillway face, an air space will form unless an outlet to the outside atmosphere is provided. Flow over a properly shaped ogee will adhere to the spillway face for heads upto several times the design head and hence no provision for aeration is usually made for spillway crest. But a situation may arise, when the operation mechanism of one or more gates may fail during the time of maximum discharge over the spillway, resulting in a free fall of the nappe over the gate crest to the spillway face downstream below. A situation like this requires aeration of the nappe to keep the sub-atmospheric above a certain minimum. Because of this purpose air vents have been provided on the spillway of Rihand dam .

15.3 <u>Air Demand</u>:

15.3.1 A relation between the air demand and the reduction of pressure beneath the nappe has been developed by Hickox[31] The relationship was developed by laboratory tests on many spillways and on a prototype (12.2 m x 6.1 m) vertical lift gate. The air capacity required to maintain the pressure reduction beneath the nappe to any desired value may be computed by this formula:

$$q_a = \frac{(C\ Ho)^{3.64}\ g^{0.5}}{P_r} \qquad (1)$$

where,

q_a = air demand in metre length of the weir in m^3/sec.

Ho = Head of water over the tope of gate in metres.

P_r = Reduction of nappe beneath the nappe in metres of water.

C = Constant depending on the ratio of the discharge over the top and beneath the gate.

If there is no discharge beneath the gate, C is equal to 0.777. If there is flow beneath the gate also C may be determined from the table given below, which gives approximate values of C for various values of U, where U is dimensionless ratio computed by the following equation:

$$U = \frac{I\sqrt{Hu}}{(Ho)^{1.5}}$$

where,

I = Height of opening beneath the gate in metres.

Hu = the head on the centre of opening beneath the gate.

Relation of air demand and co-efficient for vertical lift crest gate to discharge beneath the gate.

TABLE

U	0	0.5	1.0	1.5	2.0	2.5	4.0
C	0.077	0.135	0.175	0.202	0.219	0.225	0.225

15.4 Air Vent Size:

15.4.1 Having determined the air capacity required to maintain a specific pressure reduction beneath the nappe the vent size may be computed by the use of hydraulic equations. Since the vent draws air from the outside atmosphere, effective head causing the flow is equal to the allowable pressure reduction beneath the nappe. The procedure of finding the air vent size may be better explained with the help of a typical case. The problem is to find the size of air vent required for a 12 m wide gate with a head over the gate as 6 m and the maximum allowable negative pressure is 1.5 m.

From the table C = 0.077 as there is no discharge beneath the gate and substituting the values in equation (1)

$$q_a = \frac{(0.077 \times 6)^{3.64} \times 9.81^{0.5}}{1.5^{1.14}} = 0.121 \ m^3/sec.$$

$\therefore$ Total air demand = 0.121 x 12 = 1.5 m^3/sec.

For the design of the air vent, the following assumptions are made:

Rougosity co-efficient of air duct (assuming rough surface)	= 0.003
Friction factor	= 0.023
Loss at entrance	= 0.2 $V^2/2g$
Loss at bend	= 0.15 $V^2/2g$
Ratio of sp. weight of water and Sp: weight of air	= 830
Length of air duct	= 50 m.

Only one air duct will be provided

Assuming a head loss of a metre of water

$$a \times 830 = \frac{V^2}{2g} + 0.2\,\frac{V^2}{2g} + \frac{0.15\,V^2}{2g} + \frac{0.023 \times 50}{d}\,V^2/2g$$

$$\text{i.e. } 830 \times 2 \times 9.81 = \left(1.35 + \frac{1.15}{d}\right)\left(\frac{1.5}{\frac{\pi}{4}d^2}\right)^2$$

$$= (1.35\,d + 1.15)\,\frac{3.66}{d^5}$$

$$\therefore \quad d^5 = \frac{4.95\,d + 42}{830 \times 19.62}$$

solving this equation the value of d i.e. the diameter of the air vent is evaluated as 0.25 m.

Another point to be borne in mind is that the velocity in the duct should not be allowed to go beyond 45 m/sec. to minimise noice.

16.0 UNDER DESIGNED CREST:

16.1 Having discussed so far about the harmful, effects of having sub-atmospheric pressures on the spillway crest the next logical question is how far the pressure can be allowed to develop below atmospheric pressure without any appreciable damage and is it absolutely essential to have spillway which will give positive pressures all the time. All the same, negative pressure has its advantages also. For, it is a well known fact that negative pressure in the region of the crest is accompanied by an increase in the co-efficient of discharge and when as a positive pressure signifies that the co-efficient of discharge is reduced. This generalisation holds good not only for changes in the profile under a given head, but for variations in the head as well. Many experiments have been conducted, notably by Rouse, Reid and Dillmann (64) to show that the maximum head may be many times as large as the design head without seperation of the nappe. The variations of co-efficient of discharge for heads other then design heads for a standard U.S. Army Corps profile (WES Profile) is shown in figure (21) So economy in the design of spillways may some times be effected by using a design head for the crest, which is less than the expected maximum head on the crest. Experiments at the U.S.Waterways Experiment station have shown that pressures only slightly below atmospheric pressure which do not produce cavitation, occur at maximum head, when the design head is not less then 75% of the maximum head. This practice is almost uniformly adopted now by U.S.Army Corps of Engineers for the design of their spillways. Even prototypes tests ~~for~~ by the U.S. Army Corps of Engineers on spillway of Chief Joseph dam which was tested for 1.1 times the design head did not show any adverse pressure variation on the crest (9).

16.2 Even though most of the organisations follow the practice of under designing a spillway to make their designs more economical, none of them does seem to take into account directly the pressure, which will occur on the spillway at the time of the design. Hence a more rational approach to the problem can be given, if the minimum pressure to be allowed is decided first and the design head fixed according to that. An approach

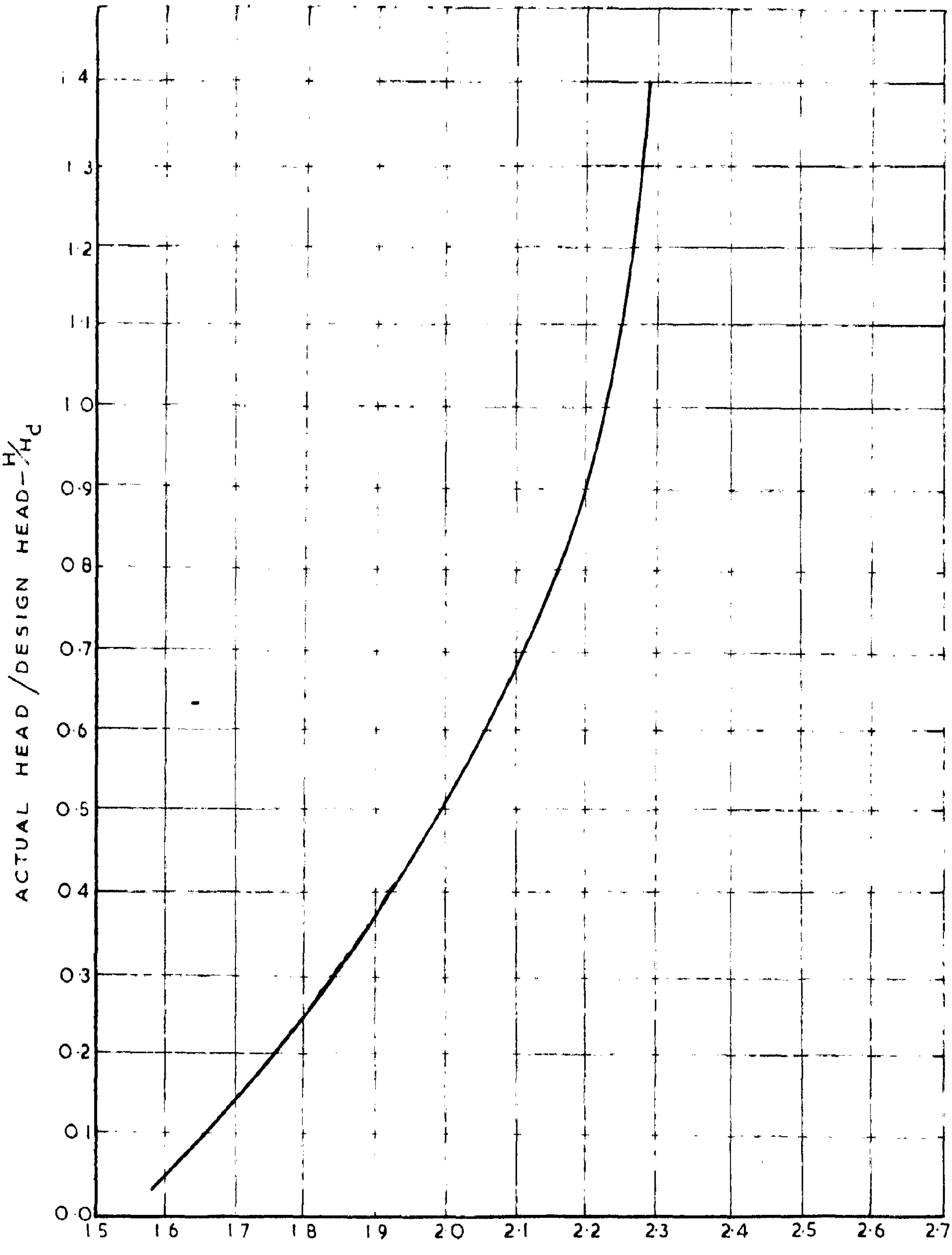

FIG 21 DISCHARGE CO-EFFICIENT OF AN OGEE-SPILLWAY

to this method been suggested by Cassidy[13]. Based on experimental results he has been able to draw curves relating $\frac{L}{Q}\left(\frac{P\ min}{w}\right)^{3/2}$ and $\frac{p\ min}{\omega WH}$; and $\frac{p\ min}{\omega WH}$ and $\frac{Q}{H^{3/2}}$

for various values of $\frac{Hd}{T}$ where, H_d is the head for which spillway crest is designed and T the height of the spillway crest above the river bed. These tests were made for the standard WES profiles. As the procedure suggested seems to simple the design outlines are very briefly mentioned here. The curves used in the design are shown in figures (22) & (23).

1. The minimum pressure to be allowed is decided first.
2. For this pressure the parameter $\frac{L}{Q}\left(\frac{-p\ min}{w}\right)^{3/2}$ is calculated.
3. Using this calculated value of $\frac{L}{Q}\left(-\frac{p\ min}{w}\right)^{3/2}$ the corresponding value of $\frac{p\ min}{\omega H}$ is read from figure (22)
4. From the value of $\frac{p\ min}{\omega WH}$, H is calculated.
5. The spillway height is now computed as knowing already the level of Maximum reservoir level from hydrological studies.
6. Using the computed value of H, the co-efficient of discharge is calculated.
7. Using the figures (22), the proper value of Hd/T can be selected to yield the required value of C and p min/ωWH
8. From this value of Hd/T, Hd can be found, as T is already known.

17.0 BOUNDARY LAYER CONTROL ON SPILLWAYS:

17.1 Seperation of boundary layer takes place in practically all hydraulic structures, which deal with flowing water. The consequent turbulence gives rise to excessive loss of energy and negative pressures created may cause cavitation.

In order to determine the effect of providing devices for boundary layer control on pressure distribution along the spillway crest. Certain experiments were done by Asthana[1]. From these experiments it was seen than an obstruction in the form of 4.5 mm. diameter rod, when placed at a particular location downstream from the crest in the Creager profile for a design head of 4.75 cm. was capable of eliminating negative pressures for heads upto 8.75 cm. It is argued that the induced turbulence due to the introduction of the rod eliminate the seperation of the boundary layer and consequent negative pressure.

17.2 However, Rouse[63] argues about this occurrence in a different manner. He points out that as the thickness of the rod was many times larger than the boundary layer thickness, it would not have been possible to have any effect on the boundary layer and at the crest the rod might have had an effect in changing the configuration of the profile, thus giving rise to different pressure distribution. So, further studies on this aspect ~~was~~ were done at the I.I.T. Madras studies were conducted on 3 profiles, designed for various design heads and the effect of having rods at different places were explored for values of H/Hd ranging from 1.5 to 2.0. The results obtained from these studies may be summed up as follows.

It was not possible to eliminate negative pressure completely. Negative pressure diminished in this area upstream of the rod and in some cases positive pressures were observed. In the area just down stream of the rod negative pressure increased. In the areas further down stream from the rod, negative pressure minimised. It was also noticed that the maximum positive and maximum negative occurred immediately up-stream and down stream of the rod respectively.

17.3 These experiments described above were not very elaborate in nature and as such no general conclusions can be arrived. However, this approach gives a completely new perpective to the problem.

CHAPTER: 6

C O N C L U S I O N S

1.0 The design of a spillway of any dam is the most important in this overall planning and design of works. Spillways are like safety valves, which if not properly designed, or in-adequate, can cause unfold miseries and havoc. A comprehensive study conducted by Water Resources Development Training Centre, University of Roorkee, indicates that a big percentage of dam failures were due to improper and in-sufficient spillway capacity. The design and type of spillway is governed to a great extent on the foundation rock continuum and types of dam. Though a number of spillways types are available, the most prevalent is the 'ogee' spillway. This type of spillway is invariably used for all projects in the country.

2.0 Extensive work has been done by many questers on profiling the spillway with as little sub-pressures as possible. The hypothesis of all these designs has been to evolve a profile conforming to the under nappe of a free falling aerated water jet over a sharp crested weir. This profile, when designed for certain design head, H_d, gives some negative pressure, when operating under a head higher than the design head or at partial gate openings. A profile accommodating these limitations would lead to a thick and un-economical profile, which would also be hydraulically less efficient. On the other hand a profile allowing some negative pressure, apart from lowering the reservoir level would also economise in the length and concrete of spillway. Different types of profiles conforming to both the criteria, viz. no negative pressure for design head and profile allowing some negative pressure for design head have been detailed and discussed in Chapter : 2,. The compilation, though not complete, however, is exhaustive and covers all that was available in the literature collected. Studies show that Creager profile is by far the most common, but his results of the U.S.Army Corps of Engineers profile are more encouraging and is recommended for adoption for future projects.

3.0 Weirs with up-stream overhang and off-sets can be used with advantage and economy, if it is structurally possible. However, spillway sections are not recommended with values of M/N (where M is the height of riser on off-set weirs and N is the horizontal off-set on off-set weirs) more than zero and less than 0.5. For these range of values of M/N, flow conditions are unstable.

4.0 The lip of the tainter gate should be located down stream of the crest point. In the case of vertical lift gates or stop log gates, proper design of the gate slot is necessary to avoid cavitation inception. The design principles of gate slots are discussed in Chapter: 5.

5.0 Different types of anti-vacuum remedial measures like proper spillway profile, aeration arrangements etc. should be planned and provided without fail for all overfall spillways. All these methods have been discussed and examplified in Chapter: 5., 6.0 Various causes of possible cavitation damage like, wrong design of spillway, rough spillway surface, improper ventilations, wrong location of gate seat and under design of spillway profile should be checked on the model before adopting on any spillway in the field.

7.0 Absorptive form lining and vacuum concreting have been suggested and used at certain places for obtaining hard and smooth spillway surface. These methods, however, are costly and require careful planning and control in construction. Better profiles and smooth surfaces can be achieved by proper use of forms and rich mix concrete on the spillway profile. This has been done practically for all the spillways in the country.

8.0 The air demand and air vent size can be evaluated with the help of HicKox's formula, which has been discussed in Chapter 5.

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