DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
WATER RESOURCES DEVELOPMENT (CIVIL)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is presented in this dissertation entitled "DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM" in partial fulfillment of requirement for award of the degree of MASTER OF TECHNOLOGY IN WATER RESOURCES DEVELOPMENT, submitted in Water Resource Development and Management Department, Indian Institute of Technology, Roorkee, is an authentic record of my own work, carried out during the period from July, 2004 to June, 2005, under the guidance of Prof. RAM PAL SINGH and Visiting Prof. B. N. ASTHANA, WRD&M, IIT, Roorkee (India).

I have not submitted the matter embodied in this dissertation for award of any other degree or diploma.

Place: Roorkee
Dated: 25 June, 2005

(NARENDRA SINGH BHANDARI)

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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LIST OF SYMBOLS

α	=	Slope angle of chute with vertical
β	=	Angle of convergence or divergence
A_1, A_2, etc	=	Horizontal dimension defining up-stream crest
B_1, B_2, etc	=	Vertical dimension defining up-stream crest
C	=	Non-dimensional coefficient of discharge
C_d	=	Coefficient of discharge
D	=	Depth of flow
F	=	Froude's number
f	=	Head loss in the flat crested weir
g	=	Acceleration due to gravity
Go	=	Gate opening
H_T	=	Total head
H	=	Head of overflow
H_d	=	Design head
H_0	=	Total head over crest of shaft.
H_e	=	Head over the crest of shaft tunnel-pipe control
H_a	=	Orifice control Head in shaft tunnel
L	=	Length parameter
L'	=	Net length of the overflow crest.
n	=	Manning's roughness coefficient
N	=	Number of piers
M	=	Riser of the crest
MCM	=	Million cubic metre
K_2	=	Variable Parameter
K_a	=	Abutment contraction coefficient
K_p	=	Pier contraction coefficient
R	=	Radius of bucket /Hydraulic radius

P	=	Absolute pressure at elbow of shaft tunnel
P_a	=	Atmospheric pressure 10.0m of water head.
P_c	=	Reduced Pressure due to centrifugal force in the tunnel elbow
P	=	Average Pressure at given section of the cross section of tunnel
P_v	=	Vapor Pressure
P_b	=	Barometric Pressure
Q_a	=	Quantity of air in chute channel
S	=	Spacing
S_0	=	Slope
T	=	Top width at water level
t	=	Thickness of flow in the funnel of shaft
Va	=	Approach Velocity
V	=	Velocity of flow in the channel
V1	=	Velocity at section 1
V2	=	Velocity at section 2
δy	=	Change in water surface elevation in trough
Z	=	Elevation of the ground
γ /W	=	Unit weight of water
π	=	A coefficient
ν	=	Kinematic viscosity

ABBREVIATIONS

PMP	=	Pancheshwar Multipurpose Project
ASCE	=	American Society of Civil Engineers
CBIP	=	Central Board of Irrigation and Power
USBR	=	United States Bureau of Reclamations
USAWES	=	United States Army Corps of Engineers Waterways Experimental Station.
JHD	=	Journal of Hydraulic Division
IS	=	Indian standard code
JHE	=	Journal of Hydraulic Engineering

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SYNOPSIS

Spillways are structures to pass surplus water safely from the reservoirs constructed on a river valley for various purposes such as power generation, irrigation, flood control, navigation waterways and for water supply to towns and cities etc. The main objective of a spillway is to pass the flood, in excess of the storage capacity in such a manner that it does not damage the down stream riverside settlements.

Spillways, therefore, have following main functions in general:

- To pass the floods safely
- To protect the down stream settlements
- To protect the upstream submergence due to level rise in the reservoir
- To regulate the downstream water as per requirements.
- To protect the main structure dam from over topping and the down stream erosion.

From engineering point of view the suitability of type of spillway, as per site condition, geology of the area, the type of dam and other parameters, are studied and examined. Hence a careful consideration of these points, before selection of the suitable type of structure is necessary.

A study of some existing projects and their spillways is made in the dissertation in order to have an insight of the effects of various factors in the selection of a particular type of spillway in each case. An attempt is also made to collect the information of some existing high head spillways regarding their performance and design features. A detailed study is made of various alternatives for Pancheshwar Dam Spillway. Pancheshwar Multipurpose Project is a bi-national Indo-Nepal dam (280.0m height) project in River Mahakali 290.0km from New Delhi and 500.0km from the Kathmandu. This dam reservoir is planned for 12.0BCM storage capacity and passing a design flood of the order of 18,000.00 cumecs at maximum reservoir level at 693.5m EL above the river bed of 412.00 EL.

The study is based on the existing literatures, codes, and existing designs of the similar structures. The following three alternatives are designed and techno-economic comparison is made.

- A conventional gated chute spillway located on left bank.
- A gated overflow section, located on left bank adjacent to the dam and discharging to a concrete lined chute with two shaft tunnels utilizing the diversion arrangement.
- An un-gated over flow section discharging into a side channel

In each case a suitable energy-dissipating device is worked out as per the site condition. A flip bucket type energy dissipator for the side channel and chute type spillways is selected depending on the hydraulic and geologic conditions and the same is designed. The shaft spillway is provided with a swirling device for energy dissipation inside the shaft tunnel. Out of the three alternatives studied, the combination of Chute and Shaft Spillway is found techno-economically feasible.

Chapter- 1

INTRODUCTION

1.1 General

Spillways are needed to pass surplus water safely from the reservoirs constructed on a river for various purposes, such as for power generation, irrigation, and flood control, navigation waterways and for water supply to town and cities etc.

A spillway is an important part of any large reservoir project. From engineering point of view its suitability as per the site condition, geology of the area and the type of dam and other parameters should be studied, keeping in view the relevant design and planning procedures. Hence a careful consideration of these points, before selection of the suitable type of structure is required.

In general overflow spillways are classified as high or low head as per IS-6943-1998 depending on whether the ratio of height of spillway crest measured from river bed to the design head is greater than or equal to or less than 1.33 respectively. High head spillways need to negotiate a large drop between reservoir level and down stream river bed.

To design a high head spillway, the main components to be designed are approach channel, control structure gated or un-gated, conveyance structure open channels or shaft tunnels and terminal structure containing energy dissipators.

Pancheshwar Multipurpose Project, the case studied is a high dam (280.0m height) scheme planned on Mahakali River, a border river between India and Nepal under the Mahakali River Treaty signed 1997. The dam site is in District Champawat, Uttranchal, India and District Baitadi, Nepal. The total catchment is 12,100.0 km square. The average flow in the river is 582.0 cumecs and the project will produce 970.0 MW of power.

This dissertation reviews the existing high head spillway structures of various types, and evaluates the techno-economic feasibility of proposed side channel spillway and compares it with various other alternatives for the Pancheshwar Multipurpose Project (PMP) high head spillway.

1.2 Objectivẽ of the study

The objective of the study of this dissertation is to propose a techno-economically feasible spillway arrangement for the proposed Pancheshwar reservoir.

1.3 Scope

The dissertation is focussed on the design of following types of high head spillways for Pancheshwar dam.

- **Gated Ogee chute spillway**
- **Shaft and gated- chute spillway combination**
- **Side channel ogee type overflow spillway.**

A ski-jump bucket type of energy dissipator for the three alternatives is also designed.

A techno-economically feasible alternative has been selected and proposed.

1.4 Organisation of the dissertation

Following are the outline of the chapters incorporated in the in detail in the study.

Chapter I entitled "Introduction" presents background of the problem along with the objective and scope of the study.

Chapter II entitled "High Head Spillways" recapitulates definitions of some high head spillways and design requirements.

Chapter III entitled "Review of high head spillways with respect to their performance" deals with performance of spillways in existing projects.

Chapter IV entitled "Pancheshwar Dam Project" gives project information and deals with reservoir routing study for Pancheshwar Dam Spillway.

Chapter V entitled "Alternative Spillway Studies" deals with design of alternative spillway options available for Pancheshwar dam.

Chapter VI entitled "Conclusion" concludes the results of the study.

Chapter 2

HIGH HEAD SPILLWAYS

Spillways are needed to pass surplus discharge safely down stream of diversion of storage dams.

Spillways are classified as high or low head depending on whether the ratio of height of spillway crest measured from the river bed to the design head is greater than or equal to or less than 1.33 respectively. The design head (H) as per IS-6943-1998 is distance measured vertically from the water surface (upstream of commencement of the draw down) to the crest elevation. It also includes the velocity of approach. High head spillways are generally provided with storage dams and low head spillways are used for diversion structures in irrigation weirs or barrages.

Following types of spillways or their combination are generally used for high head reservoirs.

- Ogee spillways
- Chute spillways
- Side channel spillways
- Shaft spillways

The description of these four types covers almost all design features of high head spillways in general. Hence a brief description of above four types is given below.

2.1 Ogee or over-flow spillways

USBR 'design of small dams' and IS-6943-1998 both define the ogee spillway as a control weir structure, which is s-shaped, or ogee shaped in profile. The upper curve of ogee is ordinarily made to confirm closely to the profile of the lower nappe of a ventilated sheet of water falling from a sharp crested weir [5]. Flow over the crest is made to adhere to the face of the profile by preventing access of air to the underside of the sheet. For discharges at design head, the flow glides over the crest with no interference from the boundary surface and attains near maximum discharge efficiency. The fig-2.1-2.2. give general elements of ogee spillways.

The profile below the upper curve of the ogee is continued tangentially along a slope to support the sheet on the face of the overflow. A reverse curve at the bottom of the slope turns the flow onto the apron of a stilling basin or into the spillway discharge channel.

The upper curve at the crest may be made either broader or sharper than the nappe profile. A broader shape will support the sheet and positive hydrostatic pressure will occur along the contact surface. The supported sheet thus creates a backwater effect and reduces the efficiency of the discharge. For a sharper shape, the sheet tends to pull away from the crest and to produce a sub atmospheric pressure along the contact surface. This negative pressure effect increases the effective head, and thereby increases the discharge.

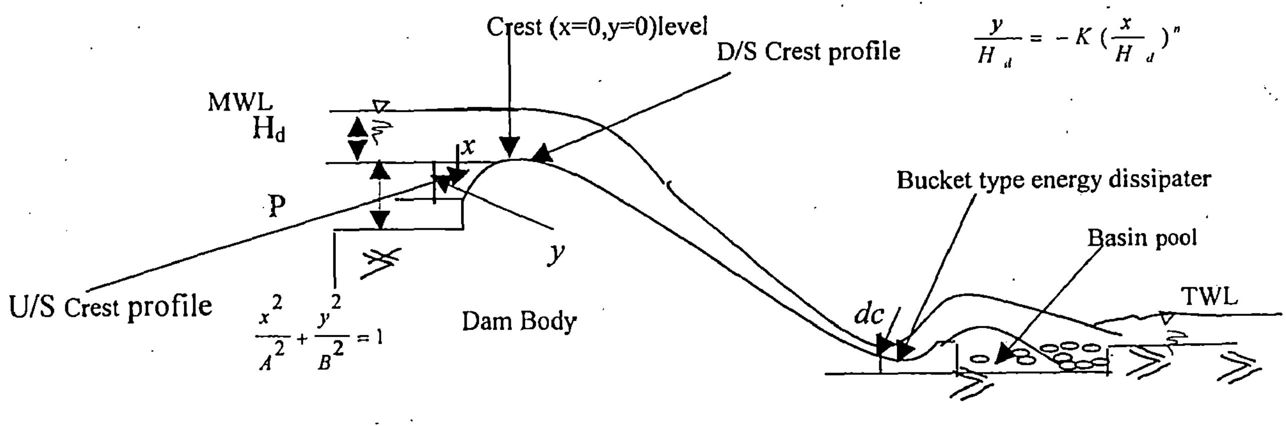


Fig-2.1. Ogee spillway elements sketch

Design Head

As per IS-6943-1998 generally design head H_d is kept in the range of 80 to 90 percent of the maximum head available to limit/remove the cavitation effect. The operation of the spillway for heads lower than design H_d, gives pressures higher than atmospheric and for heads higher than H_d, the pressure would be sub-atmospheric.

At the same time the coefficient of discharge would be affected. The extent of sub- atmospheric pressure for an under designed spillway (higher heads then design heads H_d) profile shall be ascertained from hydraulic model studies for any specific case.

Ogee crest and its apron may comprise a spillway, such as overflow portion of concrete gravity dam, or ogee crest may be control structure for some other type of spillways, such as side channel, chute spillways, etc. Because of its high discharging efficiency, the nappe shaped profile is used for most spillway control crests.

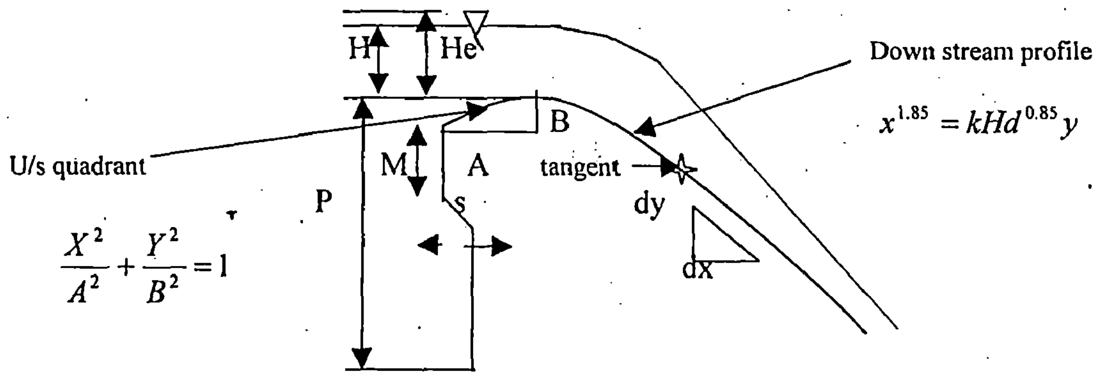


Fig- 2.2 as per IS-6934-ogee spillway

Upstream quadrant

The upstream quadrant of the crest may conform to the ellipse:

$$\frac{x^2}{A^2}+\frac{y^2}{B^2}=1 \text{--} 2.1$$

The magnitudes of constant parameters A, and B, are determined with reference to parameters P/Hd, from the available IS-6934-1998 fig-2 graphs inclosed in page no..97

Down stream profile

The downstream profile of the crest may conform to the equation:

$$\frac{y}{H_d}=-K\left(\frac{x}{H_d}\right)^n \text{--} 2.2$$

Where, n is taken as 1.85, H_d is net design head over crest and the K is a variable parameter. Magnitude of K is determined with reference to the parameter P/H_d given in IS-6934-1998 fig-2 page no 97

Ogee spillway is mostly used with gravity dams (concrete or masonry) as in Bhakra dam, Indira Sagar dam and Sardar sarovar dam. It can also be used in earth or rock fill dams with a separate structure. It has high discharging efficiency, which is an advantage over other spillways. The foundation required must be stable and sound. Ogee type crest is used as control structure in almost all type of spillways. It can be used at concrete saddle dam spillway structure in earth dams or as a central concrete spillway in a earth dam in a wide valley.

2.2 Chute spillway

As per USBR 'Design of small dams', a spillway whose discharge is conveyed from the reservoir to the downstream river level through an open channel placed either along a dam abutment or through a saddle might be called a chute, open channel, or trough type of spillway fig-2.3. These designations can apply regardless of the control device used to regulate the flow. Thus, a spillway having a chute-type discharge channel, though controlled by an overflow crest, a gated orifice, a side channel crest, or some other control device, might still be called a chute spillway. In most cases however, the name is applied when the spillway control is placed normal or nearly normal to the axis of an open channel, and where streamlines of flow, both above and below the control crest, follow the direction of axis.

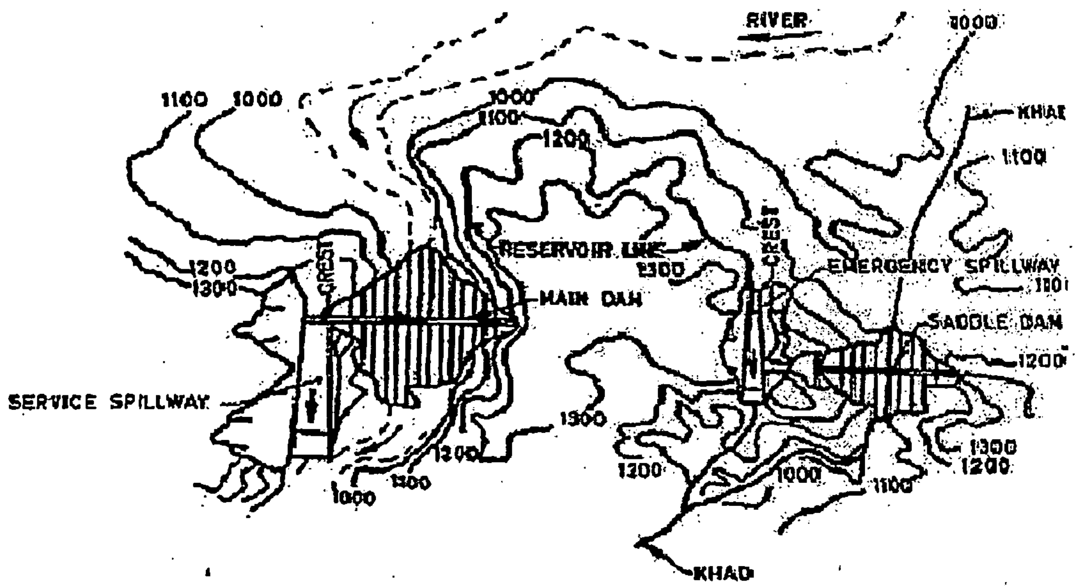


Fig. 2.3 Layout plan of a service and auxiliary (emergency) chute spillway [4(iv)]

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. The simplest form of the chute spillway has a straight centerline and is of uniform width. Most often, either the axis of the entrance channel or that of the discharge channel must be curved to fit the alignment of the topography. In such cases, the curvature is confined to the entrance, if possible, because of the low velocities. The fig. 2.3-2.5 give general plan and elements of chute spillway.

Where discharge channel must be curved, its floor is some times super elevated to guide the high velocity flow around and thus avoiding a piling up of flow to the outside of chute. Chute spillway profiles are usually influenced by the site topography and by sub surface foundation conditions. The control structure is generally placed in line with or upstream from the centerline of the dam.

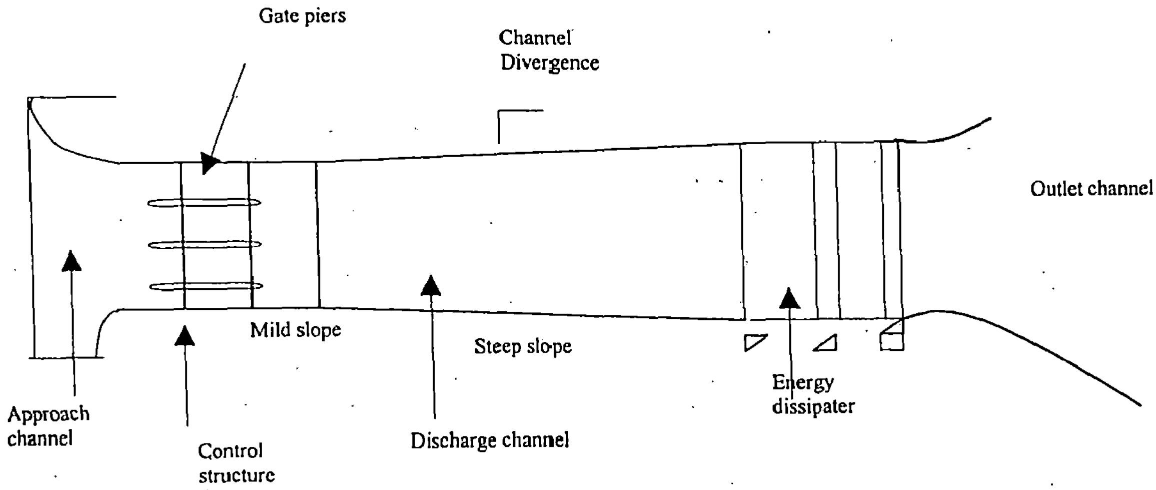


Fig. 2.4 Chute spillway general plan-Sketch

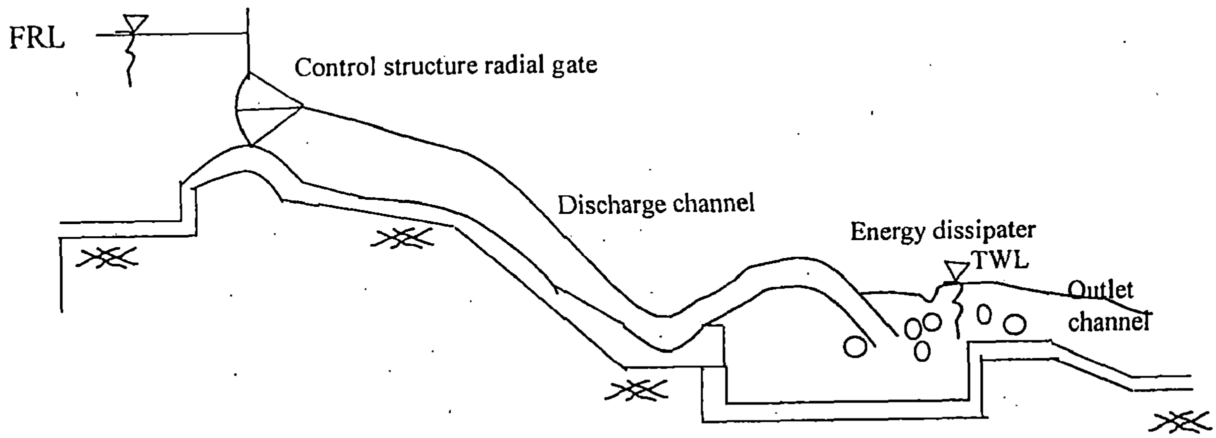


Fig.2.5 Typical chute spillway section sketch

Bed profile

As per IS-5186-1994-6.5.4.1, depending upon the site condition initial slope of channel is kept mild (in the range of 1in 4 to-20) and then a steep slope of 1 in 2 or less can be applied. For the profile of discharge channel, sharp convex or concave curves should generally be avoided. Convex curve should be sufficiently flat to maintain positive pressures. The curve should confirm the radius given by the equation below [4(v)]

$$Y = -x\tan\alpha - \frac{X^2}{4K(d+h_v)\cos^2\alpha} \text{ ---2.3}$$

Where, X and Y are the co-ordinates from the mild to steep slope and θ is the upstream slope angle with horizontal. K is safety factor and d+hv is the specific energy at the section.

The radius of concave curve should not be less than $2*\frac{WdV^2}{pg}$.

Where, W is unit weight of water, g is acceleration due to gravity, d is depth of flow, V is velocity and p is permissible intensity of dynamic pressure. In no case the radius should be less than 10d except at the toe of the crest where it may be 5d.

Convergence and Divergence

The side transition for convergence or divergence is provided at an angle given by the following equation by USBR 'Design of Small Dams'[5]:

$\tan\beta = \frac{1}{3F}$ Where, β = angle of convergence or divergence; and F = Frauds number given by $\frac{V}{\sqrt{gd}}$ Where, V = average of the velocities at the beginning and end of transition; g is acceleration due to gravity; and d = average of the depths of flow at the beginning and end of the transition.

Chute cavitations:

High velocity of flow causes cavitations. Therefore, as per IS-12804-1989 cavitation is formation of gas phase within liquid. To avoid and minimize it the surface is to be finished smooth with no abrupt or gradual irregularities. Using resistant materials like steel liner, epoxy concrete, epoxy mortar, fibrous concrete can also check the cavitations.

Channel chute Aeration

As per IS-12804-1949 the devices are needed to introduce air in the flow at regular interval of the chute channel so as to reduce the channel bed erosion and damages due to cavitation of high velocity flow [[4(iii)]]. Shape and size of the aerators are decided on a hydraulic model. Air demands on the basis of the satisfactory performance of existing structure is found to be 10-cumec/m width of chute as a rough estimate.

The chute spillway has been used with earth fill dams more often than any other type. Factors influencing the selection of chute spillways are the simplicity of their design and construction, their adaptability to almost any type of foundation condition, and the overall economy often obtained by the use of large amounts of spillway excavation in the dam fill. Chute spillways have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay. The examples are Ramganga dam spillway in Uttranchal, India and Terbela dam Pakistan etc.

2.3 Side channel spillway

As per the IS-5186-1989, Side channel spillway is one in which the control is placed along the side of and approximately parallel to the upper portion of the spillway discharge channel. Flow over the crest falls into a narrow channel trough adjacent to the weir, turns at approximately right angles, and then continues into the main discharge channel.

Figure 2.6 below give general features of side channel spillway.

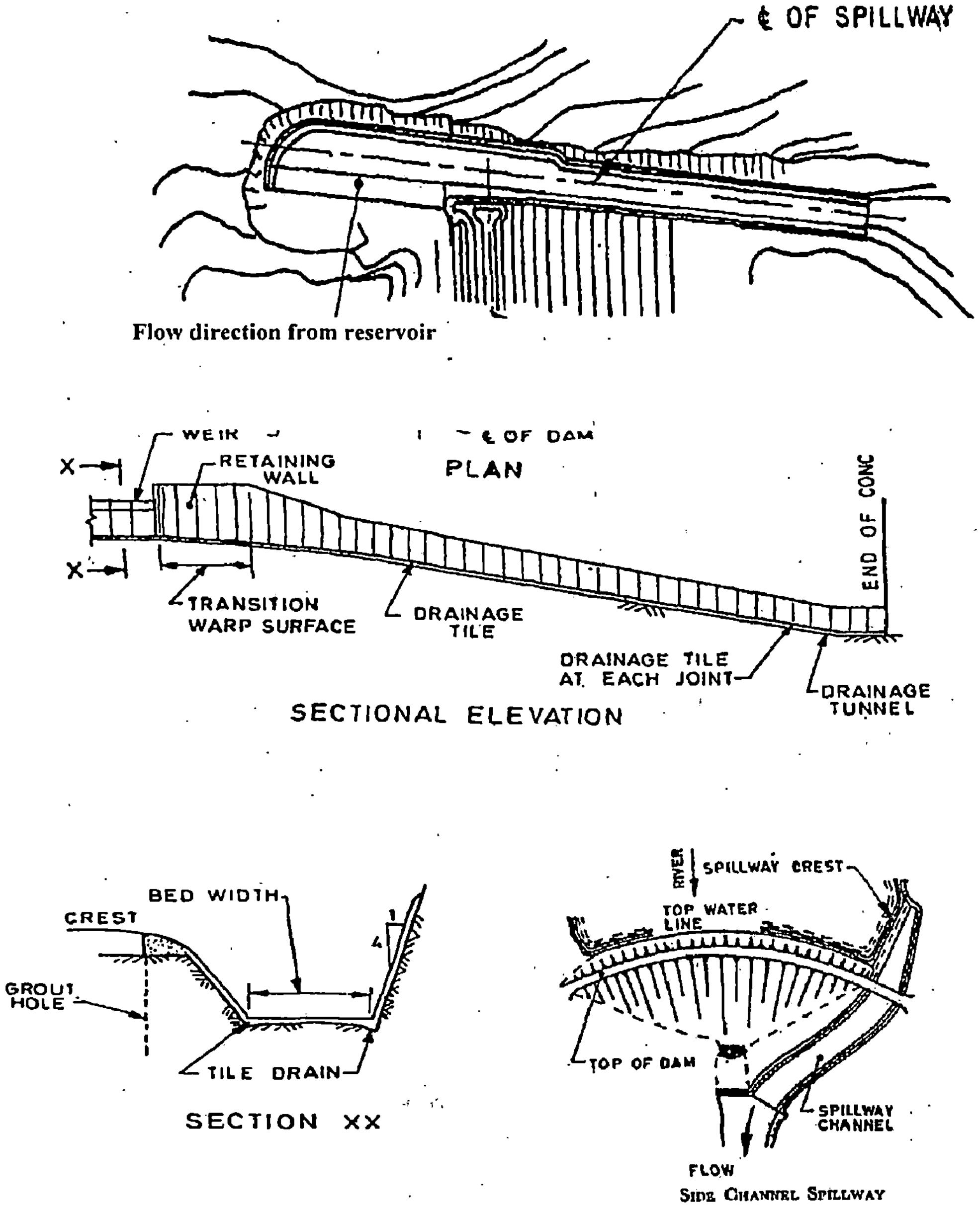


Fig.2.6 IS-5186-1994 side channel spillway [44]

Flows from the side channel can be diverted into an open discharge channel or into a closed conduit or inclined tunnel as the case may be as per the site location. Flow into the side channel might enter on only one side of the trough in case of any steep hillside location, or on both sides and over the end of the trough if it is located on a gently sloping abutment. There is a sub-critical flow in the side channel and supercritical flow in the main discharge chute. The design of side channel spillway involves certain special hydraulic and economic factors. Julian Hinds (1926) was the first ever who formulated different aspects of side channel spillway [22].

A brief description of some elements of side channel spillway is given below.

Control Structure

IS-5186-1984 describes control structure that limits or prevents outflows below fixed reservoir levels and regulates releases when the reservoir rises above that level. Control structure in plan may be straight, semicircular, or U-shaped. The crest may be gated or ungated. The overflow section may be ogee shaped or broad crested. The crest profile should be designed to suit the conditions of gate operation.

Trough

As per IS-5186-1984, side channel trough is placed approximately parallel to the upper portion of discharge channel or perpendicular to the dam axis along the abutments. For the hydraulic design of side channel trough, it assumed that:

1. The entire energy of flow at crest is dissipated through its intermingling with channel flow.
2. The only force producing motion (axial velocity) in the channel is due to water surface slope.

The drop in water surface profile is given by:

$$\text{Or } \Delta y = \frac{Q_2}{g}\frac{(V_1+V_2)}{(Q_1+Q_2)}\{(V_2-V_1)+\frac{V_1(Q_1+Q_2)}{Q_2}\} \quad \text{------------------------2.4}$$

Where Δy is the drop in water surface between two sections of the channel and Q_1, V_1, Q_2 and V_2 are the discharge and velocity at the beginning and the end of the trough section taken. A trapezoidal cross-section with minimum width to depth ratio is recommended for the side channel trough.

If the width to depth ratio is large, the depth of flow in the channel will be shallow, resulting in poor diffusion of increasing flow with the channel flow and with greater bed width; the excavation will increase. The side slopes should be the steepest angle at which the material will safely stand. It may be lined with concrete anchored directly to the rock.

Control Section

As per IS-5186-1984, the control section is a section downstream of side channel trough where the depth of flow changes from sub critical to supercritical.

The best location of a control section is usually at point where bed slope has to be in cut to keep the channel near to the ground fig-2.6.. The critical depth at the control is given by d_c

$$d_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \quad \text{--2.5}$$

Where, q is the discharge per unit width of the section and g is the acceleration due to gravity.

This Side Channel type of spillway is preferred where a long overflow crest is desired in order to limit the intensity of discharge, where the abutments are steep, where desired flood discharge is high for relatively shorter period of time. This type is suitable where abutments are in rock and stable. Though hydraulically less efficient, side channel can have long crest using duckbill type overflow crests.

2. 4 Shaft tunnel or Morning Glory spillway

USBR 'Design of small dams' describes the Shaft tunnel as a vertical shaft and a horizontal tunnel combination. It has a funnel like control crest and a vertical shaft following a horizontal tunnel, generally a diversion tunnel. Morning glory is called because it shows that flower like shape on operation. The intake may be gated for flow regulation and debris control coming along the flow. Standard crest type is used generally for the tower intakes and flat crest type is used for the overflow spillways in narrow gorges where space is available.

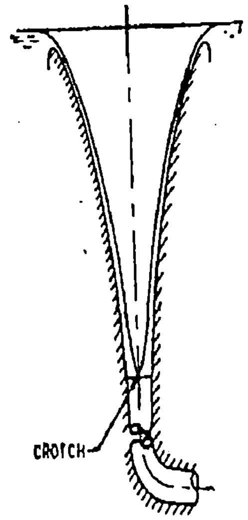


Fig 2.7. (a) Standard crested shafts[22]

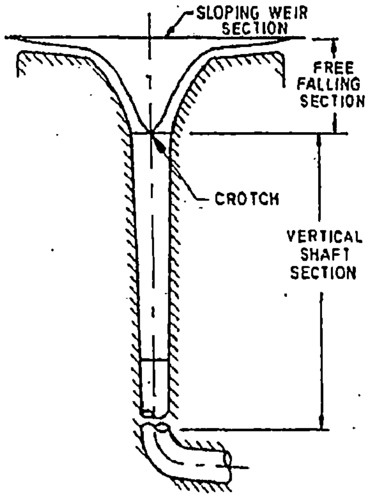


Fig-2.7 (b) flat crested shaft[22]

Types of weirs

- Standard crested.
- Flat crested.

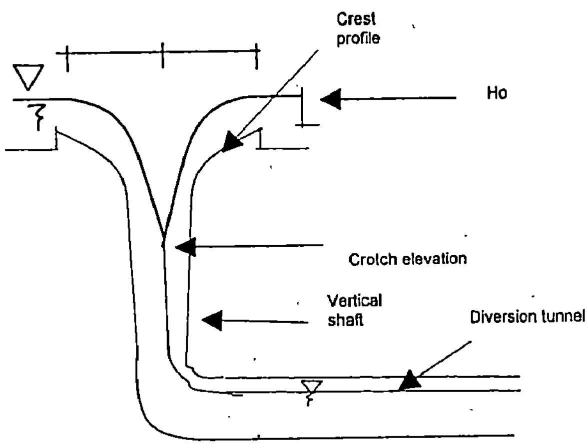


Fig-2.8. Crest control tunnel flow [5]

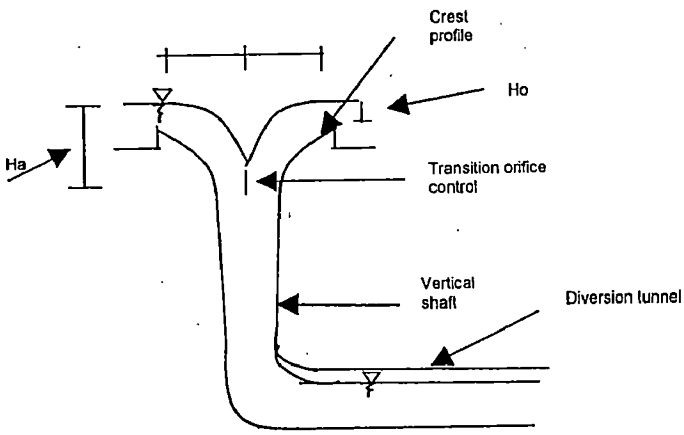


Fig-2.9 Orifice control of the [5]

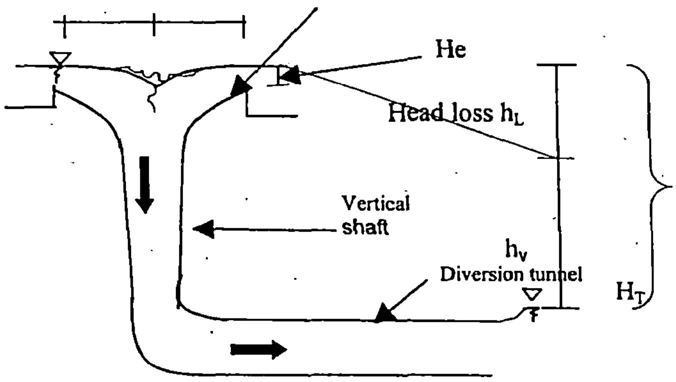


Fig-2.10 Pipe- control shaft- tunnel [5]

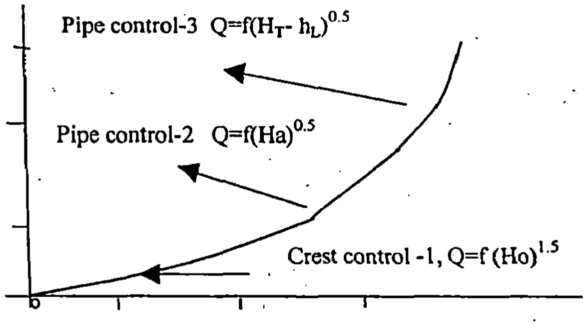


Fig-2.11 Relation of the flow in shaft [5]

The standard crest has a smaller diameter since its coefficient of discharge is greater than that of a flat crest as per USBR. Other requirements being same, the standard crest is more advantageous in case of the spillway is a tower type. On the other hand the flat crested spillway has a funnel structure before the drop. Therefore it is preferred where spillway is excavated in rock.

Discharge characteristics of the drop inlet spillway as per USBR 'Design of small dams' vary with range of head, which finally is governed by the pipe flow condition as shown in figure 2.8-2.11. In this type of spillway, maximum capacity is attained at low head and the increase in discharge beyond design head is little corresponding to large increase in the water level as control shifts to the outlet where discharge is proportional to square root of (H_T-h_L), where H_T is the total head and h_L is the head loss in the shaft.[5]

Standard crested weir

For small heads flow over drop-inlet spillway is governed by characteristics of crest discharge. The vertical transition beyond the crest will have partially full flow and it will cling to the shaft tunnel. As discharge increases, the overflowing annular nappe will become thicker and eventually the nappe flow will converge into a solid vertical jet. The point making annular nappe to solid vertical jet is called crotch and after the formation of solid jet, a boil will occupy the region fig 2.7(a).

When the flow confirms weir characteristics, the design discharge, Q is given by

$$Q = CLH^{\frac{3}{2}} \quad \text{------} \quad 2.6$$

Where,

C is coefficient of discharge; L is the length of the circular crest and H is head over the crest. C varies with the length of crest and head over the crest and a relation is given by the USBR design of small dams.

As per USBR, for ogee crest free weir flow governs and remains free fall up to a point of the ratio of H/R <0.45, where H is the head over crest and R is the radius of the shaft, then it changes into submerged flow for H/R ratio equal to or greater than 1. From 0.45 to 1 flow remains in transition. For H/R =0.4 boiling point (crotch) is at the crest level.

Beyond vertical shaft is the elbow portion where flow changes the direction and the velocity is highest at the point. To see the negative pressure at this point (P) $=P_a + (p - P_c)$ where P is absolute pressure at bend, P_c reduced pressure due to centrifugal force and p is average pressure at the given cross section of elbow. P_a is the atmospheric pressure 10m of water head. If $p - P_c$ is negative then cavitations occurs in the tunnel. The pressure at the sections elbow P when reaches to vapor pressure, is unable to increase velocity of flow, thereby discharge in shaft reduces.

Flat crested morning glory

As per Creager Justin and Hinds 'Engineering for Dams', the required capacity discharge (Q) is to be passed at a maximum head h fig-2.12. Discharge intensity can be found from weir formula, $q= ch^{3/2}$, where c is coefficient of discharge and h is the head over the crest (c may be taken as 1.6 conversion to the metric units)[22]. Then, The radius of the flat crest $R_s = \frac{Q}{2\pi q}$, where, R_s is the radius of the crest, Q is the flood discharge and q is the discharge per unit length. The drop down curve has minimum length of 2h and head loss at crest is f is given as 0.04h.

A model test in this regard is necessary to check the satisfactory performance of the shaft. Head loss in the shaft may be taken as 0.1H. Profile can be plotted by Kurtz equation given as[3]:

$$y + 0.36h_1 = \frac{(x + 0.36h_1)^2}{4.56h_1} \qquad 2.7$$

Where, x and y are the co-ordinates from the free fall point and h_l is the elevation of the water surface profile from the bed of the free fall point.

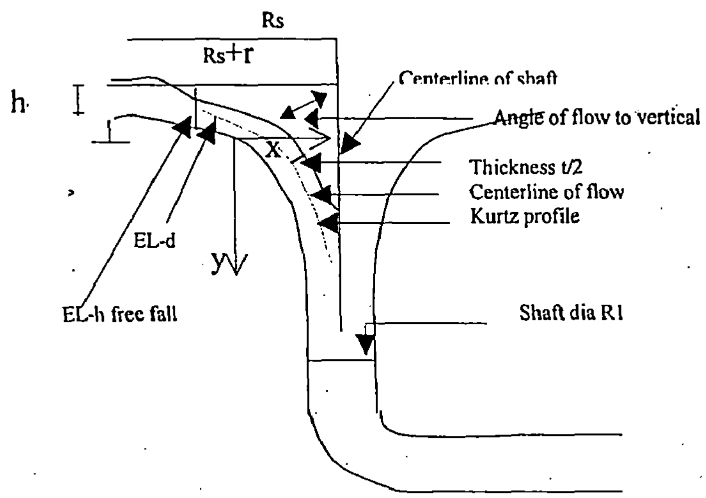


Fig. 2.12 flat crested – flow profile[22]

The shaft tunnel type of spillways can be used very advantageously in dam sites in narrow canyons. Here the minimum discharging capacity is attained at relatively low heads. This characteristic makes the spillway ideal where the maximum spill is to be limited. It is undesirable where a flood more then design capacity is to be passed. Hence it can be used as a service spillway in conjunction with an emergency spillway. It has difficulty of air-entrainment in a shaft, where downstream is free flow (not a submerged flow) at outlet. For such conditions diversion conduit is to be kept 75 percent flow as per tail water condition.[2] It can be adopted in conjunction with chute or side channel spillway where the topography permits the economy.

2.5 Energy Dissipators

The velocity of water passing through a high head spillway is so high that it is liable to cause erosion of the bed of the channel or river immediately below the toe of the structure. This high velocity is on account of the difference of head between the headwater and the tail water elevations. This high energy of flowing water, therefore, has to be killed, before the water is discharged in the tailrace channel or to the river.

In most of the methods of dissipation of excess flow energy, the underlying concept is to create turbulence at desired locations. The shear stresses generated in turbulent flows are able to dissipate the energy at a very rapid rate.

Some of the most common methods of energy dissipation below high head spillways are:

- Hydraulic Jump Type Stilling Basin
- Roller Bucket Type Energy Dissipator
- Ski-jump Bucket Type Energy Dissipator

The shaft swirling flow [3] at the junction of vertical shaft with a tunnel is found to dissipate energy. It is being applied in Tehri dam. Another method of high head energy dissipation is splitter bucket type, which can be applied in a high ogee spillway. It has been finalized on the model study for Lakhwar dam. IS code-10137-1982, 11527-1985 and 7365-1985 describe the hydraulic parameters for the selection of the type of dissipators and their hydraulic designs.

Type of high head spillway and energy dissipators of various projects.

Table 2.1 Summary showing the chute spillway used:

Sn	Project	Spillway capacity	Drop	Max velocity	Energy dissipater	Date of constr.
1	Tehri Spillway[1]	5480.0 cumecs	215.0m	60.0m/s	Stilling basin baffle wall	Under const.
2	Ramganga Spillway[7]	7607.0 cumecs	100.0m	40.0m/s	Stilling basin	1990
3	Terbela dam Spillway[15]	42000.0 cumecs	137.0m	50.0m/s	Stilling basin	1975
4	Beas dam spillway[19]	12380	76.2.0m	48.0m/s	Stilling basin	1975

Table 2.2 Summary showing the shaft tunnel spillway constructed:

Sn	Project	Spillway capacity	Drop	Max velocity	Shaft dia. Energy dissipating device	Date of constr.
1	Tehri Spillway[1]	7560.0 cumecs	215.0m	60.0m/s	12.0m/ Swirling flow at junction	Under const
2	Hoover dam shafts[21]	5665.0 cumecs	180.0m	56.0m/s	16.0m/ flip bucket Plunge pool	1936
3	GlenCanyon Spillway[20]	4000.0 cumecs	174.0m	52.0m/s	12.5m/ "	1964
4	Hungry Horse[1]	1910.0 cumecs	104.0m	43.0m/s	9.4 / Plunge pool	1953

Hoover dam has side channel spillway combination.

Table 2.3 Summary of the Ogee spillway constructed for high head in the region:

Sn	Project	Spillway capacity	Drop	Max velocity	Type	Date of constr.
1	Bhakra dam spillway[8]	8212.0 cumecs	155.0m	54.0m/s	Horiz'l basin	1970
2	Indera Sagar spillway[8]	83534.0 cumecs	90.0m	42.0m/s	Slotted Roller	Under const
3	Sardar Sarober[9] spillway	86000.0 cumecs	140.0m	50.0m/s	Stilling basin &Flip bucket	Under const

Chapter 3

REVIEW OF HIGH HEAD SPILLWAYS WITH RESPECT TO PERFORMANCE

3.0 General

High head spillways had problems in the past due to inadequate capacity, cavitation damage, surface erosion and lifting and sweeping away of the basin floor by hydrodynamic forces caused by pressure fluctuation, erosion and crack formation due to temperature change in the spillway chute bed, side walls and damage due to vegetation during no flow time and debris circulation during high flood.

3.1 Causes of Damage/ Failure of Spillways

Main causes of high head spillway damage and failure are categorised below.

1. Inadequate capacity of the spillway.
2. Cavitation due to high velocity
3. Temperature variation during operation causing freezing and thawing
4. Scouring, side erosions and pitting
5. Uplift due to inadequate drainage
6. Debris erosion and cutting
7. Hill slides along chute and side channel and jump basin

Spillways are critical to the safe operation of every dam and must be inspected and monitored. Many problems that occur at spillways may not be visible until damage or failure occurs. This is particularly true with problems that develop under water parts or tunnels and shafts.

Fig-3.1 -3.5 give some examples photo view of the spillway damages.

Following table describes the spillway damage/failures of some dams.

Table number 3.1

Dam spillway	Type of spillway	Energy dissipater	Failure/damage caused by	Dam height
Pit-6 dam USA[3]	Ogee	Hydraulic jump basin	Chute and baffle block eroded and side walls pitted	40.60m
Pit-7 dam USA[3]	Ogee	Hydraulic jump basin	Chute and baffle block eroded and side walls pitted	60.97m
Pandoh (Beas) dam India[19]	Chute	Hydraulic Jump basin	Abrasion, scouring and erosion	76.2.00m
Matatila India[3]	Chute	Hydraulic Jump basin	Cavitation damage in the appurtenances	31.00m
Malpaso dam Mexico[3]	Chute	Hydraulic jump basin	Uplift forces caused by turbulent pressure fluctuation	138.00m
Dwarshak Dam USA[3]	Chute	Hydraulic jump basin	Erosion of basin side wall and floor	219.00m
Libby Dam USA[3]	Chute	Hydraulic jump basin	Erosion of basin side wall and floor	136.00m
Terbela dam Pakistan [15]	Chute	Hydraulic jump basin	Erosion of basin side wall and floor chute concrete erosion due to high velocity flow cavitation	143.30m
Glean canyon USA [17]	Tunnel shaft	Impact pool	Erosion due to high velocity flow cavitations	174.00
Hoover dam USA [15]	Side Channel tunnel shaft	Impact pool	Erosion due to high velocity flow cavitation	180.00m

3.2 Chute Spillway Cases

After few years of operation of spillway, most of the appurtenances of the basin of 31.00 m high Matatila Dam India were observed damaged and eroded [[3]]. The concrete of sidewalls of the basin was found pitted. Cavitation was considered responsible for this damage and required repair works, which are carried out annually with additional safety factors.

Fig.3.1 Large spillway structural cracks with vertical displacement
Of a dam spillway Indiana USA [[14]]

Philip B Williams in his paper titled "Three Gorges Dam safeties! 1992" concludes 'operating experience with extremely large flows through high head chute spillways has not been good. At the Tarbela Dam (Pakistan), and the Glen Canyon and Hoover Dams (U.S.), extremely high velocities and pressures caused cavitation and erosion which threatened the structural integrity of the dam and necessitated serious and costly repairs'.

In the sub Himalayan region, there are few high head spillways constructed so far namely Pandoh Dam (Beas dam) chute spillway and Ram Ganga Dam chute spillway. Considerable abrasion /erosion of the spillway chute and flip bucket was observed as reported in publication number-4, report number-9 march 2004 [[19]]. Dam Safety committed advised to have a regular inspection of the sill of the crest to clear the debris etc. when reservoir level falls below 890.016m along the gate. The condition of the exposed portion of the spillway like walls, piers, chute, flip bucket and Brest wall was found with no sign of any distress. A costly maintenance is a regular work of the department for the damage due to cavitations, variation in temperature and also due to high velocity flows during high floods by epoxy mortar and concrete every year. To overcome these problems extensive modelling exercise is done at UPIRI and results of the modelling are new design like splitter bucket arrangements for the chute spillway flip buckets energy dissipation.

The damage due to high velocity flow in chute spillway in Indiana USA is shown in fig 3.1-3.3 as examples. These damages are due to high velocity flow cavitation and inadequate quality of concrete.

Fig. 3.2 showing the spillway concrete damage of a dam spillway Indiana USA [14]

Fig. 3.3 Showing spillway extensive cracking and general deterioration as a result of freeze- thaw effect- Of a dam spillway Indiana USA [14]

Concrete is generally used for construction of spillways. Mainly rich concrete is used in different grades for selected areas such as facing, erosion resistant layers on spillways and energy dissipation systems, piers, guide walls etc. The performance of concrete made with aggregates containing certain active materials and exposed to aggressive waters and with aging effects is the main concerns. The crack developed due to differential temperature and cooling, damages the surfaces of the chute spillway.

The strength of mass concrete increases approximately 40% in 5 years to that of 28 days strength. In the case of Idukki dam in Kerala, constructed during 1969 to 1974, the over cored samples collected in 1995 gives average modulus of elasticity of about 0.400×10^6 Kg/cm^2, against an initial value of 0.150×10^6 Kg/cm^2. So at initial phase of operation it is necessary to take extra care in operating spillway with high floods [8].

The cutting in the floor is due to the erosion effect of the high velocity flow. For the measure to be taken where gully cutting is deep and gully floors are steeply inclined, the structural works become necessary. With very severe and deep gully heads, the construction of expensive concrete works in drops spillways; chute spillways or flumes may be necessary. The purpose of such devices is to control gully head scouring and undercutting by conveying and discharging inflowing run-off over and away from the active erosion site and protecting the site itself from further scouring by providing some form of armouring and energy dissipation.

3.3 Shaft Tunnel Spillway Cases

Hydraulic jump basins and plunge pools have some inherent problems. As per Thomas J. Rhone, "These can include abrasion damage due to circulating debris in the basin, cavitation damage to the appurtenances due to high-velocity flow, or damage to the walls due to vibration. Generally, but not always, these factors can be predicted and corrected by model studies. Cavitation damage by high-velocity flow in tunnels and chutes has also been a problem. Aeration slots have been added to most of the reclamation's tunnel spillways to prevent cavitation. The severe erosion damage that occurred at Hoover, Yellowtail, and Glen Canyon Dams are examples of near catastrophes leading to a major change in design concepts due to cavitation" [16].

In the Arizona tunnel spillway total capacity 11340 m^3/s as early as the winter of 1941 at Hoover Dam when operated for 116 days, there was suspicion of the vulnerability of concrete damage caused by high velocity flow in tunnel spillways. This spillway operation resulted in a large hole in the tunnel spillway elbow 14 m deep, 9 m wide and 35 m long. The damage was thought to initiate at a "misalignment" of the tunnel invert just above the elbow. The damage was caused by high velocity flow passing over the roughness and leading to bubble formation (similar to boiling water) in the flow. When the bubbles collapsed, high-energy shock waves were generated damaging the concrete. This phenomenon is referred to as cavitation formation and damage. In the 1940's backfilling with river rock and then covering with a thick layer of high quality concrete repaired the damage. The concrete surface had a very fine finish, almost terrazzo, to prevent reoccurrence of the cavitation.

The cavitation damage problem surfaced again in June and July 1967 when the tunnel spillway at Yellowtail Dam discharged for 20 days at 425 m3/s. By July 14 it was evident that there was a problem in the tunnel spillway. When drained and inspected a hole 2 m deep, 6 m wide and 14 m long was discovered. In earlier laboratory investigations, the introduction of as little as 7.5% air into the water flow eliminated damage associated with cavitation on concrete surfaces. The first installation of an aerator in a tunnel spillway was at Reclamation's Yellowtail Dam [16].

In 1983, high runoff in the Colorado River basin created the need to pass flood flows through tunnel spillways Glen Canyon Dam fig.3.4. The damage is due to cavitation. The resulting damage was so extensive at Glen Canyon Dam's two tunnel spillways that $42,000,000 (1985 costs) and a year of reconstruction was required to repair the spillways and install an aerator in each tunnel.

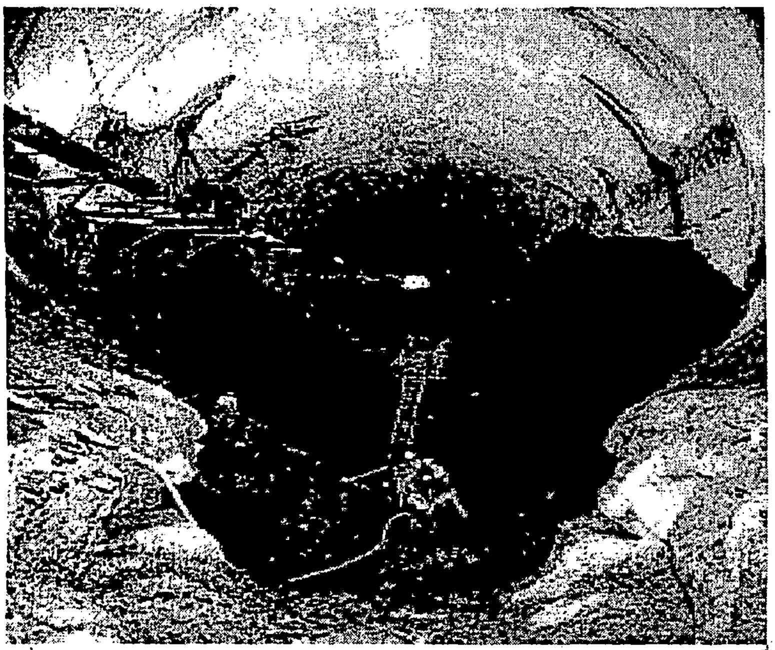


Fig. 3.4 -Damage to Glen Canyon Dam left spillway in 1983.The "big hole" was 11 meters deep [16].

Fig. 3.5 Riser lacking a trash rack can clog and cause dam overtopping- Of a shaft spillway intake Indiana USA [14]

To overcome these problems of cavitations extensive modelling exercise is done at UPIRI and results of the modelling are new design like creation of swirling flow in the shaft tunnel elbow, which is being used in Tehri spillway shafts. Another special-purpose basin is the hollow-jet valve basin. Outlet works controlled by slide gates can use Basin II, Basin III, or a plunge pool; but due to its unusual jet shape, this valve seemed to require a unique basin. It has worked exceptionally well at Boysen Dam in Wyoming, Falcon Dam in Texas, and Yellowtail Dam in Montana,

3.4 Ogee spillway cases

The damage /failure in an ogee spillway is generally at the gate openings, along the downstream glacis due to high velocity cavitations and failure/damage to the hydraulic jump basins, side walls and floor as a result of pressure fluctions.

Two concrete gravity dams namely Pit 6 and Pit 7 of Pit river hydroelectric project (USA) are provided with hydraulic jump basins with chute and baffle blocks and end sill as energy dissipation. The damage was found in chute block and basin floor [3].

In the Gandhisagar dam spillway Rajasthan India [8] (56.7m head) a maximum scour of 10.67m at a distance of 76.20m to 91.44m from the ski-jump bucket is noticed. In Vaitarna dam Maharastha India (72.4m head) with flow of 1415.00 cumecs the flow jet washed away the high overburden in front of the bucket. Subsequent model study recommended installing sufficient aerators in the central 5 spans of the spillway buckets with 35-degree lip angle.

To overcome these problems extensive modelling exercise is necessary to design and the necessary structures.

The side channel spillway performance is not studied here, as cases are similar to chute spillway flow characteristics in the side channel flow.

The Chinese statistics [13] showing dam failure and cause of failure in percentage is given below in tabular form. It shows that actual spillway damage / failure is small in comparison to total dam failure in China as precaution measures are handled effectively.

Table number 3.2

No	Causes	Percentage
1	Overtopping, including	51.5
	1) insuffiency of spill facilities	42.0
	2) extreme flood exceeding design criteria	9.5
2	Piping and other seepage problems, including	29.1
	1) Piping in dam body	22.7
	2) Piping at foundations	1.3
	3) Piping around spillway	0.6
	4) Piping around tunnel	4.5
3	Other structural problems	9.4
	1) Slope, slide of dam body	2.6
	2) Quality trouble in spillway	6.0
	3) Quality trouble in tunnel	0.8
4	Poor management, including	4.2
	1) Decrease of reservoir standard for flood control due to over storage prior to flood season	1.1
	2) Poor maintenance and operation	1.3
	3) Temporary bag dam on spillway crest failed to remove in time	0.5
	4) Nobody in charge of management	1.3
5	Others including	4.6
	1) Spillway blocked due to blank slide in reservoir	1.7
	2) Digging breach on dam face for discharging	2.3
	3) Poor planning of general layout of project	0.6
6	Unknown	1.2

Dam failure in china- table from ICOLD bulletin no 109 Hydrocoop.org-2003

A model study of respective high head spillway becomes necessary before finalizing the design to access probable damage failure area and to adopt the suitable measures of prevention of such damages. Adopting required concrete strength, adopting required thickness of the discharge channel and sidewalls, adopting enough free board and applying side slope as per the site requirement could minimize these problems of spillway damage/ failure. The suitable devices for energy dissipation for high head spillway shafts like swirling device tested at UPIRI for the shaft tunnel and also URBR recommendations vide special-purpose hollow-jet valve basin can be adopted for jet energy dissipation. It is necessary that any device proposed may be tested in the model before adoption.

Chapter 4

PANCHESHWAR DAM PROJECT

4. General

Pancheshwar Multipurpose Project (PMP) is a high dam bi-national project between India and Nepal. Under the Mahakali River Treaty between Nepal and India, Pancheshwar Multipurpose Project is planned as high rock fill dam. The location is 500 km (aerial distance) for Kathmandu and 290 km approximately from New Delhi on the a border River Mahakali in Baitadi District, of Nepal and Champawat District, of Uttranchal, India.

4.2 Project Details

4.2.1 Project area

The Project is area is located in the Sub-Himalayan mountainous region. Total catchment area is 12100.00 square km. There are 7 tributaries that meet in Mahakali River before it reaches to the Pancheshwar dam site. Fig- 4.1 shows the watershed of the Pancheshwar Multipurpose Project.

4.2.2 Hydrology

(i) Precipitation/ Climate

From the observed records the precipitation in the catchment ranges between 1000-2000mm annually with a minimum of 1075 mm. From the isohyetal maps over the catchment, the annual average precipitation is estimated as 1620 mm. 75% of this rainfall occurs during monsoon month of June to October. The maximum observed average temperature at the dam site is 30 degrees centigrade and the minimum is 13 degrees.

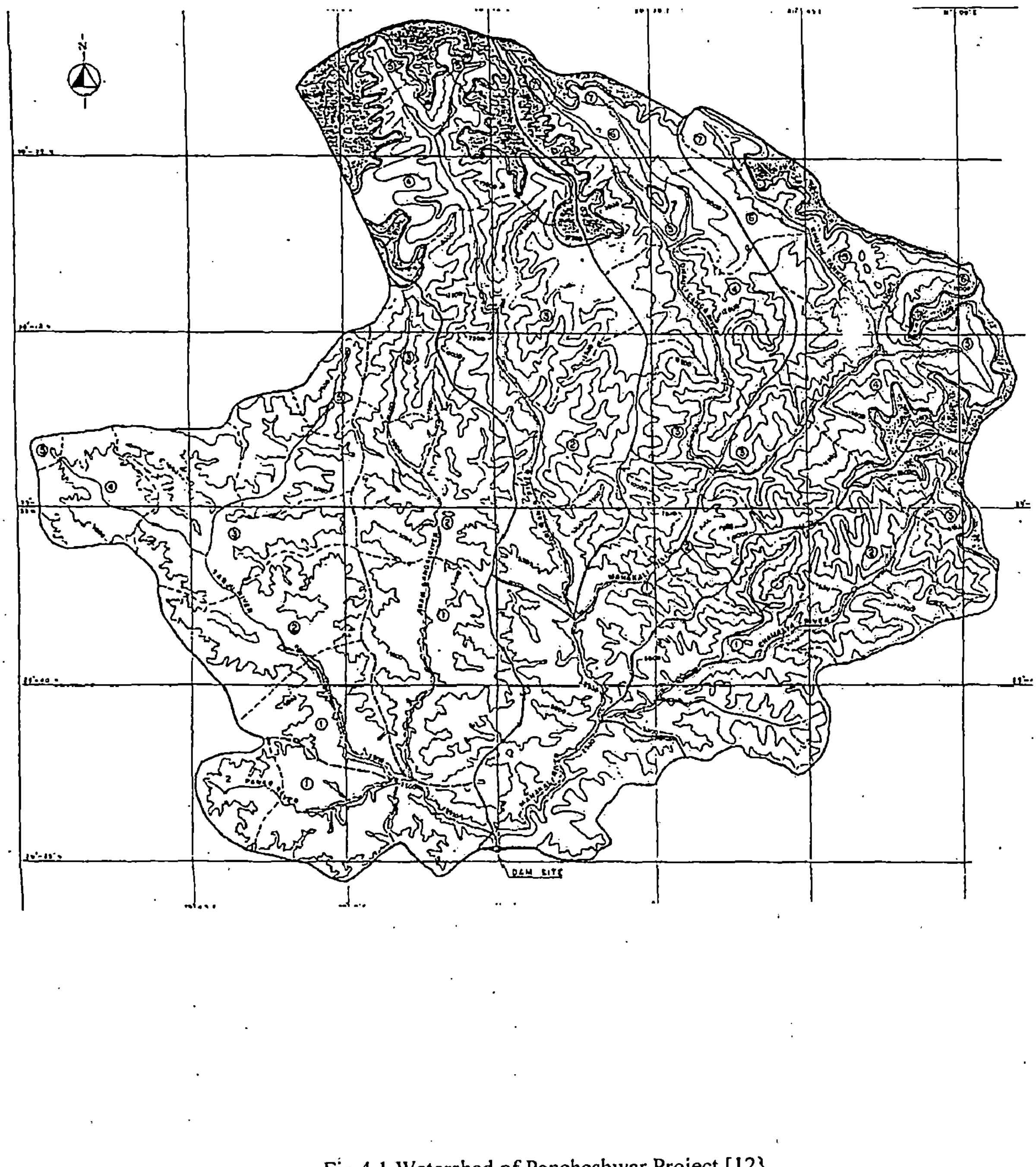


Fig.4.1 Watershed of Pancheshwar Project.[12}

(ii) Average wind velocity and sunshine hours

The average wind velocity for 3 hour is 28.0 km/h and for 24 hours average, it is 8.70 Km /h. Peak wind velocity considered is 28.0 Km/h. The maximum-recorded sunshine duration is 10 hours and 30min.

(iii) Run off available

The daily average month wise discharge in cumecs is as below.

Table no. 4.1

Average monthly flow data:											
Months	**Q**	**Months**	**Q**	**Months**	**Q**	**Months**	**Q**	**Months**	**Q**	**Months**	**Q**
January	165	March	156	May	335	July	1338	Sept.	1198	Nov.	277
February	148	April	203	June	634	August	1798	October	534	Dec.	195

The yearly distribution of average flow is given in the annexure-2 page no 96 and the average discharge of 32 years is **582.0cumecs.** The annual daily maximum flows at Pancheshwar are given following in the table:

Table no.4.2

Annual Max Daily flow at Pancheshwar site								
Year	1962	1963	1964	1965	1966	1967	1968	1969
Discharge m^3/s	4249	4656	4096	1891	3147	4232	4622	4537
Year	1970	1971	1972	1973	1974	1975	1976	1977
Discharge m^3/s	6183	6343	4715	4622	3892	6839	3241	4181
Year	1977	1978	1979	1980	1981	1982	1983	1984
Discharge m^3/s	3914	2874	3616	3812	3892	9241	2884	4174
Year	1985	1986	1987	1988	1989	1990	1991	1992
Discharge m^3/s	3694	3174	3362	3473	3455	3231	3223	5430

(iv) Flood Estimation

Flood estimate for the Pancheshwar dam site: Flood estimate is based on the three-parameter log normal distribution and is given in the following table.

Table no.4.3

S. N	Return period	Flood estimate (cumecs)
1	X_2	3924.5
2	X_5	4879.78
3	X_{10}	5468.64
4	X_{20}	6007.98
5	X_{50}	**6678.68**
6	X_{100}	7166.81
7	X_{500}	8266.66
8	X_{1000}	8732.87
9	X_{10000}	10275.46

(v) Probable maximum flood

Clark's method is used for calculating unit hydrograph from the observed flood of July 9, 10 – 1990 on the river Mahakali at Pancheshwar.

From the one hour unit hydrograph and the and time distribution of 72 hours and taking into account the 0.3 cm/h initial infiltration for 4 hours and then considering a 0.1 cm/h infiltrations and using the storm of 28, 29, 30 September 1924, the probable max flood calculated is 18174.82m^3/s which is found close to a value calculated by using CWC guidelines.

The design flood adopted is **23,627m^3/s** is 1.30 times of calculated pmf [[12]].

4.2.3 Geology of dam site

Mica Schist with thin bands of Quartzite occurs upstream and down stream of the damsite, bounding an 800m wide band of higher grade metamorphic rocks comprising augen gneiss, intrusive granite and granitized schist and quartzite. Contacts between units are generally closed and oriented parallel to the schistosity-foliation and the regional structure.

Weathering extends to a depth of about 10m near the river level, increasing to 50m or more in the upper parts of the abutments. A 20m thick alluvial layer covers the bedrock in the river bottom near dam axis. No significant faults are mapped in the vicinity of dam site.

At depth, the rock of the Pancheshwar dam site can be considered fairly sound, in particular, the granitized schist and augen gneiss are good to very good rocks, where large cavities can be excavated without resorting to special support measures.

4.2.4 Spillway site

Dam site and spillway site located on left bank adjacent to dam abutment has geology similar to dam site. The spillway is proposed to be laid on the sound rock. The depth of overburden on top of weathered rock is, as indicated by the investigation reports of the project, around 15.00 m and to sound bedrock, it is about 50.00 m. The excavation required to fit the spillway layout varies from 30.00 m to 70.00 metres at the discharge channel area. At control section the depth of excavation is 70.00 m and at stilling basin plunge pool the depth of excavation is around 60.00 m.

The three alternatives considered are located on the same left abutment of the Pancheshwar Dam and has similar geology. The shafts geology is similar to dam axis geology.

4.3 Salient features of Project

The salient features of the project are given in the following table.

Table4.4

	Salient Features of the Pancheshwar Dam Project	
Sl. No.	**Details**	
1	**Hydrology**	
	Drainage area	12,100 Sq.Km
	Average flow	582.0 cumecs
	Design flood (pmf)	23,500.0 cumecs
	Sediment load	64 million tons/year
2	**Reservoir**	
	Normal Max. Water level (NMWL)	680.0 EL
	Minimum operation level	615.0 EL
	Reservoir area at NMWL	134.0 Sq Km
	Live Storage	6.56 billion cubic meters (BCM)
	Dead Storage	5.7 BCM
3	**Dam**	
	Type	Rockfill with central clay core
	Slopes	2.2:1 U/S, 1.9: 1 D/S
	Crest EL	695.0 EL
	Maximum Height	315.0 m
	Crest Length	860.0 m
	Volume	62.7 million cubic meters
4	**Diversion**	
	Diversion flow	8,000 cumecs
	Tunnel Number and Length	Two 1,500.0 m
	Tunnel Dimension	12.20m wide 17m high Horseshoe
	U/S Coffer Dam	Rockfill 71.0 m high
	D/S Coffer Dam	Rockfill 40.0 m high
5	**Spillway**	
	Type	Side Channel, Free Overflow
	Ogee crest EL	680.0 EL
	Maximum Flood Level	693.30 EL
	Maximum Discharge	18,000.0 cumecs
6	**Power Facilities**	
	Arrangement	Two Underground powerhouses one each side
	Net design Head	236.80 m
	Potential (base load)	970 MW
	Potential at 0.2 plant factor	4,850 MW
	Average Energy Production	10.671GWh/year

4.4 Flood routing

4.4.1 General

To design a spillway first step is to check the expected flood that has pass over the designed section. The constant flow is required by the irrigation systems down stream for which down stream storage for re-regulation of flow is required. The normal maximum water level for the plant is selected at 680 masl by the project.

Flood routing and reservoir operation studies are generally aimed at deciding suitable normal reservoir level for combined the power and irrigation requirements as well as for design of spillway.

4.4.2 Basic data:

(i) From the reservoir contour map following area capacity data are worked out.

Reservoir Area and capacity [12]

Elevation (m)	Area (sq.km.)	Volume (106m)	Elevation (m)	Area (sq.km.)	Volume (106m)
700	131.60	13830.70	550	36.50	1854.40
690	123.70	12553.90	540	32.00	1511.70
680	116.10	11354.80	530	27.80	1212.40
670	108.60	10231.50	520	23.90	954.00
660	101.00	9181.80	510	20.20	733.70
650	94.30	8203.40	500	16.80	549.00
640	87.50	7294.30	490	13.60	397.10
630	80.90	6452.20	480	10.80	275.20
620	74.60	5674.80	470	8.20	180.30
610	68.40	4960.00	460	6.00	109.50
600	62.50	4305.40	450	4.00	59.60
590	56.80	3708.80	440	2.40	27.50
580	51.40	3167.80	430	1.20	9.40
570	46.20	2680.00	420	0.30	1.70
560	41.20	2243.00	410	0.00	0.00

(ii) Flood hydrograph

From the unit hydrograph flood hydrograph is derived as given below [12].

Flood hydrograph ordinates											
Time hours	0	4	8	12	16	20	24	28	32	36	40
Discharge Cumecs.	1000	1350	1880	2660	3838	6230	8090	10080	11960	12700	14110
Time hours	44	48	52	56	60	64	68	72	76	80	84
Discharge Cumecs.	17260	21130	**23270**	22355	20160	18735	18960	19245	19080	17625	15895
Time hours	88	92	96	100	104	108	112	116	120	124	128
Discharge Cumecs.	13260	12085	10665	8860	7555	6865	4985	4035	3405	2620	2390
Time hours	132	136	140	144							
Discharge Cumecs.	2025	1990	1610	1315							

4.4.3 Flood Routing Studies Results

The out flow from the reservoir depends on the selected discharging facilities that is type of spillway selected and operation mode i.e. gated or un-gated. So for proposed side channel spillway, an overflow section un-gated is considered at EL 680.00m and the routing is carried out given in annexure-1 (a) page 90-91. The results of the study are as below.

The side channel width	Peak flood routed	Side channel capacity	Max. Head required above 680.0 masl
175.00 metre (trough width 70.00 m)	23270.0 cumecs	17700.00 cumecs	13.55 meters

Further, a chute spillway alternative with gated control is analyzed. The spillway out flow from the routing studies is calculated and given in annexure-1b page 92-93.

Results of the calculation are given below in the table. The section of the chute is selected on the basis of trials keeping the crest of the gated chute at EL 660.00 m. limiting the gate height to 20.00 m only.

The Chute channel width	**Peak flood routed**	**Chute channel capacity required**	**Max. Head required above 680.0 masl**
76.75 metre (4 bays of 16 m width)	23270.0 cumecs	15350.00 cumecs	9.1 metres

To optimize the spillway flow and also to arrive at an economic design another alternative of a chute spillway combination with two vertical shafts is analyzed. The chute spillway is proposed to pass a flood of one in 50 years return period adopted as 8500.00 cumecs. The shafts designed one in each abutment will pass 3500.00 cumecs at a head of 5.0 m over its crest at El 684.00 m. Routing studies given in annexure-1c page 94-95 show that this discharge will pass through chute when reservoir level will rise to EL 684.00 m. Beyond this level the shafts will start functioning. In this study the outflow will be 18000.00 cumecs with reservoir level at EL 690.10 m.

The Chute channel width plus shafts	**Peak flood Routed**	**Max. Flood passed**	**Max. Head required above 680.0 masl**
60.0 metre plus two shafts of 15.0 m dia.	23270.0 cumecs	18000.00 cumecs	10.1 meters

Summary of Reservoir Routing Analysis

The table 4.5 below gives the summary of the routing exercise.

Type spillway	Length **of spillway crest**	**Peak flood**	**Chute channel escaping capacity**	**Max. Head required above 680.0 masl**
Side channel	175.00meters	23270.0	17700.0 cumecs	13.55meters
Chute single	76.75meters	23270.0	15350.0 cumecs	9.1 meters
Shaft and chute	60.0 meters	23270.0cumecs	(8000.00+2x3500) cumecs	10.1 meters

4.4.4 Effect of alternative spillway arrangement on dam height

The dam height proposed is up to EL 700.0 m is based on provision of side channel spillway. The chute spillway option requires dam height up to EL 695.00 m (taking 5.0 m free board). The option of shaft and chute spillway combination will require dam height up to EL 694.0 m.

4.4.5 Flood Peak Moderation

It is seen from the analysis that if the dam height is kept up to EL 700.0m the flood flow in the river can be reduced considerably. Following table shows the results of the reduction of peak flows of various return periods for the shaft and chute channel option.

Table 4.6

Flood routing through the spillway- summary of flood protection results.

Reservoir already spilling at beginning of flood at EL 680.0m

Recurrence interval years	Peak inflow m3/s	Peak outflow m3/s	Peak reduction		Maximum water level
			m3/s	%	**EL,m**
2	5101.85	5,791.00	00	00	682.90
5	6343.71	5860.00	483.71	7.61	680.30
1	7109.23	5860.00	1249.23	17.57	680.30
20	7810.37	5860.00	1950.37	24.79	680.30
50	8682.28	6050.00	2632.85	30.32	680.75
100	9316.85	6200.00	3116.85	33.45	681.20
500	10746.66	6200.00	4546.66	42.30	681.20
1000	11352.74	6475.00	4877.74	43.00	681.95
10000	13358.11	6725.00	6633.11	49.65	682.75
PMF	23270.00	18000.00	5270.00	22.65	690.10

The table indicates that there is a potential for the flood storage and the storage can be utilized for flood protection down stream of the dam.

Chapter 5

ALTERNATIVE SPILLWAY STUDIES

5.1 General

This chapter deals with the design aspects and techno-economic comparison of spillway alternatives studies in chapter four.

5.2 Side channel spillway design

Hydraulic design of the side channel spillway is dealt in this section. The design procedure is based on the USBR "Design of small dams", IS—5186-1994 and 'Engineering for Dams' by Creager Hind and Justin [22].

5.2.1 Ogee crest profile for side channel

As per IS 6934-1998 the Ogee profile consists of two quadrants, the upstream quadrant and the downstream quadrant. From the routing analysis once the design head Hd is fixed, the crest geometry can be worked out. The shape is of lower nappe profile of a fully ventilated thin-plate weir. This profile would generally result in atmospheric pressure along the entire spillway surface at design head H_d. The pressure will be higher then atmospheric for lower heads, and sub-atmospheric for lower then H_d.

From the routing studies from (chapter 4)

Maximum head over the crest is 13.55 m.

Maximum spillway discharge is 17700.0 cumecs ≈ say 18000.0 cumecs. For this the hydraulic design is carried out and is given below.

(i) Crest Length and the Design Head (H_d)

From the basic equation of discharge of sharp crested weir as per the IS-6943-1998, the discharge is given by $Q = \frac{2}{3}\sqrt{2g}CL_eH_e^{\frac{3}{2}}$ -----------------------------------5.1

Where, Le is the effective length of the section, H_e is the head over the crest, and C varies in between 0.611 to 0.75. The parameter $\frac{2}{3}\sqrt{2g}C$ is coefficient of discharge Cd, which varies from 1.8 to 2.21.

(ii) Effect of Approach depth and upstream slope and the head over the crest on coefficient of discharge C_d

Assuming the height of crest from approach channel to be 5.0 m, excavating approach channel from EL 675.0 m and maximum head over crest H ≈ He= 13.55 m from reservoir routings, the coefficient of discharge from IS-6934- 1998 figure 2-3 given in annexure-3, is 2.11. Taking Cd as 2.11, the length of the crest required is:

$$18000 = 2.11\,\text{x}\,\text{Le}\,\text{x}\,13.55^{\frac{3}{2}}$$

From which Length of the crest is 171.03 m. Now considering the total crest length as 175.0 m (Le=172.30 m) and the approach height as 5.0 m, the head over the crest is from the same weir formula He is given by:

$$H_e = \left(\frac{18000.00}{2.11\text{x}172.30.0}\right)^{\frac{2}{3}} = 13.50\text{ m.}$$

The approach channel section is taken as trapezoidal and its side slopes are at 1:1 in rocks. The A (area), and the (P) perimeter of the channel can be calculated as 3579.75 sq.m and 227.33 m respectively. The hydraulic radius of the channel section $R=\frac{A}{P}$

Where n is channel rugosity coefficient and v is the approach velocity, L is the length of approach channel as guided by the topography of the site *(100.0 m),* is calculated as 15.75. The head loss due to approach

$$h_f = \frac{n^2 v^2 L}{R^{\frac{4}{3}}}$$

Where, v=Q/A worked out 5.03 m/s and n the rugosity coefficient for concrete lined channel is taken as 0.018. The head loss h_f calculated is equal to 0.02 m. The total energy line (TEL) is 680.0+13.36+ approach velocity head loss = 680.00+13.50+1.29-0.02 =694.77 m. Hence, head over crest H_e is 694.65-680.0=14.77 m. Now design head H_d = H_e – velocity head = 14.77-1.29=13.48 m. The coefficient of discharge (C_d) for approach depth 5.0 m and H_d 13.48 and upstream crest slope 1:1, is 2.13. Check for discharge 18000.0=2.13x172.30x $H_e^{1.5}$, which is equal to 13.42. Therefore taking H_e as 13.42m and velocity head as 1.29m **H_d** is equal to 13.42-1.29-.02= 12.11 say **12.20 m** and H_e is **13.49 m.**

Approach channel and crest length:

Crest length L_c = 175.00 m

Approach channel length of = 100.00 m

Approach channel Side slope = 1H: 1V

(iii) Effect of the submergence on the value of C_d

The design of the ogee weir is such that its coefficient of discharge C_d will not be affected by the down stream tail water level. For this the condition as per IS-6934-1998-item 4.2.6 and fig 5A and 5B given in annexure-4, the condition is $\frac{h_d + d}{He} \geq 1.7$. Now h_d +d is the depth below TEL 693.49 required is 22.93 m minimum, say EL 670.56 at maximum (elevation is from the efficiency consideration of the crest). The trough hydraulics will finally govern the elevations. The down Elevation of the ogee toe at station +175 m is 645.0 m

(iv) Ogee Profile

Using IS-6943-1998 figure-2 and from the following equation, the upstream and down stream profile can be drawn.

$$\frac{X_1^2}{A_1^2}+\frac{Y_1^2}{B_1^2}=1 \text{ For upstream curve profile} \quad \text{-- 5.2}$$

$$X_2^{1.85}=K_2 Hd^{0.85} Y_2 \quad \text{For down stream ogee profile--------------------------------5.3}$$

Parameters A_l =2.89 m and B_l= 1.68 m in equation 5.2 and K2=2.06 and design head H_d = 12.20 in equation 5.3 The profiles can be plotted by considering the centre at crest assuming (0,0) co-ordinates.

Hence upstream profile is:

$$\frac{X_1^2}{2.89^2}+\frac{Y_1^2}{1.68^2}=1$$

X	Y
0	1.68 m
1	1.58 m
2	1.22 m
2.90	0.00 m

The downstream profile can is given by:

The profile is : $Y_2=\frac{X_2^{1.85}}{17.44}$

X	Y
0	0
1	-0.06
2	-0.21
5	-1.13
10	-4.06
15	-8.59
20	-14.63
25	-22.11
30	-30.98
31.65	-34.21

5.2.2 The Side Channel Trough Design

As per IS- 5186-1994, the cross-sectional shape of Side Channel Trough is influenced by the overflow crest on one side and by bank conditions on the opposite. A trapezoidal cross-section with minimum width depth ratio is recommended for the trough. However, the minimum width should commensurate with both practical and structural aspects. If width to depth ratio is large, the depth of flow in the channel will be shallow, resulting poor diffusion of energy in the channel flow, moreover with greater bed widths the excavation will increase.

Because and turbulence and vibrations inherent in side channel spillway, trough should be set well into the original rock foundation and sufficiently inside the cliff for safety. The side slope should be trimmed to steepest angle (*USBR design of small dams recommendation 0.5H: 1V)* at which material will safely stand when lined with concrete anchored to firm rock.

Considering all these, to suit the flow expected from the routings the bed width ranging from 10.0 m to 60.0 m is tried. A bed width of B=60.0 m at the bottom toe downstream along the dam crest is found sufficient for the flow intermingling and safe passage. To reduce excavations the trough can be kept at a minimum bed width of 10.0 m at upstream of the ogee weir and expanded in straight line to 60.0 m at the control structure. The slope in the trough bed can be provided as per *IS-5186 recommendations of 0.01.*

Hence bed width in upstream is kept 10.00 m and in down stream it is 60.00 m. It is provided in a length 200.00 m

According to Julian Hind's criteria for side channel, the most economical section of channel may be worked out as below.

Assume steepest side slope possible in the strata from the stability considerations and a minimum bed width for working or equipment considerations. Then assume depth of flow and work out A (area of control section) and T (top width at water level) [5].

$$h_v = \frac{m}{m+1}\frac{A}{2T} \quad \text{-----------------} 5.4$$

$$Q = A\sqrt{2gh_v} \quad \text{-----------------} 5.5$$

Where h_v is the velocity head, m is a constant varying from 0.5 to 1 and A is the area of cross-section, T is the top width of water surface, g acceleration due to gravity and Q' is the discharge in the section.

Calculations

A= (B+0.5xh) h= (60.0+0.5x25) x 25 = 1812.50 sq.m. Where height h is assumed 25 m sde slope is 0.5:1 and B = 60.0 m.

T = (B+2x0.5xh) = 60+ 2 x 0.5 x 25 =85.0 m. Hence hv, the velocity head from equation 4.3.4 is $= h_v = 5.33$ m. For discharge from equation 5.5:

$$Q = 1603.13\sqrt{19.62x5.33} = 18{,}536.50 \text{ cumecs} > 18000.0 \text{ cumecs ok.}$$

Water surface profile

Critical depth at the control section is given by equation [5]:

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

where, q is discharge per unit width of the channel = Q/B = 257.14 cumecs/m and d_c =18.89 m. The critical velocity $V_c = q/d_c$ is =13.61 m/s and h_{vc} is head loss at the control section = $(V_c)^2/2g$ =9.44 m.

As per 'USBR' the side channel flow assumes that the entire flow energy over the crest is dissipated through it's intermingling with the channel flow and therefore has no assistance in moving water along the channel. Water surface profile [4i]is therefore, given by the equation in the length of dx along the reach of trough; by dy as:

$$dy = \frac{Q_1}{g}\frac{(v_1 + v_2)}{(Q_1 + Q_2)}\{(v_1 - v_2) + v_2\frac{(Q_2 - Q_2)}{Q_2}\} \quad \text{----------------------------5.6}$$

As per USBR, assuming a transition loss from the end of the side channel toe to the control section (to provide losses due to contraction, to diffusion of flows not effected in the side channel properly and friction to friction losses) equal to 0.2 of the difference of velocity heads between the ends of the transition.

The flow characteristics [5] at the down stream end can be obtained from Bernoulli's theorem, which can be expressed as:

$$d_{0+175} + hv_{0+175} = d_c + h_{vc} + 0.2(h_{vc} - hv_{0+175}) \text{ -- } 5.7$$

where, d_{0+175} = water surface depth at section 0+175, hvc is the head loss due to velocity head at that section, dc is the critical depth of flow at control section and h_{vc} is the head loss due to velocity head at control section.

The equation 5.7 is solved by trial and error and a depth (station+175 m) 22.5 m is found satisfying the equation. Calculations for the profile of the side channel flow using the equations 5.6 and 5.7 are given in table 5.1. The computed water surface profile is shown below.

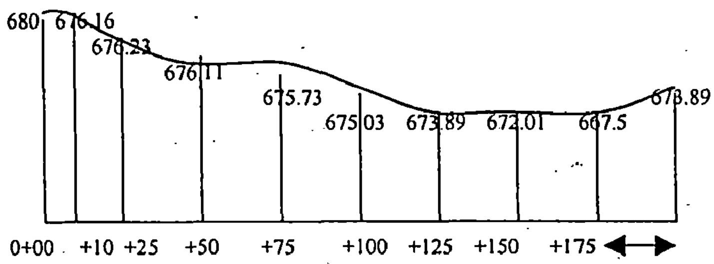


Fig. 5.1 Water surface profile in side channel trough

Side Channel spillway water surface [5] computation: Table 5.1

Stn	dx	Bottom Elev.	Trial dy	Water Surface	(d)	(A)	(Q)	(v)	(Q_1+Q_2)	$\frac{Q_1}{g(Q_1+Q_2)}$	(v_1+v_2)	(v_2-v_1)	(Q_2-Q_1)	$\frac{(Q_2-Q_1)}{Q_1}$	v_2*(15)	(13)+(16)	dy=(11)*(12)*(17)	Remarks
	m	m	m	m	m	m^2	m^3/s	m/s	m^3/s		m/s	m/s					m	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0+175		645.00		667.5	22.5	1603	18000	11.23										
0+150	25	645.25	4.50	671.75	26.75	1963	15429	7.86	33429	0.047	19.089	3.368	2571	0.17	1.87	5.24	4.70	high
	25	645.25	4.76	672.01	27.01	1986	15429	7.77	33429	0.047	18.998	3.46	2571	0.17	1.87	5.33	4.76	ok
0+125	25	645.50	1.00	72.76	27.76	2051	12857	6.27	28286	0.046	14.038	1.502	2571	0.20	1.55	3.06	1.99	high
		645.50	2.13	673.89	28.89	2151	12857	5.98	28286	0.046	13.747	1.79	2571	0.20	1.55	3.35	2.13	ok
0+100	25	645.75	1.00	674.64	29.64	2218	10286	4.64	23143	0.045	10.615	1.34	2571	0.25	1.49	2.83	1.36	high
	25	645.75	1.39	675.03	30.03	2253	10286	4.57	23143	0.045	10.543	1.41	2571	0.25	1.49	2.91	1.39	ok
0+75	25	646.00	1.00	675.78	30.78	2321	7714	3.32	18000	0.044	7.8894	1.241	2571	0.33	1.52	2.76	0.95	low
	25	646.00	0.95	675.73	30.73	2316	7714	3.33	18000	0.044	7.896	1.23	2571	0.33	1.52	2.76	0.95	ok
0+50	25	646.25	0.75	676.23	31.23	2362	5143	2.18	12857	0.041	5.5081	1.153	2571	0.50	1.67	2.82	0.63	low
	25	646.25	0.63	676.11	31.11	2351	5143	2.19	12857	0.041	5.518	1.14	2571	0.50	1.67	2.81	0.63	ok
0+25	25	646.50	5.00	680.86	35.86	2795	2571	0.92	7714	0.034	3.1078	1.268	2571	1.00	2.19	3.46	0.36	low
	25	646.50	0.37	676.23	31.23	2362	2571	1.09	7714	0.034	3.277	1.10	2571	1.00	2.19	3.29	0.37	ok
0+10	15	646.65	0.20	676.28	31.28	2366	1543	0.65	4114	0.038	1.7408	0.437	1029	0.67	0.73	1.16	0.08	low
	15	646.65	0.08	676.16	31.16	2355	1543	0.66	4114	0.038	1.744	0.43	1029	0.67	0.73	1.16	0.08	ok

Note:

- The maximum water surface at the upstream end of the channel, as per 'USBR design of small dams' 2/3 of the head over the crest to avoid the excessive submergence and achieve desired coefficient discharge.
- Assuming different bottom widths, different channel slopes and varying control sections can make efficient channel design.

5.2.3 Transition Portion

The channel section requires transitions narrower or wider then either crest or terminal structure. The sidewall convergence in discharge channel should be made gradual to avoid cross-waves ride ups on the walls and uneven distribution of flows across the channel. Similarly the rate of divergence of the sidewalls as per IS-5186-1994 should be gradual to ensure the flow to be spread uniformly over the entire width of the channel.

As per USBR 'Design of Small Dams'; the side transition for convergence or divergence should be provided at an angle given by the following equation:

$$\tan\beta = \frac{1}{3F} \text{ --5.8}$$

Where β angle of convergence or divergence and F is Froude number given by:

$$F = \frac{v}{\sqrt{gd}}$$

Where, v is average velocity at beginning and end of the transitions, g is the acceleration due to gravity and, d is the average depth of flow at beginning and end of the transition. Let the depth d1 be at the beginning of the transition equal to 8.8 m, d2 is depth at the end of transition equal to 34.47m (from control section design P+H_d, P is 10.0 m,H_d= $(18000/(2.13*70)^{0.667})$) and v_1 velocity at beginning of transition 11.23 m/s and v_2 velocity at the end that is the control section, then, $v = \frac{v_1 + v_2}{2} = \frac{11.23+13.61}{2} = 12.42$m/s and d=28.48 m.

The Froude number is equal to $F = \frac{12.42}{\sqrt{9.81*28.48}} = 0.743$, from the equation 4.3.8 β the maximum angle of divergence is equal to 24.16°. This transition is between the toe of the crest and the control section is provided in a length of 70.0 m. and width varying form 60.0 m to 70.0 m.

5.2.4 Control section

The control section d/s of side channel trough where flow velocity changes from sub critical to super critical, constricting the channel sides or elevating the channel bottom may provide the control section. The best location of control section is usually at a point where bed slope has to be steep to keep channel on the ground. Special site conditions may require another location. Flows from the side channel can be directed into an open discharge channel or into a closed conduit or inclined tunnel.

The discharge computations for upstream and downstream profile of the control section can be done similar to the ogee overflow crest. The discharge is given by the equation: $$Q = CdLeH^{\frac{3}{2}}$$

Where, C_d coefficient of discharge varies from 1.8 to 2.21 in SI units. L_e is the effective length control section and H is the head over the control section. Considering approach depth of water at the control section be 10.0 m to suit the head up in the side channel for intermingling, the head over crest taking Cd equal to 2.2 in the above weir formula comes out to 23.87 m. Now checking for approach depth correction, for P/Hd= 0.42 and upstream slope 1:1 *the value of coefficient of discharge Cd from IS-6943-1998 fig-2 is 2.13.* There is no downstream submergence effect to the discharge coefficient.

5.2.5 Discharge Channel

The flow released from the control structure is conveyed to the terminal structure or river bed below dam in an open channel excavated along the ground surface. The profile may be variably flat or steep; the cross-section may be variably rectangular or trapezoidal. It may be wide or narrow, long or short.

As per IS-5186-1994, the dimension of the channel is primarily governed by hydraulic requirements, but selection of the profile; cross-sectional shape, etc are influenced by the geological and topographical characteristics of the site.

Considering the above criterion the trapezoidal section is adopted for design and calculations are given in the table number 5.2.2

Table number **5.2**

Calculation Table of velocity and water depth on chute channel [11] at 40m intervals along the alignment 70m wide at side slope 0.5:1.

SN	Distance from the start of 1:10 slope.	Length L on the alignment.	Drop in bed	Bed level at this cross-section	Assumed depth (d).	V=(q)/d that is 257.14/(d)	Velocity head ha = $v^2/2g$	Specific energy = d + $v^2/2g$	Bed level +Sp. Energy =(5)+(9)	Area =B+0.5*d)8d that is =(70+0.5*F6)*F6	Perimeter P = B+d	Hydraulic Radius R = A/P	$R^{4/3}$	$Sf=n^2v^2/R^{4/3}$	Average Sf	L*Sf= hf	Actual TEL	Froude number F
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	0	0	0	641.00	10.333	24.89	31.56	41.90	682.90	776.7	80.3	9.67	20.60	0.0097	—	—	682.90	2.47
2	40	40	4	637.00	9.75	26.37	35.45	45.20	682.20	730.0	79.8	9.15	19.15	0.0118	0.0108	0.4302	682.47	2.70
		40	4	637.00	9.709	26.48	35.75	45.46	682.46	726.8	79.7	9.12	19.05	0.0119	0.0108	0.4335	682.46	2.71
3	80	40	4	633.00	9.2	27.95	39.82	49.02	682.02	686.3	79.2	8.67	17.80	0.0142	0.0131	0.523	681.94	2.94
		40	4	633.00	9.21	27.92	39.73	48.94	681.94	687.1	79.2	8.67	17.82	0.0142	0.0131	0.522	681.94	2.94
4	120	40	4	629.00	8.799	29.22	43.53	52.33	681.33	654.6	78.8	8.31	16.83	0.0164	0.0153	0.6123	681.33	3.15
5	150	30	3	626.00	8.534	30.13	46.27	54.81	680.81	633.8	78.5	8.07	16.19	0.0182	0.0173	0.5193	680.81	3.29
				Surface slope changed														
6	190	40	20	606.00	7.11	36.17	66.67	73.78	679.78	523.0	77.1	6.78	12.84	0.0330	0.0256	1.0236	679.78	4.33
7	250	60	30	576.00	5.9542	43.19	95.06	101.01	677.01	434.5	76.0	5.72	10.23	0.0591	0.046	2.7621	677.01	5.65
8	290	40	20	556.00	5.4659	47.04	112.80	118.27	674.27	397.6	75.5	5.27	9.17	0.0782	0.0686	2.7458	674.27	6.42
9	330	40	20	536.00	5.0985	50.43	129.64	134.74	670.74	369.9	75.1	4.93	8.38	0.0983	0.0883	3.5314	670.74	7.13
10	370	40	20	516.00	4.8113	53.45	145.58	150.40	666.40	348.4	74.8	4.66	7.78	0.1190	0.1087	4.3472	666.40	7.78
11	410	40	20	476.00	4.3233	59.48	180.31	184.63	660.63	312.0	74.3	4.20	6.77	0.1693	0.1441	5.7658	660.63	9.13
12	450	40	20	436.00	3.9795	64.62	212.81	216.79	652.79	286.5	74.0	3.87	6.08	0.2225	0.1959	7.8346	652.79	10.34

Cross-section and Bed Slope

The width of the section is (B) 70.0 meter. As per USBR, the area of the section varies as per the depth of the section and the side slopes are considered 0.5H: 1V. Velocity at control section is maintained (critical) 13.30 m/s, where critical depth is (d_c) 18.89 metres. Specific energy at control section $(d+\frac{v^2}{2g})$ 27.90 metres is from the table 4.3.4. Energy line EL at this section is 682.90 metres. To have no effect on the coefficient of discharge C_d from downstream submergence we have IS-6943-1998: $\frac{h_d+d}{H_e} \geq 1.7$, where, h_d head loss from TEL to the toe and d is the depth of flow, H_e is the head over crest. The parameter (h_d+d) is 41.60. The toe elevation can be adopted at (682.90-41.60) 641.30 metres. The toe is kept at EL 641.00 m. At the change of slope, area of the section is 1688.68 sq.m and perimeter is 88.89 metres. The minimum slope as per Manning's equation is:

$s=\frac{n^2v^2}{R^{\frac{4}{3}}}$ is equal to 1/886 to maintain the supercritical flow. A steeper slope is adopted.

The first slope is adopted as per site condition as 10H: 1V up to a distance of 150.0 metres and this follows a steeper second steeper slope 1.66H: 1V to lead the flow in the bucket.

5.2.6 Vertical Curves

The vertical curves as per IS-5186-1994, the convex curve [4v] at the junction of mild to steep slope is:

$$y=-x\tan\alpha-\frac{X^2}{4K(d+h_v)\cos^2\alpha} \quad \text{-----------------------------------5.9}$$

Where, at elevation 626.0 metres from the calculated table 5.2.2 specific energy $d+h_v$ is 54.81 and α is upstream slope angle equal to 5.71°, k is safety factor 1.5. Hence, the curve is in the form: $y = -\frac{x}{10} - \frac{x^2}{328}$ as the angle small $\cos^2\alpha$ is nearly equal to 1. This gives the curvature (approximate) at the junction and a circle drawn at two ends gives the radius of the curve radius R. This curve is finally adjusted on the model study. The approximate curve radius at the convex curve is R comes out to be 162.0 m.

5.2.7 Sidewall height and free board

IS-5186-1994 gives the free board (FB) in discharge channel as [4v]:

$$FB = 0.61 + 0.0378vd^{\frac{1}{3}} \quad \text{-----5.10}$$

where, v is the velocity at the section and d is water depth. From the table v and d are taken.

At the toe of the control section, sidewall height required is 2.66 m; total height of wall is 13.0 metres. At the junction of the vertical curve it is 2.33 m and total height of the wall is 11.13 m and at bucket level free board is 4.27metres, total height wall 8.60 m.

5.2.8 Energy Dissipator

The tail water depth of the downstream river and the conjugate depth of flow play important role in deciding the type of the dissipaters. Here conjugate depth d_2 is calculated as:

$$d_2 = \frac{d_1}{2}\{(\sqrt{1+8F_1}) - 1\} \quad \text{-----5.11}$$

Where, d_1 is inflow depth at toe of channel, 3.88 m and F1 is 10.73 at the toe of the slope. The conjugate depth (d_2) from equation 4.3.11 is 56.96 m. It is more then tail water depth available in the river. For this case, as per IS- 10137-1982 where tail water depth lies below the conjugate depth d_2 for all discharges, following three alternates are most common for energy dissipation.

- Sloping Apron Type Stilling Basin
- Baffle wall and Subsidiary dam Type Energy Dissipater

- Ski-jump Bucket Type Energy Dissipater

The alternative of sloping apron below riverbed is uneconomical for the case as excavation below the river bed is large. The second alternate is also not feasible to have a subsidiary dam of height more than 30.0 m

As per IS-7365-1985 when the tail water depth is insufficient for formation of hydraulic jump and bed of the channel downstream comprises sound rock which is capable of withstanding impact of high velocity jets, the trajectory type of buckets are adopted [4vii]. A ski-jump bucket type energy dissipater is hence proposed to dissipate the energy of incoming flow.

(i) Trajectory Invert Elevation

As per the IS-7365-1985 invert of the trajectory bucket is generally kept as near to ground elevation as possible satisfying the criterion of submergence. For the Pancheshwar site, the bucket invert can be put in the glacis, as there is sound and firm bed available down stream of the spillway outlet. This option also reduces the length of the discharge channel. The invert is put at EL 475.0 m.

(ii) Shape and Radius of Bucket

The shape is generally considered circular. A circular shape is adopted in this case also. To maintain the concentric flow and to avoid the tendency for the water to spring away from the bucket, the radius has to be substantially large so the floor pressure does not alter the streamline distribution of flow. As per IS-7365-1985, the radius should not be less then 3 times maximum depth of flow in the bucket (d_1) to avoid separation tendencies in the bucket. Another criterion is from the same code [4vii]:

R= 0.6 to0.8 times $\sqrt{HH_5}$ where, H is geometric mean of depths over control i.e 24.47 m and H_5 is total fall from control section pool and jet surface elevation on the bucket. A higher value of the coefficient 0.8 is used in absence of model tests for design. It is 682.90-479.32=203.58 m. The radius of the bucket worked out is R is 56.46 m. Radius of the bucket adopted is 50.0 m.

(iii) Length of Trajectory and plunge pool dimensions

The length of the trajectory is given by the formula from IS-7365-1985 as:

$$\frac{X}{H_v} = \sin 2\varphi + 2\cos\varphi\sqrt{\sin^2\varphi + \frac{Y}{H_v}} \text{ --------------------------------- } 5.12$$

Where, ϕ is the lip angle and H_v is the velocity head at the lip and equal to 180.44 m using the toe velocity at EL 475.00 m from table 5.2.2 and Y is the lip elevation – tail water elevation equal to 70.0 m. The length (X) calculated is 406.0 m. This length is slope as the bucket elevation and plunge pool elevation difference is 70.0 m. The jet will fall at length L= 405-(70+2a)=247.0 m, where, 'a' the vertical distance of the parabolic trajectory can be worked out by IS-7365-1985 as:

$$a = \frac{v_a^2 \sin^2\phi}{2g} = 45.11 \text{ m}$$

Where, V_a is the velocity at the bucket lip and ϕ is the lip angle. The length of trajectory and side slope fixes the plunge pool dimensions. The plunge pool is provided at EL 410.0 m and of a safe distance away from the toe of the bucket it has burms and side slope 1H: 1V. Width equal to channel section, the length can be 400.0 m and 150.0 m away from the bucket toe. The elevations of plunge pool are 400.00 at bucket toe end and 422.0 m at the tail end where excavations in rocks as per site condition are reduced. The bucket dimensions need verification from model studies.

(iv) Scour depth and pool free board

The scour depth on bed of the impact pool as per IS-7365-1985 is given as:

$$d_s = m(qH_4)^{0.5} \text{ -- } 5.13$$

Where, d_s scour depth in metres, m is constant and is 0.36 for minimum scour, 0.54 for probable scour under sustained spillway operation and 0.65 for ultimate scour and H_4 is the reservoir pool elevation (655.00+24.47) – bucket end sill elevation EL (490.0). Therefore, minimum scour depth is 79.46 m from the tail water level. Pool bottom is provided at elevation 400.0 m to give sufficient depth for energy dissipation. However, to prevent basin floor damage floor is to be properly anchored to rock up to a sufficient depth.

The sidewalls free board of the plunge pool is given by IS-5186-1994 as:
FB $= 0.1\ (v_1+d_2)$, where, v_1 is incoming velocity to the pool and d_2 is conjugate tail water depth. FB is worked out 11.64 m above the water level of the pool.

5.2.9 Cavitations control in the chute channel

As per IS-12804-1989 cavitation is formation of gas phase within liquid. The successive formation and collapse of vapour bobbles in a stream of liquid, which results from pressure changes within the stream due to changes in velocity of flow causes cavitation and damage to channel surface [4iii].

For higher velocities of water the problem of cavitation becomes more critical. For velocities around 30.0 m/s the pressure field becomes very sensitive. Cavitation occurrence is also related to discharge concentration. The damage occurs when vapour bubbles form in the void/partial vacuum created by high velocity water tends to breakaway from the concrete surface while jumping over the irregularities.

(i)Necessity of Aeration

Data: Maximum water level at EL 679.47 m

Intensity of discharge 257.14 cumecs/m

EL of the chute spillway section taken 606.0 m

Depth of flow at the section 7.11 m

Velocity at the reference 36.17 m/s

The downstream slope angle ϕ =26.57°

As per the IS-12804-1989 the flow cavitation index K is given by:

$$k = \frac{d\cos\varphi + \frac{P_b}{\gamma} - \frac{P_v}{\gamma}}{\frac{v^2}{2g}} \quad \text{-----5.14}$$

Where, d is of depth flow at the section, P_v/γ is vapour pressure taken as 10.356 m of water and P_b/γ is barometric pressure at 20°c and is taken as 0.223 m of water and v is the velocity at the section. The flow index is 0.25. It will be less than 0.25 for 50% discharge. The aerators are needed when the index reaches below 1.7 and the protection to expected cavitation damage is essential as per the code. The first aerators are given at a distance 40.0 m toe of the mild to steep slope change point of the chute channel.

(ii) Quantity of air

As per the IS-12804-1989, the airflow discharge Qa is given by [4iii]

$$Q_a = \frac{Bv^3 Cos^3\varphi (So - Tan\varphi)^2}{4g} \quad \text{-----5.15}$$

where, B=c/c distance of piers or abutments in m, v is the velocity of flow, Q_a is airflow discharge in cumecs, ϕ is the angle between the ramp and the horizontal. S_o is the spillway slope and g is acceleration due to gravity. Assuming ramp angle of 22° and then calculation is done for the B=24.5 m (clear width of piers) and v the velocity 36.17 m/s of section.

Q_a = 216.91 m/s assume the supply velocity of air 30 m/s the area of the air passage = 216.91/30= 7.23 sq.m. The size of the groove is 2.70 m x 2.70 m.

As this being chute aeration the width can be taken as 70, which gives the air demand of 619.75 cumecs and the area required is 4.5 m x 4.5 m.

(iii) Spacing of the aeration device

One general assumption for spacing required is in terms of average velocity in the flow, which is 70.0 m along the chute from the first aerator.

As per IS-12804-1989 the spacing can is given as:

$$S = 3.5d(\frac{dv_a}{v})^{\frac{1}{4}} \quad \text{--5.16}$$

Where, s is the spacing in metres, d is the d depth of flow in me in metres; Va is the mean velocity in metre per seconds and v is the kinematic viscosity of water (0.000001 m^2/s). For the chute considered the spacing considering mean depth of two section at EL 606.0 m and 576.0 m of 6.53 m and mean velocity at the two sections 39.68 m/s the spacing required worked out is 2900.0 m which indicates no requirement of another aeration point. But as per the IS-12804-1989, spacing should be decided only after the model study of the proposal for deciding the number of aerators, size and actual quantity of the air required to be mixed at different expected discharges during the operation of the spillway.

A model study is required to find the number of the aerators and quantity of the air required to mix for different discharges expected to pass from the proposed spillway.

5.2.10 Figures/Drawings

The layout plan and details of side channel spillway structure/ component parameters are shown in Figs. 5.2 and sheet no. 1a & 1b page no. 60-62.

5.2.11 The summary of the design parameters of side channel spillway is given in the following table.

Table number 5.3

Descriptions	Details
Maximum water level	EL 693.55 m
Maximum discharge	18000.00 cumecs
Design Head H_d	12.20 m
Crest Elevation	EL 680.0 m
Approach channel	EL 675.00 m length 100.00 m and width 175.00 m
Side channel trough	Slope 0.01 length 175.00 m width 10.00-60.00 m side slope 1:1
Transition Zone	Length 70.00 m width 60.00 m-70.00 m
Control zone	EL 655.00 m, 3.50 m wide two piers. Clear width 70.00 m
Discharge channel	0.01 and 0.6 slopes for 150.0 m and 300.0 m Trapezoidal section, vertical curve radius 162.0 m and height of side wall 8.60 m-13.00 m
Bucket	Radius 50.0 m and lip angle 30-31 degree EL490.00
Plunge Pool	Width 70.00 and length 400.0 m
Aeration tower at	40.0 m from end of first slope

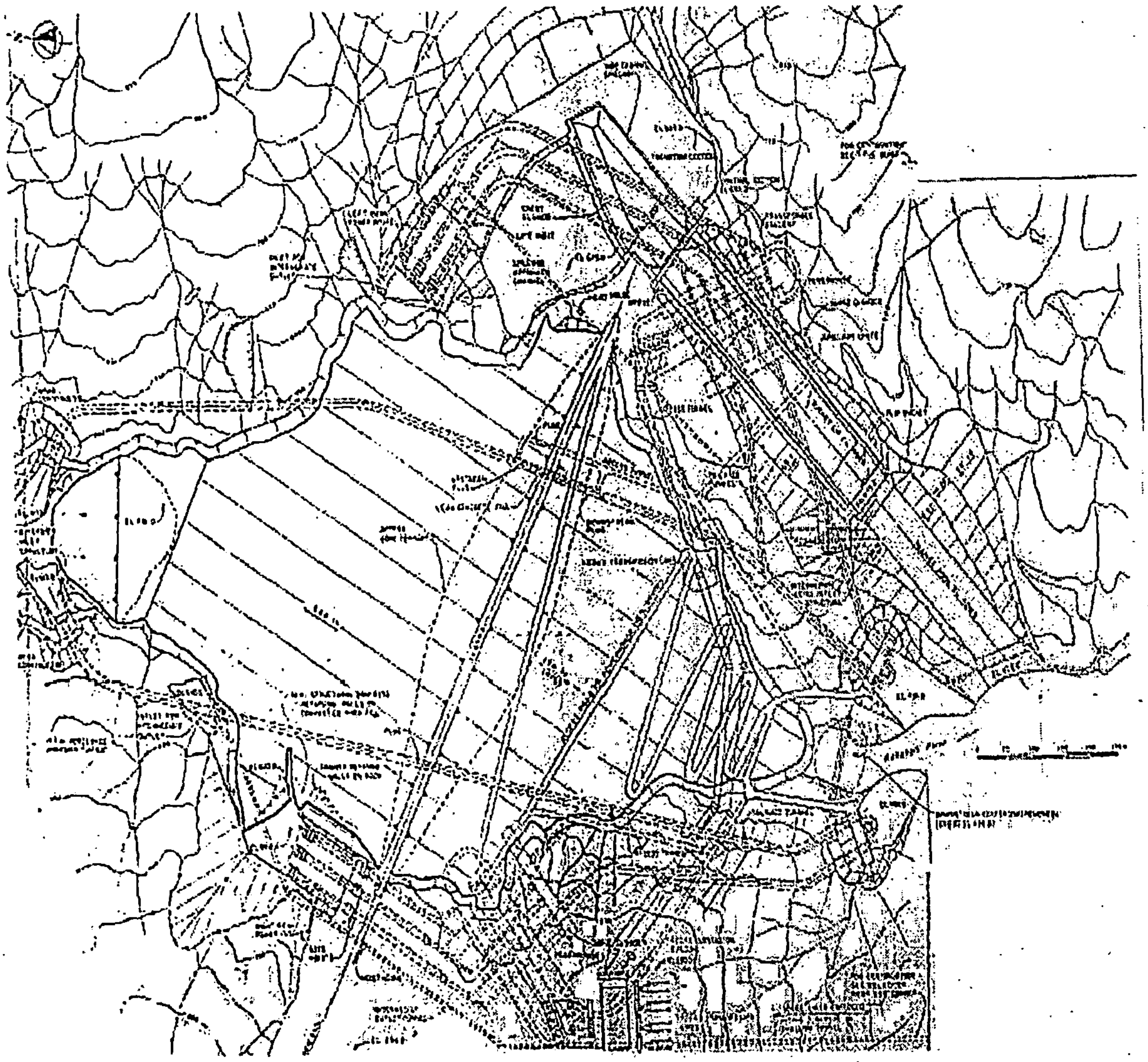
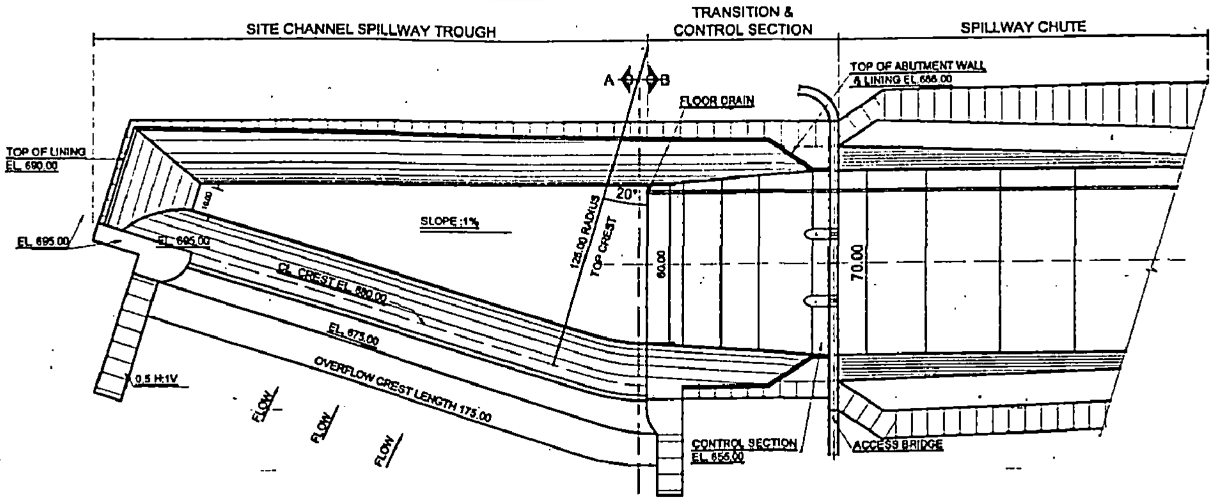
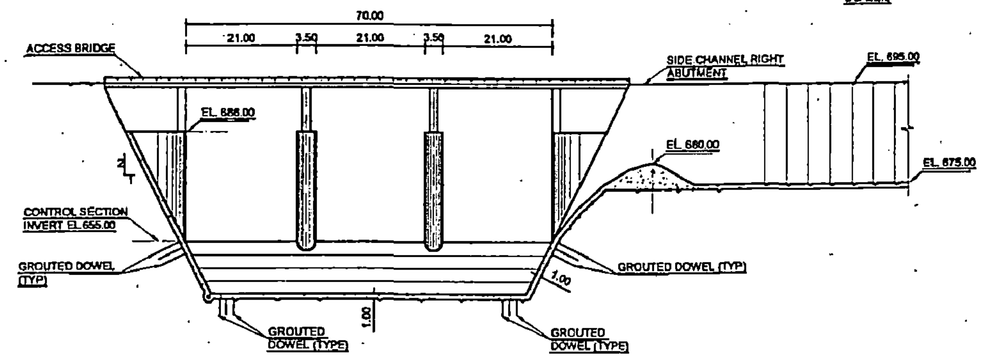


Fig. 5.2 Side Channel spillway layout plan [12]



SIDE CHANNEL SPILLWAY PLAN

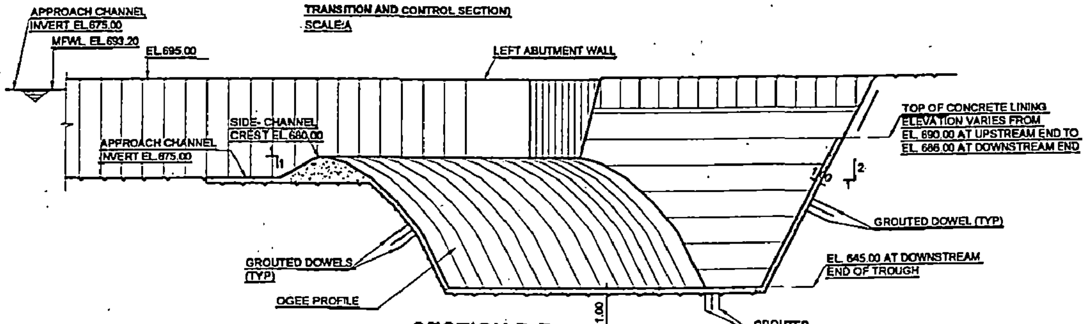
SCALE:B



SECTION A-A

(LOOKING DOWNSTREAM THROUGH TRANSITION AND CONTROL SECTION)

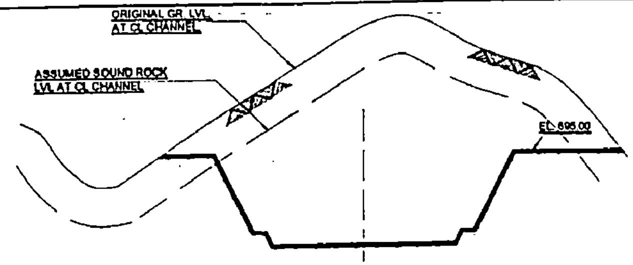
SCALE:A



SECTION B-B

(LOOKING UPSTREAM INTO SIDE CHANNEL SPILLWAY THROUGH)

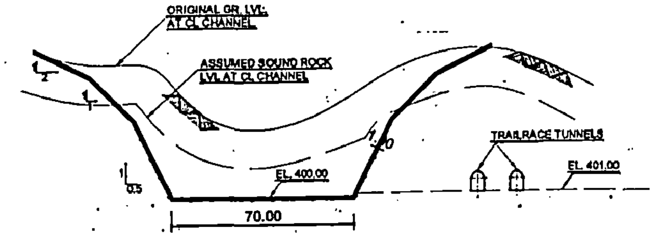
SCALE:A



TYPICAL SECTION THROUGH CHUTE

LOOKING DOWNSTREAM

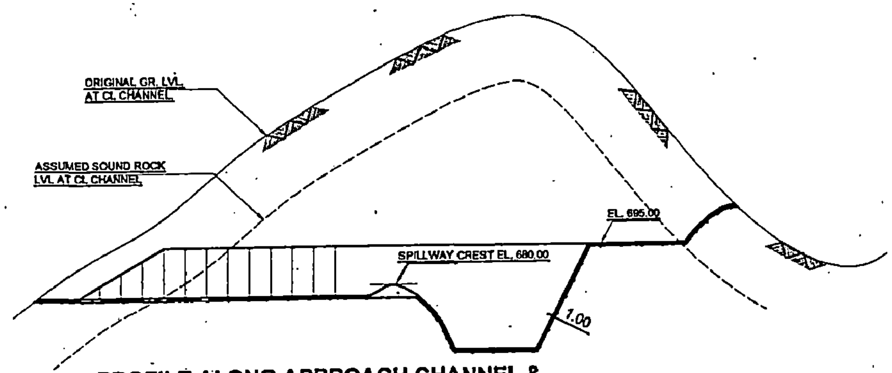
SCALE:B



TYPICAL SECTION THROUGH PLUNGE POOL

LOOKING DOWNSTREAM

SCALE:B



PROFILE ALONG APPROACH CHANNEL & THROUGH SIDE CHANNEL SPILLWAY

LOOKING DOWNSTREAM

SCALE:B

SCALE-C 40	80	120	160	200	240
SCALE-B 20	40	60	80	100	120
0 SCALE-A 10	20	30	40	50	60
SCALE-A0 5	10	15	20	25	30

NOTE: ALL DIMENSIONS ARE IN METERS

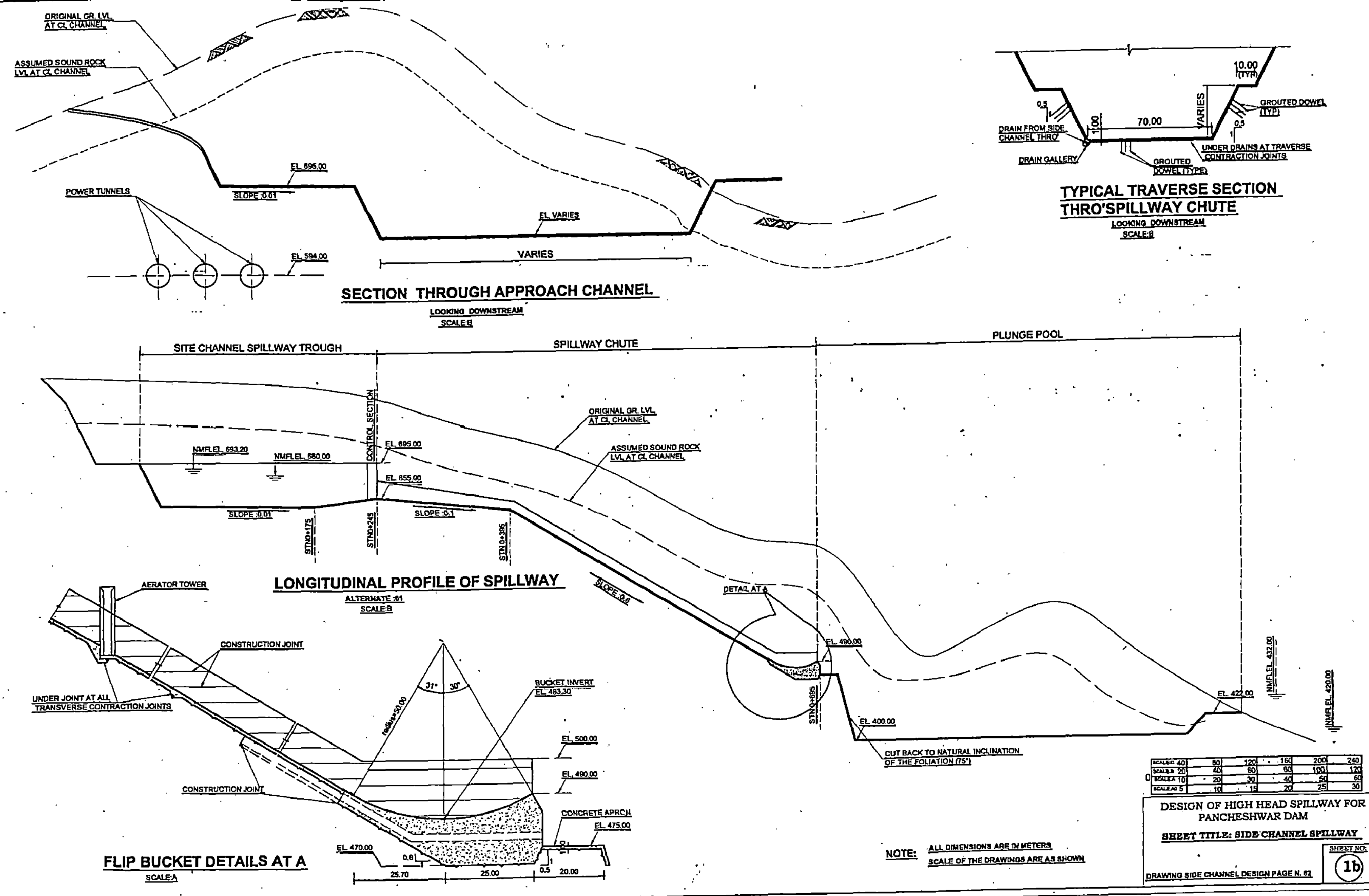
SCALE OF THE DRAWINGS ARE AS SHOWN

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: SIDE CHANNEL SPILLWAY

DRAWING SIDE CHANNEL DESIGN PAGE N. 61

SHEET NO: 1a



SCALE:C 40	80	120	160	200	240
SCALE:B 20	40	60	80	100	120
0 SCALE:A 10	20	30	40	50	60
SCALE:A0 5	10	15	20	25	30

NOTE: ALL DIMENSIONS ARE IN METERS
SCALE OF THE DRAWINGS ARE AS SHOWN

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: SIDE CHANNEL SPILLWAY

SHEET NO. 1b

DRAWING SIDE CHANNEL DESIGN PAGE N. 62

5.3 Chute Spillway

5.3.1 General

The Chute spillway design option is considered to check and compare the difference of volume of works with respect to a Side Channel design proposed by the Pancheshwar dam project. The design is mainly based on the USBR "Design of small dams" and IS—5186-1994.

As per IS-5186-1994 any type of spillway selection site topography, discharge outflow, and the site geology [4v]. A control section gives the length of crest required from the routing and surcharge over the crest along the reservoir. After selection of control section of the spillway, flood routing is carried out to define the **head over the crest** and **maximum** outflow from the spillway. The head over the crest is found around 24.77 metres for the chute spillway and maximum overflow is 15350.00 cumecs from table 4.2.5

5.3.2 Design of Chute Spillway

Hydraulic design of chute spillway as per IS-5186-1994 consists of:

- An approach channel
- Control Structure
- Discharge channel
- Energy Dissipater or Terminal Structure
- Outlet channel

(i) Approach Channel

As per IS-5186, where the spillway is located through an abutment an approach channel is required to draw the water form the reservoir and convey it to the control structure. To minimize head loss through the channel and to obtain uniformity of flow over the spillway crest, approach velocities are to be limited and channel curvatures and transitions need to be made gradual.

From the flood routing table number 4.2.5, the spillway capacity required to pass the probable maximum flood of 23,270 cumecs is 15350 cumecs. The rest of the discharge is contributing to surcharge the reservoir upstream of the dam. The control structure, a gated one it tried with various *widths* and gate heights. *A section with gate height 20.0 m and 16.0 m width, four bays with a clear span of 76.75 m having pier width of 4.25 m (similar to adopted in Tehri) is found finally suitable by placing crest elevation at 660.00 m to pass the design flood.*

The approach channel width at control is same crest length of 76.75 m. This width can be adjusted upwards as per the site necessity and approach curves. The head over the crest decides the depth of approach channel and control section type gives the coefficient of discharge.

It is a low crest the C_d worked out as 2.18 and head over crest to pass 15350 cumecs flood has worked out as 23.0 m. hence upstream water level is EL 683.00 m.

The bed elevation of the approach channel can be kept at 652.00 to have the smooth flow at the control. Total depth of flow in the approach is 31.00 m. The side slope is 1H: 1V in the approach channel. The section area (A) is equal to 3340.25 sq.m and perimeter of the section is 164.43 m. The approach velocity (v_a) Q/A is equal to 4.60 m/s and the velocity head (ha) is 1.08 m. Head loss in the approach channel by Manning's equation h_f is given as [11]

$$h_f = \frac{n^2 v^2 L}{R^{\frac{4}{3}}} \quad \text{--}5.17$$

where, n is the Manning's roughness (rugosity) coefficient and v is the approach velocity L is the length of the channel (considered approximately 100.0 m as per the site) and R is the hydraulic radius. The head loss is equal to 0.01 m.

Approach channel parameters are summarised below:

B= 76.75 m

D= 31.00 m

Side slope= IH: 1V

Area of c/s=3340.25

Head loss in channel= 0.01 m

Bed Elevation= 652.00 m

Now upstream Total Energy line 683.00+1.08-0.01=684.07 m

Total Head over crest is H_e =684.07-660.00=24.07m and design head H_d (H_e-h_a)= 22.99 m.

Coefficient of discharge

As per IS-6943-1998 fig (3) and (4) for ogee control weir, the correction for coefficient of discharge (C_d) is calculated as [4(vi)]:

- For P/H_d = 8/22.99=0.35 where P, the approach height and Hd is design head,
- For H_e/H_d =24.07/22.99=1.05,
- For upstream slope 1H: 1V,
- And assuming the high head spillway section and no effect of tail water depth, the coefficient of discharge (C_d) is 2.17. very near to assumed value of 2.18. With this value of C_d, H_d, the head over the crest in 23.70 m and total head is 24.77 m.

(ii) Control Section

A control section as per IS-5186-1994 is placed generally normal to the discharge channel in the head reach. It limits or prevents the outflows below fixed reservoir levels and also regulates releases when reservoir rises above that level. Pancheshwar dam a gated Chute Spillway (4 bay each 16 m wide separated by 4.25 m wide piers fitted with 20 m high gates) is adopted by trial and error along the left abutment of the dam axis appromately15.0 m away from the dam top. Refer chapter 4.

The control is considered a low ogee over flow in section. As per 'USACWES', the downstream crest profile is to follow as [11]:

$$\frac{h_a}{H_e} = 0.04$$

It lies between 0.08and 0.12 range and

$$\frac{P}{H_e} = 0.32$$

It lies between 0.58 and .012 ranges. For the ranges the downstream crest profile is:

$$X^{1.747} = 1.905\, H_e^{0.747} Y \text{ -- } 5.19$$

where, x and y are (0, 0) at the crest. For no submergence criteria, as per IS-6934-1998-item 4.2.6 and fig 5A and 5B, the condition is $\frac{h_d + d}{H_e} \geq 1.7$. And h_d is the head loss from the reservoir, d is the depth of flow at the toe and H_e is head over the crest of ogee overflow including velocity head. Now $(h_d + d) \geq 1.7 \times 24.77 = 42.11$ m. Therefore, the elevation of the ogee curve where it meets the down stream mild slope is = upstream TEL- (hd+d)=684.77-42.11=642.66 m. The toe of the ogee chute adopted at **642.50** m. The maximum ordinate y of the downstream curve for equation 4.4.3 $Y = X^{1.747}/20.95$ is found from crest and toe elevations =660-642.5=17.5 m and at this point X= 29.36≈30.0 m, the downstream crest profile is given below:

Downstream crest profile $\quad Y = \dfrac{X^{1.747}}{1.905 H_e}$

X	Y
0	0
5	-0.79
10	-2.67
15	-5.41
20	-8.95
30	-17.5

The upstream profile of the control section follows an ellipse confirming the ordinates as per "USACEWES" for ha/He =0.08:

Upstream crest profile ordinates (0,0) at crest of the spillway:

X/He	Y/He	X	Y
0.00	0.00	0.00	0.00
0.02	0.0004	0.50	0.01
0.06	0.0035	1.49	0.09
0.10	0.0101	2.48	0.25
0.15	0.0235	3.72	0.58
0.195	0.042	4.83	1.04

Hence the Control section parameter are worked out as below:

Length of the crest clear = 76.75 m

Elevation of crest = 660.00 m

Gate size = 20 m height X 16 m width

Piers = 4 number 4.25 m width

Discharge intensity q = 200 cumecs/m

Critical depth as per USBR is $(d_c) = \sqrt[3]{\frac{q^2}{g}}$ 15.98 m

and critical velocity is q/dc = 12.52 m/s.

The reverse curve at the toe of the control can be given by a radius equal to $2H_e$= 49.00 m.

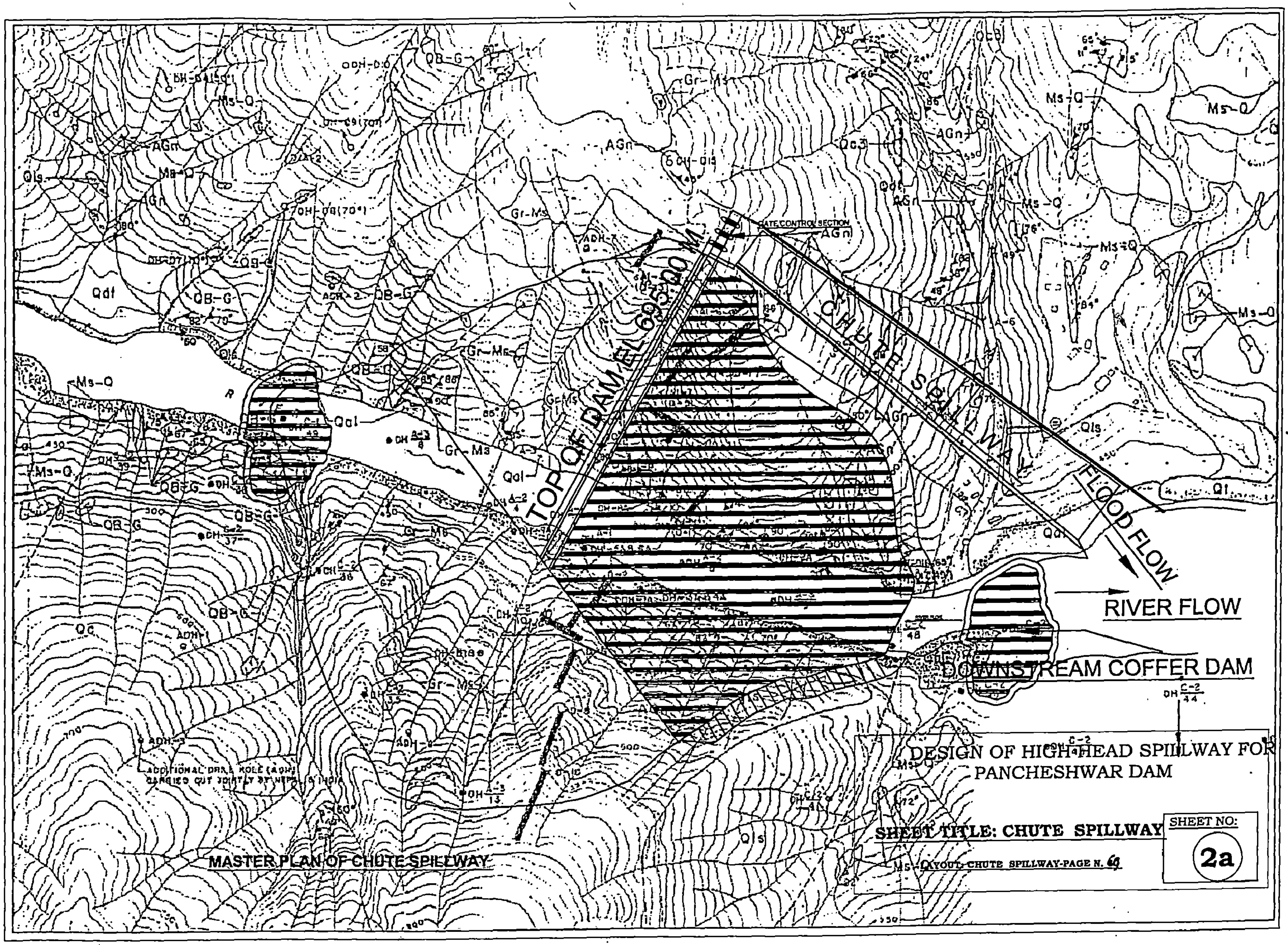
(iii) **The design procedure for the rest of the components of the chute spillway is same as that described for side channel spillway. Hence these are not described again. The summary of the parameters so worked out for the chute spillway is given below.**

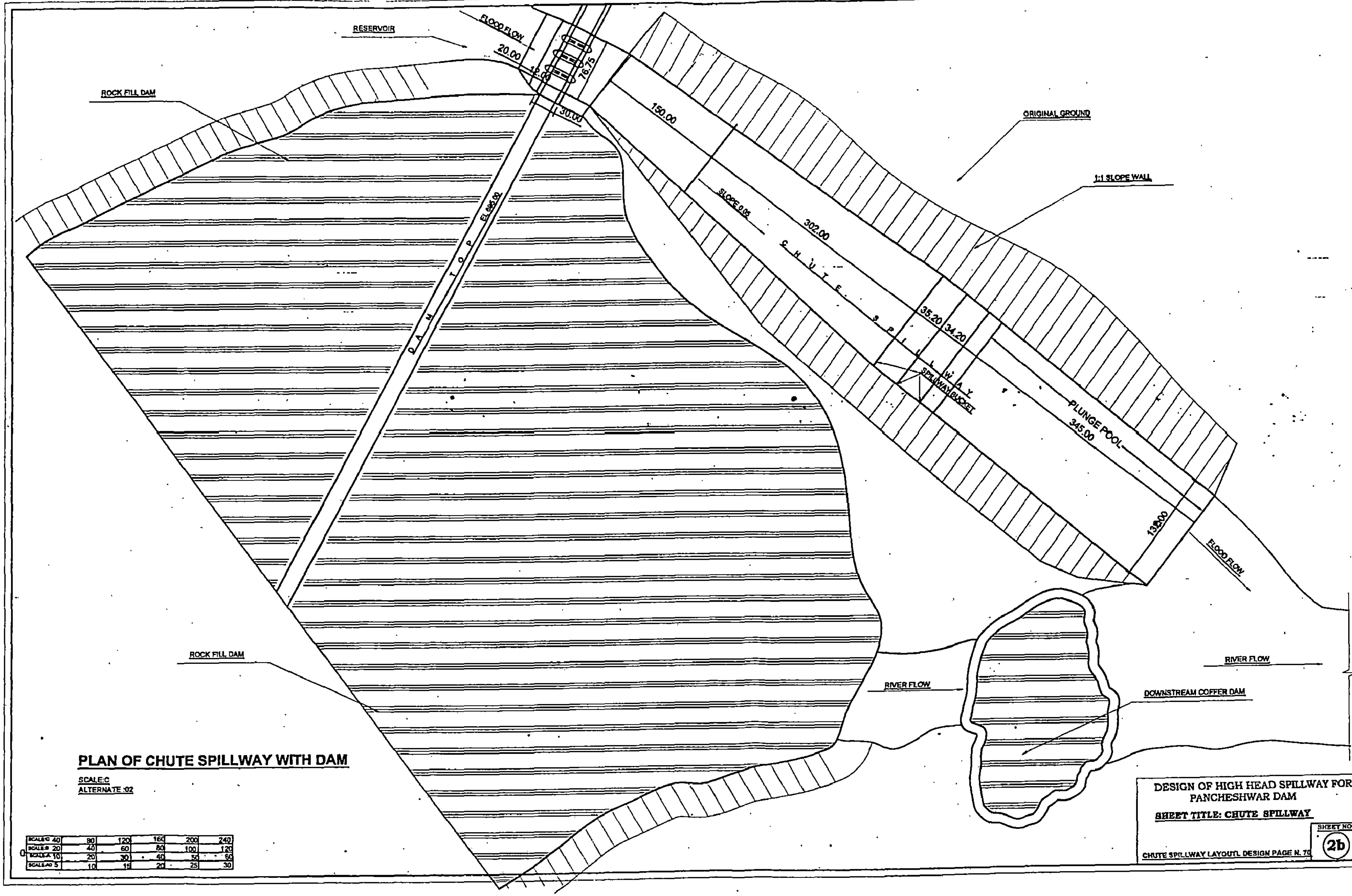
Summary of the design for chute spillway option

Table number 5.4

Descriptions	Details
Maximum water level	EL 689.00 m
Maximum discharge	15350.00 cumecs
Design Head H_d	23.70 m
Crest EL	660.00 m
Approach channel	EL 652.00 m length 100.00 m and width 76.75.00 m
Control	Crest length 76.75 m Width 30.00 m Gate size 20 x 16 m Ogee control. 4.25 m width 3 no. Piers
Transition Zone	Length 452.00 m Width 76.75.00 m-130.00 m
Discharge channel	0.05 and 0.5 slopes for 150.0 m and 302.0 m respectively. Vertical curve at the distance of 150.0 m from toe of crest. Radius 165.00 m height of side wall 7.50 m-10.5 m
Bucket	Radius 58.0 m and lip angle 35 degree EL 490.00 m
Plunge Pool	Width 130.00 and length 250.0 m
Aeration tower at	40.0 m from end of first slope

5.3.3 The figures in sheet no. 2a, 2b and 2c in page no. 69 to71show layout and various components of chute spillway dimensioned as per design described above.


TOP OF DAM EL 695.00 M
CHUTE SPILLWAY
GATE CONTROL SECTION
FLOOD FLOW
RIVER FLOW
DOWNSTREAM COFFER DAM
MASTER PLAN OF CHUTE SPILLWAY
DESIGN OF HIGH HEAD SPILLWAY FOR
PANCHESHWAR DAM
SHEET TITLE: CHUTE SPILLWAY
SHEET NO:
2a
LAYOUT-CHUTE SPILLWAY-PAGE N. 69



PLAN OF CHUTE SPILLWAY WITH DAM

SCALE:C
ALTERNATE :02

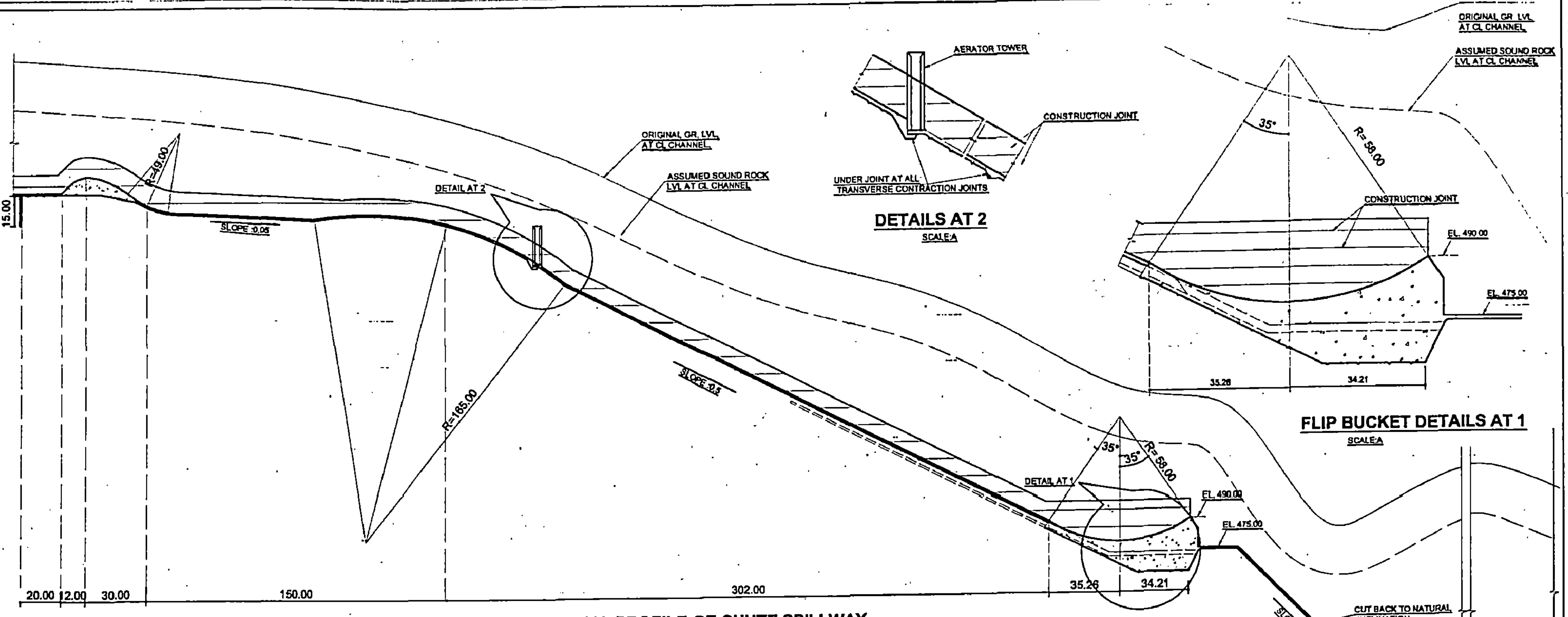
SCALE:C	40	80	120	160	200	240
SCALE:B	20	40	60	80	100	120
SCALE:A	10	20	30	40	50	60
SCALE:AD	5	10	15	20	25	30

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: CHUTE SPILLWAY

CHUTE SPILLWAY LAYOUT, DESIGN PAGE N. 70

SHEET NO:
2b



DETAILS AT 2

SCALE:A

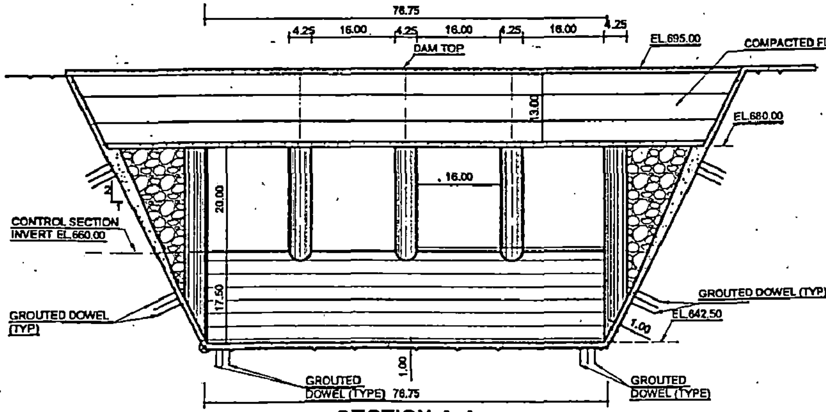
FLIP BUCKET DETAILS AT 1

SCALE:A

LONGITUDINAL PROFILE OF CHUTE SPILLWAY

ALTERNATE :02

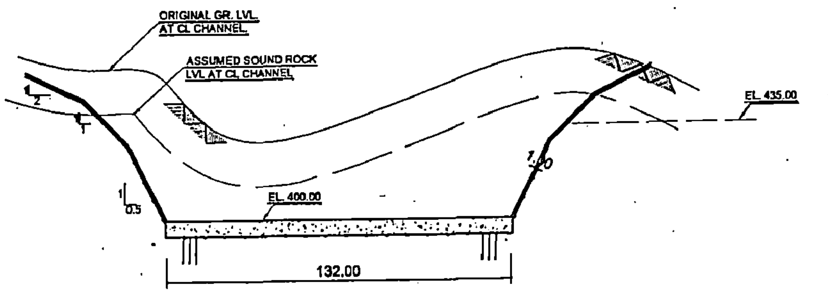
SCALE:B



SECTION A-A

(LOOKING UPSTREAM OF CONTROL SECTION)

SCALE:A



TYPICAL SECTION THROUGH PLUNGE POOL

SCALE:B

NOTE: ALL DIMENSIONS ARE IN METERS
SCALE OF THE DRAWINGS ARE AS SHOWN

SCALE C 40	80	120	160	200	240	
SCALE B 20	40	60	80	100	120	
0 SCALE A 10	20	30	40	50	60	
SCALE D 5	10	15	20	25	30	

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: CHUTE SPILLWAY

SHEET NO. 2c

CHUTE SPILLWAY, DESIGN PAGE N. 71

5.4 Chute and shaft spillway combination

5.4.1 Chute spillway general

Pancheshwar dam site is in narrow gorge in the hilly mountainous region. The project has a provision for diversion of flood from the two-diversion tunnels (horse shoe 12.0 m dia.) one along each bank. To utilize diversion tunnels, the combination of chute and shaft tunnel spillway is proposed for the Pancheshwar dam as an alternate of the dissertation study.

The flood routing study given in chapter four define the total **head over the crest** 20.37 m and **maximum** outflow (18000.0 cumecs) from the spillway for the combination of spillway structures.

The chute spillway will be used as service spillway and one in 50 year return period design flood is considered for calculations in table number 4.1.3. The design discharge Q is around 8500.0 cumecs [6].

A section with gate height 16.0 m and 12.0 m width, four bays with a clear span of 60.00 m having pier width of 4.0 m is found finally suitable by placing crest elevation at 664.00 to pass the design flood [1].

The design procedure for the rest of the components of the chute spillway is same as that of side channel spillway as described in chapter 5.3 above. The details of the chute spillway such as approach channel, control section, discharge channel, chute aeration and energy dissipators are designed.

The summary of the design parameters so worked out for the chute spillway of this option is given in section 5.4.6. The figures in sheet no. 3a, 3b and 3c show the plan of the chute and shaft spillway and detail dimensions of the chute spillway.

The model study of the design is required to verify the worked out parameters.

5.4.2 Shaft -Tunnel Morning Glory

5.4.2. General

The proposed design is an alternate option in which, two shaft tunnels are used to pass the floodwater in excess of the service chute spillway. Conventional Chute and Shaft Tunnel combination will pass the flood of one in 10,000 years or more. The design discharge form both shafts is 7000.0 cumecs. The chute spillway passes the flood of one in 50 years return period. The shaft operates at EL 684.00 m for the present proposal. Shafts are similar in both banks hence their design is same.

5.4.3 Crest Shape

As per "USBR Design of small dams" flat crested shape (circular in open) is generally adopted for the narrow gorges [[5]]. In Pancheshwar dam also the same shape is considered and two shafts one on each abutment are proposed to utilize the diversion tunnels having horseshoe shape and size of 17.0 m depth and 12.0 m width.

Assuming Q_p the maximum discharge, which is to be allowed from one tunnel shaft, is 3500.0 cumecs and from the reservoir operation studies the maximum head hv, can be taken as 5.0 m considering the spill level at 684.00 EL of shaft crest.

The diversion tunnel (one) is designed to pass 4000 cumecs of water at a head of 48 m. Considering the dia of tunnel as 17 m and its area as $A= 0.8293D^2$, the maximum velocity in the tunnel spillway is of the order of 14.60 m/s, which will occur for a short period.

5.4.3 Design of the shaft tunnel

Now considering coefficient of discharge C_d as 1.6 for flat (flat crest is selected to suit the ground condition of narrow gorge and 'USBR' conversion to metric units [[22]]) crested weir of the shaft, h_v as the head over crest 5.0 and Q_p the flood discharge to be passed from the shaft 3500.0 cumecs from the reservoir routings and from USBR design of small dams- the intensity of discharge over the crest is given as [5]:

$q = C (h_v)^{1.5}$, where, C is coefficient of discharge, h_v is the head over the crest. $q=1.6*(5.0)^{1.5}=17.99$ cumecs/m. It is high discharge intensity for tunnel shaft and may occur at certain interval of time.

Length of crest

Adopting a circular shape of the crest at EL 684.00 and keeping it at 4.0 m from the ground line. The length of the crest $L = Q_p/q = 3500.00/17.99 = 195.66$ m, where Q is the design discharge and q the discharge intensity. The radius of the crest, (Rs) $=L/2\pi$ $=31.14$ m say 32.00 m. The total radius of the crest funnel is $(R) = R_s+1 = 33.00$ m

For Pancheshwar dam, maximum water level is EL 689.00m. The upstream head loss f of the flat crested weir is given as:

$$f = h_v\{1-(1.6/1.7)^{\frac{2}{3}}\} \text{------------------------------} 5.20$$

where, h_v is the head over the crest.

$=0.20$ m. Therefore h_v-f = 4.80 m

Hence energy line TEL at crest level of the shaft is 688.80 m for maximum discharge during the flood.

5.4.4 Morning glory hole profile

As per the Creager Justin and Hinds [22], the bed elevation from the energy line is computed as 'd' where,

$$d=(h_v-f)(Rs/R1)^{2/3} \text{------------------------------} 5.21$$

Where, R_s is the radius of crest of the shaft, R_1 is the radius of the profile where depth is required, and h_1 the elevation of the profile of the flow from the bed elevation $= 2d/3$.

Now, for profile of the centre line of free fall flow curve as per Kurtz is given as:

$$y + 0.36h_1 = \frac{(x + 0.36h_1)^2}{4.56h_1} \quad \text{--5.22}$$

Where, h_l is the depth of flow at the free fall surface. The thickness of the flow is given by t, as per Creager, Justin and Hinds as:

$$t = \frac{Q}{2\pi R_1 \sqrt{2g(y + 1.5h_1)}} \quad \text{--5.23}$$

Where, the radius of the funnel shaft is R1, and y+1.5h1 is the velocity head.

Profile calculation

Table number 4.6.1

Distance from crest m	Shaft line EL m	Centreline flow profile m	Profile thickness t/2
0.00	684.00	688.80	-
16.00	681.18	679.65	2.31
23.00	677.61	679.25	2.39
23.50	677.18	678.84	2.49
24.00	676.69	678.41	2.60
24.50	676.17	677.95	3.05

The profile is plotted and found to follow the nappe flow to the shaft tunnel where it converges at the crotch. The approximate crotch elevation is EL 665.00 m

The profile and the design are shown in the drawing sheet number 4.

5.4.5 Dimension of the shafts

The shaft radius as per USBR is:

$$R = \frac{0.268 Q_p^{0.5}}{h_v^{\frac{1}{4}}} \quad \text{--5.24}$$

Where, the Q is the maximum discharge to be passed and h_v is the head of water at the section taken. The shaft diameter is fixed not more then 15.0 m in the Himalayan range.

This is mainly due to the earthquake prone area. The profile plotting gives that the diameter of the shaft will govern 15.0 m at elevation 670.0 m. The shaft is proposed to have varying diameter from 15.0 m to 13.0 m at EL 670.00 m and at the elevation 420.0 m and then it is made eccentric, to develop a swirling motion at elevation 420.0 m where it changes direction [6,17] `A slope of 10H: 1V is given to join the diversion tunnel at a distance of around 200.0 m at elevation 400.0 m.

Velocity at the bottom of the shaft is 68.92 m/s, which is a high velocity, which may cause cavitation damages in the walls of the shaft. An energy killing device is proposed as is adopted in the Tehri Dam spillways and it is shown is fig. 5.3 below and on sheet 4 page no. 81. However, the design has to be evolved in a model and various parameters like the requirement of air supply etc. are finalized.

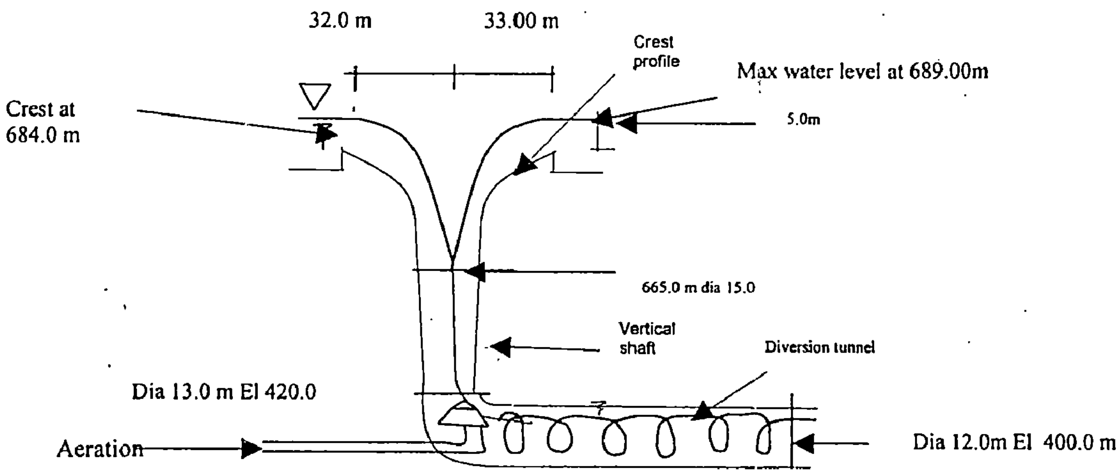


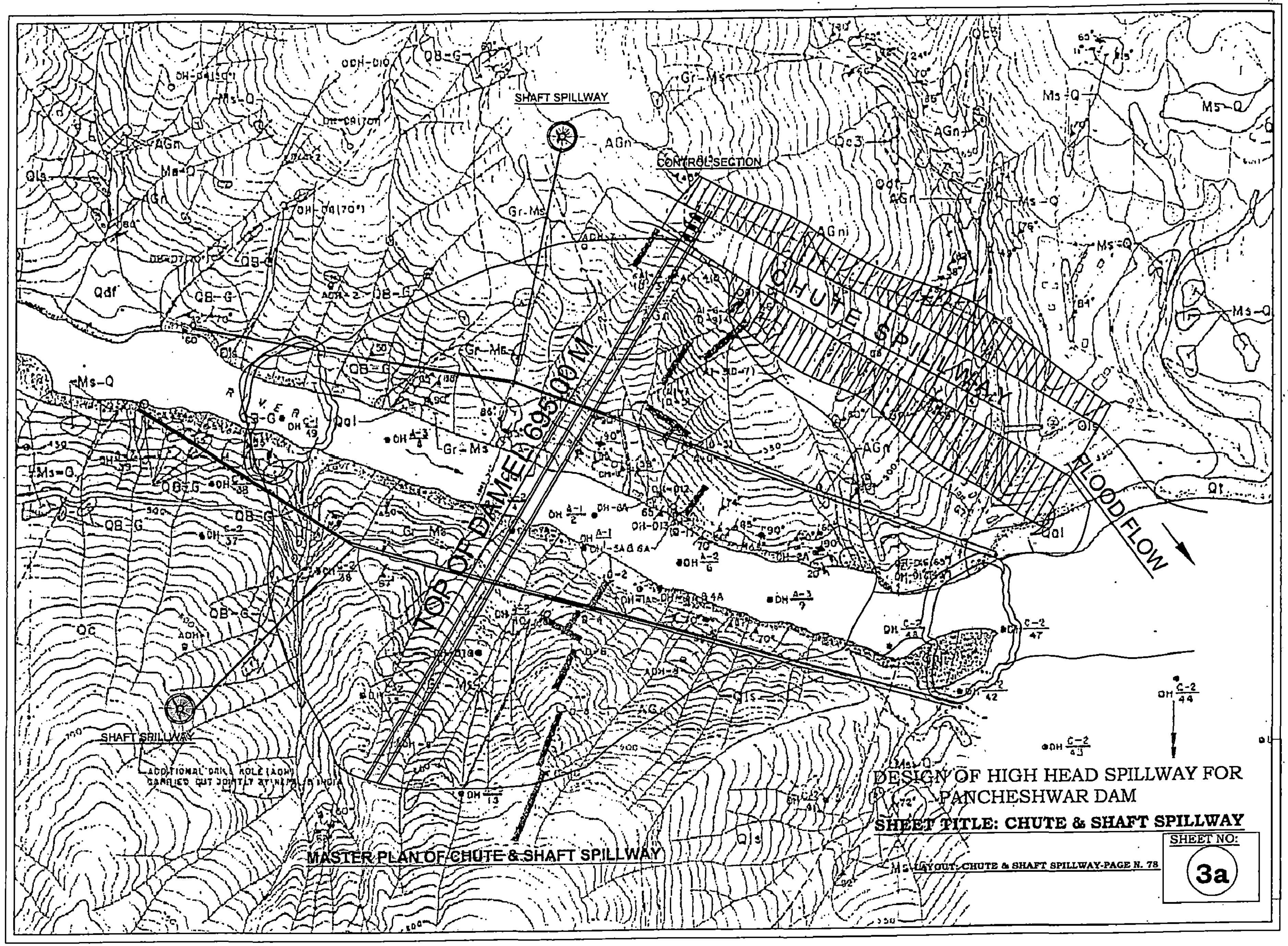
Fig. 5.3 Shaft Tunnel Sketch

5.4.6 Summary of the design parameters for chute and shaft spillway

Table no.5.5

Descriptions	**Details**
Chute spillway	**Parameters**
Maximum water level	EL 689.00 m
Maximum discharge	8500.00 cumecs
Head over crest H_d	19.70 m
Crest at EL	664.00m
Approach channel	EL 656.00 m length 50.00 m and width 60.00 m
Control Section	Ogee control Crest length 60.00 m width 25.00 m, Gated 16 m x12 m. 4.0 m width Piers 3 nos.
Transition Zone	Length 460.00 width 60.00 m-110.00 m
Discharge channel	0.05 And 0.5 slopes for 150.0 m and 310.0 m respectively. Vertical curve at the toe of mild slope radius 170.0 m, maximum height of side wall 6.50 m-8.50 m
Bucket	Radius 53.0 m and lip angle 35 degree EL 487.50 m
Plunge Pool	Width 110.00 and length 250.0 m
Aeration tower at	40.00 m from end of first slope
Shaft-tunnel	
Design flow	2 X3500.00 cumecs
Crest at	EL 684.00 m
Head over the crest	5.00 m
Radius of shaft intake	33.00 m
Diameter of the main shaft	Varying from 15.0 m to 13.0 m
Crotch elevation	At EL 665.0 m
Maximum water level	EL 689.0m
Tail water level	EL 420.0 m
Velocity at the bottom	68.92 m

5.4.3 The figures in sheet no. 3a, 3b, 3c and 4 in page no. 78 to 81 show layout and various components of chute and shaft spillway dimensioned as per design described above.

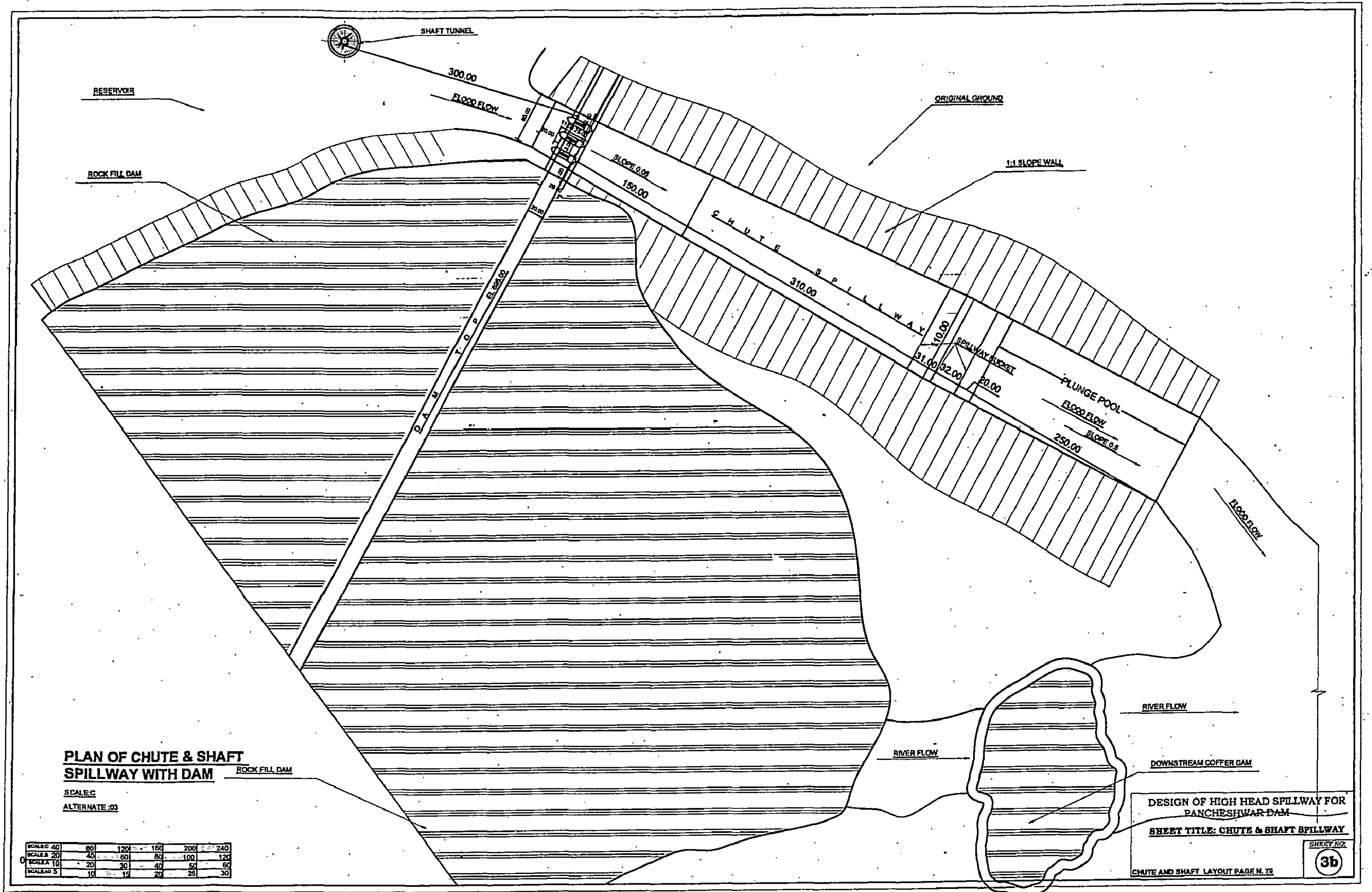


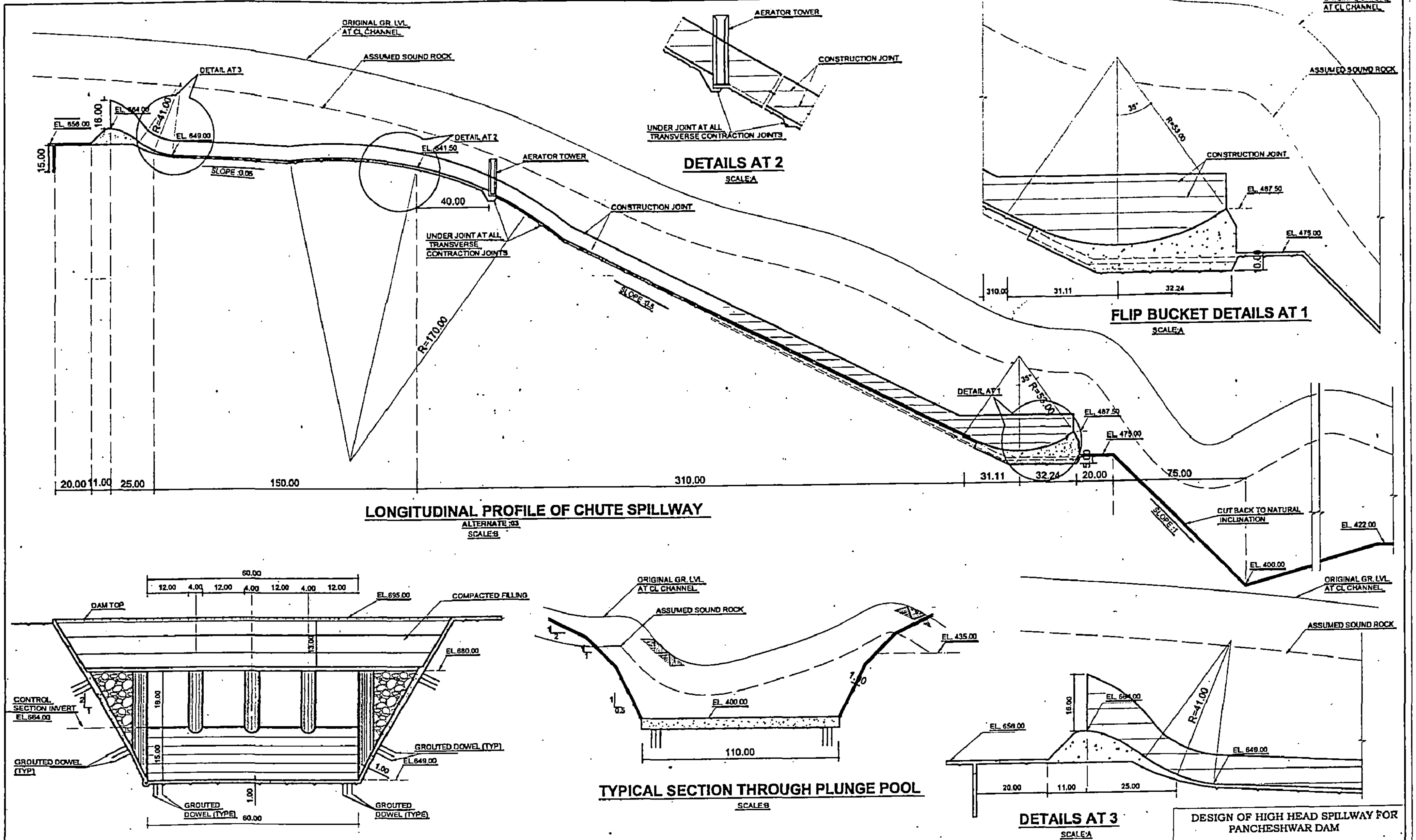
DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: CHUTE & SHAFT SPILLWAY

SHEET NO: **3a**

LAYOUT: CHUTE & SHAFT SPILLWAY-PAGE N. 78


SHAFT TUNNEL
300.00
RESERVOIR
FLOOD FLOW
ORIGINAL GROUND
1:1 SLOPE WALL
ROCK FILL DAM
SLOPE 0.05
150.00
CHUTE SPILLWAY
310.00
DAM TOP El 695.00
110.00
31.00
32.00
SPILLWAY BUCKET
20.00
PLUNGE POOL
FLOOD FLOW
SLOPE 0.5
250.00
FLOOD FLOW
RIVER FLOW
RIVER FLOW
DOWNSTREAM COFFER DAM
PLAN OF CHUTE & SHAFT SPILLWAY WITH DAM
ROCK FILL DAM
SCALE:C
ALTERNATE :03
SCALE:C 40 80 120 160 200 240
SCALE:B 20 40 60 80 100 120
SCALE:A 10 20 30 40 50 60
SCALE:AO 5 10 15 20 25 30
DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM
SHEET TITLE: CHUTE & SHAFT SPILLWAY
SHEET NO:
3b
CHUTE AND SHAFT LAYOUT PAGE N. 79



SCALE:C 40	80	120	160	200	240
SCALE:B 20	40	60	80	100	120
SCALE:A 10	20	30	40	50	60
SCALE:AO 5	10	15	20	25	30

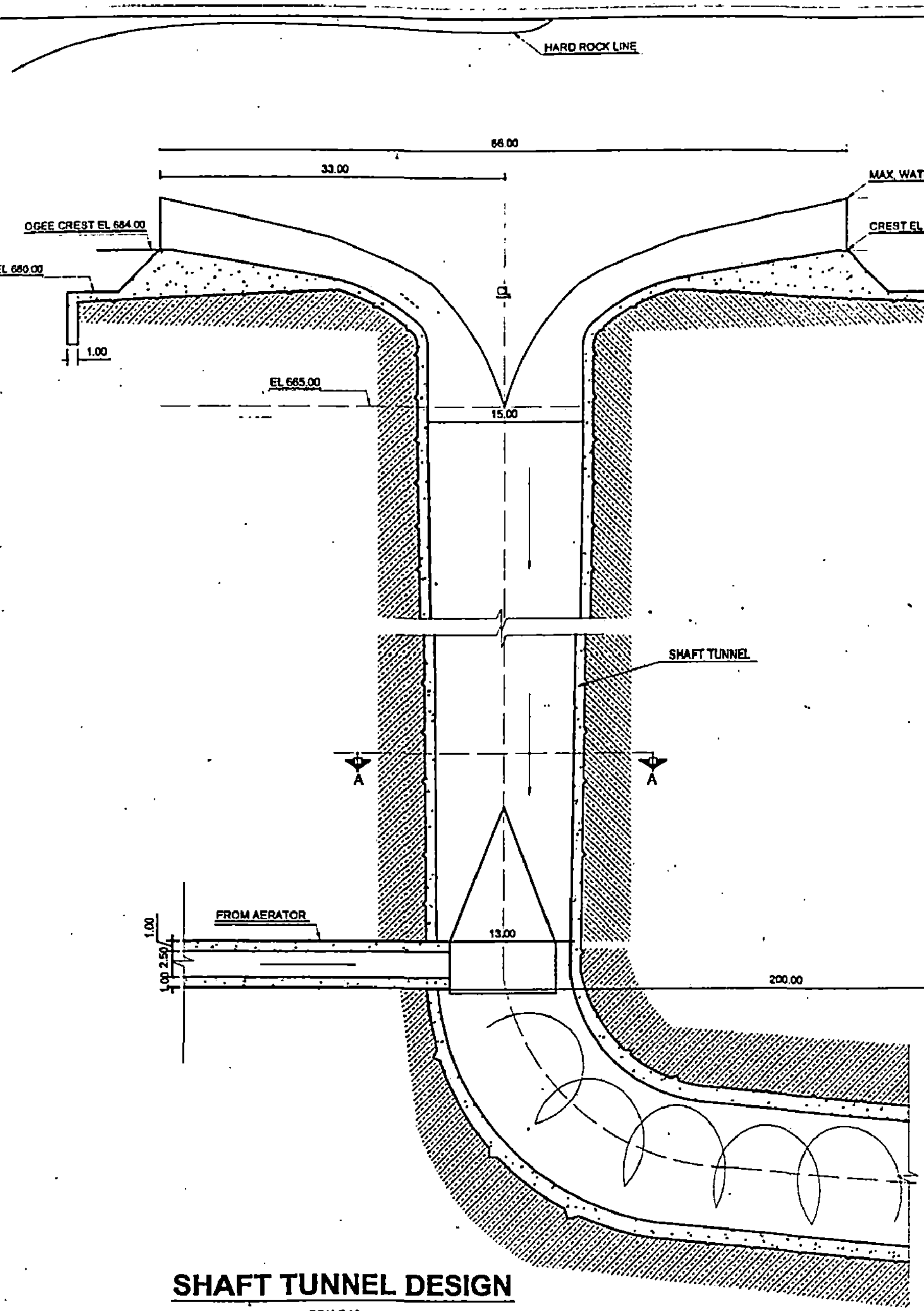
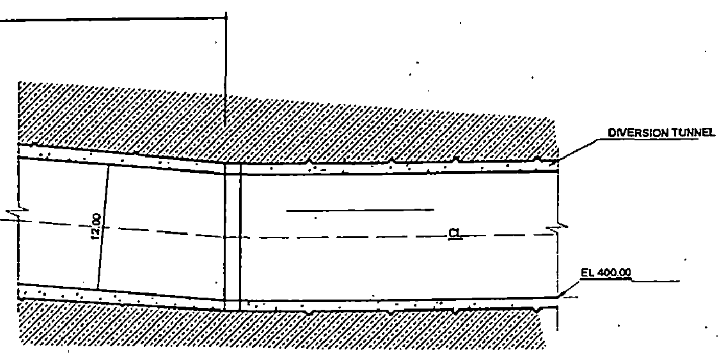
NOTE: ALL DIMENSIONS ARE IN METERS
SCALE OF THE DRAWINGS ARE AS SHOWN

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: CHUTE SPILLWAY

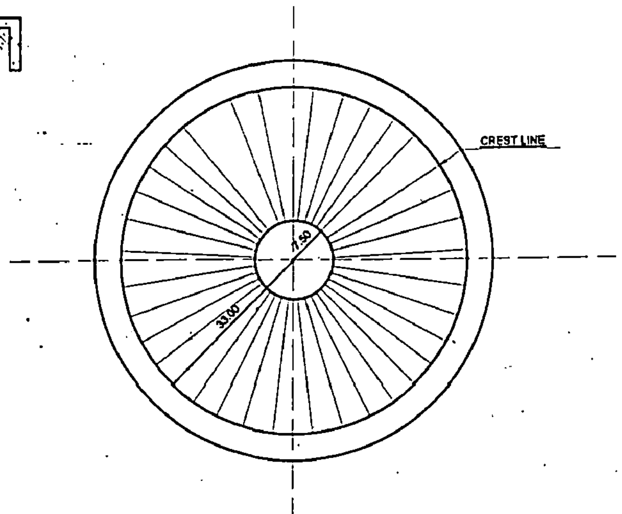
CHUTE SPILLWAY OF C&S DESIGN PAGE N. 80

SHEET NO: 3c

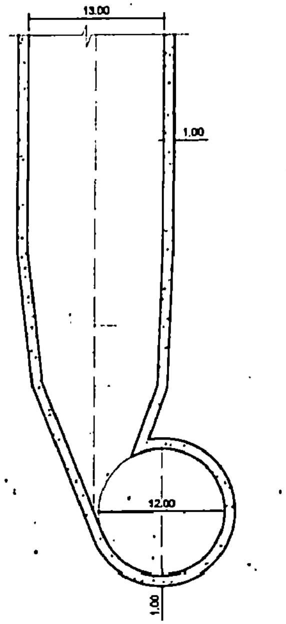
SHAFT TUNNEL DESIGN

SCALE:A0



PLAN OF SHAFT TUNNEL

SCALE:A



SECTION AT A-A

SCALE:A0

NOTE: ALL DIMENSIONS ARE IN METERS
SCALE OF THE DRAWINGS ARE AS SHOWN

SCALE:C	40	80	120	160	200	240
SCALE:B	20	40	60	80	100	120
SCALE:A	10	20	30	40	50	60
SCALE:A0	5	10	15	20	25	30

0

DESIGN OF HIGH HEAD SPILLWAY FOR PANCHESHWAR DAM

SHEET TITLE: SHAFT SPILLWAY

SHEET NO: 4

SHAFT TUNNEL SPILLWAY DESIGN PAGE N. 81

5.5 Comparison of quantities and cost for different options of spillways

The design is laid down on the contour map of the site. The calculations are made from available contour maps and exact volume will have to be surveyed as per design. The shafts are laid vertical and a 200.0 m length of the horizontal shaft is kept 10H: 1V to join the diversion tunnel. Two major items of works earthwork and concreting have been worked out and compared.

5.5.1 Volume of earth works

Following table gives the volume of earthwork for the options designed.

A) Side channel spillway

Earthwork quantity

Details area	Sectional area average	Length	Volume Cu.m
Trough	23370	265	6193050
Chute first slope	15060	180	2710800
Chute second slope	8885	250	2665500
Plunge pool	7050	250	2432250
		Total	14.0016 MCM

B) Chute spillway

Earthwork quantity

Details area	Sectional area average	Length	Volume Cu.m
Control area	13922.5	65	904962.5
Chute first slope	15077.5	150	2261625
Chute second slope	8285	250	2502070
Plunge pool	6950	345	2501250
		Total	8.1699 MCM

C) Chute spillway in combination of shaft tunnel

Earthwork quantity

Details area	Sectional area	Length	Volume Cu.m
Control area	9050	60	543000
Chute first slope	10540	150	1581000
Chute second slope	6540	373	2439420
Plunge pool	5070	345	1749150
			6.3126 MCM
Shaft tunnel volume	Circular F		**125663**
	Shafts		**714172**
		Sub total	**0.8498 MCM**
		Total	**7.1524 MCM**

5.5.2 Volume of concrete works

Following table gives the volume of concrete for the options designed.

A) Side channel spillway

Concrete quantity

Details area	Length	Volume Cu.m
Approach	100	**27870**
Trough	175	**42595**
Transition	70	**10850**
Control pier		**9800**
Chute first slope	180	**16311**
Chute second slope	250	**31439**
Buckets	50.7	**27860**
Plunge pool	376	**67680**
	Total	**231166**
		0.2312MCM

B) Chute spillway concrete works

Concrete quantity

Details area	Length	Volume Cu.m
Approach	100	**17675.00**
Control section ogee	30	**20820.00**
Simple section	12	**2109.00**
Control pier		**12750.00**
Chute first slope	150	**15919.50**
Chute second slope	302	**39488.01**
Buckets	69.5	**77562.00**
Plunge pool	376	**90992.00**
	Total	**277315.50**
		0.2773MCM

C) Chute and shaft spillway concrete works

Concrete quantity

Details area	Length	Volume Cu.m
Approach	40	**5907.00**
Control section ogee	30	**11375.00**
Simple section	35	**1705.00**
Control pier		**8000.00**
Chute first slope	150	**12739.50**
Chute second slope	302	**36570.00**
Buckets	69.5	**36902.00**
Plunge pool	376	**75200.00**
Shaft funnel		**11489.00**
Shaft		**86480.00**
	Total	**283367.00**
		0.2833 MCM

Summary for comparison

The rates for hard rock excavations and hill cutting are taken from the recently used rates of Asi ganga hydropower project district Utrakashi district of Uttranchal (India) etc. and a ten percent increase is added to it. Rate are not analysed and considered only for a comparison. The rates per cu.m of open excavation IRs 210.00, for shaft excavation IRs 800.00 and for tunnel excavations IRS 550.00 and for M-20 concrete IRs 4300.00.

Works

Table number 5.5.1

Items	Earth works	Concrete
Side channel	14.00 MCM	0.2312 MCM
Single Chute	8.1699 MCM	0.2773MCM
Chute and shaft	7.1524 MCM	0.2833MCM

Costs

Table number 5.5.2

S.No.	Items	Costs in IRS Million
1	Side channel	3934.00
2	Single Chute	2908.00
3	Chute and shaft Tunnel	3110.00

Chapter 6

CONCLUSION

6.1 Pancheshwar Dam is a 280.0 m high earth and rock fill dam. This study of the project is limited to techno-economic suitability of spillway. Preliminary designs of three alternative spillways have been prepared and then costs have been worked out and are given in chapter 5. Following table gives itemwise quantities of work for the three spillways.

Table 6.1

Details of Works Items	Side Channel Spillway	Chute Spillway Single	Shaft and Chute combination
1) Earth works	14.00 MCM	8.169 MCM	7.152 MCM
2) Concrete works	0.2312 MCM	0.2773 MCM	0.2833 MCM
3) Side wall height	13.00 m Large	10.50 m Large	8.50 m Small
4) Width of the channel	70.0 m	(76.75-130) Large	(60-110) Small
5) Width of the Plunge pool	70.0 m	130.0 m Large	110.0 m Small
6) Type of control	Ungated	Gated	Gated
7) Depth of flow	Maximum	-	Minimum
8) Channel for Spillway	Main service Single	Main service Single	Service only and shaft
9) Environmental effect	-	-	Low

6.2 It is seen from the table 6.1 that the volume of earthwork is about 6.0 million cubic metres more in side channel spillway as compared to chute spillway. The dam is rock fill and if the excavation from the side channel can be used as dam fill material, the side channel option may be considered otherwise it is costlier than other options.

6.3 From the reservoir routing study, it is seen that gated chute and shaft tunnel and gated chute alternatives give low dam height in comparison to a side channel alternative.

6.4 The energy dissipation arrangement in side channel and single chute spillways will be more expensive than a combination of chute and shaft spillways because of the provision of a swirling device proposed in the shafts.

6.5 In case of shaft and chute combination the diversion tunnels will be utilized.

6.6 In view of the above advantages based on the study presented in this report, it is inferred that spillway arrangement comprising shaft and chute is better for Pancheshwar Dam Project.

Future studies

1 Detailed design of Shaft- Chute Spillway should be carried out.

2 The designs should be tested on a model before adoption.

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ANNEXURE-1a

Flood Routing Studies for Pancheshwar Spillway

(Pulse method is used) For side channel Spillway

Sn	Flood hydrograph Time- hour Hours	Inflow (I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
	0	1000			11.485		681.00	290.5
1			1175	0.017	11.483	11.500	681.10	340
	4	1350						
2			1615	0.023	11.495	11.518	681.25	400
	8	1880						
3			2270	0.033	11.512	11.545	681.45	525
	12	2660						
4			3249	0.047	11.538	11.584	681.85	750
	16	3838						
5			5034	0.072	11.574	11.646	682.30	1025
	20	6230						
6			7160	0.103	11.631	11.734	683.00	1625
	24	8090						
7			9085	0.131	11.711	11.842	683.85	2350
	28	10080						
8			11020	0.159	11.808	11.967	684.80	3300
	32	11960						
9			12330	0.178	11.919	12.097	685.85	4550
	36	12700						
10			13405	0.193	12.031	12.224	686.80	5775
	40	14110						
11			15685	0.226	12.141	12.367	687.90	7450
	44	17260						
12			19195	0.276	12.260	12.536	689.30	9600
	48	21130						
13			22200	0.320	12.398	12.717	690.65	11900
	52	23270						
14			22812.5	0.329	12.546	12.875	691.75	14000
	56	22355						
15			21257.5	0.306	12.673	12.979	692.45	15450
	60	20160						
16			19447.5	0.280	12.757	13.037	692.85	16150
	64	18735						
17			18847.5	0.271	12.804	13.076	693.10	16650
	68	18960						
18			19102.5	0.275	12.836	13.111	693.30	17000
	72	19245						
19			19162.5	0.276	12.866	13.142	693.47	17550
	76	19080						
20			18352.5	0.264	12.889	13.154	693.55	17700
	80	17625						
21			16760	0.241	12.899	13.140	693.45	17500
	84	15895						
22			14577.5	0.210	12.888	13.098	693.20	16975

Flood Routing Studies for Pancheshwar Spillway

(Pulse method is used)

Sn	Flood hydrograph Time- hour Hours	Inflow (I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
	88	13260						
23			12672.5	0.182	12.853	13.036	692.85	16200
	92	12085						
24			11375	0.164	12.803	12.966	692.55	15700
	96	10665						
25			9762.5	0.141	12.740	12.881	691.80	14100
	100	8860						
26			8207.5	0.118	12.678	12.796	691.20	12925
	104	7555						
27			7210	0.104	12.610	12.714	690.60	11800
	108	6865						
28			5925	0.085	12.544	12.629	690.00	10775
	112	4985						
29			4510	0.065	12.474	12.539	689.30	9600
	116	4035						
30			3720	0.054	12.401	12.454	688.65	8650
	120	3405						
31			3012.5	0.043	12.330	12.373	687.95	7500
	124	2620						
32			2505	0.036	12.265	12.301	687.45	6700
	128	2390						
33			2207.5	0.032	12.205	12.237	686.95	6000
	132	2025						
34			2007.5	0.029	12.150	12.179	686.50	5375
	136	1990						
35			1800	0.026	12.102	12.128	686.10	4850
	140	1610						
36			1462.5	0.021	12.058	12.079	685.70	4400
	144	1315						
37			1157.5	0.017	12.015	12.032	685.30	3900
		1000						

From the reservoir routing studies it is clear that a side channel spillway of capacity 17700 cumecs is required for the peak flood of 23270 cumecs. It will have a Surcharge in the reservoir of 13.55 m at peak flow.

ANNEXURE-1b

Flood Routing Studies for Pancheshwar Spillway

Chute alternate a clear span of 76.75m crest at EL 660.00m.

(Pulse method is used) For Chute Spillway size 20x16

Sn	Flood hydrograph Time- hours Hours	Inflow(I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
1	0	1000				11.354	680.00	0.00
			1175.00	0.017	11.354	11.371	6810.15	0.00
2	4	1350						
			1615.00	0.023	11.371	11.394	680.30	0.00
3	8	1880						
			2270.00	0.033	11.394	11.427	680.45	0.00
4	12	2660						
			3249.00	0.047	11.427	11.474	680.70	0.00
5	16	3838						
			5034.00	0.072	11.474	11.546	681.10	6600.00
6	20	6230						
			7160.00	0.103	11.451	11.554	681.15	6800.00
7	24	8090						
			9085.00	0.131	11.456	11.587	681.35	8000.00
8	28	10080						
			11020.00	0.159	11.472	11.631	681.60	9650.00
9	32	11960						
			12330.00	0.178	11.492	11.669	681.85	11200.00
10	36	12700						
			13405.00	0.193	11.508	11.701	682.05	12175.00
11	40	14110						
			15685.00	0.226	11.526	11.751	682.45	12400.00
12	44	17260						
			19195.00	0.276	11.573	11.849	683.25	12800.00
13	48	21130						
			22200.00	0.320	11.665	11.985	684.45	13375.00
14	52	23270						
			22815.50	0.329	11.792	12.121	685.50	13825.00
15	56	22355						
			21257.50	0.306	11.922	12.228	686.35	14225.00
16	60	20160						
			19447.50	0.280	12.023	12.303	687.00	14500.00
17	64	18735						
			18847.50	0.271	12.094	12.365	687.50	14700.00
18	68	18960						
			19102.50	0.275	12.154	12.429	688.00	14925.00
19	72	19245						
			19162.50	0.276	12.214	12.490	688.50	15125.00
20	76	19080						
			18352.50	0.264	12.272	12.536	688.90	15275.00
21	80	17625						
			16760.00	0.241	12.316	12.558	689.10	15350.00
22	84	15895						
			14577.50	0.210	12.337	12.547	688.96	15300.00

Flood Routing Studies for Pancheshwar Spillway

(Pulse method is used)

Sn	Flood hydrograph Time- hour Hours	Inflow(I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
	88	13260						
23	-		12672.5	0.182	12.326	12.509	688.65	15175.00
	92	12085						
24			11375	0.164	12.290	12.454	688.20	15025.00
	96	10665						
25			9762.5	0.141	12.238	12.378	687.60	14750.00
	100	8860						
26			8207.5	0.118	12.166	12.284	686.80	14425.00
	104	7555						
27			7210	0.104	12.076	12.180	686.00	14075.00
	108	6865						
28			5925	0.085	11.977	12.063	685.15	13700.00
	112	4985						
29			4510	0.065	11.866	11.930	683.90	13125.00
	116	4035						
30			3720	0.054	11.741	11.795	682.80	12625.00
	120	3405						
31			3012.5	0.043	11.613	11.657	681.80	10900.00
	124	2620						
32			2505	0.036	11.500	11.536	681.05	6175.00
	128	2390						
33			2207.5	0.032	11.447	11.479	680.70	4000.00
	132	2025						
34			2007.5	0.029	11.421	11.450	680.55	3100.00
	136	1990						
35			1800	0.026	11.405	11.431	680.40	2400.00
	140	1610						
36			1462.5	0.021	11.397	11.418	680.35	2025.00
	144	1315						
37			1315	0.019	11.389	11.407	680.25	1450.00

chute design taken is 20x16 having Go=20m and 16 is the bay width 4bays and 3pier of 4.25m. From the reservoir routing studies it is clear that a ogee chute spillway of capacity 15350 cumecs is required for the peak flood of 23270 cumecs. It will have a surcharge in the reservoir of 9.10 m at peak flow.

ANNEXURE-1c

Flood Routing Studies for Pancheshwar Spillway

Chute alternate a clear span of 60.00m crest at EL 664.00m.

(Pulse method is used) For Chute Spillway and shaft combination

Sn	Flood hydrograph Time- hou Hours	Inflow(I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
1	0	1000				11.354	680.00	5791.00
			1175.00	0.017	11.312	11.329	<680	0.00
2	4	1350						
			1615.00	0.023	11.329	11.352	680.00	5791.00
3	8	1880						
			2270.00	0.033	11.269	11.302	<680	0.00
4	12	2660						
			3249.00	0.047	11.302	11.349	680.00	5791.00
5	16	3838						
			5034.00	0.072	11.265	11.338	<680	0.00
6	20	6230						
			7160.00	0.103	11.338	11.441	680.30	5860.00
7	24	8090						
			9085.00	0.131	11.356	11.487	680.75	6050.00
8	28	10080						
			11020.00	0.159	11.400	11.559	681.20	6200.00
9	32	11960						
			12330.00	0.178	11.469	11.647	681.95	6475.00
10	36	12700						
			13405.00	0.193	11.554	11.747	682.75	6725.00
11	40	14110						
			15685.00	0.226	11.650	11.876	683.85	7040.00
12	44	17260						
			19195.00	0.276	11.774	12.051	685.25	8375.00
13	48	21130						
			22200.00	0.320	11.930	12.250	686.75	10750.00
14	52	23270						
			22815.50	0.329	12.095	12.424	688.00	13250.00
15	56	22355						
			21257.50	0.306	12.233	12.539	688.90	15250.00
16	60	20160						
			19447.50	0.280	12.319	12.599	689.40	16400.00
17	64	18735						
			18847.50	0.271	12.363	12.635	689.65	17050.00
18	68	18960						
			19102.50	0.275	12.389	12.664	689.85	17525.00
19	72	19245						
			19162.50	0.276	12.412	12.688	690.00	17975.00
20	76	19080						
			18352.50	0.264	12.429	12.693	690.05	18000.00
21	80	17625						
			16760.00	0.241	12.434	12.675	689.95	17800.00
22	84	15895						
			14577.50	0.210	12.419	12.629	689.60	16900.00

Flood Routing Studies for Pancheshwar Spillway

(Pulse method is used)

Sn	Flood hydrograph Time- hou Hours	Inflow(I) Cumecs	(I1+I2)/2 Cumecs	I bar * del t BCM	S-O*del t/2 BCM	S+O*del t/2 BCM	Elevation	Outflow
	88	13260						
23			12672.5	0.182	12.386	12.568	689.15	15800.00
	92	12085						
24			11375	0.164	12.341	12.504	688.65	14650.00
	96	10665						
25			9762.5	0.141	12.293	12.434	688.10	13400.00
	100	8860						
26			8207.5	0.118	12.241	12.359	687.55	12300.00
	104	7555						
27			7210	0.104	12.182	12.286	687.05	11300.00
	108	6865						
28			5925	0.085	12.123	12.209	686.45	10250.00
	112	4985						
29			4510	0.065	12.061	12.126	685.85	9175.00
	116	4035						
30			3720	0.054	11.994	12.047	685.20	8250.00
	120	3405						
31			3012.5	0.043	11.929	11.972	684.25	7300.00
	124	2620						
32			2505	0.036	11.867	11.903	684.05	7125.00
	128	2390						
33			2207.5	0.032	11.800	11.832	683.50	6950.00
	132	2025						
34			2007.5	0.029	11.732	11.761	682.85	6750.00
	136	1990						
35			1800	0.026	11.664	11.690	682.30	6575.00
	140	1610						
36			1462.5	0.021	11.595	11.616	681.20	6375.00
	144	1315						
37			1157.5	0.017	11.524	11.541	681.05	6150.00

From the reservoir routing studies it is clear that a Ogee Chute & shaft tunnel combine discharges 18000 cumecs during the peak flood of 23270 cumecs. It will have a surcharge in the reservoir of 10.05 m at peak flow.

ANNEXURE-2

The yearly distribution of average flow at Pancheshwar site

		Months with average daily discharge cumecs.												
Sn	Year	Jan.	Feb	Mar	Apr	May	June	July	Aug	Sept.	Oct.	Nov.	Dec	Average
1	1962	223	212	228	252	331	752	1252	2427	1712	639	309	212	712
2	1963	162	136	165	203	324	672	1526	2381	1476	484	282	205	668
3	1964	161	134	125	176	211	435	1481	1686	1480	534	279	207	576
4	1965	166	155	163	217	255	485	876	1076	725	321	215	165	402
5	1966	129	125	119	134	237	472	1038	1874	854	331	210	160	474
6	1967	128	110	107	131	176	366	1155	1937	1121	426	247	189	508
7	1968	168	152	167	182	294	695	1497	1842	918	466	261	189	569
8	1969	166	145	148	183	352	587	1172	1767	1564	594	303	209	599
9	1970	172	151	145	190	257	620	1648	1748	999	548	296	214	582
10	1971	174	159	174	241	273	1207	1763	2169	1520	611	361	250	742
11	1972	197	193	187	198	365	464	1193	1273	1235	447	280	200	519
12	1973	177	162	195	274	429	930	1579	1668	1387	1212	399	251	722
13	1974	200	172	158	204	240	398	989	1667	921	482	272	202	492
14	1975	178	171	176	260	396	1317	1568	1859	1585	659	330	236	728
15	1976	184	169	157	198	326	502	937	1435	1112	404	248	184	488
16	1977	154	141	124	144	238	422	1479	1793	1066	454	276	204	541
17	1978	171	165	206	252	484	758	1500	2318	1181	490	281	216	669
18	1979	169	174	161	223	397	546	1131	1245	499	280	182	150	430
19	1980	130	110	133	181	300	556	1591	2053	1091	436	248	180	584
20	1981	155	138	148	197	321	470	1503	1826	937	556	302	206	563
21	1982	171	161	230	280	391	724	1176	1928	1141	415	253	182	588
22	1983	161	135	129	208	361	475	902	1324	2359	1385	363	216	668
23	1984	167	182	174	202	494	972	1484	1398	1226	403	229	168	592
24	1985	147	121	117	151	310	483	1351	1872	1232	1053	473	284	633
25	1986	235	153	142	226	402	822	1979	1813	850	431	287	209	629
26	1987	160	155	139	183	276	575	1026	1493	1242	384	222	155	501
27	1988	117	108	130	219	465	605	1789	2145	920	440	283	176	616
28	1989	182	128	140	170	329	534	1066	1833	1175	427	244	173	533
29	1990	134	121	179	244	476	668	1659	1981	1261	477	246	173	635
30	1991	155	130	157	221	430	705	1279	1876	1075	381	212	152	564
31	1992	123	119	114	152	258	444	901	2022	1284	398	207	136	513
	Average	165	148	156	203	335	634.2	1338	1798	1198	534	277	195	**582**

Annexure -3

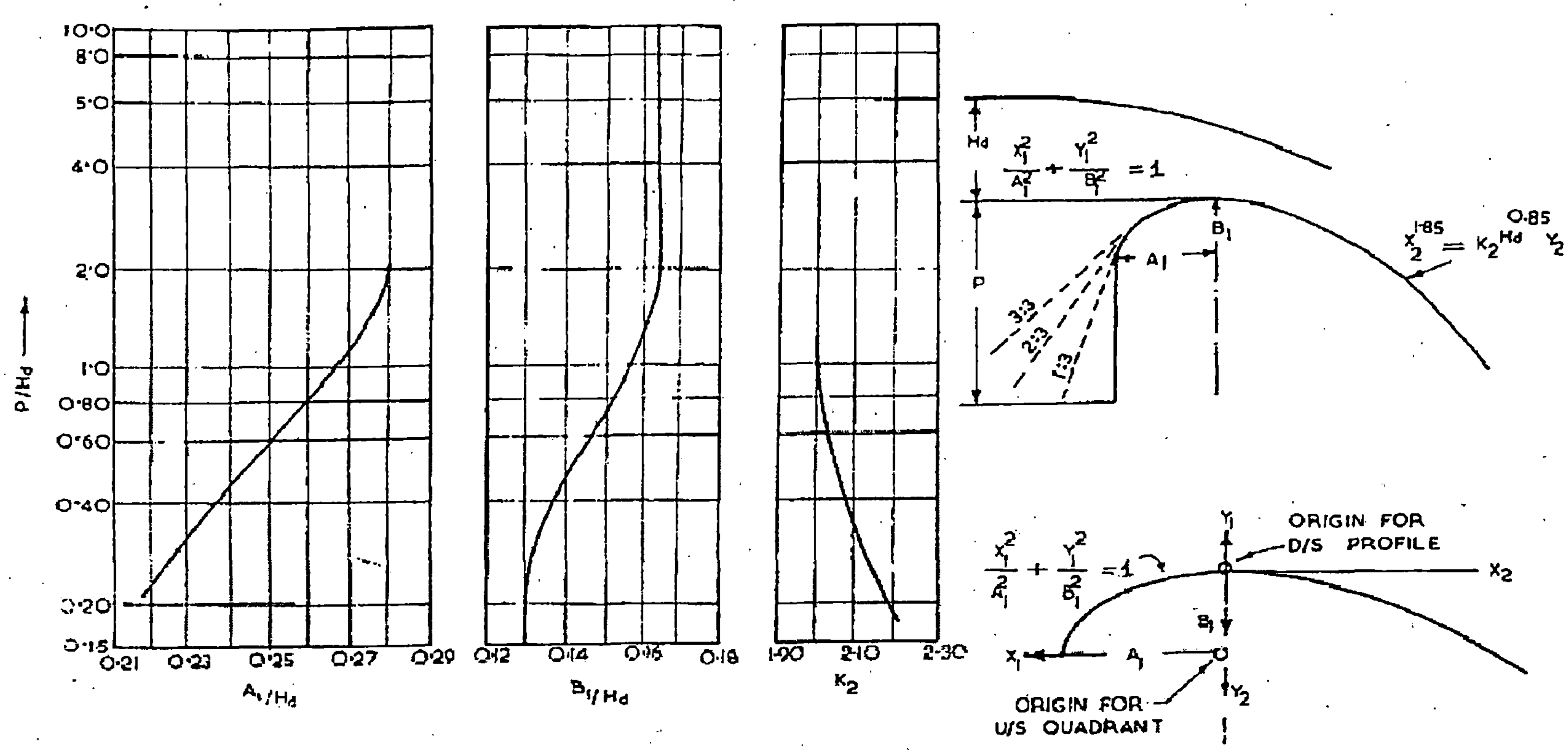


FIG. 2 OVERFLOW SPILLWAY CREST — DESIGN PARAMETERS

Annexure -3

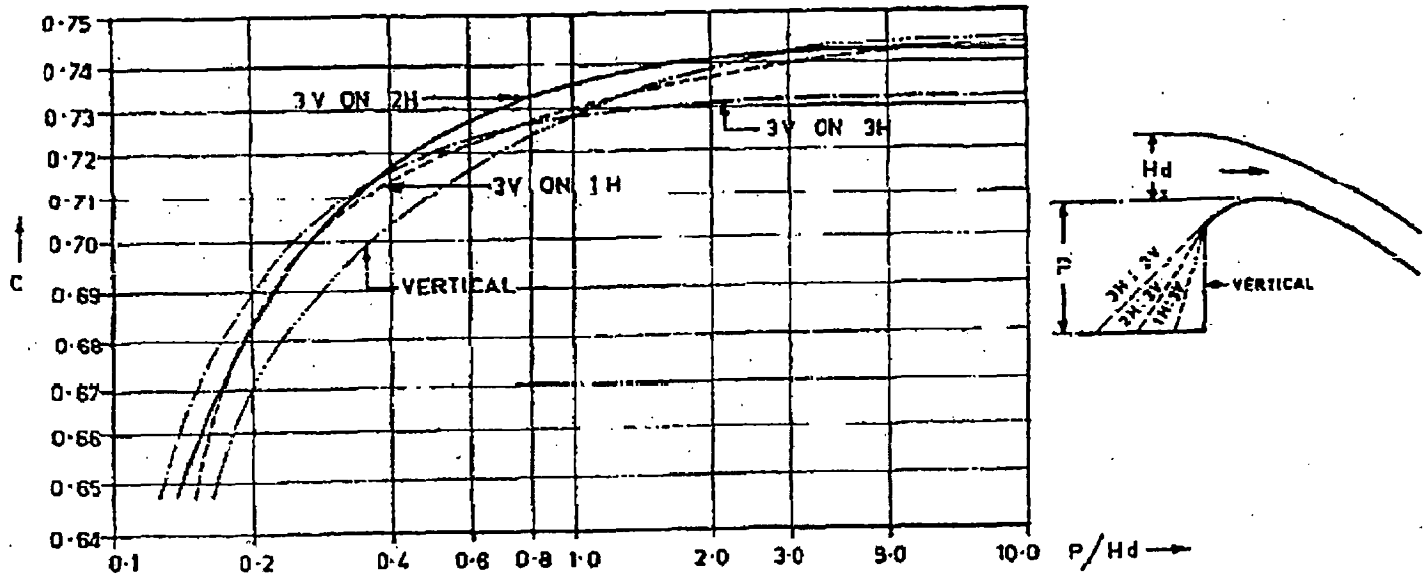
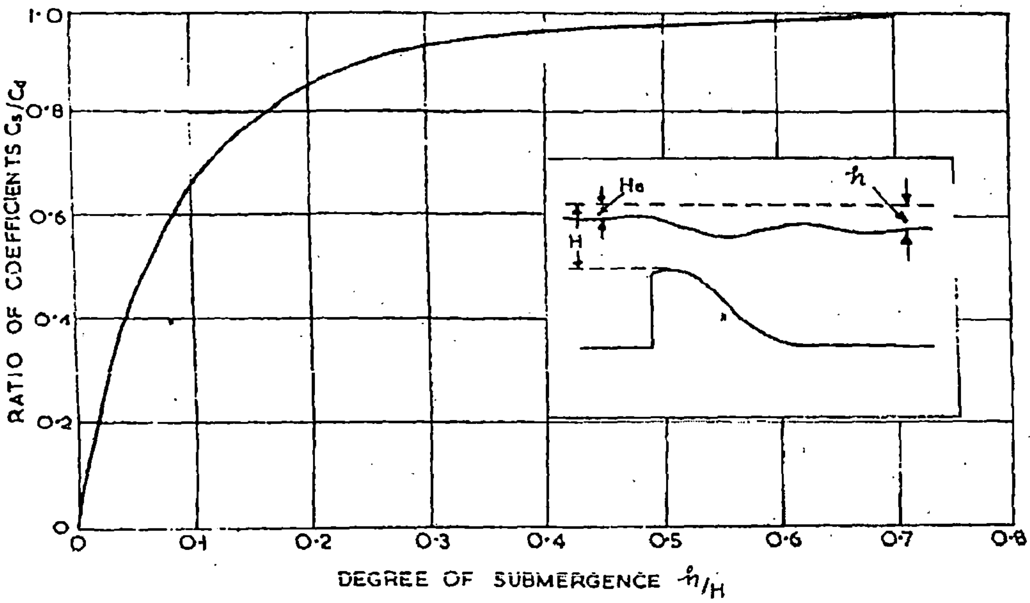
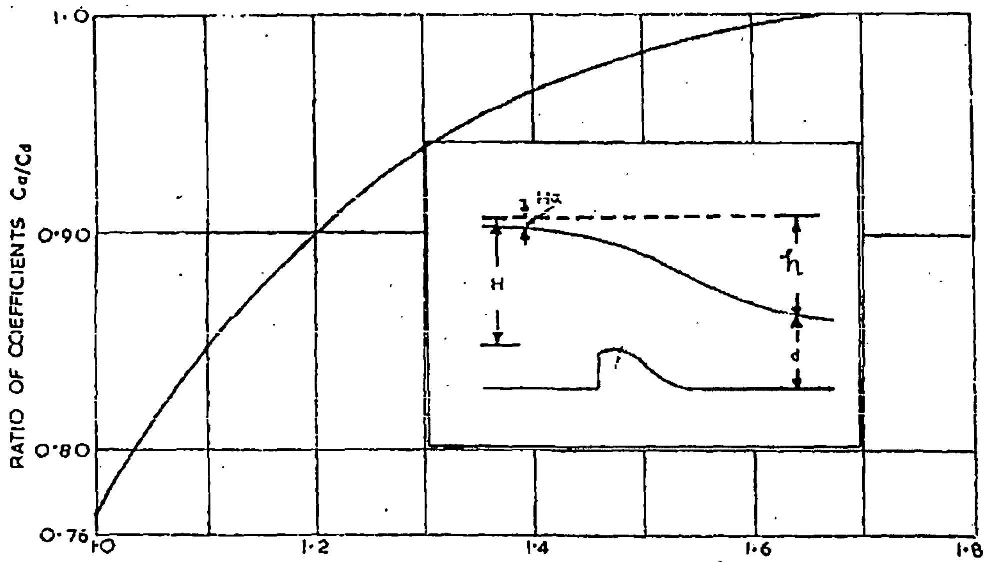


FIG. 3 DISCHARGE COEFFICIENT FOR DESIGN HEAD

Annexure-4



5A EFFECT OF TAIL WATER ON DISCHARGE COEFFICIENTS



5B EFFECT OF APRON ELEVATION ON DISCHARGE COEFFICIENTS

FIG. 5 COEFFICIENT OF DISCHARGE