STEPPED SPILLWAY MODEL INVESTIGATIVE STUDY

BY

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MAY, 1990

THESIS DECLARATION

I hereby declare that this thesis has been composed by myself and that it is a record of my own research work. It has not been accepted in any previous publication for a higher degree.

All the sources of information which are not my own originals are specifically acknowledged by means of references.



Kadai Modu Gubio

CERTIFICATION

This project thesis entitled "Stepped Spillway Model Investigative Studies", by Kadai Modu Gubio, meets the regulations governing the award of the degree of Master of Science (M.Sc) of Ahmadu Bello University, and is approved for its contribution to Scientific and Literary presentation.

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ABSTRACT

In this study, a physical hydraulic model investigation was conducted to evaluate the performance of a stepped overflow spillway. The spillway has a standard ogee profile with continuous steps cut into the spillway weir down—stream face just below the crest to the toe. The experiment was carried out on three models of the spillway weir, two of the models were of the same scale ratio 1:1OO of the prototype, one model has the stepped down — stream face while the other is unstepped, which was used to compare their test results. Discharges were allowed to pass over each of the weirs and readings were noted and compared. The third model is a bigger model of 1:50 which is also of the stepped face type. The investigation demonstrated that this stepped spillway is quite effective at dissipating energy and smooth flow transition from the spillway crest to the stepped face is easily achieved.

TABLE OF CONTENTS

		PAGE
Declaration		i
Certification		ii
Acknowledgement		iii
Abstract		iv
List of Figures		vii
List of Tables		ix
1.	INTRODUCTION	
1.1	Objective of Study	1
1.2	Justification of Study	1
1.3	Scope of the Study	2
1.4	Spillway and Functions	4
2.	LITERATURE REVIEW	9
2.1	Energy dissipation by stilling basins	9
2.2	Energy dissipation by stepped spillway weir	12
2.3	Hydraulic models	16
3.	THEORY OF ENERGY DISSIPATORS	18
3.1	Energy dissipating by stilling basins	18
3.2	Hydraulic jump formation	20
3.3	The jump concept and theoritical investigations	27
3.4	Basic characteristics of the jump	31
3.5	Jump application in stilling basin design	33
3.6	Design of models	42
3.7	Froude number criterion	43
3.8	Planning and construction of models	45
4.	FEATURES OF BIU DAM AND SPILLWAY	
4.1	Dam location	50
4.2	Hydrology of Biu Area	52
4.3	The Biu Dam spillway and components	61

5.	EXPERIMENTAL SET-UP	64
5.1	Experimental Facilities	64
5.2	Experimental models	65
5.3	Measurements and Computation	67
6.	ANALYSIS AND DISCUSSION OF RESULTS	86
6.1	Analysis and discussion of results	86
7.	CONCLUSION AND RECOMMENDATION	95
7.1	Conclusion	95
7.2	Recommendation	98
	References	103

LIST OF FIGURES

		PAGE:
CHAPTER ONE:		
1.2	Separate Types of Spillway	5
1.3	Types of Stilling Basins	8
CHAPTER TWO:		
2.1	Ski Jump Bukcet Disspator	11
2.2	Concept of Flow over Rough Surface	15
CHAPTER THREE:		
3.1(a)	Deflector Bucket Details	21
3.1(b)	Biu Spillway Section	22
3.2	Forms of the Hydraulic Jump	26
3.3	Theory of the Hydraulic Jump	29
3.4	Head Lost by the Jump	33
3.5	Jump formation illustration	36
3.6	Basin type determination curve	38
3.7	Sloping Apron stilling lBasin	40
3.8	Illustration of flow over steps	48
CHAPTER FOUR:		
4.1	Biu Dam Location Map	51
4.2	Biu Dam Cross-Sections	53
4.3	Section of Water intake Well	54
4.4	Frequency Analysis of Max day - Precipitation	59
4.5	Biu Spillway and Chute Plan	60
4.6	Hydrograph	62
CHAPTER FIVE:		71
5.1	First step size model B type	71
5.2	Second step size model B type.	72
5.3	Final step selected size from test model B type in experiment.	73

5.4	Model C type used to determine flow transition pattern.	74
5.5	Model A, unstepped type used in experimental comparison with Model B of Fig. 5.3.	75
5.6	Model–Toe velocities curves	84
5.7	Prototypical Toe–velocities Curve	85

LIST OF TABLES

CHAPTER FIVE

5a.	Step sizing test result at Q_m = 1 l/s.	76
5b.	Step sizing test result at Q_m = 3 l/s.	77
5c.	Step sizing test result at Q_m = 5 l/s.	78
5.1	Result of Model C under test	79
5.2	Model A Toe-Velocity Results	80
5.3	Model B Toe-Velocity Results	81
5.4	Corresponding Prototypical values for Model A results.	82
5.5	Corresponding Prototypical values for Model B results.	83

4.1	Rainfall Data at Biu.	97

CHAPTER ONE

INTRODUCTION

1. OBJECTIVE AND SCOPE OF STUDY

1.1 Objective of Study:

The objective of this study is to investigate the effects of introducing steps on the down stream face of a spillway weir. The characteris - tics investigated are the ability of the stepped faced weir in dissipating the energy of flowing water over the steps, its effectiveness in flow transition from the smooth surface profile at and below the spillway crest to the first few steps on the spillway face and training wall heights to contain the flow along the stepped spillway face.

1.2. Justification of Study:

If it can be shown that the amount of energy that can be dissipated is more by the steps introduced on a spillway experimentally and using models as done in this investigation, stepped spillways can be used to replace most of the existing methods of energy disspation structures which are rather costly and time consuming during construction. An example is the Biu dam spillway with an expensive chute stilling basin for energy dissipation.

1.3 Scope of the Study:

For the study, experiments were conducted in the laboratory comparing the stepped and unstepped model weirs of an ogee spillway. Weir models used were made of plywood and perspex materials for the various scale ratios. Two of the models were 1:100 scale ratio to the prototype that is stepped 1:100 model and an unstepped 1:100, tested using same discharges within a certain range to study their toe velocity ratings. The other model tested was the 1:50 scale ratio which was used to assess the flow transition of water from the unsteppedportion of the weir to the few steps introduced. Other parameters observed during the experiment are the head of water over weir crest, toe velocity, discharge passing over the weir.

In this experimental study, the ogee spillway considered was proposed for the Biu dam area, therefore, the range of discharge used and geological foundation conditions are as for the Biu dam. Except that site was not modelled exactly because of inavailability of space and appropriate equipment. But in any case, the scope of the study can give some general view and consideration for a stepped type of spillway.

Specific Energy – The hydraulic energy in a channel is defined as the energy per kilogram of water at any section of a channel measured, with respect to the depth of flow and the velocity head, usually expressed as:

$$E = Y + \frac{V^2}{2g}$$

where

E = Specific energy,

Y = Depth of flow,

V = Velocity of flow at section,

g = Acceleration due to gravity.

Critical State, Subcritical and Supercritical Flows – For a given specific energy, there are two possible depths, the low stage and the high stage which alternate, but at the critical state, these two depths apparently become one which is known as the critical depth (Yc). When the depth of flow is greater than the critical depth, then the flow is subcritical while when the the depth of flow is lessthan the critical depth, the flow is supercritical.

Design Flood and Probable Maximum Flood (PMF) – PMF is the flood that may be expected from the most severe combination of critical metrological and hydrological conditions that are reasonably possible in the region. And design flood is the maximum discharge that a spillway can

handle without causing hazard to the structure.
(Ref: U.S. Dept. of Agriculture).

1,4 Spillways and Functions:

The primary functions of spillway is to release surplus water from reservoirs in order to prevent overtopping and possible failure of the dam. Spillways are often designed so that they may also be used as out - let works with relatively frequent operation. The spill- way structures should provide sufficient discharge capacity to pass the spillway design flood, as modified by available reservoir storage without exceeding the maximum permissible water surface elevation in the reser- voir and without endangering other structures of the dam. It may be advantageous to separate a spillway into sections to be used for specific purposes. For example, provision may be made for a section of spillway to be used as outlet works, and for passing floods of reasonably expected magnitude together with an auxiliary section of spillway which would come into use only for the passage of extremely large floods which would otherwise overtop the dam. Types of separate spillways are as shown in Fig. 1.2.

In the process of releasing excess water from the reservoir over the spillway weirs, the spillway and dam

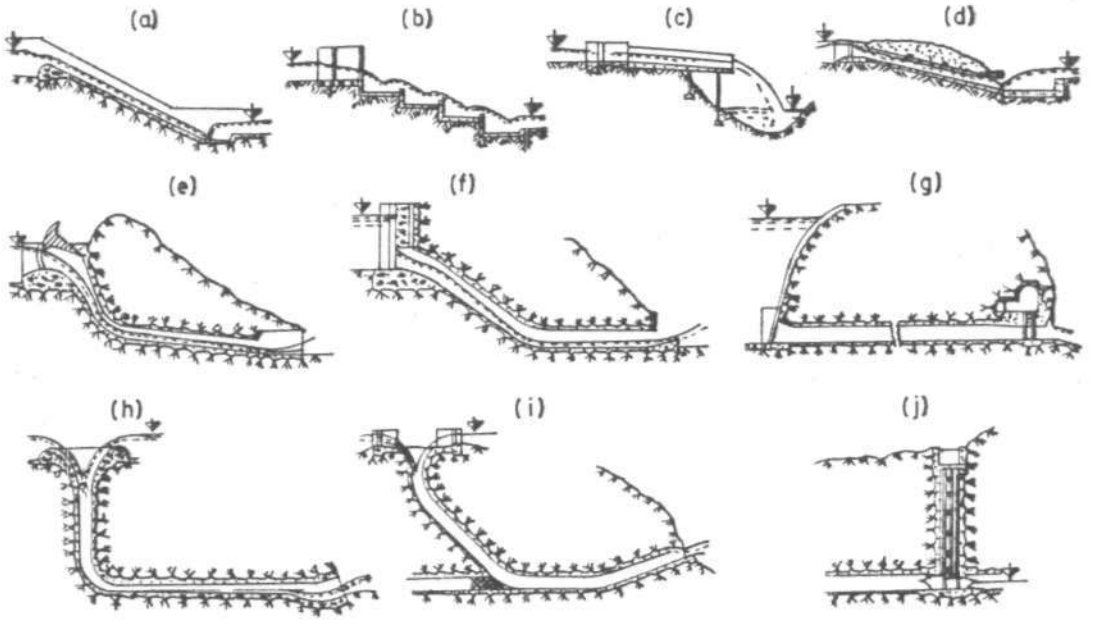


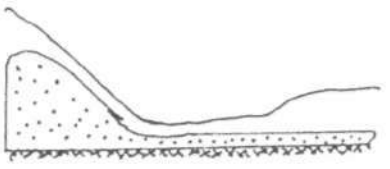
Fig.1.2 Types of Separate Spillway
Ref. (Grishin M.M.)

itself are endangered by the high velocity flows emanating from the weirs which will lead to scour and erosion at the bottom. To overcome this effect and to protect the structures from scour and erosion hazards, construction of various types of hydraulic jump stilling basins are always made as part of the whole project as can be seen in Fig. 1.3. The construction of these basins requires excavations to set the stilling basins at some elevations to form and accommodate the hydraulic jump to dissipate the energy of the high velocity water emanating through the weirs. In addition to the excava - tion, a lot of reinforced concrete has also to be used in the basin construction, especially within the energy dissipating length.

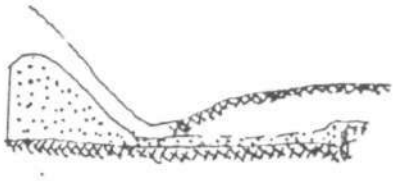
The design of suitable spillway structure is an important part of the development of reservoir projects and requires the initiative and resourcefulness of the designer to correlate the many factors involved in a satisfactory manner. In addition to hydraulic features, the determination of a desirable location and type of spillway necessitates consideration of geological and structural features and limitations. The character and strength of foundation materials frequently determine the general types of spillways that may be used and the

degree of protection against scour that should be provided in returning the discharge to its natural course.

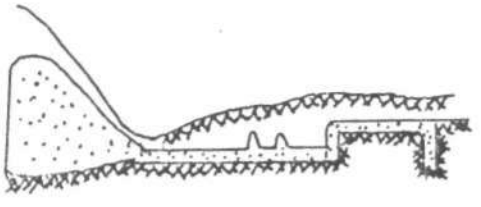
The scientific bases for the structural design of hydraulic structures is well established. In contrast, the nature of the flow transition from the spillway crest to the stepped face and the energy dissipation phenomenon makes it difficult to treat by regorous deductive reasoning, rather it is necessary to perform experiments and analyse the resulting data and from this analysis one can establish a relationship among the parameters and deduce some conclusion.



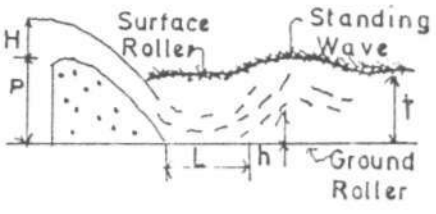
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WITHOUT BAFFLES AND END SILL



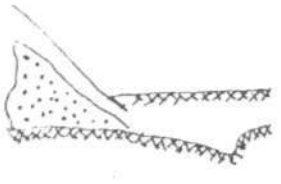
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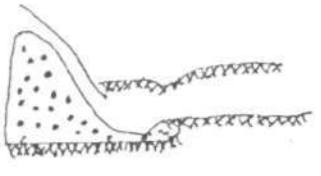
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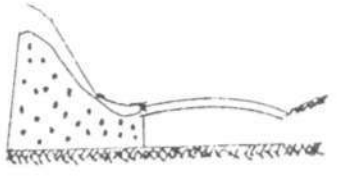
ROLLER BASIN



SLOPING



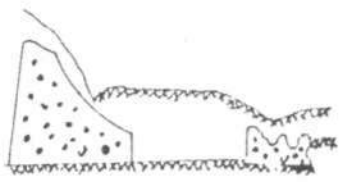
ROLLER BUCKET



FLIP BUCKET



DEFLECTOR BUCKET



HIGH DEFLECTOR BUCKET
WITH SUBDAM

Fig. 3.1 Types of Stilling Basins

Ref: "Hydraulic design of spaceways" Army Corps of Engineers. Cap. 3 – 1953.

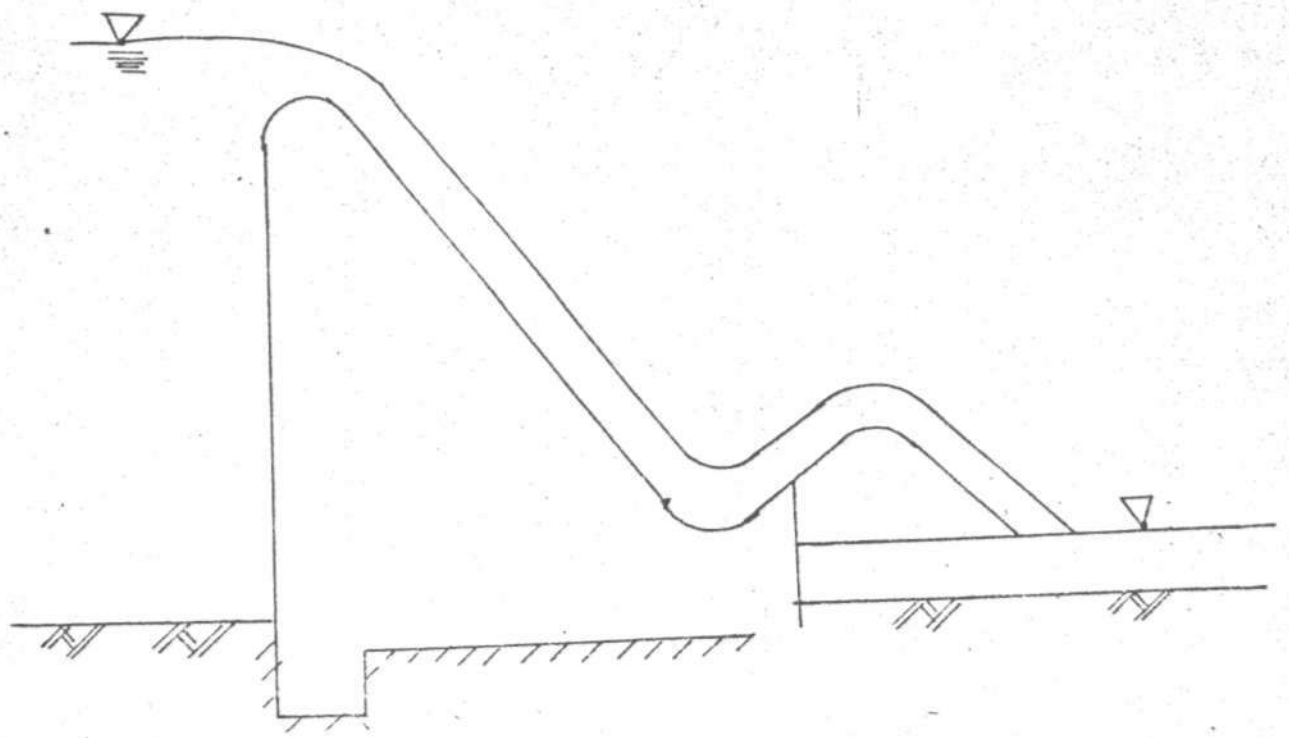
CHAPTER TWO
LITERATURE REVIEW

2. A problem which frequently occurs in hydraulic engineering practice is the need to spread and dissipate the energy of a concentrated, fast flowing stream of water, so that it does not cause scour below spillway weirs and dams results from the erosive power the water acquires in flowing over spillways. Protection against scour is afforded by ensuring that the high velocity flows do not come into contract with the channel bottom or by directing the flow as far away from the structure as possible. Usually, a combination of these methods is used. The subject of channel retrogression below dams is not a recent discovery, but was frequently discussed by irrigation engineers in India, during the early nineteenth century. One of such failure was reported by Islam Enquiry Committee on failure of the Islam weir, dated December 10, 1929. Gilbert, G.K.(1914) also found that the scouring capacity of a stream increases with an increase in discharge, with a decrease of bed material size and with an increase in velocity.

2.1 Energy Dissipation By Stilling Basins:

The present tendency is to dissipate the surplus energy of the flow in some form of energy dissi - pator. Towards this, a number of studies and approaches

were carried out in the past in designing an appropriate energy dissipator structure. One of the earliest structures was the stilling basin which was presented by the Bureau of Reclamation (1942) United States. The stilling basin was defined as a structure in which all the energy – reducing action is confined. In a stilling basin, kinetic energy is converted into turbulent energy and ultimately into heat. The stilling basin was first used on the Cochiti Diversion Dam, located on the Rio-Grande (North Maxico); Newcomer, A.W. (1942); it was a rectangular canal made of concrete for specific length and width which determined experimentally by model studies. Feldt, (1945) carried out a different approach at the State University, Iowa. He studied a roller type of the stilling basin which he claimed to be more economical to be used because of relatively short basins as compared to the horizontal apron mentioned earlier. Another study was also conducted by Andre, C. (1951), a French designer, when designing the Mareges Dam; this was the ski-jump type stilling basin. These types are purposely designed to spread the high velocity flow leaving the spillway, so that surrounding air can attack the largest possible area and move the erosive impact of the water away from the toe of the weir. The various



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Fig. 2.1 Ski Jump Bucket Dissipator
Ref. (Edward A.E.)

stilling basin types are shown earlier in Fig. 1.3, except ski-jump which is shown in Fig. 2.1.

The stilling basin method of energy dissipation is based on the hydraulic jump phenomenon. The hydraulic jump frequently occurs in open channel flow, when a liquid at high velocity discharges into a zone of lower velocity, a rather abrupt rise develops in the liquid surface and is accompanied by violent turbulence, eddying, air entrainment and surface undulations.

When this happens, a large amount of energy or head loss is experienced and thus the phenomenon can be used to serve as an effective dissipator of kinetic energy.

2.2 Energy Dissipation By Stepped Spillway Weir:

Another method of dissipation of energy of high velocity flowing water from and over the spillway weirs is to introduce steps on the down stream face of the weir such that the water passes from one step to another, on to several steps before reaching the weir toe. In this process, the velocity of the fact flowing water will be reduced as reported by many investigators earlier. The first attempt for this type of weir was made by Garsmann and Madden (1923). In their study, they intoduced a series of steps on the down stream face of an ogee weir of Gilboa Spillway which resulted in

dissipating the energy by contact with the successive steps and reaching the bottom with insufficient energy to erode the stream bed. Earlier, Beebe and Reigal (1917) also studied the effect of reducing steps on a sloping apron to the velocity of scouring water over it, they also concluded that the steps are capable of reducing its energy successively until it reaches the bottom of the apron. The latest work on a stepped spillway as an energy dissipator was the one carried out and reported by Robert (Dec. 1985).

Morris (1955) investigated flow past surfaces with large uniformly spaced surface roughness elements and defined three regimes: (1) isolated roughness flow, (2) Wake-interference flow and (3) Quasi-smooth flow and also assumed that the loss of energy in turbulent flow over a rough surface is due to the formation of wakes behind each rough element as the steps may be referred to here.

Isolated-Roughness Flow - This type of flow prevails when the roughness elements like the steps are completely developed and dissipated before the next element is reached. Under such a condition of flow transition at each element, a stable rolling vortex develops and rotates for abrief period and then returns

to the main flow to proceed on down to the next element. In such a flow, the ration Y/λ may be taken as a significan correlating parameter influencing the flow nature; where Y = rise and λ = spacing distance.

Wake Interference Flow – This kind of flow results when the elements (or steps) are placed such that the wake and vortex at each element will interfere with those developed at the following element resulting in intense and complex vorticity and turbulence unimportant but the spacing is of major importance and another important parameter here is the depth of flow over the steps or elements; this will control in part, the vertical extent of the surface region of abnormal turbulence. The ratio, Y/λ , will be a significant parameter.

Quasi–smooth Flow – This occurs when the steps are so close together that the flow essentially skim the crest of the elements and large roughness projec – tions are absent and the surface acts hydraulically smooth. Here also the ratio, Y/λ will be a significant parameter.

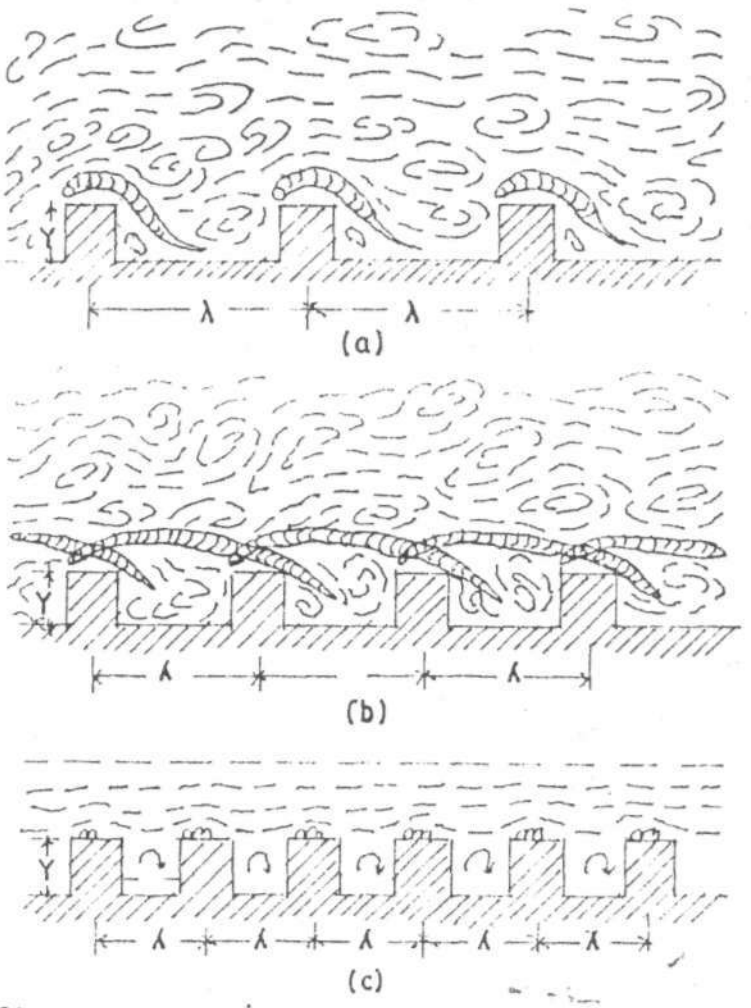


Fig. 2.2 Sketches showing coucept of three basic types of rough-surface flow: (a) isolated-roughness flow; (b) wake-interference flow; (c) quasi-smooth flow.

2.3 Hydraulic Models:

The use of hydraulic models has become standard practice in design procedures, particularly where large structures are involved or where problems are encountered that are not susceptible to an analytical solution. Hydraulic models afford a means of checking expected prototype performance and devising modifications when difficulties are encountered. They also aid in the development of better and more economical designs. Models for solution of problems encountered in spillway design can be made to different sizes and scales, depending upon the type of problem and facilities for investigation. They can be made to small scale, encom - passing large areas for study of a spillway, while large scale models may be required for reproducing sections of the structure under study for evaluation of other para - meters related with weir performance.

Data computed for discharge over spillway weirs and their models show that even with very small models the scale effect is small.

Experiments conducted by (U.S) Department of the Army Corps of Engineers (1953), on a weir of ogee cross-section, under different heads and on its 1:50 scale model, the weir coefficient for the model was about 2% greater than that of the full size weir.

Also, experiments by the U.S. Bureau for Reclamation (1955) on a series of models of ogee cross section to scales of 1:25 and 1:50, were tested under different heads from 150mm to 40mm shows that the model coefficient is about 1.3% greater than that of the original when the scale is 1:25 and 2.8% for 1:50 scale, so long as upstream conditions are accurately reproduced, the coefficient of the weir and prototype may be expected to agree within 2.5%.

The results of model test on round topped or ogee section weirs can be relied upon as giving results for which only a very small scale correction needs to be made, so long as the smaller head in the model is not less than 6mm (0.25inch) as reported by Gibson (1934). Also Karlsvebe (1929) earlier studied and made comparative measurements on models of weirs to different scales of brass and woods for weirs having semi-circular crests the scales varied between 1:100 and 1:5, the results showed that no deviations from the law of similarity could be traced.

CHAPTER THREE

THEORY OF ENERGY DISSIPATORS

3. Energy Dissipation:

A lot of work and experiments so far have been reported to have been done on the methods of dissipation of energy of high-velocity flows emanating from over the spillway weirs in the last two chapters.

Basically, the methods were reduced to two although different types of stilling basin can be seen in Fig. 1.3, because all these basins use the same phenomenon of hydraulic-jump formation to dissipate the energy of flow, while the second approaches is the introduction of continuous successive steps on the downstream face of the weir to avoid stilling basin construction.

3.1 Energy Disspating By Stilling Basins:

The object of spillway design is to provide a safe and adequate structure for the least combined cost of spillway and dam. Many types of spillways have been used throughout the world and usually the design of each has varied quite radically to meet the probelm at hand. The best type suited for protection against scour at a particular location depends largely upon the relationship between the existing tail water

depth, the depth required to form a hydraulic jump and the geological nature at site. One of the most important consideration in the design of a spillway section is the method of controlling erosion of the river bed at the downstream toe of the dam.

An enormous amount of energy must be dissipated before the flow reaches the natural stream bed. The structure which contains the energy dissipating action is commonly known as a stilling basin. It employs the hydraulic jump formation phenomenon in dissipating the energy. Other types of hydraulic energy dissipators used in conjunction with spillways are roller or trajectory buckets. On this spillway, the deflector bucket is used at the end of the chute to further reduce the energy by trajectory method as shown in Fig. 3.1.

The hydraulic jump is a useful means of dissipating excess energy in supercritical flow; it is therefore, normally confined within the stilling basin. The bottom of the basin is paved to resist scouring, consequently accessories to control the jump are usually installed in the basin end as the deflector bucket is being provided for the Biu dam spillway chute. The bucket serves two purposes, namely; it improves the

the disspation function of the basin and also stabilizes the jump action.

3.2 Hydraulic Jump Formation:

Hydraulic jump may be expected to form under any of the following conditions:

(i) At the base of a hydraulic structure where supercritical flow plunges into a pool.

(ii) In a long flume or channel where high velocities can no longer be sustained by a flat slope.

(iii) At a constriction in a flume where supercritical velocities are reduced to subcritical velocities.

(iv) At the junction of a canal of channel where supercritical flow passes through critical to the subcritical stage.

(v) Where the flow under a sluice gate is supercritical and impinges on a subcritical pool.

(vi) The jump may occur in pipelines or conduits where the grade changes cannot sustain supercritical flow.

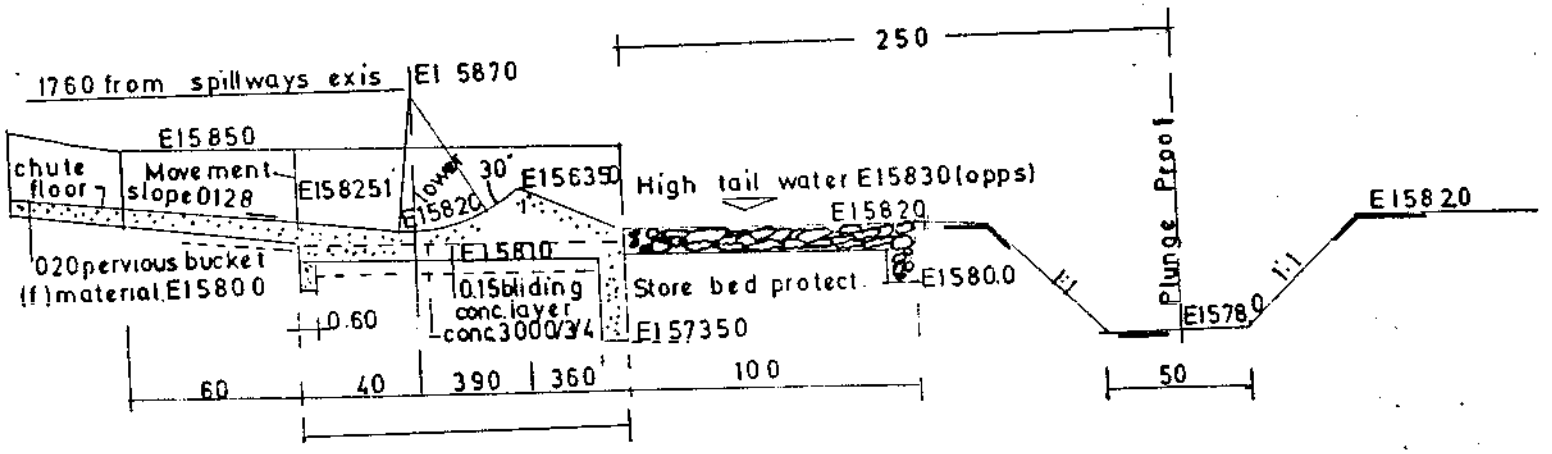


Figure 3.1(a): DETAILS OF DEFLECTOR BUCKET AND DOWN STREAM PROTECTION

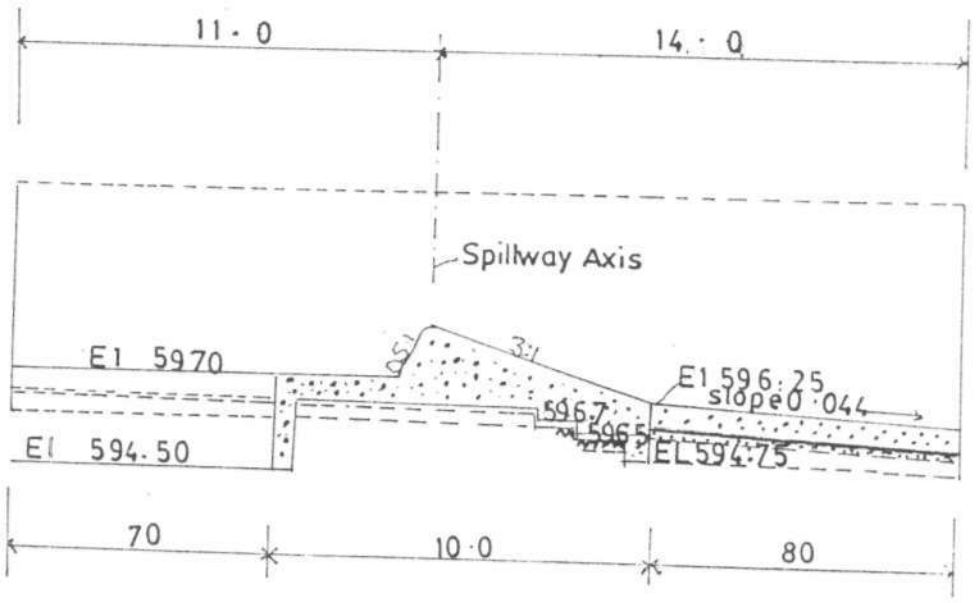


Figure 3.1(b): Spillway Section

(ii) In a long flume or channel where high velocities can no longer be sustained by a flat slope.

(iii) At a constriction in a flume where supercritical velocities are reduced to subcritical velocities.

(iv) At the junction of a canal or channel where supercritical flow passes through critical to the subcritical stage.

(v) Where the flow under a sluice gate is supercritical and impinges on a subcritical pool.

(vi) The jump may occur in pipelines or conduits where the grade changes cannot sustain supercritical flow.

The first two conditions apply to the type of structure under discussion in this work. The hydraulic jump was first investigated experimentally by Bidone, an Italian in 1818, from then many Scientists have studied the phenomenon and it was discovered that for a supercritical flow in a horizontal rectangular channel, the energy of flow is dissipated through frictional resistance along the channel resulting in a decrease in velocity and an increase in depth in the direction of flow.

The first two conditions apply to the type of structure under discussion in this work. The hydraulic jump was first investigated experimentally by Bidone, an Italian in 1818, from then many Scientiests have studies the phenomenon and it was discovered that for a supercritical flow in a horizontal rectangular channel, the energy of flow is dissipated through frictional resistance along the channel resulting in a decrease in velocity and an increase in depth in the direction of flow.

There are about five forms of the hydraulic jump which may occur on a horizontal apron, and which may be encountered in the design of energy-dissipation devices. It is important to note that the energy dissipation and internal characteristic of the hydraulic jump vary considerably with each form. According to the studies of U,S. Bureau of Reclamation (1953), these types can be conveniently classified according to the Froude number. The Froude number expresses the ratio between the gravitational and internal forces as $F_1 = V_1/gy_1$

F_1 of the incoming flow, Fig. 3.2, as follows:

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

2. For F_1 = 1 – 1.7, the water surface shows undulations and the jump is called an UNDULAR JUMP.

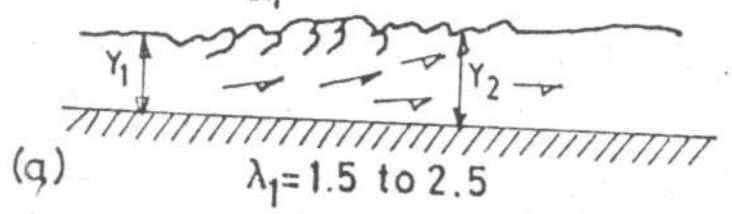
3. For F_1 = 1.7 – 2.5, a series of small rollers develops on the surface remains smooth. The velocity throughout is fairly uniform, and the energy loss is low. This jump may be called a WEAK JUMP.

4. For F_1 = 2.5 – 4.5, there is an oscillating jet entering the jump bottom to surface and back again with no periodicity. Each oscillation produces a large wave of irregular period which, very commonly in canals, can travel kilometers doing unlimited damage to each banks and repraps. This jump may be called an OSCILLATING JUMP.

5. For F_1 = 4.5 – 9.0, the down stream extremity of the surface roller and the point at which the high velocity jet tends to leave the flow occur at practically the same vertical section. The action and position of this jump are least sensitive to variation in tail water depth. The jump is well balanced and the performance is at its best. This jump may be called a STEADY JUMP.

$\lambda_1 = F_1^2$ is defined as the Kinetic flow factor,

while, $F_1 = \frac{V_1}{\sqrt{gY_1}}$ is as previously defined.



(a) $\lambda_1 = 1.5$ to 2.5 surface undulations



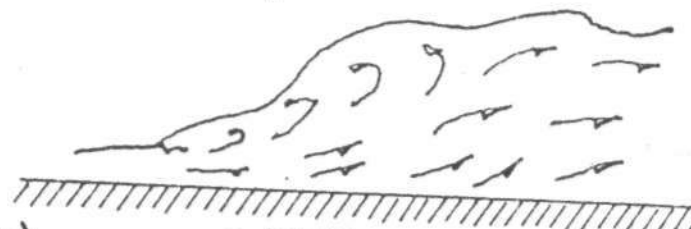
(b) $\lambda_1 = 2.5$ to 6.0 pre – jump



(c) $\lambda_1 = 6.0$ to 20.0 transition



(d) $\lambda_1 = 20$ to 100 Range of good jumps



(e) $\lambda_1 > 100$ choppy jump

Fig. 3.2 Forms of the Hydraulic Jumps
Ref: (Chow V.T.)

6. For F_1 = 9.0 and larger, the high-velocity jet grubs intermittent slugs of water rolling waves down stream, and a rough surface can prevail. The jump action is rough but effective since the energy dis - sipation may reach 85%. This jump may be called a STRONG JUMP.

3.3 The Jump Concept and Theoretical Investigations:

Momentum formular - In the past, various formulars have been employed to express the hydraulic jump concept but most of the formulas did not yield correct result because they were dimensionally incor - rect when experimented and is now universally accepted without doubt. The hydraulic jump may be concisely be stated as below:

Let abfe in Fig. 3.3 represent a mass of water moving through the hydraulic jump. In a short period of time, the mass of water will move to the position cdhg and the following characteristics will be observed.

(i) The water entering at ab is transparent and has a uniform high velocity.

(ii) The water leaving at gh is also transparent but in comparison to ab has relatively low velocity.

(iii) It will be observed that between c and e the surface rises rapidly and is greatly disturbed by spraying and spattering. The entire mass edef appears milky and this suggests the presence of much internal impact. In passing from the position cehg, the moving mass of water losses much of its momentum and consequently there is a reduction in velocity. The rate of momentum lost must be equal to the unbalanced force acting to retard the motion of the moving mass in accordance with Newton's Second Law of Motion.

The following assuptions are made in the analysis:

(i) The channel is rectangular in shape having parallet side constraints.

(ii) A horizontal floor exists.

(iii) All friction losses are neglected.

(iv) The hydraulic jump is assumed to take place instantenously.

(v) Streamline flow exists immediately before and after the jump.

In Fig. 3.3 below, let Y_1 and V_1 represent the depth and velocity upstream of the jump and P_1 the

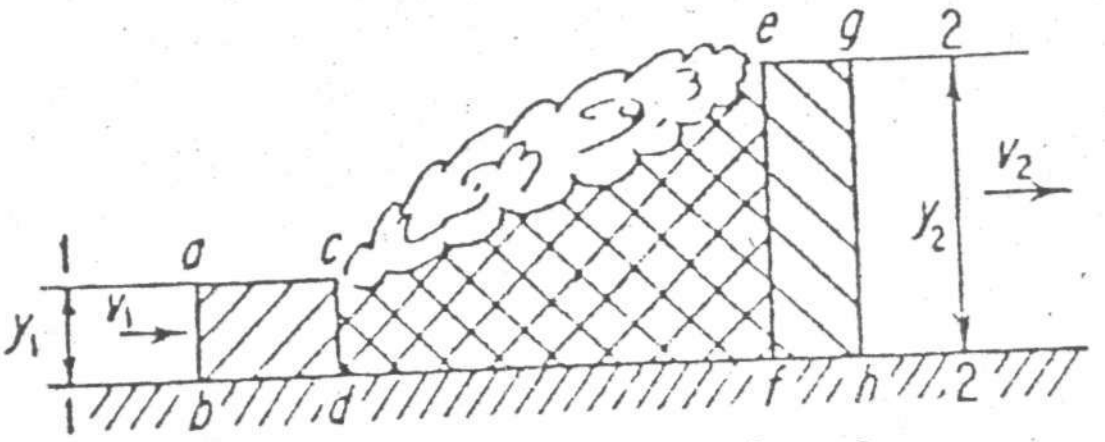


Fig.3.3 Theory of the Hydraulic Jump
Ref. (Chow. V.T.)

total hydrostatic pressure over the vertical section cd, and V_2 and Y_2 the velocity and depth after the jump and P_2 the total hydrostatic pressure after the the jump formation and energy loss. Take a channel width of unity for simplicity. The change in hydrostatic pressure P from Section 1 to 2 is computed from the total depth at each section.

$$\Delta P = P_2 - P_1 \quad \text{.....(3.1)}$$

and for rectangular channel of unit width

$$P_2 - P_1 = \tfrac{1}{2} \gamma (Y_2^2 - Y_1^2) \quad \text{.....(3.2)}$$

the change in momentum M will be computed from the velocity main-stream itself. Thus,

$$\Delta M = \frac{\gamma q}{g} (V_1 - V_2) \quad \text{.....(3.3)}$$

In accordance with Newton's Second Law of Motion, the change in hydrostatic pressure must be equal to the change in momentum.

$$\Delta P = \Delta M$$

$$\frac{\gamma q V_1}{g} + P_1 = \frac{\gamma q V_2}{g} + P_2 \quad \text{.....(3.4).}$$

substituting for $p_2 - p_1$ from equation (3.4) into equation (3.2) for rectangular channel, equation (3.2) becomes

$$Y_2^2 - Y_1^2 = 2\, q/g\, (V_1 - V_2)$$

but $V = q/Y$

$$\therefore\ Y_1 Y_2 (Y_1 + Y_2) = 2V^2\, Y^2/g \qquad \text{.....(3.5)}$$

Solving for Y_2 in term of Y_1 and V_1, equation (3.5) becomes

$$Y_2 = -\ \frac{Y_1}{2} + \sqrt{\frac{2V_1^2}{g}\, Y_1 + \frac{1}{4}\, Y_1^2} \qquad \text{.....(3.6)}$$

Kennison in 1916 simplified the momentum formula by equation (3.5) by Y_1^2 and re-arranged the equation to obtain

$$\left(\frac{Y_2}{Y_1}\right)^2 + \frac{Y_2}{Y_1} = \frac{2V^2 1}{gy_1} \qquad \text{.....(3.7)}$$

By plotting $V_1/\sqrt{gY_1}$ versus Y_2/Y_1, Kennison obtained a straight line relationship having the equation

$$Y_2 = V_1 \sqrt{Y_1/4} - 0.45\, Y_1$$

3.4 Basic Characteristic of the Jump:

Loss in Energy – The loss of energy in the jump is equal to the difference in specific energies before and after the jump which is usually represented in the following form:

$$\Delta E = E_1 - E_2 \qquad \text{.....(3.8a)}$$

$$H_L = H_1 - H_2 \qquad \text{.....(3.8b)}$$

Where H_L = Head loss through energy dissipation by the hydraulic jump and H_1 and H_2 are total head before and after jump. This can be proved when a relation - ship between the initial depth and the sequent depth of a hydraulic jump on a horizontal floor in a rectangular channel is derived and the difference between the specifc energy at the two sections considered.

The relationship between Y_1 and Y_2 is derived as follows: Figure 3.4.

The external forces of friction and the weight effect of water in the hydraulic jump on a horizontal floor are negligible. Because the jump takes place in a relatively short distance, the slope angle of a horizontal floor is zero.

$$\frac{Q^2}{gA_1} + \bar{Z}_1A_1 = \frac{Q^2}{gA_2} + \bar{Z}_2A_2 \quad \ldots\ldots(3.9)$$

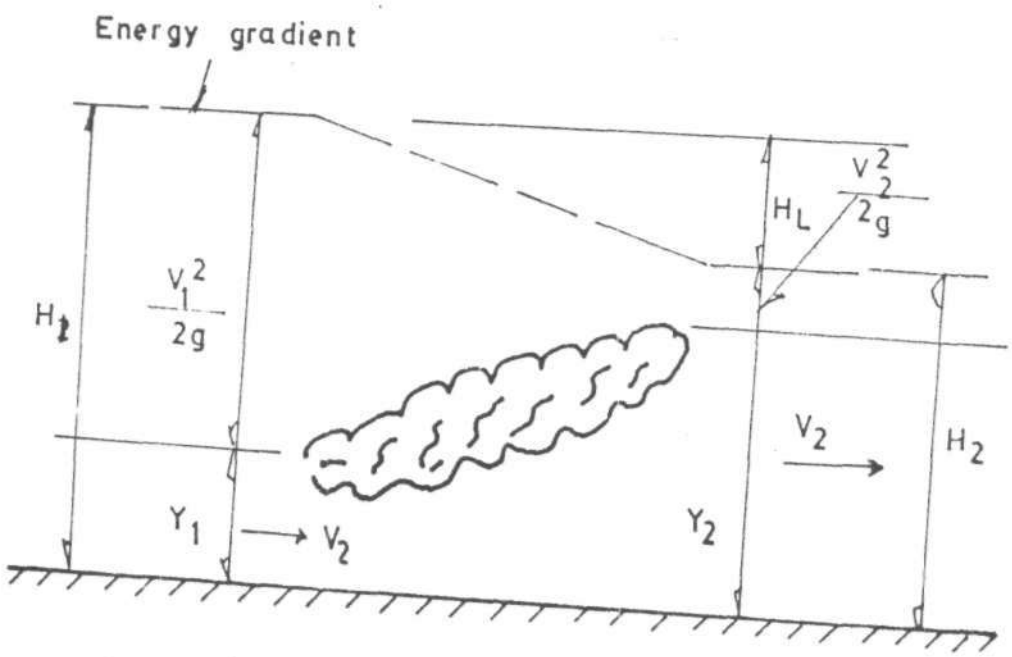
For a rectangular channel of width b, Q = discharge, $V_1A_1 = V_2A_2$, where A_2 and A_2 are cross-section areas at the respective sections; $A_1 = by_1$ and $A_2 = by_2$ and depth of centroids are $\bar{Z}_1 = Y_1/2$ and $\bar{Z}_2 = Y_2/2$

The specific forces at section 1 and 2 before and after the jump can therefore, be considered equal substituting these values and using

$F_1 = \frac{V_1}{gy_1}$ and $F_2 \frac{V_2}{gy_2}$ where

F = Froude number, the above equation becomes ..(3.9)

$$(\frac{Y_2}{Y_1})^3 - 2F_1^2 + 1)(\frac{Y_2}{Y_1}) + 2F_1^2 = 0 \quad \ldots\ldots\ldots(3.10)$$



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Fig. 3.4 Head Lost by the Hydraulic Jump
Ref. (Edward A.E.)

Factoring, we obtain

$$\left(\frac{Y_2}{Y_1}\right)^2 + \left(\frac{Y_2}{Y_1} - 2F_1^2\right)\left(\frac{Y_2}{Y_1} - 1\right) = 0 \quad \text{.....(3.11)}$$

Let

$$\left(\frac{Y_2}{Y_1}\right)^2 + \frac{Y_2}{Y_1} - 2F_1^2 - 0 \quad \text{.....(3.12)}$$

the solution of which gives

$$\frac{Y_2}{Y_1} = \tfrac{1}{2}\sqrt{1 + 8F_1^2} - 1 \quad \text{.....(3.13)}$$

Now the energy loss ΔE which is same as H_L while $H_1 = E_1$

and $H_2 = E_2$

$$E_1 = Y_1 + \frac{V_1^2}{2g}$$

and

$$E_2 = Y_2 + \frac{V_2^2}{2g}$$

Specific energy at respective sections

$$\therefore \Delta E = Y_1 + \frac{V_1^2}{2g} - Y_2 - \frac{V_2^2}{2g} \quad \text{.....(3. 14)}$$

from equation (24)

$$\frac{2V_1^2}{g} = \frac{Y_1 Y_2}{Y_1}\left(1 + \frac{Y_2}{Y_1}\right) \quad \text{.....(3.15)}$$

also

$$\frac{2V_2^2}{g} = \frac{Y_2 Y_1}{Y_2}\left(1 + \frac{Y_1}{Y_2}\right) \quad \text{.....(3. 16)}$$

Substituting these in equation (3. above,

$$Y_1 + \frac{Y_1 Y_2}{4Y_1}\left(1 + \frac{Y_2}{Y_1}\right) - \left(Y_2 + \frac{Y_2 Y_1}{4Y_2}\left(1 + \frac{Y_1}{Y_2}\right)\right) = E$$

$$\therefore \Delta E = Y_1 + \frac{Y_2}{4}\left(1 + \frac{Y_2}{Y_1}\right) - Y_2 - \frac{Y_1}{4}\left(1 + \frac{Y_1}{Y_2}\right) \text{.....(3 .17)}$$

Solving the above equation (3.

$$= \frac{4Y_1Y_1^2 + Y_1Y_2^2 =+ Y_2^3 - 4Y_1Y_1^2 - Y_2Y_1^2 - Y_1^3}{4Y_1Y_2}$$

But the numerator above is

$$(Y_2 - Y_1)^3$$

Since

$$\therefore \quad E = \frac{(Y_2 - Y_1)^3}{4Y_1Y_2} \quad \text{.....(3.18)}$$

Energy loss equation in terms of Y_1 and Y_2.

3.5 Jump Application in Stilling Basin Design:

The hydraulic jump is a sudden and turbulent passage of a flowing liquid from a low stage below critical depth to a high stage above critical depth during which the velocity changes from supercritical to subcritical. Like all expanding flows which produce an increased pressure downstream, the hydraulic jump is accompanied by large and violent eddies and therefore invariably involves a loss of kinetic energy. The hydraulic jump is, therefore, a very useful instrument of energy dissipation, its application is as illustra - ted in Fig.3.5.

$Y_2 = \frac{Y_1}{2}$ (- 1 + /1 + $8F_1^2$) as proved earlier, $L = K(Y_2 - Y_1)$

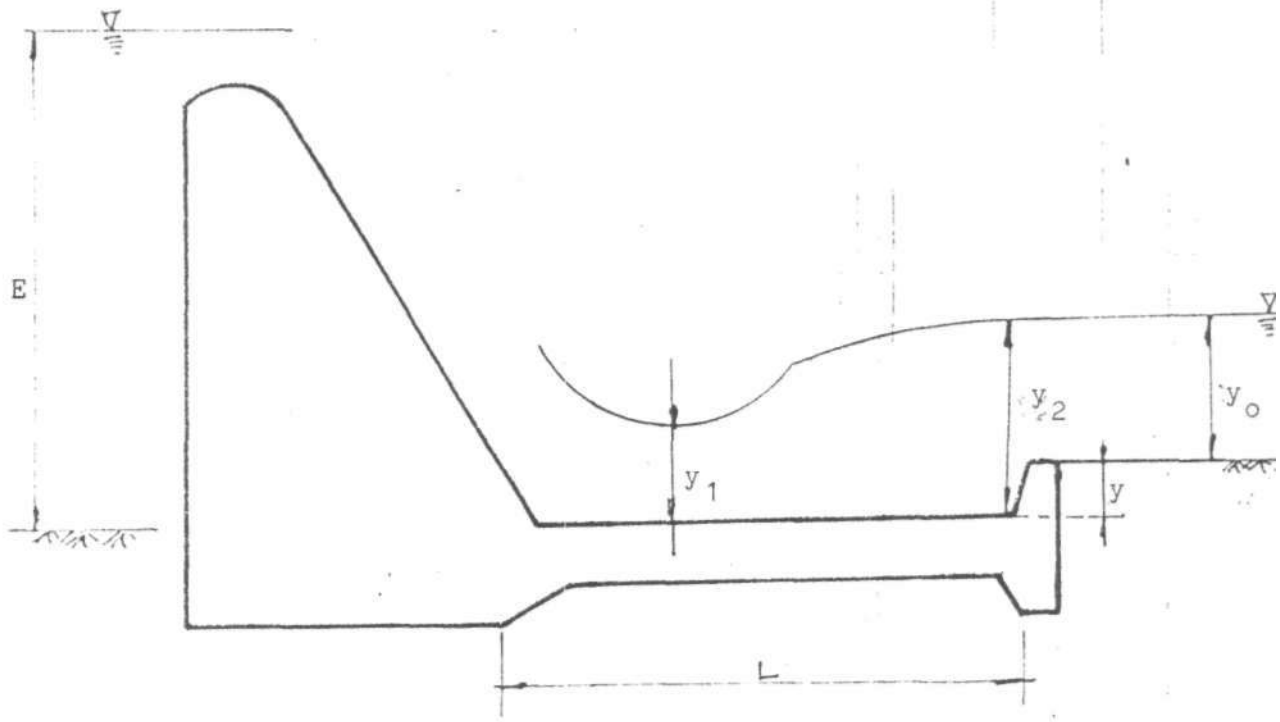


Fig. 3.5 Jump Formation Illustration
Ref. Design of Small Dams

Where K is a constant given by $K = Y_2/Y_1$, but this should be greater than three (3) for all conditions of flow to form hydraulic in all cases where there is potential scour hazard.

The location and character of the jump depend on tail water elevation as illustrated by curves A, B, C and D in Fig.3.6.

Tail Water Conditions – As stated earlier, the tail water elevation is of paramount importance in selecting the appropriate basin to contain the energy dissipating stream channel. From previous studies, four possibilities of the relationship between Y_2, Y_0 and the discharge Q were considered for selecting the energy dissipating portion as illustrated in Fig.3.6 as curves A, B, C, and D (Design of Small Dams 1977).

A. Tail Water depth Y_0 is less than conjugate Y_2 (but greater than critical depth). This is the most frequent case is design. The flow will continue below critical depth for some distance down stream from the toe of the dam. Because of energy loss due to friction the depth of flow will gradually increase until a new depth is reached which is conjugate lwith the tail water depth. At that point the hydraulic jump will occur. In this situation, the jump would be swept away from the apron.

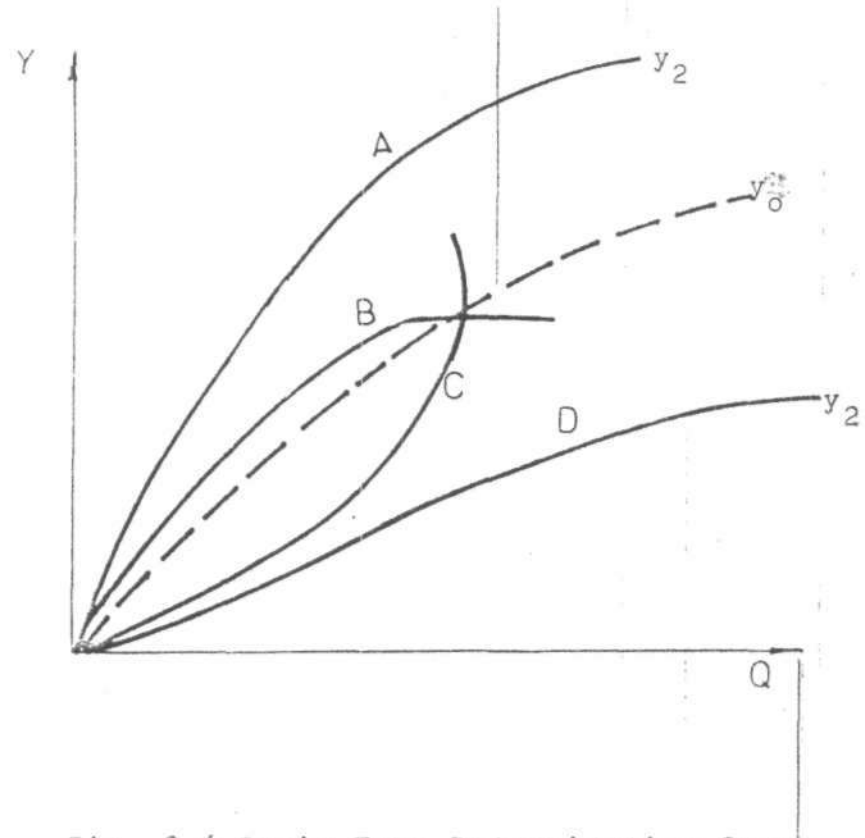


Fig. 3.6 Basin Type Determination Curve
Ref. (Designs of Small Dams)

In order to ensure the jump at the toe of the spillway, the following measure may be adopted:

1. Lower the level of the apron and form stilling basin so that the increased depth (tail water depth plus the depth of the basin) is greater than Y_2 at all states of flow. Refer to Fig.3.7.
2. If the spillway is low and the tail water almost sufficient to form a jump, loaffles or sills may be used to dissipate the energy.
3. Provide a large spillway crest, this will decrease the depth of flows over the spillway and in turn the conjugate depth, Y_2, also will be decreased.
4. If the stream bed is of solid rock, an upturned bucket may provide satisfactory protection (it will through the overflow up and out so that it strikes the stream at a safe distance below the dam as shown in Fig.2.1.

B. Tail water depth not sufficient to cause a jump at low flows and is too great during high flows. The following measures may be adopted:

1. A sill which will increase the tail water depth sufficiently at low flow rates may be

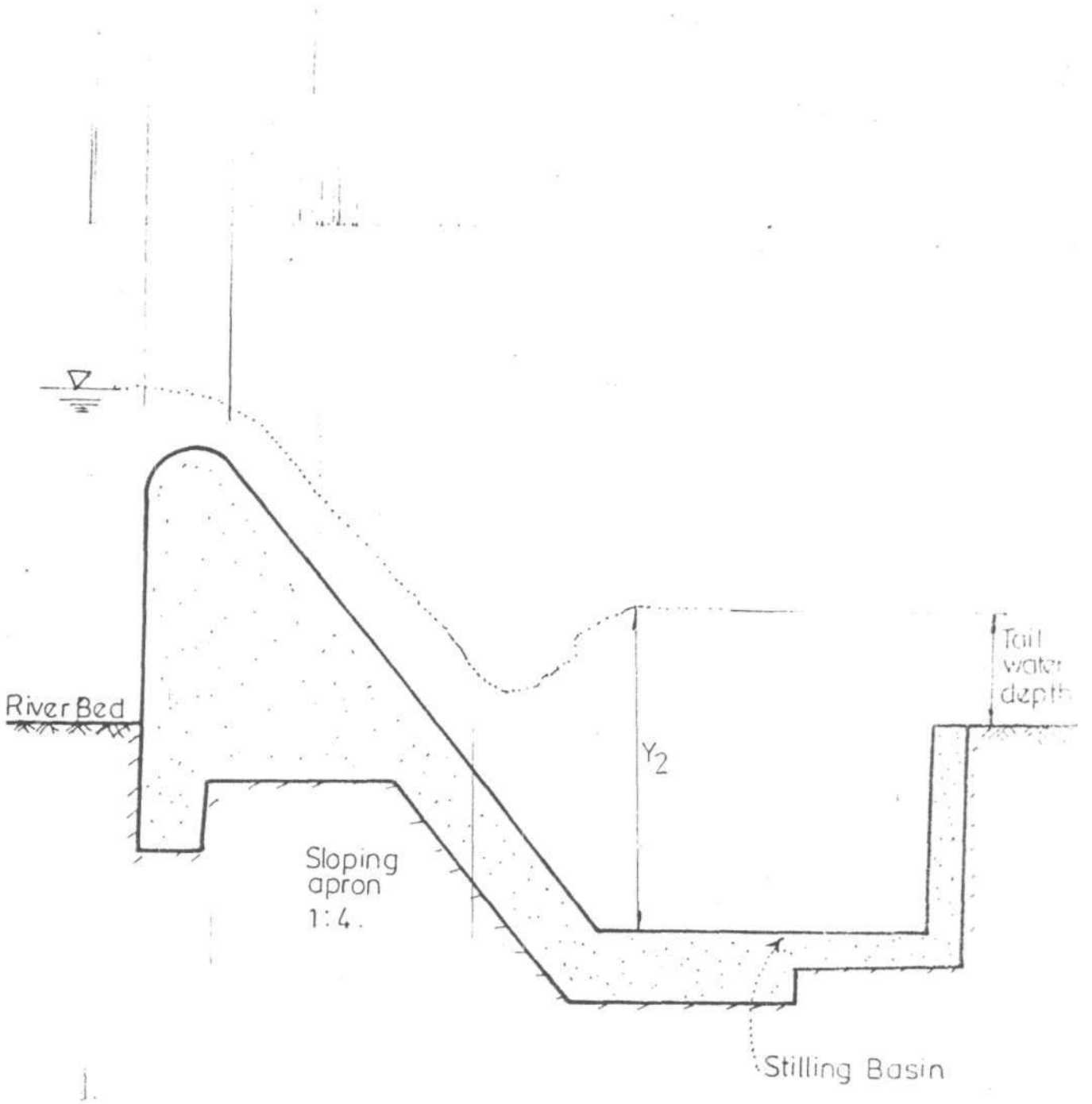


Fig. 3.7 Slopping Apron Stilling Basin
Ref (Design of Small Dams)

combined with a sloping apron to provide satisfactory jump at high discharge.

2. If the velocity is not too high, baffles piers may be used to break the flow.

C. Conjugate depth Y_2 is below tail water level Y_0 at low flows and above at high flows. A lowered apron (stilling pool) will hold the jump at high discharge.

D. Tail water depth greater than the conjugate depth Y_2. The jump will be drawned by the tail water and will occur upstream of the toe spillway. Energy dissipation will be unsatisfactory and water may continue at high velocity along the channel bottom for a considerable distance. This problem may be overcome by the following methods:

1. A sloping apron above the stream bed such that it would produce a satisfactory jump at all discharges as shown in Fig.3.7.
2. Use an apron with an unturned bucket which will deflect the overflow up through the tail water. A roller will form downstream of bucket but it will tend to move scoured bed material towards the dam, thus preventing serious scour at the toe of the dam.

3.6 Design of Models:

The design of a model must be related to that of the prototype that its behaviour under test will reproduce accurately prototype conditions. To satisfy this requirement, it is necessary that geome - trical similarity of boundaries be maintained and that the forces affecting fluid motion in model and proto - type bear a constant ratio to each other. That is flow particles in model cnd nature must move in geometrically similar patterns in proportional times. The requirement for geometrical similarity is satisfied when the model is constructed undistorted to the linear scale adopted.

Fulfilment of the requirements of dynamic similarity makes more rigorous demands. Forces capable of influencing fluid motion in any system are gravity, fluid viscosity, elasticity and surface tension. Certain combination of these forces as they affect the inertia of the fluid particles have been expressed in the form of dimensionless numbers known as Froude (gravity), Reynolds (viscosity), Weber (surface tension) and Carchy (elasticity). It is first necessary to determine which of the dynamic forces dominated the problem under study, then scale ratios are selected and

and the model is operated such that the effect of other forces becomes secondary. In particular, all spillway problems are gravity forces dominated and dynamic similitude is attained by designing the model in accordance with the Froudian relationship. Thus denoting model and prototype values by the subscripts M and P, respectively, and the ratio of model value to prototype value by the subscript.

3.7 Froude Number Criterion:

The flow behaviour over the weir is governed by gravity forces and as such the Froude number criterion is applicable. Here, the inertial and gravity forces are considered to be the only dominating forces in the fluid motion. From Newton's Second Law of Motion, the inertial force, F_i is

$$F_i = Ma \qquad \text{.....(3.19)}$$

Where M = mass of the element of fluid and has dimension (M), a = acceleration with which the element of fluid is moving and has a dimension of L/T^2, where L = length parameter and T = time parameter. The ratio of the inertial forces acting on homologous elements of fluid in the prototype and model can be defined thus:

$$\frac{F_p}{F_m} = \frac{M_p \; L_p/T^2 \; {}_p}{M_m \; L_m/T^2 \; {}_m}$$

Also, the gravity force, F_g, is given by

$$F_g = Mg \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.20)$$

Where M = mass of the fluid element and has dimension (M), g = acceleration due to gravity and has dimension L/T^2.

The ratio of gravity forces, which is determined by the weight of the mologous fluid elements involved is defined thus:

$$\frac{F_p}{F_m} = \frac{M_p g_p}{M_m g_m}$$

$$\lambda_{F_g} = \lambda_p \lambda_g \lambda_L^3$$

Since the inertial force and gravity forces are considered dominant in the fluid motion, the ratio of these forces acting on homologous particles in the model and its prototype is defined by Froude law as below:

$$\frac{(F_i)_p}{(F_g)_p} = \frac{(F_i)_m}{(F_g)_m} = 1$$

$$\therefore \frac{F_i}{F_g} = 1 = \frac{\lambda_\rho \cdot \lambda_L^4 \cdot \lambda_T^2}{\lambda_\rho \cdot \lambda_g \cdot \lambda_L \lambda_g \lambda_L} \frac{\lambda_{Ve}^2}{} = 1$$

which implies

$$\frac{V_m}{\sqrt{g_m L_m}} = \frac{V_p}{\sqrt{g_p L_p}} \quad \text{for } g_m = g_p \Rightarrow \frac{V_m}{V_p} = \sqrt{\frac{L_m}{L_p}}$$

$$\text{Or } V_r = L_r^{\frac{1}{2}}$$

discharge scale λ_Q, Q = Velocity x area.

$$\therefore \lambda_Q = \lambda_V \times \lambda_L^2 = \lambda_L^{\frac{1}{2}} \times \lambda_L^2 = \lambda_L^{5/2}$$

Equally the discharge per unit width q has a scale ratio

$$\lambda_q = \lambda_L^{3/2}$$

3.8 Planning and Construction of Models

The first step in designing a model is the selection of scale such that similarity of the occurance being studied is perserved in the model. Customary practice is to start with geometric similarity, once the geometric scale of the model to the prototype has been fixed according to available space and other necessary instruments to be utilized during test, the requirement for the dynamic similarity may be used to determine other model scales. The following ranges of scale have been successfully used in many model studies and are recommended for use as a point of departure in the determina - tion of scales in any given case. Spillways for dams may be studied on models with scale ratios in the range from 1:30 to 1:100, Bradly (1949), J.E. Warhork (1945) also used scales of 1:30 to 1:100 models when studying the progress reports on the sacramater San Jaoquin Dam, Colarado.

3.81 Model Scale Determination:

The following procedure was used in the determination of models for this study.

Models A and B

Specific energy head for prototype (E_p) = 25.0m

$E_p = h_p + H_p$ where h_p = prototype height and H_p design head over crest of spillway weir.

Flume depth d_f = 30 cm

Assumed Vertical Scale ratio V_r = 1:100

Computed Vertical Scale ratio = $\frac{dF}{E_p}$ = $\frac{30\ cm}{25 \times 100}$

Assumed vertical scale of 1:100 is satisfactory so that some allowance will also be left over the weir as recommended earlier by Karlsrebe (1929) that head over the model should not be less than 0.25 inch.

Horizontal Scale ration $H_r = V_r$ = 1:100

Maximum prototype discharge Q_p = 500 m /S

Maximum Pump discharge Q_{pump} = 10 l/S.

Required discharge of model in accordance with Froude law, which says $Q_r = L_r^{5/2}$ as expressed under section 3.7, the L_r which is the length ratio is equal to $\frac{1}{100}$, therefore, $Q_r = (\frac{1}{100})^{5/2}$ and since $Q_r = \frac{Q_m}{Q_p}$.

$$Q_m = Q_p L_r^{5/2}$$

$$= 500 (\frac{1}{100})^{5/2}$$

Q_m = 5 l/s which implies $Q_m < Q_{pump}$.

Model spillway hieght (h_m)

$$= h_p \times V_r$$

$$= 25 \times \frac{1}{100}$$

$$= 0.25m.$$

The assumed scale ratio of 1:100 for models A and B is satisfied by both the flume depth and a available pump capacity and the following sizes as computed above are proposed for the model construction.

Prototype hieght = 25m, model hieght = 25cm

Prototype dischare ≠ 500 $m^{3/S}$ modle discharge = $5^{l/s}$

pump maximum discharge = 10 $^{l/s}$ $5^{l/s}$

Flume height (depth) = 30 cm .25 cm.

3.8.2. Step Sizing:

The step sizes to be introduced on the models for the test were taken according to a formular presented by Chteren Liht (1982). According to this formular, the appropriate step length (L_s) should be equal to the sum of the falling length (l_f) plus the distance between the minimum water depth and the maximum water depth which he called the backwater length (l_w) and the distance (l_r) all expressed on Fig. 3.8. The distance (l_r) is related to the height he used in his formular, according to Chteren Liht, $l_r = h_c\ (2 - 2.5)$ which means l_r is in the range between 2 h_c to 2.5 h_c. Therefore $L_s = l_f + l_w + l_r$.

Procedure For Computation of Ls

1. The procedure for the determination of an appropriate step length is to assume a step size of say 4 cm on the model which is 400 cm on the prototype.

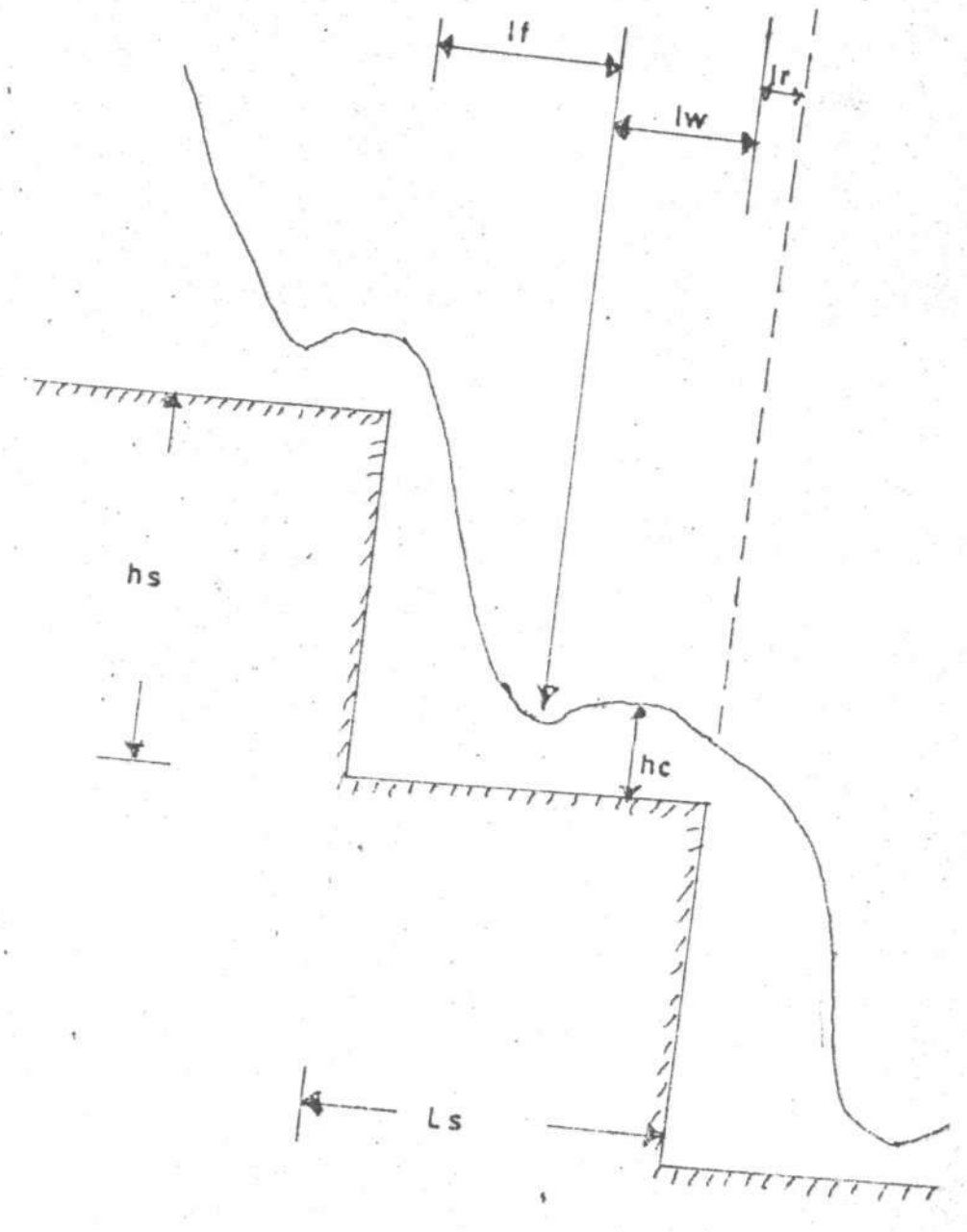


Fig. 3.8
Illustration of flow over steps

2. Assume a discharge say of 1 l/s; 3 l/s and 5 l/s so as to cover the range of discharges to be used during test.

3. Start the pump and adjust on the 1 l/s position and observe the nature of flow over the steps and try to make note of the parameters, l_f, l_w and h_c, as shown on figure.

4. Take the approximate values of the l_f, l_w and l_r and repeat the experiment for the discharges of 3 l/s and 5 l/s respectively until a step size is reached such that $Ls = l_f + l_w + l_r$, precaustion is to be taken such that the following conditions are satisfied:

(i) The most suitable step size should not be greater than the sum of l_f, l_w and l_r otherwise a hydraulic jump may be developed over the step.

(ii) And also if the step size is less than the sum of l_f, l_w and l_r, high turbulence flow will occur on the whole step which may result in the development of very high kinetic energy (Chterin Liht 1982).

CHAPTER FOUR

FEATURES OF BIU DAM AND SPILLWAY

4.1 Dam Location:

The town of Biu (10^{o} 37' N, 12^{o} 12' E), is a local government headquarters and is situated at some 200 Kms south of Maiduguri, the capital of Borno State, as shown on the location map, Fig. 4.1.

The dam site is located within the Duvana river gorge between the small villages of Tsahuyun and Burakum. A recently constructed dry weather road, about 5 km long leads to the dam site from Tum village situated on Gombe Road, 10 km from Biu. For water supply to Biu town and its surroundings, the dam site was initially selected by the Borno State Water Board and consultants were commissioned for the detailed studies.

While selecting the type of dam, onsideration was given to site topography, geology and foundation conditions, availability of construction materials and economy. Possibility of providing a concrete gravity dam was studied, but presence of compressible clay deposit to a significant depth did not allow this type of structure at the site. From further studies, it was concluded that an earthfill embankment with modified homogenous section is best suited type of dam for the site. The crest level of the dam is fixed at El.602.00

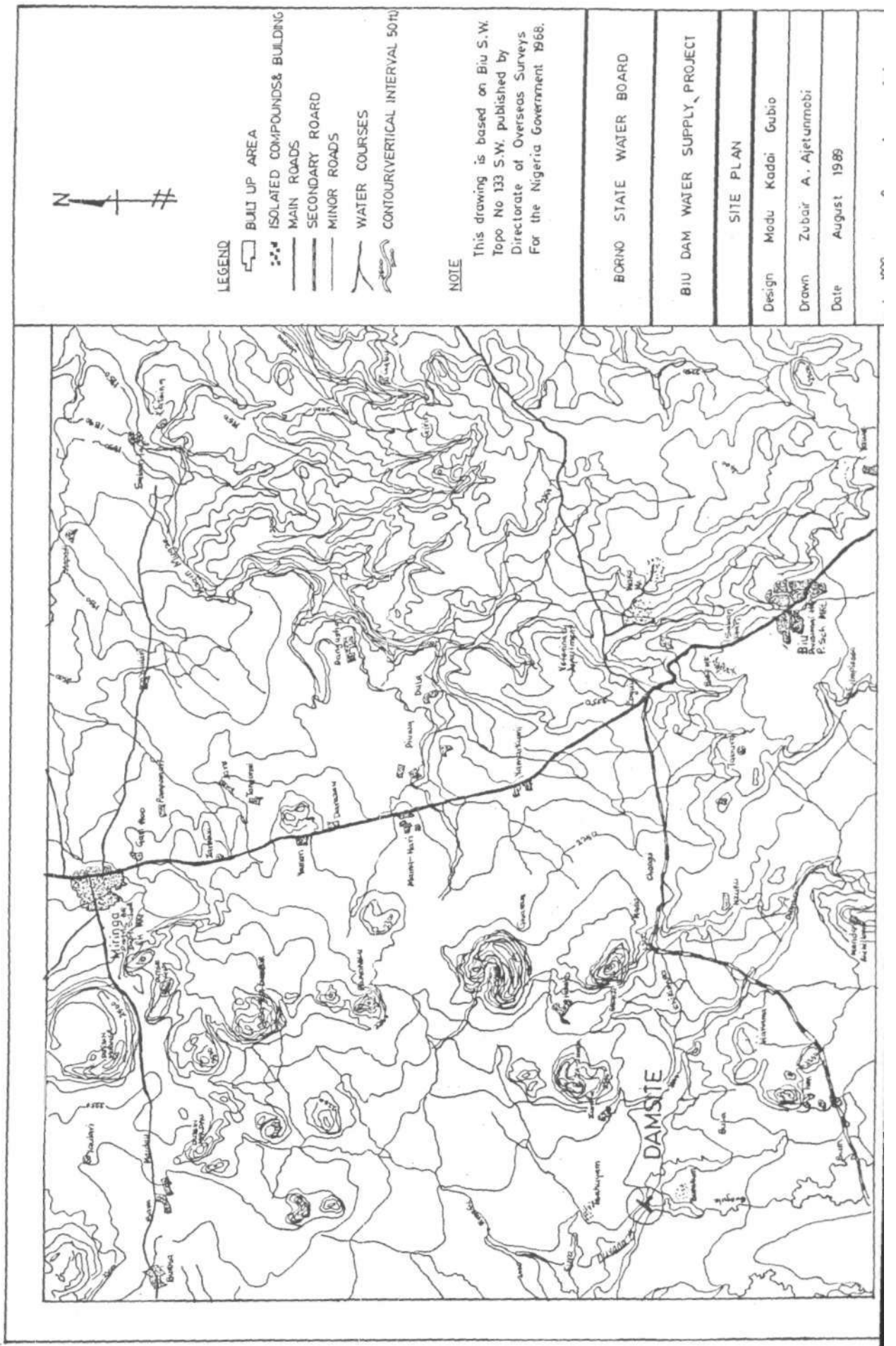
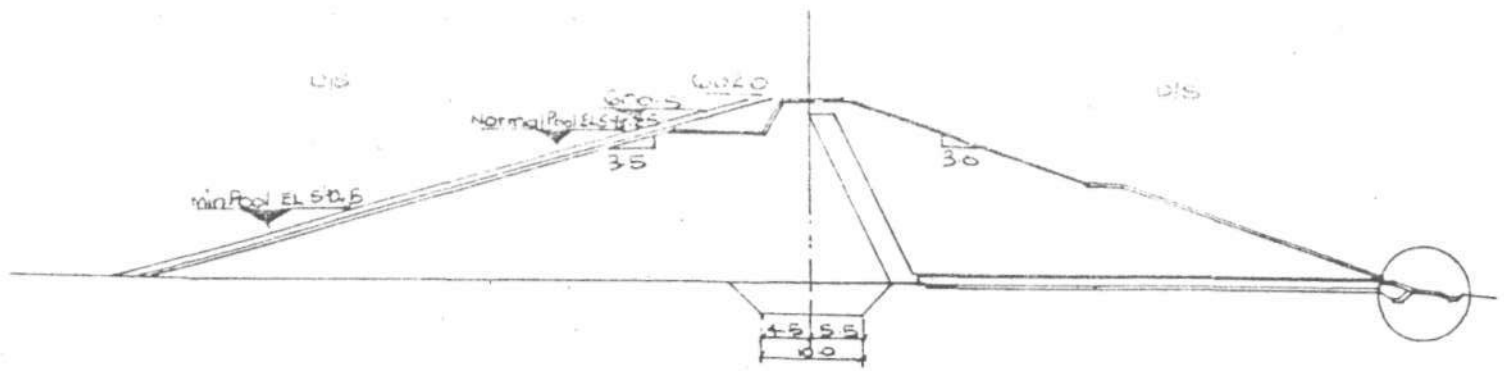

N
LEGEND
BUILT UP AREA
ISOLATED COMPOUNDS& BUILDING
MAIN ROADS
SECONDARY ROARD
MINOR ROADS
WATER COURSES
CONTOUR(VERTICAL INTERVAL 50ft)
NOTE
This drawing is based on Biu S.W. Topo No 133 S.W. published by Directorate of Overseas Surveys For the Nigeria Government 1968.
BORNO STATE WATER BOARD
BIU DAM WATER SUPPLY PROJECT
SITE PLAN
Design Modu Kadai Gubio
Drawn Zubair A. Ajetunmobi
Date August 1989
DAMSITE

Fig. 4.1

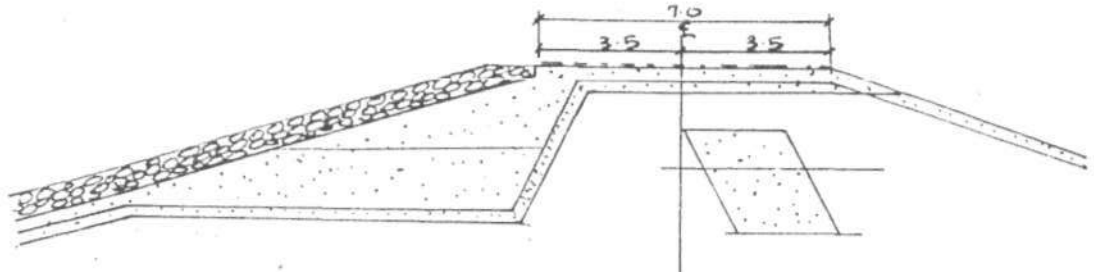
and it will create a reservoir having a total capacity of 11.1 million cube metres with a normal pool elevation of 600.50, and a live storage of 9.6 million cube metres with minimum pool elevation 590.5. This storage in conjunction with the normal run-off of the river, will be able to supply a perennial outflow of 0.26 w.m/sec. or about 5 mgd of water. The dam ;consists of an earth embankment made of compacted homogenous fill as silty clay and clay which will provide impervious barrier to water. The embankment has alv:3.5H upstream slope and a lv:3H downstream slope as shown in Fig.4.2. As part of the dam, a wet type intake is also located near the toe of the upstream side to extract raw-water from the reservoir to treatment plant. The intake features are as shown in Fig. 4.3. Other componets of the dam are: spillway ogee type weir, chute plus deflector bucket and an approach channel.

4.2 Hydrology of Biu Area:

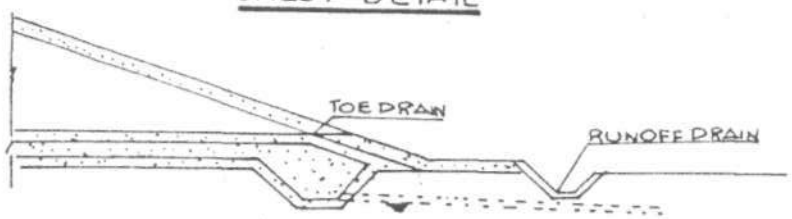
The area surrounding Biu has an undulating topography sloping mainly towards north and north - west. A number of small streams drain this area and pass near and through the town. Duvana river is one of such streams and it is proposed to utilized this



TYPICAL EMBANKMENT SECTION



CREST DETAIL



TOE DETAIL

Figure 4.2: BIU DAM CROSS SECTION

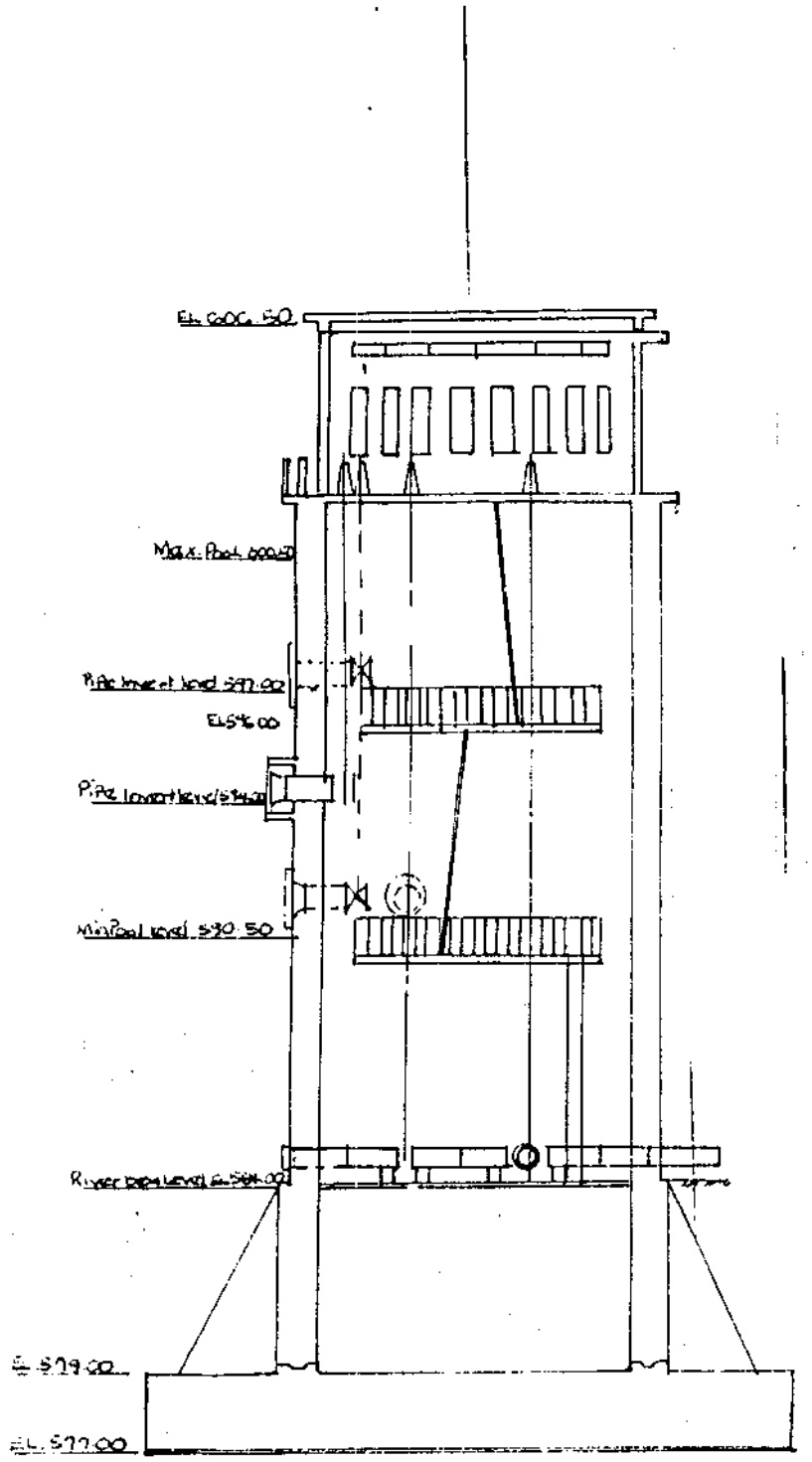


FIG. 4.3: SECTION OF WATER INTAKE WELL

river as a source of water supply to Biu through construction of a dam and storage reservoir at Tum. The catchment area of Duvana river is a non perennial river and flows only during the rainy season. The storage reservoir will hold the river run off during the rainy season to provide a continuous supply to Biu, Miringa and the villages enroute.

4.2.1 Climatology

The climate of the area is typically "Sahelian" with a summer rainy season from April to November (Maximum in August) and a completely dry winter season. The mean temperature is around 28°C with around 35°C and minimum 22°C.

4.2.2 Annual Rainfall:

Rainfall records for Biu which lies in the catchement area of Duvana river are available for a period of 37 years. Records of rainfall for Garkida and Askira are also availabe for 14 and 10 years respectively. These records have been collected and and analyzed to amire at various hydrological para - meters required for estimating the yield structures. A summary of rainfall at Biu and nearby stations is given in Table 4.1.

The average annual rainfall at Biu is around 950 mm which does not seem to be critically low as far as amount is concerned. The distribution of rainfall is, however, not uniform and at times the interval between two consecutives rains may be as long as seven months. As the Duvana river gets all of its run off as a result of rainfall, it remains dry most of the year and as such cannot be relied on for domestic water supply without the construction of a storage reservoir.

4.2.3 Monthly Rainfall:

The rainy season in Biu commences in April. Occassionally, there are light showers in March which hardly produce any run off. Most of the annual rainfall occurs from May to September, and morethan 75 percent of total annual rainfall is concentrated in this period; maximum rainfall occurs in August, whereas in October, the rainfall starts decreasing. The period from November to February is virtually dry.

4.2.4 Daily Rainfall:

The average number of rainy days in a year at Biu is about 62, mostly in months from May to September, while the maximum day rainfall usually occurs in July, August and September. The maximum

day rainfall in the period of record (1952 – 1988) is 96 mm on 4th September, 1974. Refer to Tab.4.1.

4.2.5 Rainfall and Run Off:

Because of lack of stream flow records at Duvana river, the rainfall data available have been used to estimate the average annual run off and the spillway design flood. Tab.4.1.gives the daily rainfall and estimated run off for the 1952 – 1988 period. The average annual run off at the dam site was estimated from the rainfall data to be about 20.214 million cubic metres.

4.2.6 Reservoir Capacity: Evaporation and Sedimentation:

The annual evaporation at lake was estimated to be about 1,078 mm by using the regional pan evapo – ration data supplied in the form of Map No. 16 "Annual Pattern of Evaporation and Precipitation".

For the sediment concentration, an intent method was emploed to estimate the sediment yield of the basin. Reseachers have related the mean annual rainfall to the annual sediment yield; one such rela – tion was that of Linsely and Kohler. Refer Hydrology For Engineers (3rd Edition). This was used in estima – ting sediment yield which comes to about 14685 tons/ year, for a catchment of 146.8 km^2; taking ;the specific weight of sediments as 1200kg/m^2, it comes to about 0.0122MCM/year. And for a normal ponded reservoir,

sediment trap efficiency is about 97.5% (Linsely – Kohler) for a capacity in flow ratio of 0.58, therefore, total sediments trapped in the reservoir comes to about 0.0119 MCM/year.

Based on the evaporation rate, sedimentation yield of the basin on a 60 years of operation, the dead storage of the reservoir was arrived at about 1.5MCM at storage level of 590.5m and maximum storage level at 598.25m with capacity of 11.1MCM and live capacity of 9.6MCM; also considered was the seepage losses at 0.883MCM.

4.2.7 Design Flood:

The Biu dam comes under intermediate dams, and for intermediate dams with significant hazard potential, the recommended spillway design flood is ½PMF – 1PMF (Ref.3); and is derived from probable maximum precipitation and run off are not available, a return period of 1,000 years was adopted for the design flood. The maximum day rainfall at Biu for 1952 – 1988 was derived from the data (Tab.4.1) with further frequency analysis lof maximum day rainfall using Gumbel distribution and the line was fitted by method of least squares with a coefficient of correlation of 0.00 (Fig.4.4). From this figure, a relationship is obtained

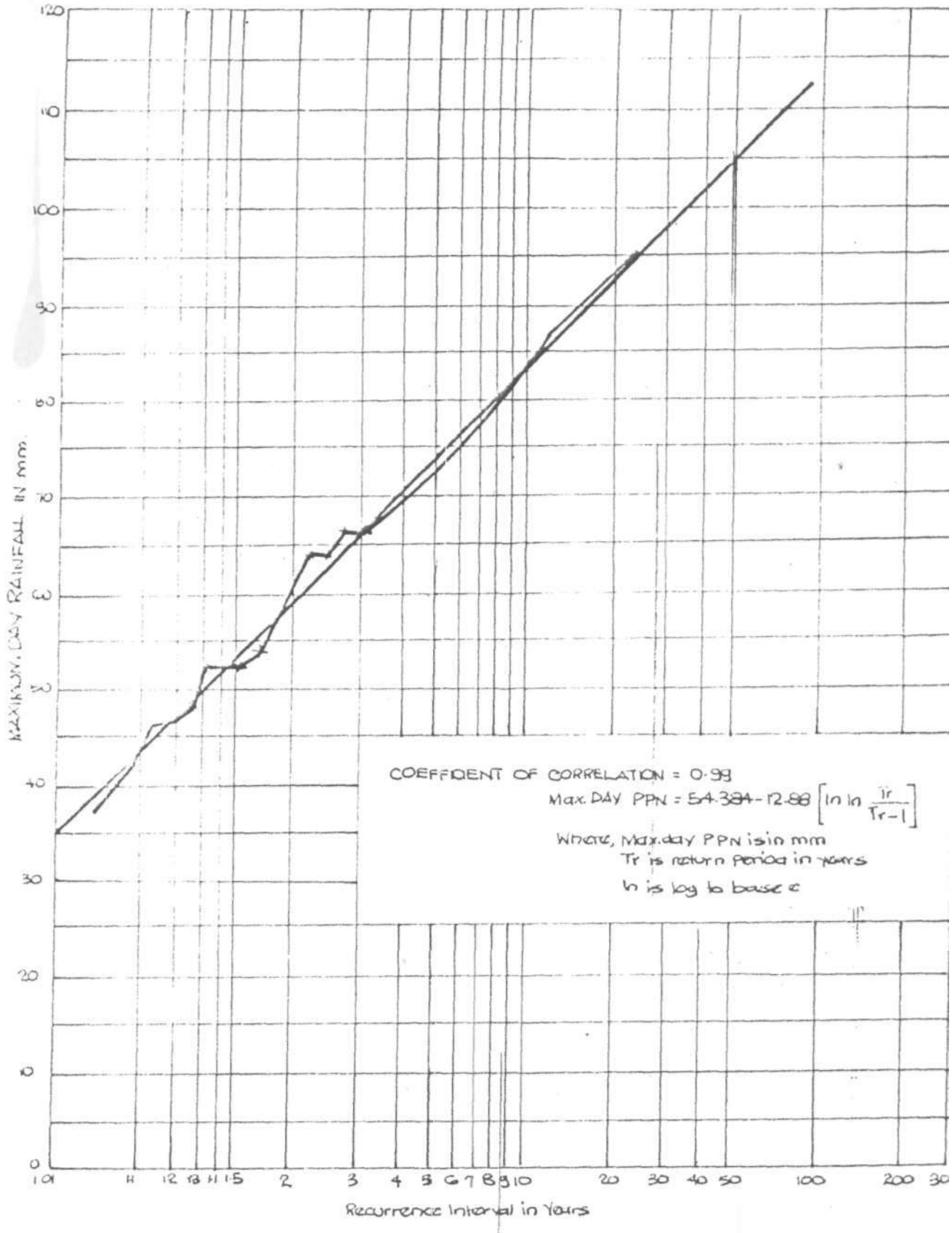


Figure 4.4: FREQUENCY ANALYSIS OF MAX. DAY PRECIPITATION

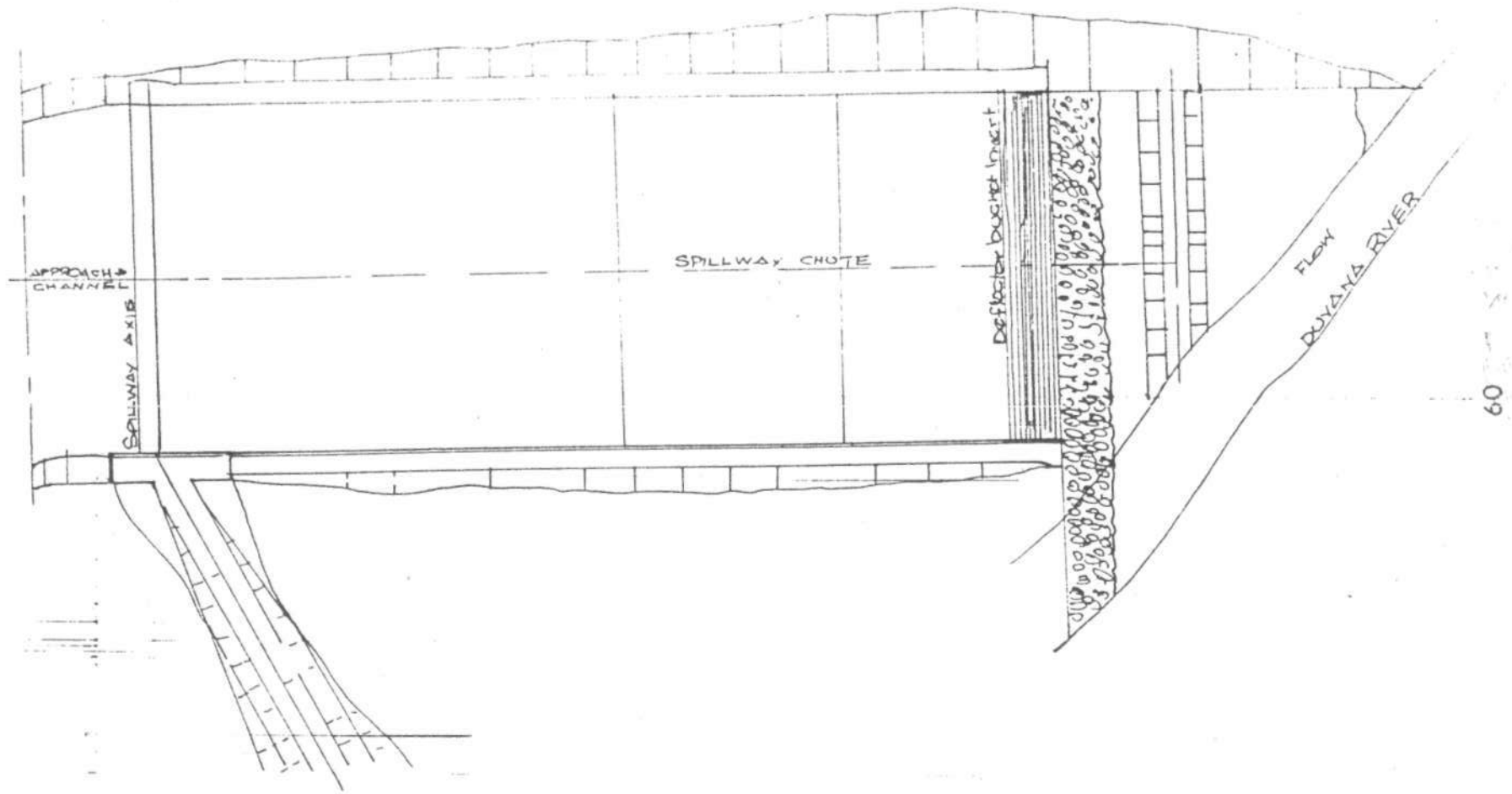


Figure 4.5: SPILLWAY AND CHUTE PLAN

for the maximum day precipitation and return period. Maximum day precipation = 54.384 − 12.88 (in $\frac{Tr}{Tn-1}$) Ref. Biu Dam Design Report by (ECAN Consultants). The maximum day precipitation corresponding to different return periods are calculated. As the design return periods of 1,000 years has been selected, the maximum day rainfall of 143 mm was adopted for design. The Sub-hydrographs resulting from hourly precipitation and the composition hydrograph are shown in Fig.4.6. The design peak in flow thus obtained is estimated at 660 cumecs.

4.3 The Biu Spillway Weir and Its Components:

In order to design the spillway of Biu dam, the design in flow hydrograph derived in the preceding section and shown in Fig.4.6. was routed through the reservoir using different spillway flow characteristics. According to the topographic conditions of the reservoir, the maximum permissible water level is 600.5. A spillway at 598.25mm crest level and 72mm crest was considered most desirable with maximum peak discharge passing through the spillways of 496 cumecs. The outflow hydrograph is shown inFFig.4.6 along side the inflow hydrograph. The dissipating components, the spillway chute measuring about 166m long and 72m wide, and the

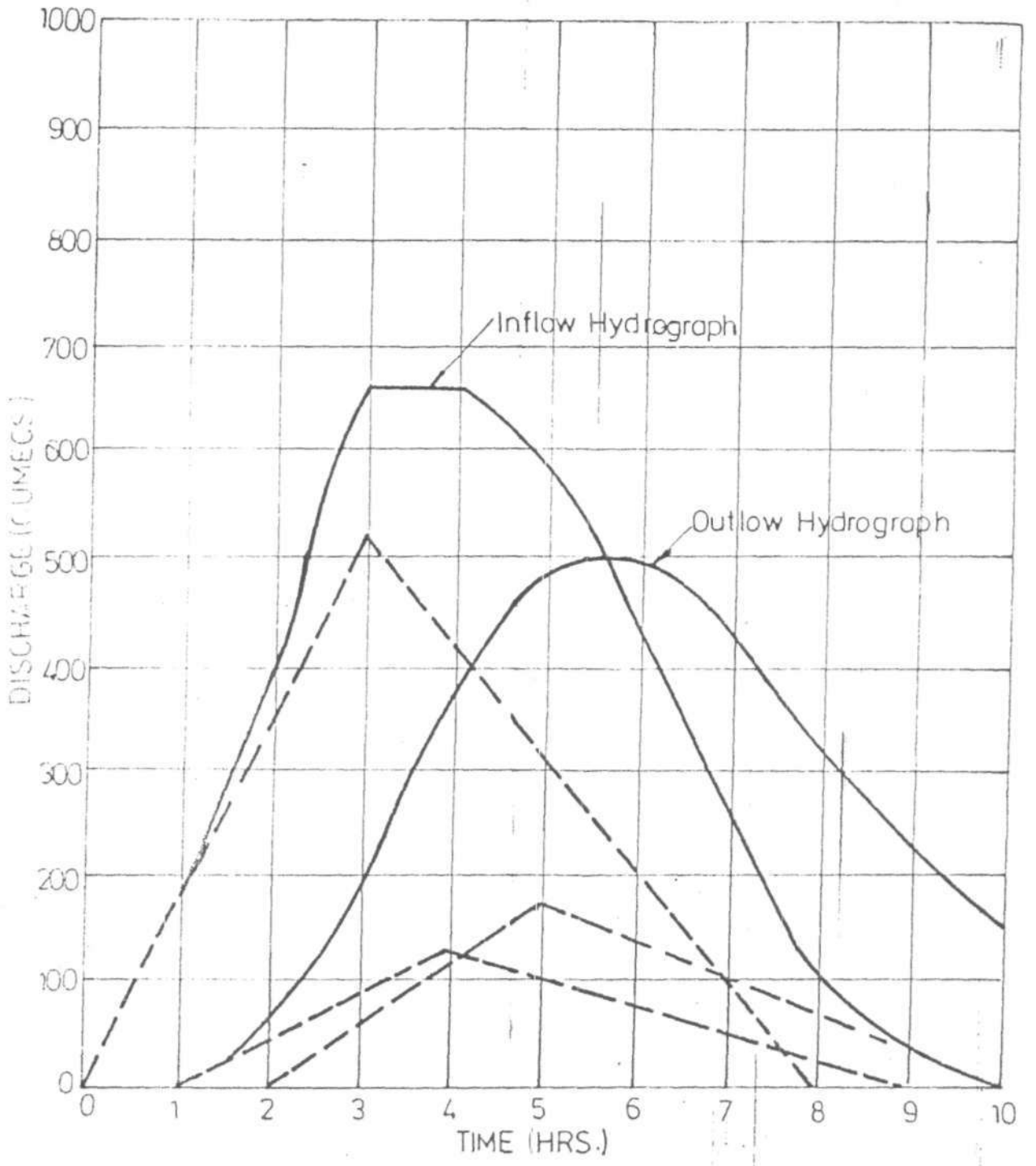


Figure 4.6: SYNTHESIS OF DESIGN INFLOW HYDROGRAPH AND ROUTED OUTFLOW HYDROGRAPH

deflector bucket, are all placed at the left flank of the earth dam, the details of which are shown in Fig. 4.5 and Fig.3.1 (a) and 3.1 (b).

CHAPTER FIVE

EXPERIMENTAL SET-UP

5.1 Experimental Facilities:

The experiment was carried out in a laboratory flume with a working length of 8m and a section measuring 20cm width by 30cm depth. The flume has a glass side which makes it possible to take readings when water is passing within the flume over some object, say weir. The flume is provided with water being pumped from a surface tank through a delivery pipe; to the delivery pipe is attached a U-tube manometer containing manometric liquid for measurement of discharge, in addition to a valve just before the U-tube manometer. As the valve is opened for water to pass through the delivery pipe in the flume, the pressure difference in the two limbs of the U-tube manometer also adjust themselves such that the pressure difference gives the value of the amount of water passing in litres per second. The computation of this can be made by the use of an empirical formula given by the manufacturer of the flume set up, Meesrs. Plint and Partners of England (1978). The formula is as below:

$$Q = \frac{1}{100} \; C \; \pi \; \frac{d^2}{4} \sqrt{\frac{2g(Sm - 1)\,H}{1 - m^2}}$$

Where:

C = Coefficient of discharge

d = Nozzle diameter (cm)

m = Area ratio d^2/D^2

D = Delivery pipe diameter (cm)

H = Pressure head fdifference across the nozzle(cm)

g = Acceleration due to gravity

sm = Specific gravity of menometric liquid.

The values for d = 7, D = 12.7 cm, m = 0.3492 and c = 0963, as given by manufacturer and mercury was used as the manometric liquid throughout the experiment. It has specific gravity value of 13.55.

Water which is pumped to the flume through the delivery pipe is discharged into another tank placed adjacent to the surface for re-circulation.

Other facilities with the flume include a stop-watch, a pilot-tube for measurement of velocity and a point-gauge for depth measurement.

5.2 Experimental Models:

The models used in this test were constructed using the procedure described under section 3.8 of Chapter Three. Three experimental models were constructed using perspex materials and 6mm thick plywood for the purpose of this investigation. Two of the models are 1:100 scale ration

which are measuring 25cm height and 20cm crest length. The models are tagged A and B. Model A is the normal ogee weir while Model B has some steps introduced on its dowstream face. The readings obtained using these two models are to be compared. The third model is of 1:50 scale ratio whose values are 50cm height but the crest length is same as the previous ones which is also the maximum width of the flume. This model was not completely used but only some portion of it which of interest to this study was utilized. In the experiment, only upper portion of this enlarged size of the previous models was considered so as to observe in a more enlarged structure, the nature of flow transition from the unstepped portion of the stepped weir to the steps.

In order to avoid abrupt changes from flow direction and production of centrifugal pressures, and also to create a smooth transition of the flow, to prevent the impact of the falling water. The surface at the spillway toe was designed as a curved bukcet using the emperical formular.

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

Where R is the radius of the curve in feet;

V = velocity in foot per second of the flow at the

toe and H is the head over weir in feet, Chow (1959) PP 884, and used in the construction of the unstepped model A as may be referred as on Fig.

The steps introduction on downstream face of model B was achieved through trial and error method using the procedure earlier described under section 3.8 of Chapter Three. However, the experimental trial steps and discharges plus position of readings are as presented through tables 5a, 5b and 5c.

Respective step sizes were tried before an appropriate size was arrived at.

5.3 Measurement and Computation:

5.3.1 Step Sizing Experiments:

For the selection of an appropriate step size that will satisfy the conditions earlier stated in the literature and the theoretical criterion stated in Chapter three, the following experimental procedure was adopted.

1. Three model discharges of 1 l/s, 3 l/s and 5 l/s are considered.
2. A proposed model with step sizes of 4cm was first adopted and the discharges of 1 l/s, 3 l/s and 5 l/s were passed over

it and the stated parameters of

L_f; L_w and L_r were observed and noted.

This step size was not satisfactory as per the results obtained in tables 5a, 5b, and 5c. The step size was increased to 5cm and the experiment was repeated as above, again the results obtained for discharge of 5 l/s was not satisfactory. Finally, the step size was increased to 5.5cm and experimented on discharges 3 l/s and 5 l/s and it was found to be satisfactory. The step of 5.5cm implying a prototype step size of 550cm was adopted for the model construction and it was used in the experiment for the determination of toe velocities of the stepped downstream face model. The three models are as shown on Figures 5.1,5.2 and 5.3 with location of examined steps.

5.3.2 Measurement:

After an appropriate step size of 5.5cm has been selected for the stepped model type, the model type was placed in the experimental flume at some 5.5 distance from the flume water in-let point so that the pumpted water is stabilized before reaching the model. For measurement of various parameters when the pump is started, the valve on

the delivery pipe for discharge measurement is being observed carefully so that a specific discharge value of interest is set for particular set of readings to be taken. For subsequent discharge values, the valve will be further opened to increase the discharge until the required discharge limit is reached. For each discharge value, a set of readings comprising of the toe velocity at the model weir was taken with the use of a pilot tube, head of water over the model weir crest using a poing gauge was also taken. For all the weir models, a set of ten discharge readings with corresponding values of head over the weir crest and toe velocities were recorded as presented in tables 5d, 5e and 5f and model shapes are also shown in figures 5.1,5.2 and 5.3.

5.4 Precautions

(i) Weir models were placed at least at a distance of 5.5m from the water inlet into the flume through the delivery pipe so that the approach flow conditions could be reflected.

(ii) To reudce errors due to turbulence, readings were not taken with each flow immediately valve is adjusted

until after the water level in the flume and the two limbs of the U-tube manometer stabilizes.

(iii) The flume was kept at a constant horizontal level throughout the duration of the experiment.

(iv) The following denominations were used here in the data collection, computation and result analysis of this experiment:

R = Is used to denote Run which inplies a certain position of delivery valve setting.

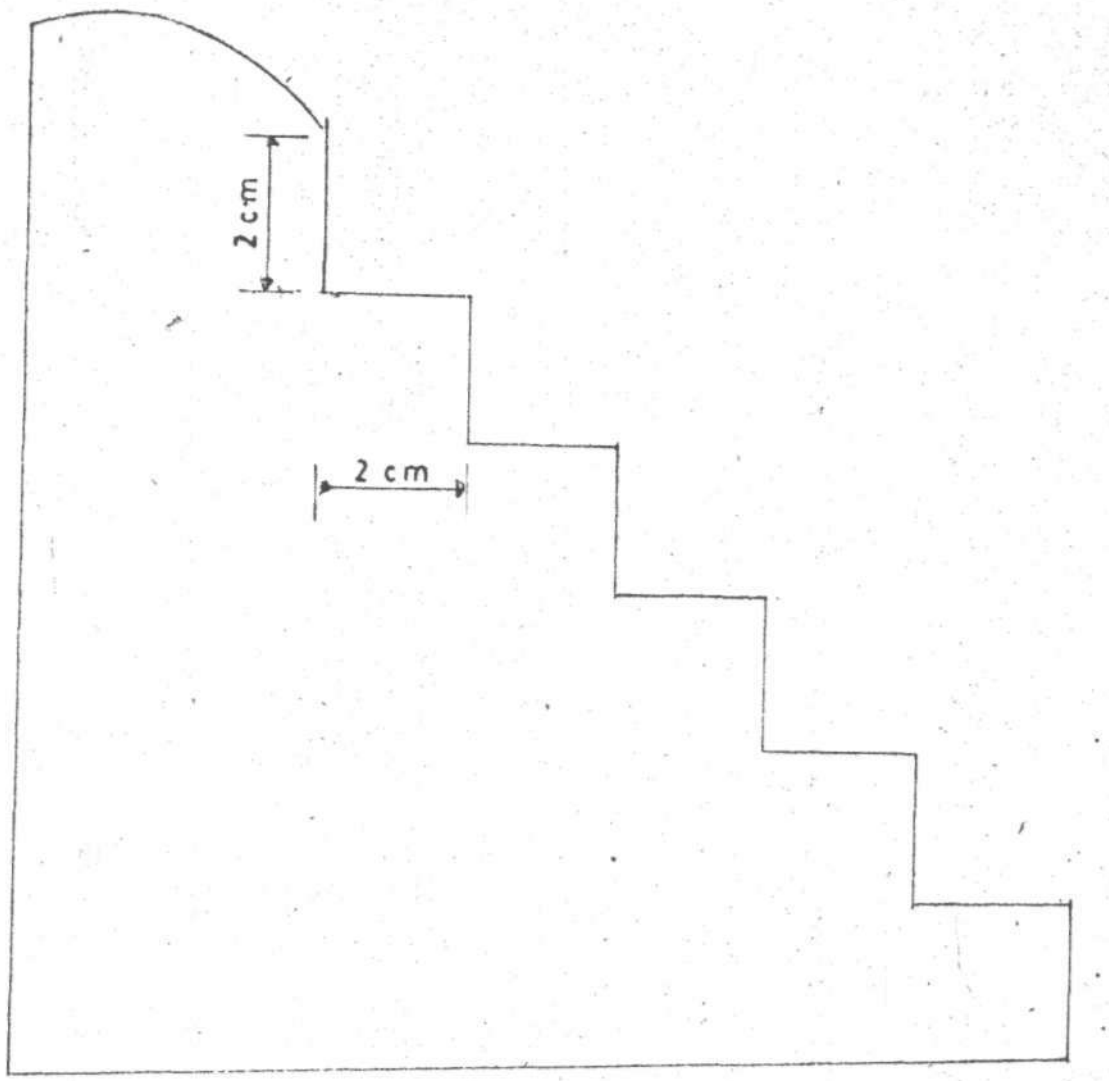
h = Head difference in water level between the two limbs of the U-tube manometer for the measurement of discharge.

H = Head of water over weir.

Q = Computed discharge value from the experiment data using the formular in litres per second.

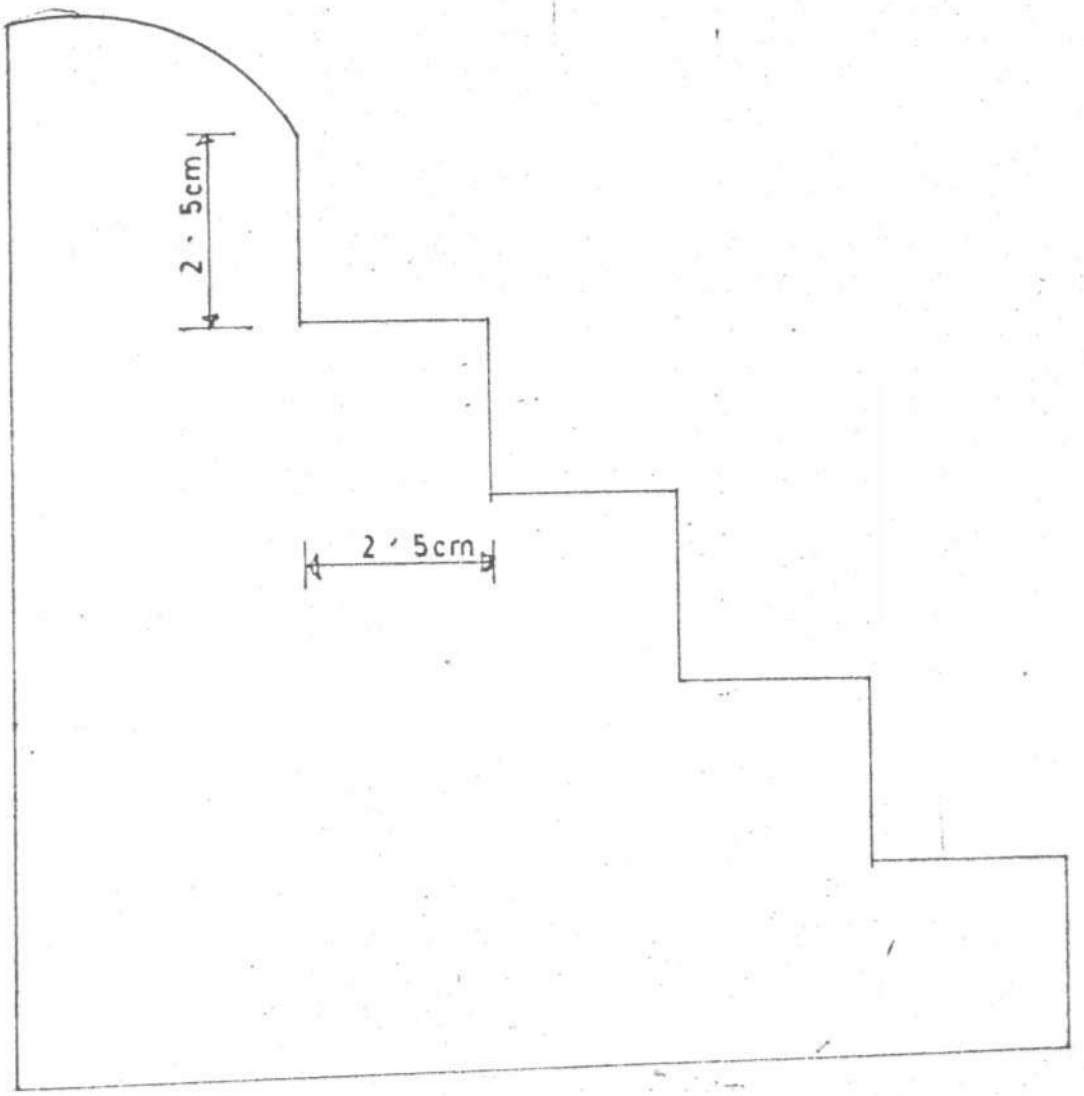
h_i = Level of water in Pilot-tube used in velocity of flow computation.

V = Computed velocity value in centimetre per second.



Model B

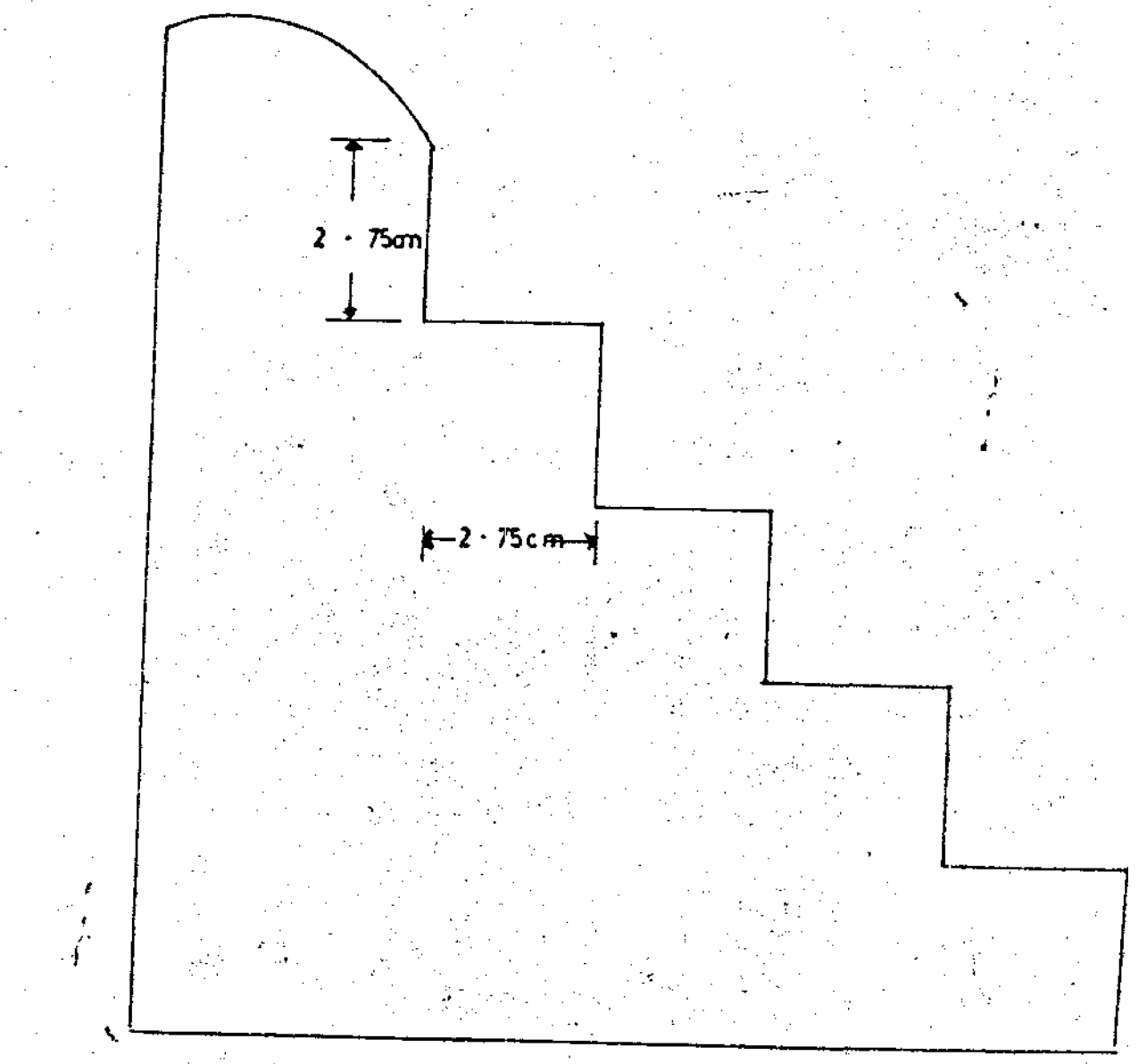
Scale 1:2 Fig. 5.1



Scale 1 : 2

Model B

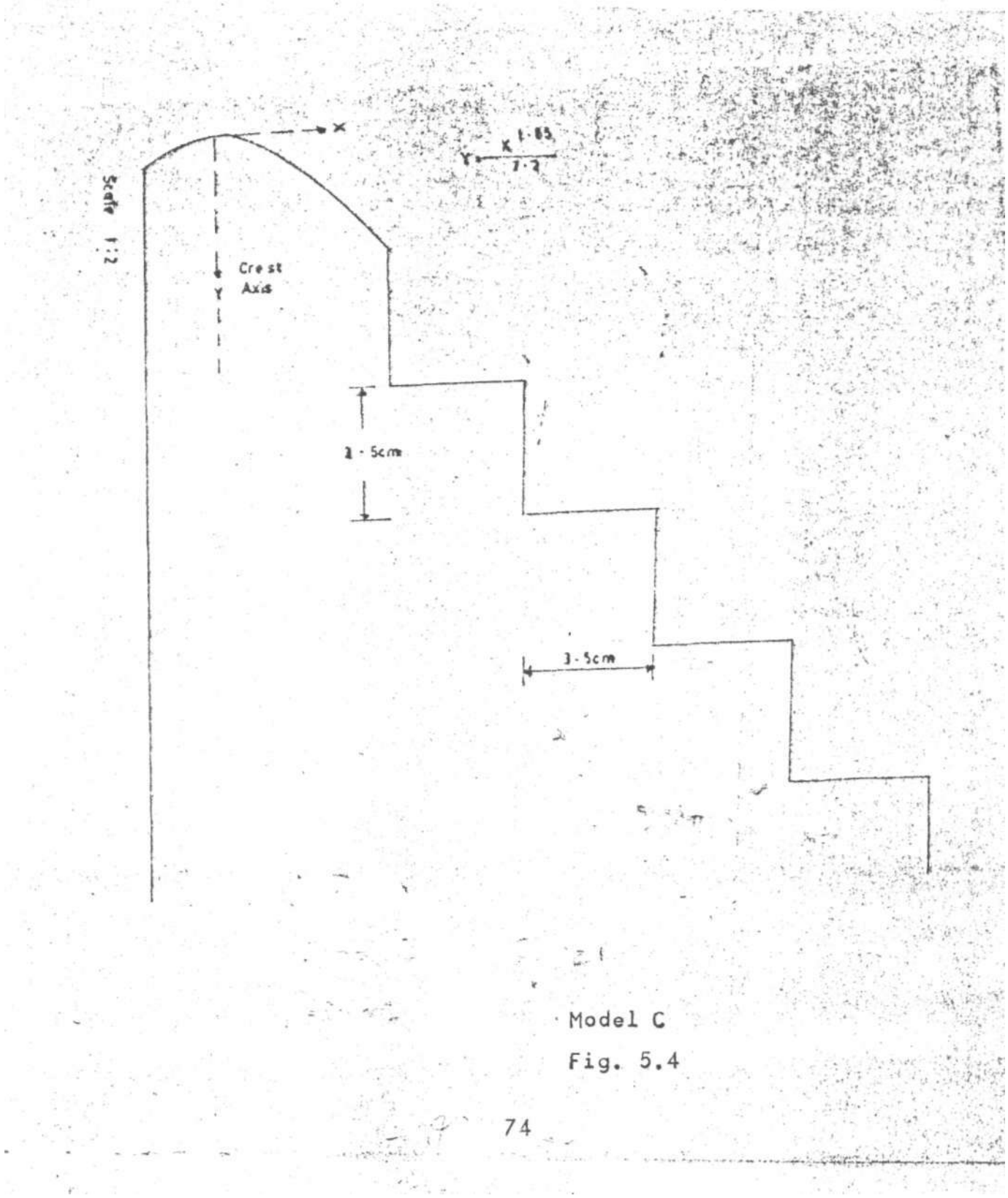
Fig. 5.2



Scale = 1 : 2

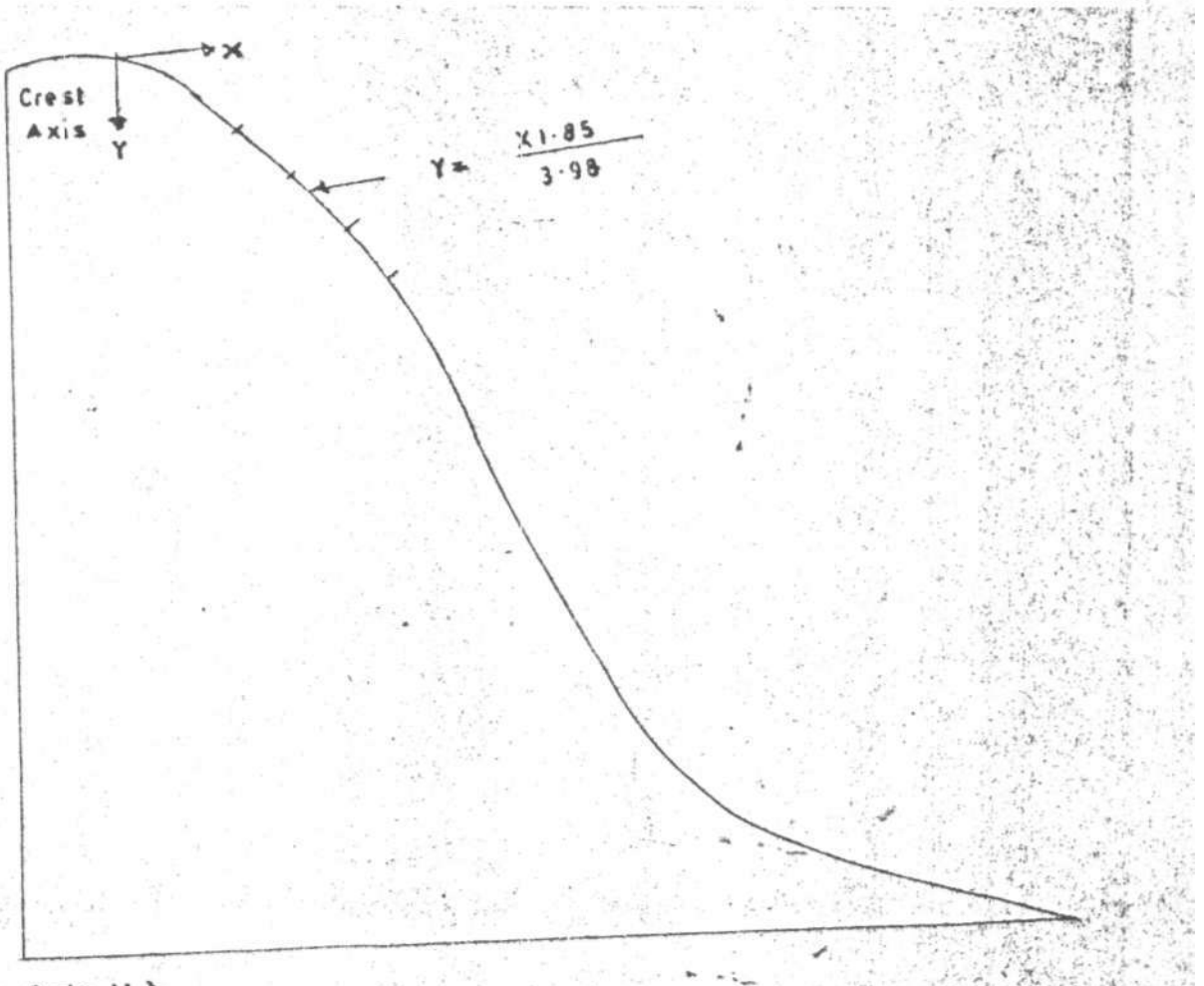
Model B

Fig. 5.3



Model C

Fig. 5.4



Model A
Unstepped Type

Fig. 5.5

5a

1. STEP SIZING TEST RESULT AT Q_m = 1 l/s

Assumed L_m = 4cm; L_p = 400cm.

No. of Readings Taken	Step Position	L_f cm	L_w cm	h_c cm	L_r cm	L_s cm	Relation $L_s \sim L_m$
1.	1	2.1	1.8	0.18	0.36	4.26	
2.	3	2.0	1.83	0.17	0.34	4.17	$L_s > L_m$
3.	4	2.2	1.83	0.18	0.36	4.36	
	Assumed L_m = 5cm; L_p = 500cm						
4.	1	2.1	1.82	0.2	0.4	4.32	
5.	3	2.0	1.84	0.2	0.4	4.24	$L_s < L_m$
6.	4	2.1	1.8	0.2	0.4	4.4	

STEP SIZING TEST RESULT AT Q_m = 3 l/s

Assumed L_m = 4cm; L_p = 400cm.

No. of Readings Taken	Step Position	L_f cm	L_w cm	h_c cm	L_r cm	L_s cm	Relation $L_s \sim L_m$
1	1	2.53	1.7	0.3	0.6	4.83	
2	3	2.5	1.75	0.3	0.6	4.85	$L_s > L_m$
3.	4	2.55	1.80	0.3	0.6	4.95	
Assumed L_m = 5cm; L_p = 500cm							
4	1	2.5	1.67	0.5	1.0	5.17	
5	3	2.51	1.67	0.5	1.0	5.18	$L_s > L_m$
6	4	2.51	1.66	0.5	1.0	5.17	
Assumed L_m = 5.5cm; L_p = 550cm							
7	1	2.5	1.68	0.5	1.0	5.18	
8	3	2.52	1.68	0.5	1.0	5.20	$L_s < L_m$
9.	4	2.51	1.67	0.5	1.0	5.18	

5c.

3. STEP SIZING TEST AT Q_m = 5 l/s

Assumed L_m = 4cm, L_p = 400$_y$cm.

No. of Readings Taken	Step Position	L_f cm	L_w cm	h_c cm	L_r cm	L_s cm	Relation $L_s \sim L_m$
1.	1	2.7	1.6	0.4	0.8	5.10	
2.	3	2.72	1.7	0.4	0.8	5.22	$L_s > L_m$
3.	4	2.7	1.65	0.4	0.8	5.15	
	Assumed L_m = 5cm; L_p = 500cm						
4.	1	2.7	1.71	0.5	1.0	5.41	
5.	3	2.74	1.68	0.5	1.0	5.42	$L_s > L_m$
6.	4	2.7	1.7	0.5	1.0	5.5	
	Assumed L_m = 5.5cm; L_p = 550cm						
7.	1	2.7	1.7	0.5	1.0	5.4	
8.	3	2.71	1.68	0.5	1.0	5.39	$L_s < L_m$
9.	4	2.71	1.68	0.5	1.0	5.39	

MODEL C SCALE RATIO 1:50

STEPPED TYPE

TABLE 5.1

R	H cm	h cm	Q l/s
1	2.0	2.0	2.1
2	2.3	3.0	2.6
3	2.4	3.3	2.7
4	2.4	3.5	2.8
5	2.5	4.0	3.0
6	2.7	4.5	3.2
7	2.7	4.7	3.2
8	2.7	4.8	3.3
9	2.7	5.0	3.3
10	2.8	5.2	3.4

R = Run

h = Head difference in water level in U-tube limbs

H = Head over weir

Q = Discharge

MODEL A

SCALE RATION 1:100

UNSTEPPED TYPE

Table 5.2

R	H cm	h cm	Q l/s	hi cm	V. cm/s
1.	1.3	0.5	1.0	100	44.3
2.	1.6	1.0	1.49	188	60.7
3.	1.8	1.5	1.82	205	63.4
4.	2.0	2.0	2.1	220	65.6
5.	2.3	3.0	2.5	225	66.4
6.	2.4	3.5	2.78	250	70.0
7.	2.7	5.0	3.3	289	75.3
8.	2.8	5.5	3.5	300	77.0
9.	3.0	5.8	4.5	370	92.0
10.	3.2	6.2	5.0	430	98.0

MODEL B

SCALE RATIO 1:100

STEPPED TYPE

Table 5.3

R	H cm	h cm	Q l/s	hi cm	V vcm/s
1.	1.4	0.4	0.94	11.6	15.1
2.	1.6	0.9	1.44	22.9	21.2
3.	1.7	1.2	1.64	24.6	22.0
4.	2.0	2.0	2.1	31.8	25.0
5.	2.3	3.0	2.58	30.9	24.6
6.	2.4	3.5	2.78	31.0	24.8
7.	2.5	4	3.0	48.9	31.0
8.	2.7	4.5	3.3	56.0	33.0
9.	2.8	5.0	3.5	62.0	34.7

CORRESPONDING PROTOTYPICAL

VALUES FOR UNSTEPPED TYPE A

TABLE 5.4 -

R	H m	Q m^3/s	V m/s
1	1.3	5.26	4.4
2	1.5	6.6	5.5
3	1.6	7.4	1.6
4	1.8	9.1	6.3
5	2.0	10.5	6.6
6	2.3	12.9	6.6
7	2.4	13.9	7.0
8	2.5	14.9	7.3
9	2.7	16.7	7.5
10	2.8	17.5	7.7

CORRESPONDING PROTOTYPICAL
VALUES FOR STEPPED TYPE B
TABLE 5.5

R	H m	Q m^3/s	V m/s
1	1.4	4.7	1.5
2	1.5	5.8	1.7
3	1.6	7.2	2.1
4	1.7	8.2	2.2
5	1.8	10.0	2.5
6	2.0	10.5	2.5
7	2.1	11.8	2.0
8	2.3	12.9	2.5
9	2.4	13.9	2.4
10	2.5	14.9	3.1

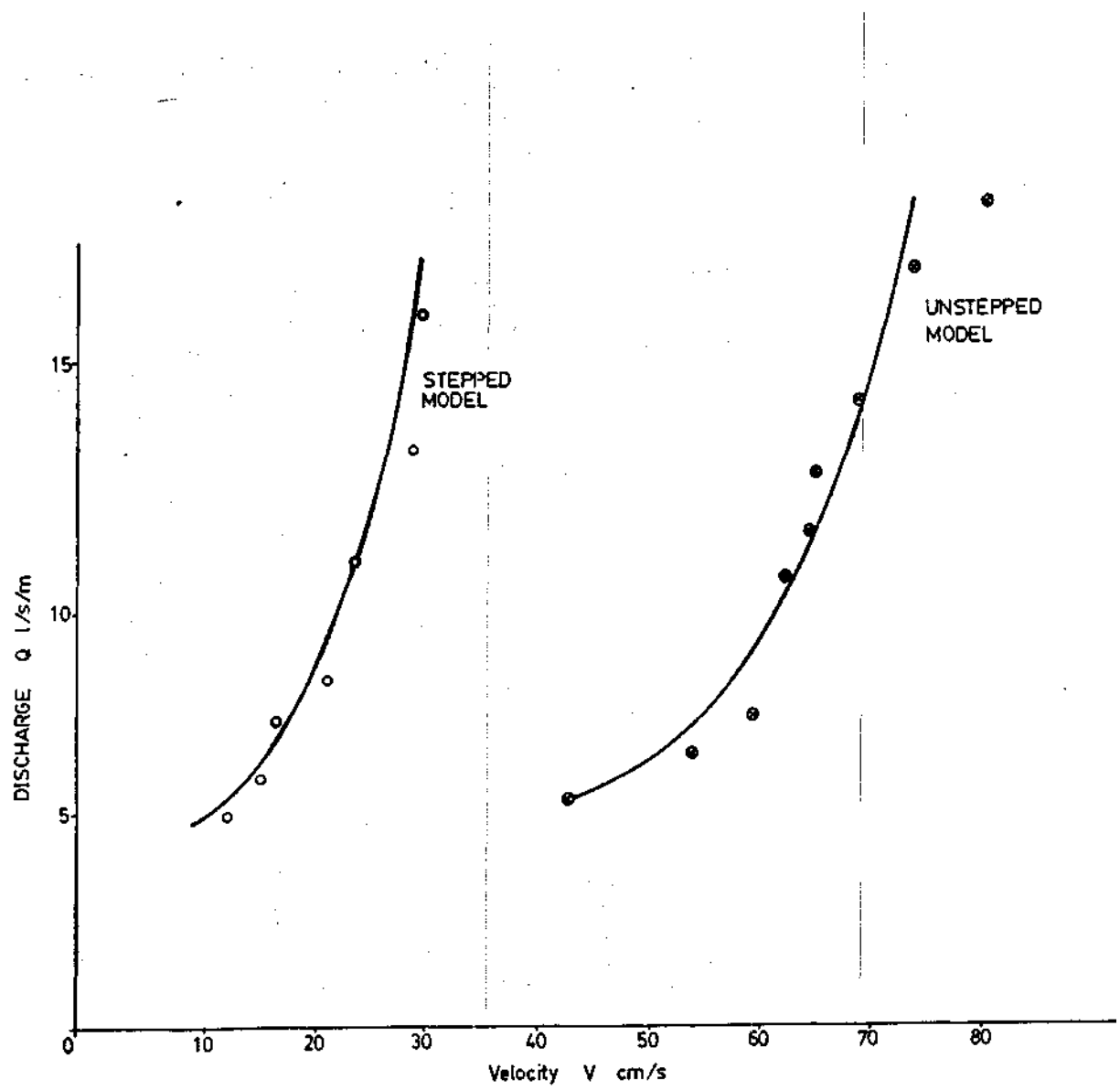


FIG. 5.6 Model – Toe – Velocities for Unstepped and stepped Spillway.

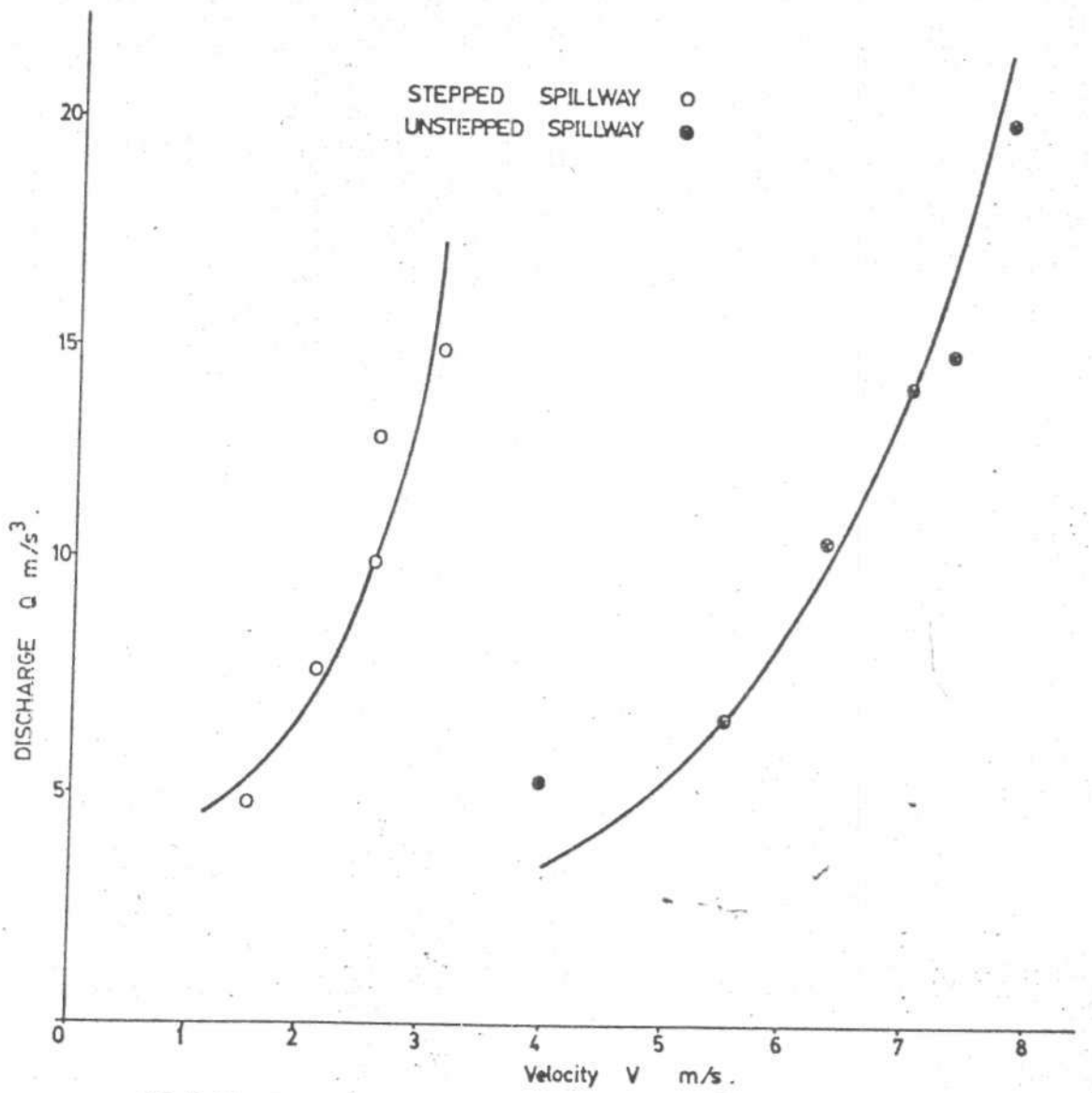


FIG. 5.7: Prototypical Toe – Velocities for Stepped and Unstepped Spillway.

CHAPTER SIX

ANALYSIS AND DISCUSSION OF RESULTS

The investigative study carried out in this project involves the construction and use of hydraulic models. The fundemental scale for any hydraulic model is the geometric scale. The choice of a suitable geometric scale depends on the type of fluide system which is to be studied and on the space available to build the model.

Once the scale of the model has been fixed according to the available space and other necessary equipment to be utilized, the requirement for the dynamic similarity may be used to determine other model scales.

These are needed in order to operate the model so that it will be dynamically similar to the prototype and so that measurements made in the model may be used to determin prototype values. It was through these procedures I arrived at this test results. And because the flow is dominated by gravitutional effects, models of free surface flow like these ones are scaled according to the Froudian Creteria.

In the course of this experiment, photographs were taken to show the flow nature over the steps and flow nature at the transition stage just before falling on the first step.

Photograph A shows the constructed model before placing in the flume for experiment, this is the model tagged B, whose experimental values were compared with model A which is the normal ogee type.

Photagraph B shows the model under experiment right in the flume with flow passing over it at a discharge of 5 l/s value. This photograph gives a very good description of what Chteren Liht (1982) says about the nature of flow over the steps and the type of parameters he described namely, falling – length; backwater length and the distance L_r which is $(2 - 2.5)h_c$, where h_c is as defined earlier to calculate the appropriate step size for some range of values of required discharge in any particular case.

Photograph C, this shows the flow pattern at low flow rates, say during summer or at times of low flow period in the life–time of the structure. High turbulence is immenant on first step but it ceases on the subsequent steps downwards due to low flow rate coming from reservoir.



PHOTOGRAPH A.



PHOTOGRAPH B.



PHOTOGRAPH C.

Generally, the two photographs taken of the experiments portray the type of flow nature described in the literature of this study over the series of steps which may result in the dissipation of energy before the flowing water reaches the toe of the weir and the nature of flow transition from one step to the next step. Photograph B is clearly showing how this pattern is taking place and is very similar in shape to what is shown on Fig.3.8 and Fig.2.2, both indicating how flow over rough surface takes place.

From the result of these models, it was observed that the toe velocity values were tremendously reduced for the same discharge of water passing over the unstepped weir, compared with the stepped weir of the same scale ratio. As can be seen from the results shown in Tables 5.2 and 5.3 for the model probable maximum discharge of 9.21/s/m, the toe velocity was reduced from 61.5cm/s for the unstepped model to 21.4cm/s for the stepped weir model, also at the model design, discharge of 6.91/s/m, the toe velocity was reduced from approximately 55cm/s to 17.5cm/s. The model A to model B toe velocity ratios thus were 2.9 and

3.1 respectively, and increase to just over 4.4 for a low model discharge of 4.5091/s/m. Consequently, the kinetic energy in the flow at stepped weir toe varies from 41% to 21% of the energy at the unstepped spillway mode toe for the range of model discharges. Below, the computation of this results are shown.

For all the runs carried out in the experiments, it was found that the introduction of the steps was able to reduce the velocity of flow of the incoming excess flow from the reservoir such that the velocity at the bottom of the weir is so negligible that scour and erosion may not occur at the toe. The curves for both the model types are drawn from the results found to give the corresponding values for velocities at the toe for a particular discharge for such a spillway shape and scale ratio, although this cannot be generalised for any spillway weir shape, but it makes a guidance for any further investigation that may be required for any spillway shape.

Therefore, the construction of this type of spillway will need minimum provision of any form of energy dissipation structure at the toe of the weir. The comparison of toe velocities for the

velocities of 2.3m/s at the probable maximum flood discharge, compared to toe velocity of 6.3m/s for a prototypical unstepped spillway. This represents a kinetic energy dissipation of about 60% which is as good as or superior to the energy dissipation achieved in a typical well developed hydraulic jump.

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATION

Conclusion:

The focus of this project is the spillway shape which was proposed in a stepped downstream face manner as against the conventional ogee unstepped type. The steps were introduced to find the possibility of either reducing grossly or completely eliminating the provision of a stilling basin dissipator at the bottom of the weir which in most cases, increase the cost of spillway by morethan 60% of its total cost. The test carried out proved that the steps were able to reduce the velocity of the flowing water over them which indicates the reduction of the total kinetic energy since the velocity reduction happens at each and every subsequent step before it reaches the toe of the weir.

With the introduction of the steps at some intervals on the wier downstream face, the flow was accompanied by some abnormal flow behaviours because now that surface composes of peaks and valleys. A turbulent flow with rolling vortex emnated on the top of the steps, velocity of flow

is likely reduced to to the roughness of the surface and also due largely to the formation of wakes behind each step. The wake and vortex at ecah step are completely developed and dis - sipated before the next step is reached, under such condition of flow at each step, a stable rolling vortex develops and rotates for a brief period and then returns to the main flow to procedd on down to the next step. The flow essentially skims the crests of the steps and large roughness projections are absent and the surface acts hydraulically smooth. As the intensity of the velocity caused by the steps increase in the direction of flow, more and more energy dissipation is achieved with successive steps continually placed up to the weir toes, so this explains why the velocity at the stepped weir toe is reduced by about 60% or more of the stepped weir for the same discharge.

Therefore, an ogee spillway weir with stepped downstream face just from below the crest to the toe can be designed to perform satisfac - torily as kinetic energy dissipator of the fast flowing water from over the weir. Consequently,

this shall minimise the construction of a stilling basin which is basically used at the toe of spillways to protect the structure from scour and erosion emanating from high velocity flows over the normal unstepped weir.

Recommendation:

Models studies on hydraulic structures is highly recommended before commencement of such project like Dams and other related structures. Through different alternatives and comparisons, the most economic and simplest alternative can be arrived at by use of models. Particularly in case of Biu dam project, there was no model study of what ever type of either the spillway or Dam and the reservoir, before the execution was started.

From this experimental study, it was d discovered that the use of a stepped spillway type could have saved both money and time for the execution of the project. A lot of excavation works could have been avoided and large sums of money could also have not been spent in the construction of the spillway chute, which is a 60cm thick reinforced concrete measuring 166m x 73m. Although it is too late to consider this change in application to Biu dam, this study can be used as a guide toward future projects. I recommend a similar studies and other model investigations for the rest of the projects of its kind before their execution.

I also suggest much more experiments to be carried out on different scale models of other dams so as to arrive at more research datas to further guide the design of steps for spillway faces, designs that will optimize response to construction and energy dissipation requirements, specifically, the optimum depression shapes, sizes and spacing for supercritical flow need to be determined.

RAINFALL AT BIU

TABLE 4.1

(Millimetres)

YEAR	ANNUAL (mm)	MAXIMUM DAY RAINFALL	DATE
1952	1100	64	23. 8.52
1953	1103	80	25. 8.53
1954	1151	60	26. 6.54
1955	1092	52	8. 6.55
1956	641	54	17. 9.56
1957	1076	53	23. 7.57
1958	1102	61	31. 8.58
1959	769	42	12. 8.59
1960	723	66	29. 9.60
1961	762	37	28. 8.61
1962	990	52	19. 9.62
1963	1185	66	25. 8.63
1964	999	53	19. 6.64
1965	1119	46	11. 8.65
1966	1064	70	21. 8.66
1967	1033	87	18. 7.67
1968	672	73	6. 7.68
1969	1095	69	15. 9.69
1970	826	46	19. 7.70
1971	873	67	24. 8.71
1972	921	64	28. 5.72
1973	665	48	10. 7.73
1974	981	96	10. 9.74
1975	1001	82	4. 8.75
1976	996	78	10. 8.76
1977	802	62	9. 9.77
1978	692	71	18. 7.78
1979	590	41	29. 8.79
1980	647	45.0	28. 5.80

RAINFALL AT BIU TABLE 4.1 (contd.)

YEAR	ANNUAL (mm)	MAXIMUM DAY RAINFALL	DATE
1981	1006	63.0	9. 9.81
1982	799	50.0	24. 7.82
1983	559	54	29. 8.83
1984	672	56	12. 7.84
1985	611	39	1. 6.85
1986	1114	80	28. 8.86
1987	671	53	5.10.87
1988	811	78	8. 6.88

TABLE 4.2

MAXIMUM AND MINIMUM MONTHLY RAINFALL (mm)

BIU DIVISIONAL OFFICE

	JAN.	FEB.	MARCH	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
Maximum	-	1	35	120	242	258	277	394	301	143	21	-
Minimum	-	-	-	-	28	47	122	96	-	-	-	-

GARKIDA MISSION

	JAN.	FEB.	MARCH	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
Maximum	-	-	30	85	128	181	318	283	243	117	6	60
Minimum	-	-	-	-	-	29	95	73	118	-	-	-

ASKIRA

	JAN.	FEB.	MARCH	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
Maximum	-	-	13	41	151	197	324	329	259	103	25	-
Minimum	-	-	-	-	19	51	105	157	114	26	-	-

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