HYDRAULIC MODEL STUDIES

FOR

IRRIGATION STRUCTURES

AND
FOR

RIVER PROBLEMS

Dissertation submitted in partial fulfilment of the requirement for the award of the Degree of

MASTER OF ENGINEERING
IN
WATER RESOURCES DEVELOPMENT

BY

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C E R T I F I C A T E

Certified that the dissertation entitled " HYDRAULIC MODEL STUDIES FOR IRRIGATION STRUCTURES AND FOR RIVER PROBLEMS " which is being submitted by Shri M.M. MAHODAYA, in partial fulfilment for the award of Degree of Master of Engineering in Water Resources Development of the University of Roorkee, is a record of the student's own work on the subject carried out by him under my supervision and guidance as the internal guide.

The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

It is further certified that he has worked diligently for a period of NINE Months from October 1966 till June 1967 for preparing the dissertation for the Master of Engineering Degree at this University.

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SYNOPSIS

In this Dissertation the author has attempted a clear exposition of the technique of Hydraulic Model Studies for Irrigation Structures and River Problems.

After briefly tracing the historical back ground of model studies, the basic theory involved, the laws governing various phenomenon in different problems, have been discussed. The different types of models involving closed conduit flows, flow with a free surface pertaining to hydraulic structures and rivers have been dealt in details. The different types of similitudes viz. Geometric, Kinematic and Dynamic similitudes and possibility of their attainments have been discussed.

An important aspect of the study is the determination of suitable scale ratios for Mobile bed river models. Critical study of the Einstein & Chien's Rational Method and Lacey's Empirical method of evaluating scales of river models have been made. Some case studies have also been made to go into the details of the model scales fixed for some river problems experimented at field research station Bahadrabad of Irrigation Research Institute Roorkee and Hydraulics laboratory of Civil Engineering Department of University of Roorkee.

The field data needed for model studies, the processing of the data, construction of models with suitable materials and the process of verification of the model have been described in details. Some instances of model and prototype conformity have been quoted and the value of hydraulic model as a valuable tool with the engineers has been impressed. Lastly, the limitations of a distorted river model and care and judgement required in interpretation of prototype behaviour from it has been pointed out and further scope of development in this field indicated.

SYMBOLS AND NOTATIONS

A - Area

B - Breadth

C - Cauchy's Number, constant in Chezy's formula

D - Diameter of the Sand grains, Depth Dimension

E - Euler's Number, Ratio of $\frac{p}{W_s}$

F - Froude's Number, Force Dimension

F',F''- Function of

F_B - Bed-factor

F_S - Side-factor

I - Slope of water level or gradient of energy head

L - Length Dimension along flow direction

L' - Radius of curvature of stream lines

M - Mass Dimension

P - Wetted Perimeter

Q - Discharge Dimension, Total Discharge

R - Hydraulic mean radius, Reynold's number

S - Slope of water surface

T - Time Dimension, Tractive force Dimension

U - Shear velocity

V - Velocity Dimension, Volume Dimension

W - Weight Dimension, Weber's number

Y - Depth Dimension

Z - Height of water level above horizontal datum

a,b,c - Linear Dimensions

d - Linear Dimension, grain diameter, derivative of a function

e - Exeggeration, Ratio of $\frac{k}{\rho}$ in Cauchy's number

f - Friction factor in Darcy-Weishbach formula, Lacey's silt factor, function of

g - Acceleration due to gravity

h - Height, Head, Depth Dimension

k - Elasticity Modulus

m - Exponent in Einstein's equations

n - Coefficient of Rougosity in Manning's formula

q - Discharge intensity per ft.

q_T - Total sediment load rate

q_B - Bed load rate

t_1 - Time scale for movement of a particle

t_2 - Time scale for sediment transport

α - Vertical exeggeration

β - Ratio of q_{Tr} / q_{Br}

γ - Unit weight of water

δ - Thickness of laminar sub layer

Δ - $\frac{\rho_s - \rho_f}{\rho_f}$

Δ_p - Pressure gradient

ρ - Mass density of water

ρ_f - Density of the fluid

ρ_s - Density of solid particles

μ - Coefficient of Dynamic viscosity for water

ν - Coefficient of Kinematic viscosity for water

φ_* - Intensity of transport

ψ_* - Intensity of shear for individual grain

σ - Surface tension

ω - Ratio of $\frac{\tau}{\rho}$

$\_\_\__m$ - Quantity pertaining to model

$\_\_\__p$ - Quantity pertaining to prototype

V.E. - Vertical exeggeration

S.E. - Slope exeggeration

HYDRAULIC MODEL STUDIES FOR IRRIGATION STRUCTURES AND RIVER PROBLEMS

CONTENTS

LIST OF PLATES

CHAPTER I

INTRODUCTION

(A) GENERAL

(B) DEFINITION OF A MODEL

(C) HISTORICAL BACK-GROUND OF MODEL STUDIES

(D) JUSTIFICATION OF MODEL STUDIES FOR PRESENT DAY HYDRAULIC PROBLEMS OF IRRIGATION STRUCTURES AND RIVERS

(E) LIMITATIONS OF MODEL STUDIES

(F) SCOPE FOR FURTHER DEVELOPMENTS IN THIS FIELD

CHAPTER 1 - INTRODUCTION

(A) General

1.1 The world has witnessed tremendous developments in the field of science and Engineering in the past 100 years. In the field of Hydraulics also, the progress has been none the less important and valuable. The "Technique of Hydraulic Model Studies" developed with the systematic and organised research work carried out in the Hydraulic Laboratories all over the world. The Technique has been so effectively developed by now that for each major work, whether it be an intricate irrigation or flood control structure or a navigation canal or harbour, experiments on a model of the prototype render great assistance and provide very useful information pertaining to the work.

(B) Defination of a model

1.2 For simplicity, a model may be defined as a replica of the main work (prototype). It is usually made smaller than the prototype for reasons of economy and ease of operation. The different classifications of the models are[1]:-

1) True Models
2) Adequate Models
3) Distorted models and
4) Dis-similar Models

In a "True Model " there is absolute similarity between Model and Prototype viz there is complete Geometric, Kinematic and Dynamic similarity in their performance and behaviour. This is an idealised concept and is possible only when model assumes the size of prototype. An " Adequate Model" is one which can accurately predict one of the characteristics of the prototype but not all of them. Thus there may exist geometric similarity but there may be no " Kinematic " or

"Dynamic" similarity. In a "Distorted Model " two different scales are adopted for length and height in order to produce the desired effect (turbulence or bed movement). Such a model apparently produces an awkward psychological effect on the observer, never the less, it is quite useful for interpreting the behaviour of prototype. The "Dis-similar Model " bears no resemblance whatsoever, with the prototype. It gives the information with regard to prototype with the help of suitable analogues viz. for flow of water through a pipe net work, an electric analogue may be made. These require good knowledge of Electrical Technology and Electronics and have been omitted from this dissertation.

(C) Historical back ground of Model Studies(2)

1.3 Model studies were started in the period of Leonardo-Da-Vinci or even earlier to study stream flow problems. 'Fargue' made the first systematic study on the model of Channel of Garonne at Bordeaux and recorded his observations. The model gave satisfactory results, inspite of the fact, that scales adopted by him were arbitrary and without any correlation. Later in 1885, Osborne Reynolds made the model of Mersey river Estuary, to study the silting characteristics. Freeman gave great encouragement to the new born technique of model studies and due to his benevolence, Hydraulic Research gained prominence in U.S.A.

In 1915, Edgar Buckingham evolved his method of "Dimensional Analysis ", which made it possible to express any flow phenomenon by substituting the dimensions of the pertinent quantities involved in ~~it~~ the phenomenon. In fact, the ' Theory of Similarity' has its basis in the principles of Dimensional similarity, which has been explained in details later. Since 1915, till to date, the Model science flourished in the progressive countries of the world, viz. United States of America, United

Kingdom, France, Germany, Japan and India.

Although in India, the Hydraulic models were constructed only about 50 years ago, but, in the short span the Technique has been so much developed, that full use of this has been made in finding solutions to various intricate problems connected with the numerous Multipurpose, Major or Medium Irrigation projects, Flood protection works, Reclamation works, Navigation in rivers. There are about 20 Hydraulic and Irrigation Research Stations functioning all over India. The credit for all this development in this field rightly goes to Sir Claude Inglis who can be rightly called the Father of Hydraulic Model Research in India. The Central Waterways, Experiment station was started at Poona near Khadakvasla in 1916. Now it can boast of being one of the best equipped Research Stations of the World, where all the different types of model experimentation work is carried out.

(D) Justification of Model studies for present day Hydraulic problems of Irrigation Structures and Rivers

1.4 Since a model serves as a visual aid and provides opportunity to the designer to carefully observe the phenomenon that is taking place, he gains confidence on the adequacy of a particular design. At times, cheaper alternatives are suggested at a moderate cost, there by saving millions worth of expenditure on an abnormally costly proposal. Models afford satisfactory solutions for problems for which mathematical solutions are either very cumbersome or not possible at all. In this sense, Hydraulic model serves as handy tool in the hands of research workers. The judgement and wisdom of the experimenter and his past experience with similar problems go a long way in interpretation of results of model studies while predicting the prototype behaviour. In view of their profound utility, there exists full justification for

Hydraulic model studies for any major problem connected with the design and operation of Hydraulic Structures or river training works. The small percentage of entire cost of the prototype, which is spent on Model studies, is not something esoteric, but, it is money well spent for general benefit and welfare of the people.

(E) Limitations of Model studies

1.5 It has been mentioned earlier that for absolute Dynamic similarity, the model should assume the dimensions of prototype only. The limitations for geometrically similar models of hydraulic structures occur when

1) surface tension force may be significant in the model
2) viscous force may act simultaneously with gravity
3) limitation in the degree of smoothness which can be attained in the model
4) special fluids having low viscosity may be used which are very costly and may be imported
5) costly equipments such as wind tunnels or water tunnels may be needed for studying problems connected with high heads giving rise to cavitation

In respect of River Models, where vertical exeggeration is adopted, banks may become steeper than the angle of repose of material and may need to be moulded as rigid. This vitiates the flow conditions. Also, 'Throw off' from guide bunds, spurs etc may be different in models. It is difficult to reproduce silting in a model corresponding to prototype. Also the model is unable to adjust to changed conditions after one flood has passed and another flood is following. Suspended load can not be accurately reproduced in a river model. A model may provide exeggerated scour holes and vortices unless adjusted previously.

1.6 Inspite of the numerous limitations indicated in the foregoing paragraphs, very high degree of Model-Prototype conformity has been obtained with the aid of Hydraulic Models for various problems governed by irrotational flow phenomenon. Successful designs of over-flow spillway crests, outlets, gates and valves have been evolved with the help of geometrically similar models. Also, numerous schemes of river improvements carried out in accordance with the findings of model studies have proved to be adequate. As a direct benefit, the flood hazard has been mitigated, greater security is warranted at a moderate cost of construction of river improvement measures.

(F) Scope for further developments in this field

1.7 Inspite of the great achievements made in this field, there remains great scope for future developments. For Geometrically Similar models, the causes of scale effects need to be investigated closely. Also the means have to be found out for eliminating the possible errors due to the scale effects while interpreting the results from model studies. For distorted models, the scope for development is enormous because of the following:

i) There is no unanimity of opinion for fixing the scales of a distorted River model

ii) The Time Scale adopted in river models is only arbitrarily fixed which needs a proper basis

iii) The science of Fluvial Hydraulics is in its infant stage and research workers hold different views on the different theories of sediment transport

iv) There is controversy regarding interpretation of results from a 'Tilted' river model

v) Instrumentation for model studies needs further refinement and standardisation

vi) Instruments for prototype measurements need to be evolved, manufactured and prototype data collected for model prototype confdrmity for fully establishing the efficacy of the Hydraulic Model studies

CHAPTER II

DIMENSIONAL ANALYSIS & PRINCIPLES OF HYDRAULIC SIMILITUDE

(A) CONCEPT OF DIMENSIONAL ANALYSIS AND ITS SCOPE

(B) BUCKINGHAM'S 'π' THEOREM AND DIMENSION-LESS PARAMETERS

(C) PRINCIPLES OF SIMILITUDE

(D) IMPORTANT MODEL-LAWS

(E) DERIVED MODEL-LAWS

(F) INTERACTION OF TWO MODEL-LAWS

CHAPTER II - DIMENSIONAL ANALYSIS & PRINCIPLES OF HYDRAULIC SIMILITUDE

(A) Concept of Dimensional Analysis and its scope

2.1 The principle of Dimensional Analysis is based on the concept, that physical quantities have specific dimensions. The properties of a body such as its length, mass, weight, viscosity, temperature, volume etc. are thus expressed by suitable dimensions. There are two different systems in which the dimension of a physical quantity may be expressed. These are

(1) Force - Length - Time (F-L-T) system also known as Engineer's system and

(2) Mass-Length-Time (M-L-T) system or Absolute system.

The difference between the two is only due to Mass and Force being considered as the Chief Physical quantity.

2.2 The application of dimensional analysis to practical problems, is based on the hypotheses, that solutions of such problems are expressible by dimensionally homogenous equations in terms of variables influencing the problem. Thus, Dimensional analysis is a necessary adjunct to principles of similarity and forms the basis of model investigation technique. Some popular Empirical Formulae such as Chezy's and Manning's formula do not satisfy the requirement of dimensional homogeneity. These indicate that they have limited usefulness as they cannot be extrapolated beyond the experimental range from which they were deduced.

(B) Buckingham's 'π' Theorem and Dimension-less parameters (3)

2.3 If a flow phenomenon is governed by 'n' variables such that $(Q_1, Q_2, Q_3 - Qn) = 0$ then from Buckingham's π Theorem, the 'n' variables can be expressed in terms of ' m ' dimensional units and the general equation can be expressed as a function of (n - m) Dimension-less parameters (π - terms) each one having (m+1) variables of which only one may change from one 'π' term to another 'π' term. Hence

$$f\ (\ \pi_1,\ \pi_2,\ \pi_3\ ---\ \pi_{n-m}\ ) = 0$$

The variables which generally influence fluid motion are :-

1) Linear dimensions defining boundary conditions
2) Kinematic and dynamic characteristics of flow
3) Fluid properties of density, specific weight, viscosity, surface tension and elasticity

The functional relationship between the variables can be expressed as below

$$f\ (\ a, b, c, d, V, \Delta p, \rho, \mu, \gamma, \sigma, K\ ) = 0$$

Thus there are eleven terms each of which can be expressed by three fundamental dimensions of Length, Mass and Time. This according to 'π' Theorem provides (11-3) = 8 Dimension-less parameters. Thus the relationship of 11 variables are transformed in to

$$F\ \left(\ \frac{a}{b}, \frac{a}{c}, \frac{a}{d}, \frac{V^2}{a.\gamma/\rho}, \frac{Va}{\mu/\rho}, \frac{V^2 a}{\sigma/\rho}, \frac{V^2}{K/\rho}, \frac{V^2}{\Delta p/\rho}\ \right) = 0 \text{ or}$$

$$F\ (\ \pi_1, \pi_2, \pi_3, \pi_4, \pi_5, \pi_6, \pi_7, \pi_8\ ) = 0$$

The above 8 dimension-less parameters can be classified in to 3 categories as follows

1) those defining the boundary conditions
2) those defining the basic flow characteristics, and
3) those involving the forces of gravity, viscosity, surface tension and elasticity

The Dimensionless parameters are the well-known Froude, Reynolds', Weber and Cauchy numbers.

1) Froude's Number (F) $= \dfrac{V^2}{a.\gamma/\rho}$
2) Reynolds' Number (R) $= \dfrac{V.a}{\mu/\rho}$
3) Weber Number (W) $= \dfrac{V^2 a}{\sigma/\rho}$
4) Cauchy's Number (C) $= \dfrac{V^2}{K/\rho}$

These have been explained in details subsequently.

(C) Principles of similitude

2.4 There are three main types of similarities. These are

i) Geometric similarity :- This exists when ratios of homologous dimensions are equal. It involves similarity of form, shape or boundaries.

ii) Kinematic similarity :- This postulates similarity of form and motion. It exists when ratios of velocities and accelerations at corresponding points are equal.

iii) Dynamic similarity :- This requires the force ratios to be equal besides satisfying the conditions for Geometric and Kinematic similarities.

2.5 Force system in fluid phenomenon includes

1) Gravity force
2) Viscous force
3) Surface tension force
4) Elastic force
5) Pressure Force

Thus, it is necessary for perfect similitude, that all the Dimensionless parameters mentioned in para 2.3 are equal for model as well as prototype. It is a highly exacting demand and cannot be satisfied in practice. Due to interaction of one or more forces, perfect dynamic similarity is unattainable. The study is consequently carried out on the basis of several partial similitudes and judgement has to be exercised in interpreting the results from hydraulic model studies.

(4)

(D) Important Model Laws

2.6 Generally before taking up the model studies, it is ascertained which force is predominent in the flow phenomenon or which is the model law governing the flow characteristics. The impor tant Model laws are:-

i) Froude's Law - If in a flow phenomenon, the force of gravity is the main acting force(this would cover the case of flow over weirs and spillways or through sluices and orifices, propagation of gravity waves etc. in an open channel) then the model law governing is the " Froude's Law ". Other forces, such as fluid friction, surface tension etc. are negligible. This requires the ratio of gravitational forces to be equal to ratio of inertial forces. Also the paths of flow lines are similar.

$$F_r = \frac{F_m}{F_p} = M_r \cdot \frac{L_r}{T_r^2} = \rho_r \cdot L_r^3 \cdot \frac{L_r}{T_r^2} = \frac{\rho_r \cdot L_r^4}{T_r^2} \quad --- \text{Eq}^n\ 1$$

Since the ratio of gravitational forces are in the ratio of weights of homologous particles, we have

$$F_r = \frac{W_m}{W_p} = \gamma_r \cdot L_r^3 = \rho_r \cdot g_r \cdot L_r^3 . \quad --- \text{Eq}^n\ 2$$

equating the two $\frac{\rho_r \cdot L_r^4}{T_r^2} = \rho_r \cdot g_r \cdot L_r^3 .$

or $T_r^2 = \frac{L_r}{g_r}$ or $T_r = \sqrt{\frac{L_r}{g_r}} = \sqrt{L_r}$ since $g_r = 1$

since velocity = Length/Time $V_r = \frac{L_r}{T_r}$

substituting Vr= $\sqrt{L_r \cdot g_r} = \sqrt{L_r}$ since $g_r = 1$

since discharge= Volume/Time $Q_r = \frac{L_r^3}{T_r} = \frac{L_r^3}{\sqrt{L_r}} = L_r^{5/2}$

These ratios can be obtained by assuming

$$F_N = \frac{V^2}{gL} \quad \text{and} \quad \frac{V_r^2}{g_r \cdot L_r} = 1$$

ii) Reynold's Law - While studying motion in compressible fluids, where the motion is mainly governed by internal frintional forces, it is necessary that ratio of viscous forces on homologous particles in model and prototype is equal to the ratio of inertia forces.

Since viscous force F= $\mu \cdot \frac{\delta v}{\delta L} \cdot A$

Fr= $\frac{F_m}{F_p} = \mu_r \cdot \frac{L_r}{T_r} \cdot \frac{1}{L_r} \cdot L_r^2 = \frac{\mu_r \cdot L_r^2}{T_r}$

equating inertial and viscous force ratios

$$\frac{\rho_r \cdot L_r^4}{T_r^2} = \frac{\mu_r \cdot L_r^2}{T_r} \quad \text{or} \quad T_r = \frac{L_r^2}{\mu_r / \rho_r} = \frac{L_r^2}{\nu_r} \quad \text{since } (\mu/\rho = \nu)$$

since Vr= $\frac{L_r}{T_r}$; $V_r = \frac{\nu_r}{L_r}$

The scale ratios can be obtained by assuming $R_N = \frac{V.L.}{\nu}$ and $\frac{V_r . L_r}{\nu_r} = 1$ for model and prototype .

iii) Weber's Law + When in a flow phenomenon, the depth of flow is unusually small, so that the forces of surface tension become significant, then the model law governing the flow phenomenon is the " Weber's Law ". It requires that the ratio of surface tension forces are equal to ratio of inertia forces on homologous particles. The scale ratio can be determined by assuming $W_N = \frac{V^2 L}{\sigma/\rho}$

and $\frac{V_r^2 . L_r}{\omega_r} = 1$ where $\omega = \sigma/\rho$

This gives $V_r = \sqrt{\omega_r / L_r}$ and $T_r = \frac{L_r^{3/2}}{\sqrt{\omega_r}}$

iv) Cauchy's Law - In some cases where motion occurences are governed by elastic forces the Cauchy's law is adopted. The scale ratios are obtained by assuming $C_N = \frac{V^2}{K/\rho}$

and $\frac{V_r^2}{e_r} = 1$ where $e = K/\rho$

This gives $V_r = \sqrt{e_r}$ and $T_r = \frac{L_r}{\sqrt{e_r}}$

2.7 On the same premise the scale ratios for Euler's law can also be found; this has been omitted as it is not commonly applicable. Due to recent advances made in the field of hydraulic research, some more dimension less numbers have been evolved viz. the ' Karman Number ', the ' Cavitation Number ' and the ' Strouhal Number '. These are applicable in special situations viz. for a rough boundary and for rough turbulent flow. These have not been dealt with.

(E) Derived Model laws

2.8 It has been mentioned earlier in para 2.2 that some empirical formulae derived from experimental observations do not satisfy the conditions of dimensional similarity. The equations are not balanced when the dimensions of physical quantities involved are substituted. For example, the Manning's Formulae is popularly used to describe velocity in open channels. It has the form

$$V = \frac{1.486}{n} . R^{2/3} . S^{1/2}$$

This formula can be used for a big natural river and a small model of the same. The scale ratios are determined thus

$$Vr = \frac{V_m}{V_p} = \frac{R_r^{2/3} \cdot S_r^{1/2}}{n_r}$$

but for a geometrically similar model Sr = 1 and therefore

$$Vr = \frac{L_r^{2/3}}{n_r}$$

If the Manning's law as well as Froude's law govern the phenomenoh

$$V_r = \frac{L_r^{2/3}}{n_r} = \sqrt{L_r} \quad \text{or} \quad n_r = L_r^{1/6}.$$

This is the ('Derived Model Law ' showing that the roughness scale ratio is equal to the sixth root of length scale ratio.

(F) Interaction of Two model laws

2.9 When the same fluid is used in model as wel as in the prototype, only one law can be satisfied for dynamic similarity. With the use of special fluids, viz. Glycerol etc. simultaneously the Froude's law and Reynold's law can be satisfied.

$$V_r = \sqrt{L_r} = \frac{\nu_r}{L_r} \quad \text{or} \quad \nu_r = L_r^{3/2}$$

It would some times become necessary to restrict the length scale ratio due to availability of a fluid of certain Kinematic viscosity. This gives Lr = $\nu_r^{2/3}$.

2.10 Fortunately in nature for any perticular problem, only one specific force is predominent compared to the rest, hence a suitable model can be prepared and the governing model law is used for model studies.

CHAPTER III

DIFFERENT TYPES OF MODELS FOR HYDRAULIC STRUCTURES AND RIVERS

(A) CLASSIFICATION OF MODELS

(B) GEOMETRICALLY SIMILAR MODELS NOT GIVING GEOMETRICALLY SIMILAR RESULTS

(C) SECTIONAL MODELS, SCALE RATIOS AND THEIR MINIMUM SIZES

(D) MODELS OF TRANSITION STRUCTURES

(E) HARBOUR AND BREAKWATER MODELS

(F) MODELS FOR CAVITATION-FREE DESIGNS OF HYDRAULIC STRUCTURES

(G) SHIP MODELS AND LOCK CHAMBER MODELS

(H) DISTORTED MODELS FOR RIVERS

(I) NEED FOR DISTORTION

(J) RIGID, SEMI-RIGID AND MOBILE BED RIVER MODELS

(K) TILTED RIVER MODELS

CHAPTER III - DIFFERENT TYPES OF MODELS FOR HYDRAULIC STRUCTURES AND RIVERS

(A) Classification of Models

3.1 There are mainly two types of models. These are :-

i) Geometrically similar models

ii) Distorted models with Rigid bed and Mobile bed models

In a geometrically similar model, all the linear dimensions are reduced to the same scale. In a distorted model two different scales are adopted for horizontal and vertical dimensions. Great reputation has been built up for model studies by the geometrically similar models, as very close dynamic similarity is attained with such models. The models are mostly made rigid, however for studying problems of energy dissipation below spill ways or out-let structures, a rigid model is adopted with a mobile bed down stream, to judge the comparative efficiency of energy dissipation by measuring relative depths of scour.

3.2 Thus it can be realised that the most vital consideration for geometrically similar models is the choice of scales and the practicability of relatively small scales. The cost of large models may be highly prohibitive to be taken lightly. In many cases great difficulty may be experienced in handling the quantity of water and making measurements as the model size is increased. Also a large model is not capable of easy modifications for studying other alternative proposals.

3.3 Since the discharge scale Qr in a geometrically similar model is $=Lr^{5/2}$, one cusec in model corresponds to about 29000 cusecs in prototype when Lr = 1/60. From considerations of the ease of handling, a model discharge of about 1 - 8 cusecs can be adopted for any model. The capacity of the pumping plant and economic considerations in use of water available from gravity flow, may also impose limitation on the maximum permissible discharge for a model study. Ordinarily maximum discharge for a model should not exceed 10 cusecs, inclusive of river models.

3.4 Some of the different problems for which the geometrically similar models are adopted are as follows:-

i) Determination of the co-efficient of discharge for weirs and spillways.

ii) Pressure distribution on spillway profile, out-lets and syphons.

iii) Standing wave studies

iv) Harbour and break-water studies

v) Cavitation in hydraulic structures

vi) Design of efficient propeller characteristics and the form of hull of a ship

vii) Determination of filling and emptying time of lock chambers in navigation canals

3.5 The spillway models are governed by the Froude's law. Since the surface in a model cannot be finished smoother than n = 0.01, there is limitation on the length scale ($n_r = L_r^{1/6}$) For proper simulation of roughness, the length scale ratio(Lr) should not be smaller than 1/60 - 1/70. Also the depth of overflow should be more than 3" to reduce the effect of surface tension and for convenience of measurements.

(B) Geometrically similar models not giving geometrically similar results [5]

3.6 In case of sharp spillway profiles, it would be found that the co-efficient of discharge increases with scale and depth of water over the spillway crest. In such cases the geometrically similar models do not give geometrically similar results and the co-efficient of discharge has to be extrapolated judiciously from results obtained from 3 or 4 models made to different scales. Also if a very small scale has to be chosen because of the large size of the structure giving model roughness $n_m < .01$ then at least 3 models should be made to different scales and the prototype co-efficient of discharge extrapolated by eliminating the scale effects.

(C) Sectional models, scale ratios and minimum sizes[6]

3.7 In order to determine the co-efficient of discharge or to note the pressure distribution on the spillway profile, a sectional model is generally made in a glass sided flume about 1'-3' wide. The model is moulded in cement concrete or cement sand mortar and surface neatly finished off with cement paste to give the correct profile and roughness. The piezometer tips are carefully installed at appropriate locations with due consideration that these are at right angles to the finished surface. The scales adopted for the sectional models are $\frac{1}{4}$ to 1/12 depending on the size of the structure. The scale ratio should be so fixed that laminar flow is avoided and value of Reynold's number is sufficiently high in the model for all discharges required to be considered. Not withstanding the previous criteria, there are some minimum sizes to be adopted in models such as -

i) models of conduits, gates, valves etc; should have diameters more than 4"

ii) models of canal structures such as a fall, aqueduct or distributary head should have base width more than 4"

Models of smaller sizes are difficult to operate and information regarding the prototype behaviour can not be dependable under such conditions.

(D) Models of Transition structures

3.8 In transition structures the flow changes from sub-critical to super-critical or vice-versa. This causes change in elevation of water surface and pressure distribution. Sometimes there is change in bed width or super- elevation in bed at bends.

Information regarding the theory and design of transitions is very defficient. Thus for problems involving both sub-critical and super critical flow, the models are necessary adjuncts. In a long flat chute, at times distortion of length or slope or both are adopted in order to counter-act the effect of high frictional resistance. This distortion is accomplished with computed losses in models and prototype. In case of a steeply sloping chute, there are chances of air entrainment which increases the depth downstream. The phenomenon of ' in-sufflation ' also needs to be simulated in the model to some extent.

(E) Harbour and breakwater models

3.9 The phenomenon of wave action being of complex nature, as also the harbour geometry and sea-bed configuration, small scale geometrically similar model serves a very useful purpose in solving the problems of design of harbours and in deciding length, alignmen and stability of breakwaters and tide protection structures. In the wave phenomenon, reflection, refraction, diffraction, atteaenuation and breaking of waves are important. In a distorted model because of steeper slope, wave reflection is considerably in excess. For a harbour having deeper and shallow portions simultaneously, wave length and depth govern the wave velocity in deeper and shallow portions respectively. Thus refraction becomes important and requir to be simulated, by Froudian criterian, in a geometrically similar model. For breaking of waves also, the ' steepness ratio ' should remain same for model and prototype and so again geometric similarity is required. The requirement of accuracy and the least count of the depth measuring instruments also govern the scale to be adopted in harbour models. These usually range from 1/50 - 1/250.

(F) Models for cavitation-free designs of hydraulic structures

3.10 In case of low pressure and high velocity of flow through structures such as conduits, intakes, siphons, sluice gates etc, cavitation starts when absolute pressure equals the vapour pre- ssure at prevailing temperature. This causes damage to the surfaces of the structures. To evolve cavitation-free designs, the model studies are conducted in a cavitation tank or tunnel in which controlled negative pressure is maintained and actual cavitation simula ted. Quick motion photographs reveal the development of cavities. The Cavitation Number = $\frac{2g(h_o - h_v)}{V_o^2}$ forms the basis of predicting the possibility of cavitation in the prototype. The cavitation number is based on the fact that the pressure changes between two points at the surface of a fluid are proportional to the changes in velocity head. For geometrically similar boundaries, cavitation occurs in model and prototype at the same value of cavitation numbe: The model procedure consists in changing the value of the cavitatio: parameter by altering the flow characteristics and when cavitation damage starts occuring in the model, as revealed by the photographs. it is surmised that in prototype also the cavitation damage would occur at the same value of cavitation number.

(G) Ship models and Lock chamber models

3.11 Models of ships are tested in Froude's tanks and for these both the Reynold's law and Froude's law need to be satisfied simultaneously. Sometimes large models of about 20 ft length are ma(and tested to get the propeller characteristics and the most efficient shape of the hull for prototype. Similarly models of navigation locks with their water passages etc. also serve as a useful guide in the design of the detailed components of locks and in thei: operation.

(H) Distorted Models for rivers

3.12 Usually, for studying problems connected with the rivers, the hydraulic model is distorted viz it's depth scale is made comparatively larger than the length scale. In other words the model provides vertically exeggerated effect. The range of exeggeration varies from 2 - 10 for one-way flowing river models. However for tidal rivers, much higher exeggeration of the order of 25 - 30 is adopted. The river model may be a rigid bed model or a mobile bed model.

(I) Need for distortion

3.13 A geometrically similar model for a river problem would give very shallow depth and low velocities incapable of reproducing scour effects. Distortion in scales in such a model is adopted to achieve some definite objective such as increased tractive force for inducing additional bed movement. Usually in a river during floods, the horizontal dimension i.e. the width is very large as compared to the vertical dimension i.e. the depth of the river. Also the river flow is invariably turbulent and river carries substantial amount of sediment during floods as bed load and suspended load. In a model due to space limitations, cost restrictions and from discharge considerations, a vertically exeggerated scale is invariably adopted, both for rigid and mobile bed models. At times distortion of slope scale may also be introduced. Due to non-availability of material, insufficient quantity of particular grain size, distortion in sediment size becomes necessary and thus a sediment scale has to be adopted. Sometimes material of different specific gravity may also be used in the model.

(J) Rigid, semi-rigid and Mobile bed river models

3.14 In a Rigid bed model, both the banks and bed of the river are moulded with hard and in-erodible material. For a semi-rigid model, the banks are made rigid, while the bed is moulded with sand of particular grain size to profile. In a mobile bed model, both the banks and the bed are moulded in sand and much difficulties are encountered while experimenting with a mobile bed model.

The rigid bed models are generally used for studying problems of flood protection works, where sediment movement can only cause error on the safe side i.e. the model gauges are slightly higher than prototype gauges. In these models the bed roughness needs to be adjusted by trial and error. Artificial roughness elements like screens, studs, strips etc. are used in models for bringing about the similitude of prototype roughness factor.

Semi-rigid and mobile bed models are usedfor studying similar problems and those connected with bank protection and other river training measures. Experience is of great essence in carrying out experiments on distorted mobile bed models. The results need careful and judicious interpretation based on observations of prototype in nature.

(K) Tilted river models

3.15 Sometimes even with the vertical exeggeration adopted for a model, adequate bed movement does not take place due to inadequate tractive force. In such cases the whole model is required to be tilted to provide additional tractive force. In a straight reach of the river, tilting may not pose much difficulty, but in a meandering river, the extra slope due to tilting has to be incorporated by raising the bed levels of templates from tail cross section upwards. In order to economise cutting and filling, tilting is adopted from the middle point of the entire reach under study. Much experience and judgement is required in the interpretation of results , when study is made with a ' Tilted Model ' of a fluvial river. In model design usually either the Lacey's slope formula is used which gives $S = 0.000547 \frac{f^{5/3}}{Q^{1/6}}$ or

$$S_r = f_r^{5/3} \cdot Q_r^{-1/6} = m_r^{5/6} \cdot Q_r^{-1/6}.$$

Some times Manning's formula in conjunction with the Froude's law is adopted to give the slope scale

$$Vr = \sqrt{Y_r} = \frac{(R_r)^{1/6}}{n_r} \cdot R^{2/3} \cdot S^{1/2} = \frac{(Y_r)^{1/6}}{n_r} \cdot Y_r^{2/3} \cdot S_r^{1/2} \quad \text{or} \quad Y_r = \frac{Y_r^{4/3} \cdot S_r}{n_r^2}$$

$$\text{Or} \quad Sr = \frac{n_r^2}{Y_r^{1/3}}$$

In India, at Poona research station tilting has been adopted for many river models and very satisfactory results obtained from them.

The detailed design of a fluvial river model needs several special considerations and the same have been treated in details in a seperate chapter hereafter.

CHAPTER IV

DESIGN OF FLUVIAL RIVER MODELS AND A CRITICAL REVIEW OF DIFFERENT THEORIES

(A) GENERAL

(B) EINSTEIN'S & NING CHIEN'S APPROACH

(C) KOMURA'S SIMPLIFIED EQUATIONS

(D) LACEY'S REGIME EQUATIONS

(E) THOMAS BLENCH'S MODIFIED REGIME EQUATIONS

(F) E.W.BIJKER'S APPROACH

(G) SCALE RATIOS TO BE FINALLY ADOPTED FOR RIVER MODELS

CHAPTER IV - DESIGN OF FLUVIAL RIVER MODELS AND A CRITICAL REVIEW OF DIFFERENT THEORIES

(A) General

4.1 It has been pointed out in previous chapter that a river model is almost always distorted to simulate existing natural conditions. The necessity arises because of the following:-

(i) From considerations of space and discharge limitations and for maintaining a high Reynold's Number at all flows, (prototype flow is usually turbulent) the length scale is kept different from depth scale ($h_r \neq L_r$)

(ii) This is also required to generate ~~to~~ sufficient tractive force to produce corresponding bed movement, at corresponding stages and discharges with available material for moulding the model bed.

(iii) At times the slope scale (S_r) of the model is not equal to $\frac{h_r}{L_r}$ $\frac{\text{(Depth Scale)}}{\text{(Length Scale)}}$ and slope exeggeration(S.E.) is less than or in excess of vertical exeggeration(V.E.)

(iv) Occasionally material of different specific gravity is used in model, in order to induce bed movement without excessive distortion.

(v) Although River Models are governed by Froude's law, (i.e. the Froude's Number for prototype and model should be same) slight departure is allowed, for the sake of adequate bed movement.

Thus, there is geometric distortion in (i),(ii) & (iii) and there is material distortion in (iv).

(B) Einstein's & Ming chien's Approach[7]

4.2 Detailed analysis of distorted river models has been given by Einstein and Chien and further modifications have been presented by Bogardi & Komura which simplify the procedure of fixation of different scales for such river models. These methods are described in succeeding paras.

The procedure consists of selecting suitable equations for

(1) Mean velocity

(2) Resistance co-efficient

(3) Critical Tractive Force

(4) Sediment transport

The equations are written in Dimensionless forms and Nine conditional Equations are obtained as deduced below:

(1) Friction criteria

A flow equation of the Manning's type is assumed such as

$$V = \frac{C\sqrt{g}}{D^m} \cdot S^{\frac{1}{2}} \cdot h^{(\frac{1}{2} + m)}$$

Where $m = \frac{1}{6}$ in Manning's formula

Value of 'm' should be determined by trial and should remain same for model and prototype. The scale relation obtained is

$$\frac{V_r^2 \cdot D_r^{2m}}{C_r^2 \cdot S_r \cdot h_r^{(1+2m)}} = \triangle V \quad \text{Or}$$

$$V_r^2 \cdot D_r^{2m} \cdot C_r^{-2} \cdot S_r^{-1} \cdot h_r^{-(1+2m)} = \triangle V = 1 \text{ for}$$

exact similitude.

(2) Froude's number criteria

The Froude numbers should be same for model and prototype for exact similarity

$$F_N = \frac{V}{\sqrt{gh}} \quad \text{Or} \quad F_r = \frac{V_r}{h_r^{\frac{1}{2}}} = \triangle F$$

Or $V_r \cdot h_r^{-\frac{1}{2}} = \triangle F = 1$; slight deviation from exactly equal Froude numbers is allowed.

(3) Sediment Transport criteria

According to this intensity of sediment transport should remain same in model and prototype. Neglecting the effect of grading.

Intensity of transport φ_* is given as

$$\varphi = \frac{q_B}{g(\rho_s - \rho_f)} \cdot \left(\frac{\rho_f}{\rho_s - \rho_f}\right)^{\frac{1}{2}} \cdot \left(\frac{1}{g\, D^3}\right)^{\frac{1}{2}}$$

When same fluid is used in model and prototype ($P_{fr} = 1$)

$$\text{Or } \varphi_r = \frac{q_{Br}}{(\rho_s - \rho_f)_r^{3/2} \cdot D_r^{3/2}} = 1$$

$$q_{Br} \cdot (\rho_s - \rho_f)^{-3/2} \cdot D_r^{-3/2} = 1$$

No deviation in intensity of transport is desired.

(4) Zero sediment load criteria

$$\text{Intensity of shear } \psi = \frac{(\rho_s - \rho_f)}{\rho_f} \cdot \frac{D}{R'b \cdot S}$$

Where R'b is hydraulic mean radius for surface drag only.

If $R'b = \eta h$ Where $\eta = 1$ Approximately

ψ should also be equal for model and prototype

$$\psi_r = \frac{(\rho s - \rho f)_r \cdot D_r}{\eta r \cdot h_r \cdot S_r} = 1 \text{ or}$$

$$(\rho s - \rho f)_r \cdot D_r \cdot \eta_r^{-1} \cdot h_r^{-1} \cdot S_r^{-1} = 1$$

only slight deviation from unity is allowed.

(5) Laminar sub layer criteria

For this the projections of grains outside the laminar sub-layer should be same for model and prototype or $\frac{\delta_r}{D_r} = 1$

Since $\delta = \frac{\nu}{U_*}$ Assuming $\nu_r = 1$ (fluid of same kinematic viscosity is used in model and prototype)

$$\delta \propto \frac{1}{\sqrt{R'b \cdot g \cdot s}} \quad \text{or} \quad \delta \propto \frac{1}{\eta^{\frac{1}{2}} \cdot h^{\frac{1}{2}} \cdot s^{\frac{1}{2}}}$$

$$D_r \cdot \eta_r^{\frac{1}{2}} \cdot h_r^{\frac{1}{2}} \cdot s_r^{\frac{1}{2}} = \Delta\delta \simeq 1 .$$

Slight deviation in value of $\Delta\delta$ from unity is allowed when the bed material is coarser than thickness of the laminar sub layer.

(6) Criteria of $\left(\frac{\text{Bed Load Rate}}{\text{Total load}}\right)$ Ratio

According to this the proportion of bed load should be same as compared to total load transported in model and prototype

$$\frac{q_{B_r}}{q_{T_r}} = \frac{1}{B}$$

or $q_{B_r} \cdot q_{T_r}^{-1} \cdot B = 1$

(7) Hydraulic time (t_1)

Hydraulic time is the time required to move a particle to a distance L with velocity $\bar{V}$ or $t_1 = \frac{L}{V}$

$$\text{or} \quad t_{1r} = \frac{L_r}{V_r} \quad \text{OR}$$

$$t_{1r} \cdot V_r \cdot L_r^{-1} = 1$$

(8) Sedimentation time (t_2)

The time t_2 is adopted for duration of individual flows, from this time scale, the hydrographs are run and with this time scale, corresponding sediment rates q_{T_r} fillup corresponding volumes of pockets. The equation is written for unit width

$$q_{T_r} \cdot t_{2r} = L_r \quad h_r \quad (\rho s - \rho f)_r \text{ OR}$$

$$q_{T_r} \cdot t_{2r} \cdot L_r^{-1} \cdot h_r^{-1} (\rho s - \rho f)_r^{-1} = 1$$

q_{T_r} is measured as submerged weight/unit time / unit width.

(9) Criterion of supplementary slope of Tilting the bed

When tilting is adopted, the vertical exeggerations is different from slope exeggeration but the ratio of two should be near about unity.

$$\frac{S_r}{h_r/L_r} = \Delta N \qquad \text{or}$$

$$S_r \cdot L_r . h_r^{-1} = \Delta N \simeq 1$$

Since no tilt is allowed in tidal model $\Delta N = 1$ exactly.

The Nine conditional equations have been given in Table 1

Einstein & Chien have given tables of exponents of scale ratio with the following ratios chosen

(i) L_r chosen

(ii) h_r chosen

(iii) $(\rho_s - \rho_f)_r$ sediment density chosen

these have been given ~~in appendix.~~ in tables 2, 3 & 4 at the end of chapter

The manner in which Einstein's equations are used for fixing scales of a river model is clearly depicted from the example given in appendix.

(C) Komura's simplified equations[8]

4.3 Komura in 1962 gave his own set of equations for designing the scales of movable bed model. He did not use Einstein's bed load theory. The advantage of Komura's equations are that two quantities can be chosen against only one in the Einstein's method. The equations are also similar, as the exponents are given as fractions. The equations are given below :

1. $S_r = L_r^{-1/4} \cdot d_r^{-1/4} \cdot S^{-1}$
2. $U_r = L_r^{3/8} \cdot d_r^{3/8} \cdot \beta^{-1/2}$
3. $t_{1r} = L_r^{5/8} \cdot d_r^{-1/8} \cdot \beta^{1/2}$
4. $q_{Tr} = L_r^{5/4} \cdot d_r^{1/4} \cdot \beta^{-5/2} (\rho_s - \rho_f)_r^{-2}$
5. $D_r = L_r^{3/4} \cdot d_r^{1/4} \cdot \beta^{-1}$.
6. $t_{2r} = L_r^{1/2} \cdot \beta^{3/2} (\rho_s - \rho_f)_r^{2}$
7. $Q_r = L_r^{17/8} \cdot d_r^{3/8} \cdot \beta^{-3/2}$

In these equations 'β' is the magnification ratio such that $\beta = 1 + (2 \times \alpha \frac{D_p}{B_p})$ where 'α' is the vertical exeggeration.

In the first instance the value of α is assumed and β is obtained. For better accuracy β value is verified.

The procedure of fixing scale ratios by Komura's method. is clearly exhibited from the example given in appendix.

(D) Lacey's Regime Theory

4.4 For a long time models have been designed as small replica of rivers based on the classical regime equations put forth by Gerald. Lacey. These are given below :

(1) $P = 2.67\ Q^{\frac{1}{2}}$

(2) $R = 0.4725\ \left(\frac{Q}{f}\right)^{1/3}$

(3) $\frac{P}{R} = 56.5\ \ Q^{1/6} \cdot f^{1/3}$ for equivalent bed movement

(4) $S = 0.000547\ f_m^{1/3}\ \frac{f_r^{4/3}}{Q^{1/6}} = \frac{f^{5/3}}{1788\ Q^{1/6}}$

Since same length scale is used for length as well as the breadth of the river, from equation (1) $L_r = Q_r^{1/2}$ or $Q_r = L_r^{\ 2}$

But the relation given above holds good for the meandering reach of the river as the meander length $M_L = Q^{\frac{1}{2}}$

For straight river course $Q = A.V = B.D\sqrt{D}$ or $Q_r = L_r\ D_r^{\ 3/2}$ models are so prepared that $D_r^{3/2} = L_r$ or Depth scale $D_r = L_r^{2/3}$ This gives vertical exeggeration $V.E. = L_r^{1/3}$

The method of fixing scale ratios for a river model by Lacey's regime equations is explained in the example given in appendix. In 1958 Lacey proposed a correction factor for supplementary slope according to it $S_r = \frac{S_m}{S_p} = \frac{E_m}{E_p}(V.E.)$ Where $E = \frac{P}{W_s} = \frac{D_m}{R} \geqslant 1$ here P is wetted perimeter, W_s is width at water surface, D_m is mean depth and R is the hydraulic mean radius. Although $E_p = 1$ for a wide and shallow river but in Model E_m exceeds 1. In the upper reaches of river higher $\left(\frac{E_m}{E_p}\right)$ ratio can be used but for lower reaches close to estuary this ratio should be unity.

(E) Thomas Blench's modified regime equations [9]

4.5 Thomas Blench introduced the effect of bed and side factors and proposed his modifications over Lacey's regime equations. Blench's equation for design of Canals with large bed load charge are as under

(1) $B = \frac{(F_B)^{1/2}}{(F_S)^{1/2}} Q^{1/2}$ where B is the average width

(2) $D = \left(\frac{F_S}{F_B^{\,2}} Q \right)^{1/3}$

(3) $S = \frac{F_B^{5/6} \quad F_S^{1/12}}{(3.63\ g/S^{1/4})\ Q^{1/6} (1 + \frac{C}{233})}$

(4) $F_B = F_{Bo} \ (1 + 0.12\ C)$ where F_{Bo} is Lacey's silt factor and F_S is the side factor and equal to $\frac{V^2}{D} = \frac{q^2}{D^3}$ where q is the discharging intensity of the river. Thus

$$F_S = Q^2 / B^2 D^3 = \frac{V^2}{B}$$

His equations also give the same scale ratios as obtained by Lacey's regime equations viz.

(i) $L_r = Q_r^{\frac{1}{2}}$

(ii) $D_r = Q_r^{1/3}$

(iii) $S_r = Q_r^{-1/6}$

(F) E.W.Bijker's approach for determining scale ratios [10]

4.6 In order to simulate river conditions in a model the two important conditions that should be satisfied are

(1) Current directions should be reproduced

(2) The amount of bed load transport should be reproduced

The scale laws are derived from Dynamic equations of motion in the direction of flow and perpendicular to the flow direction.

$$\frac{\delta V^2/2}{\delta s} + g \frac{\delta z}{\delta s} + g \frac{V^2}{c^2 y} = 0 \quad \text{Eq. 1}$$

$$\frac{V^2}{r} + g \frac{\delta z}{\delta p} = 0 \quad \text{Eq. 2}$$

Expressing in terms of scale ratios which are reverse of the normal it is obtained from equations 1 and 2

$$\frac{V_r^2}{L_r} = \frac{Z_r}{L_r} = \frac{V_r^2}{C_r^2 \cdot Y_r} \text{ in flow dir}^n \qquad \text{Eq. 3.}$$

$$\frac{V_r^2}{L'_r} = \frac{Z_r}{L_r} \text{ in transverse dir}^n \qquad \text{Eq. 4.}$$

Thus we have from equations 3 and 4

$$\frac{Z_r}{L_r} = \frac{V_r^2}{L_r} = \frac{V_r^2}{C_r^2 Y_r} = \frac{V_r^2}{L'_r} \quad \text{or} \quad V_r^2 = Z_r.$$

Here Z_r pertains to reproduction of water levels and not water depths. Also $V_r^2 = \dfrac{C_r^2 \cdot Y_r \cdot Z_r}{L_r}$, where Z_r/L_r is slope scale or replacing Z_r/L_r by I_r, we have the following relation.

$V_r^2 = C_r^2 \cdot Y_r \cdot I_r$. This can also be deduced from Chazey's formula.

For reproducing current directions $L_r = L'_r$ since $L'_r = C_r^2 \cdot Y_r$, it is evident that the scale of radius of curvature is affected by roughness scale. From equation(3) $C_r^2 = L_r/ Y_r$ but this condition may or may not be satisfied when $C_r^2 \lessgtr L_r/Y_r$

In these cases the model is either too smooth or too rough and adjustment is required by providing artificial roughness. When the model is too smooth it would show large radius of curvature and when too rough small radius of curvature would be there. It also become evident that the scale of roughness is independent of velocity scale which only affects the slope scale Z_r / L_r.

The slope scale may be different from vertical exeggerations Y_r / L_r but usually $\dfrac{Z_r}{L_r} \simeq \dfrac{Y_r}{L_r}$. The effect of different scales ratios for slope and depth are that the water level changes due to velocity variations are not reproduced on depth scale which results in too large depths on upstream side and too small depth downstream of a reach. When tilting is adopted by giving supplementary slope

$$I_m = I_p \left(\frac{L_r}{Z_r} - \frac{L_r}{Y_r} \right)$$

The scale of velocity governs the bed configuration and affects bed load movement. The scale of bed load transport T_r hence should remain constant throughout the entire reach of the river and should be independent of velocity and depth variations.

The normal form of the formula representing the bed load is $\dfrac{T}{F'(\Delta d^{3/2} g^{1/2})} = F''\left(\dfrac{\Delta d}{\mu Y I}\right)$ in which the terms are

T = transport in m^2 /sec per unit of time and unit of width

$\Delta = \dfrac{\rho_s - \rho_f}{\rho_f}$

ρ_s = density of grain

ρ_f = density of water

Y = water depth

I = gradient of the energy head, or in the case of uniform flow, the slope of the water level

μ = a ripple coefficient showing what percentage of bed stress is due to grains and what percentage is due to ripples

$$\mu = \left(\frac{C_{bottom}}{18 \log 12 \frac{h}{d_{50}}} \right)^{3/2}$$

The function $F'(\Delta^{1/2} g^{1/2} d)$ is of simple nature but F'' is complex and it should be assumed that

$$F''\left(\frac{\Delta d}{\mu Y I}\right)_m = F''\left(\frac{\Delta d}{\mu Y I}\right)_p$$

Thus we get $(\Delta d)_r = \mu_v . Y_v . I_v$ or $Y_r \, I_r = \dfrac{(\Delta d)_v}{\mu_v}$ From above relations

$T_r = F'(\Delta^{1/2} . d^{3/2} . g^{1/2})$ we get from this

$V_r^2 = C_r^2 . Y_r . I_r = C_r^2 . \dfrac{(\Delta d)_v}{\mu_v}$

Time scale $t_r = L_r . Y_r / T_r = \dfrac{\text{Area scale}}{\text{Intensity of transport}}$

This time scale should checked by means of a case history study in the model. The procedure of fixing scale ratio is explained from the example given in appendix.

(G) Scale ratios to be finally adopted for river models

4.7 In the foregoing paras different approaches for fixing scale ratios for river model have been presented. These ~~add~~ at best serve as initial guide lines. However the scale ratios initially chosen for model design need to be modified according to similation produced while running and testing the model for verification of past events

EXPONENTS OF RATIOS FOR MODEL LAWS FOR RIVER MODELS WITH SEDIMENT MOTION

	Scale Ratio											Δ Values							
Eqn	L_r	h_r	V_r	S_r	D_r	$(\rho s - \rho f)$	q_{B_r}	q_{T_r}	t_{1_r}	t_{2_r}	B	C_r	ηr	ΔV	ΔF	Δd	ΔN		
1	-	-(1+2m)	2	-1	2m	-	-	-	-	-	-	-2	-	-1	-	-	-	=	1
2	-	-1	2	-	-	-	-	-	-	-	-	-	-	-	-2	-	-	=	1
3	-	-	-	-	-3	-3	2	-	-	-	-	-	-	-	-	-	-	=	1
4	-	-1	-	-1	1	1	-	-	-	-	-	-	-1	-	-	-	-	=	1
5	-	1	-	1	2	-	-	-	-	-	-	-	1	-	-	-2	-	=	1
6	-	-	-	-	-	-	1	-1	-	-	1	-	-	-	-	-	-	=	1
7	-1	-	1	-	-	-	-	-	1	-	-	-	-	-	-	-	-	=	1
8	-1	-1	-	-	-	-1	-	1	-	1	-	-	-	-	-	-	-	=	1
9	1	-1	-	1	-	-	-	-	-	-	-	-	-	-	-	-	-	=	1

Note: The values of B, C_r, η_r & m are determined for auxiliary computations. Δ Values may be chosen to suite the conditio

MODEL RATIOS FOR OPEN CHANNEL FLOWS WITH SEDIMENT MOTION
LENGTH RATIO, L_m CHOSEN

Symbol	h_r	L_r	$(\rho_s - \rho_f)_r$	C_r	η_r	B	ΔF	$\Delta\delta$	ΔN	ΔV
h_r	..	$\frac{m+1}{4m+1}$	..	$\frac{-2}{4m+1}$	$\frac{-m}{4m+1}$	..	$\frac{2}{4m+1}$	$\frac{2m}{4m+1}$	$\frac{-(m+1)}{4m+1}$	$\frac{-1}{4m+1}$
V_r	..	$\frac{m+1}{2(4m+1)}$	..	$\frac{-1}{4m+1}$	$\frac{-m}{2(4m+1)}$	..	$\frac{2(2m+1)}{4m+1}$	$\frac{m}{4m+1}$	$\frac{-(m+1)}{2(4m+1)}$	$\frac{-1}{2(4m+1)}$
S_r	..	$\frac{-3m}{4m+1}$	..	$\frac{-2}{4m+1}$	$\frac{-m}{4m+1}$	..	$\frac{2}{4m+1}$	$\frac{2m}{4m+1}$	$\frac{3m}{4m+1}$	$\frac{-1}{4m+1}$
D_r	..	$\frac{2m-1}{2(4m+1)}$	..	$\frac{2}{4m+1}$	$\frac{-(2m+1)}{2(4m+1)}$	..	$\frac{-2}{4m+1}$	$\frac{2m+1}{4m+1}$	$\frac{1-2m}{2(4m+1)}$	$\frac{1}{4m+1}$
$(\rho_s - \rho_f)_v$	..	$\frac{3(1-2m)}{2(4m+1)}$	..	$\frac{-6}{4m+1}$	$\frac{3(2m+1)}{2(4m+1)}$	..	$\frac{6}{4m+1}$	$\frac{2m-1}{4m+1}$	$\frac{3(2m-1)}{2(4m+1)}$	$\frac{-3}{4m+1}$
q_{B_r}	..	$\frac{3(1-2m)}{2(4m+1)}$	..	$\frac{-6}{4m+1}$	$\frac{3(2m+1)}{2(4m+1)}$	..	$\frac{6}{4m+1}$	$\frac{6m}{4m+1}$	$\frac{3(2m-1)}{2(4m+1)}$	$\frac{-3}{4m+1}$
q_{T_r}	..	$\frac{3(1-2m)}{2(4m+1)}$	..	$\frac{-6}{4m+1}$	$\frac{3(2m+1)}{2(4m+1)}$	1	$\frac{6}{4m+1}$	$\frac{6m}{4m+1}$	$\frac{3(2m-1)}{2(4m+1)}$	$\frac{-3}{4m+1}$
t_{1_r}	..	$\frac{7m+1}{2(4m+1)}$	..	$\frac{1}{4m+1}$	$\frac{m}{2(4m+1)}$	..	$\frac{-2(2m+1)}{4m+1}$	$\frac{-m}{4m+1}$	$\frac{m+1}{2(4m+1)}$	$\frac{1}{2(4m+1)}$
t_{2_r}	..	$\frac{5m+2}{4m+1}$	..	$\frac{-2}{4m+1}$	$\frac{-m}{4m+1}$	-1	$\frac{2}{4m+1}$	$\frac{-(2m+1)}{4m+1}$	$\frac{-(m+1)}{4m+1}$	$\frac{-1}{(4m+1)}$

DEPTH RATIO h_r CHOSEN

Symbol	h_r	L_r	$(\rho_s - \rho_f)_r$	C_r	η_r	B	ΔF	$\Delta\delta$	ΔN	ΔV
L_r	$\frac{4m+1}{m+1}$	-	-	$\frac{2}{m+1}$	$\frac{m}{m+1}$	-	$\frac{-2}{m+1}$	$\frac{-2m}{m+1}$	1	$\frac{1}{m+1}$
V_r	$\frac{1}{2}$	-	-	-	-	-	1	-	-	-
S_r	$\frac{-3m}{m+1}$	-	-	$\frac{-2}{m+1}$	$\frac{-m}{m+1}$	-	$\frac{2}{m+1}$	$\frac{2m}{m+1}$	-	$\frac{-1}{m+1}$
D_r	$\frac{2m-1}{2(m+1)}$	-	-	$\frac{1}{m+1}$	$\frac{-1}{2(m+1)}$	-	$\frac{-1}{m+1}$	$\frac{1}{m+1}$	-	$\frac{1}{2(m+1)}$
$(\rho_s - \rho_f)_r$	$\frac{3(1-2m)}{2(m+1)}$	-	-	$\frac{-3}{m+1}$	$\frac{3}{2(m+1)}$	-	$\frac{3}{(m+1)}$	$\frac{2m-1}{(m+1)}$	-	$\frac{-3}{2(m+1)}$
q_{B_r}	$\frac{3(1-2m)}{2(m+1)}$	-	-	$\frac{-3}{m+1}$	$\frac{3}{2(m+1)}$	-	$\frac{3}{m+1}$	$\frac{3m}{m+1}$	-	$\frac{-3}{2(m+1)}$
q_{T_r}	$\frac{3(1-2m)}{2(m+1)}$	-	-	$\frac{-3}{m+1}$	$\frac{3}{2(m+1)}$	1	$\frac{3}{m+1}$	$\frac{3m}{m+1}$	-	$\frac{-3}{2(m+1)}$
t_{1_r}	$\frac{7m+1}{2(m+1)}$	-	-	$\frac{2}{m+1}$	$\frac{m}{m+1}$	-	$\frac{-(m+3)}{m+1}$	$\frac{-2m}{m+1}$	1	$\frac{1}{m+1}$
t_{2_r}	$\frac{5m+2}{(m+1)}$	-	-	$\frac{2}{m+1}$	$\frac{m}{m+1}$	-1	$\frac{-2}{m+1}$	$-\frac{(3m+1)}{m+1}$	1	$\frac{1}{m+1}$

SEDIMENT-DENSITY RATIO

Symbol	h_r	L_r	$(\rho_s - \rho_f)_r$	C_r	η_r	B	ΔF	$\Delta \int$	ΔN	ΔV
L_r	-	-	$\frac{-2(4m + 1)}{3(2m - 1)}$	$\frac{-4}{2m - 1}$	$\frac{2m + 1}{2m - 1}$	-	$\frac{4}{2m - 1}$	$\frac{2}{3}$	1	$\frac{-2}{2m - 1}$
h_r	-	-	$\frac{-2(m + 1)}{3(2m - 1)}$	$\frac{-2}{2m - 1}$	$\frac{1}{2m - 1}$	-	$\frac{2}{2m - 1}$	$\frac{2}{3}$	-	$\frac{-1}{2m - 1}$
V_r	-	-	$\frac{-(m + 1)}{3(2m - 1)}$	$\frac{-1}{2m - 1}$	$\frac{1}{2(2m - 1)}$	-	$\frac{2m}{2m - 1}$	$\frac{1}{3}$	-	$\frac{-1}{2(2m - 1)}$
S_r	-	-	$\frac{2m}{2m - 1}$	$\frac{2}{2m - 1}$	$\frac{-2m}{2m - 1}$	-	$\frac{-2}{2m - 1}$	-	-	$\frac{1}{2m - 1}$
D_r	-	-	$-\frac{1}{3}$	-	-	-	-	$\frac{2}{3}$	-	-
q_{B_r}	-	-	1	-	-	-	-	1	-	-
q_{T_r}	-	-	1	-	-	1	-	1	-	-
t_{1_r}	-	-	$\frac{-(m + 1)}{3(2m - 1)}$	$\frac{-3}{2m - 1}$	$\frac{4m + 1}{2(2m - 1)}$	-	$\frac{2(2-m)}{2m - 1}$	$\frac{1}{3}$	1	$\frac{-3}{2(2m - 1)}$
t_{2_r}	-	-	$\frac{-2(5m + 2)}{3(2m - 1)}$	$\frac{-6}{2m - 1}$	$\frac{2(m + 1)}{2m - 1}$	-1	$\frac{6}{2m - 1}$	$\frac{1}{3}$	1	$\frac{-3}{2m - 1}$

CHAPTER V

SCALE RATIOS ADOPTED FOR SOME RIVER MODELS AND COMMENTS

(A) SCOPE OF THE CASES STUDIED

(B) TRAINING GREAT GANDAK RIVER FOR PROTECTION OF 'B' BUND AND WESTERN GANDAK CANAL

(C) SITING OF A ROAD BRIDGE ACROSS GANGA RIVER NEAR PATNA TOWN

(D) TRAINING OF RAPTI RIVER FOR PROTECTION OF MOLONY BUND

(E) TRAINING OF GANGA RIVER NEAR RISHIKESH FOR WATER SUPPLY OF ANTI-BIOTIC PLANT OF ALL INDIA PHARMACEUTICALS LIMITED

(F) CRITICAL REVIEW OF SCALES ADOPTED FOR RIVER MODELS STUDIED

(G) UNDESIRABLE EFFECTS PRODUCED WITH LARGE DISTORTION ADOPTED IN RIVER MODELS

CHAPTER V - SCALE RATIOS ADOPTED FOR SOME RIVER MODELS AND COMMENTS

(A) Scope of the cases studied

5.1 A few River Problems that were being studied at Field Research Station Bahadrabad and in the Hydraulics Laboratory of Civil Engineering Department, University of Roorkee were studied in details for getting a clear picture of procedure adopted for fixation of scale ratios, adjustments, verifications and the procedure of experimentation with different proposals for solving the river problems. These case studies are briefly described here under and comments offered on each of them. A general note on limitations of Distorted models is also given at the end.

(B) Training Great Gandak River for protection of 'B' Bund[11] and Western Gandak Canal

5.2 The problem is one of flood protection of an area behind Bund 'B' in Gorakhpur District from the dangerous embayment of the Great Gandak on the right. It also threatened to breach the Western Gandak Canal. Since River Training in the upper reach has the effect of transferring the attack lower down, the protection of Naraini - Chitauni flood embankment also became a part of the study.

The river reach in question is just below the Bhainsalotan Barrage across Gandak, hence the maximum flood discharge of 8.5 lacs Cfs was adopted for this investigation for getting the dominant flood discharge of 6.0 lacs cfs. The hydrograph of the year 1960 was modified for being used in the model and is indicated in ~~appendix.~~ PLATE 2. River cross sections at about 12 places on the river were taken to determine the water surface slope. Curves of water levels Vs $AR^{2/3}$ were plotted for these cross sections, and for the same discharge, the vertical intercepts gave the water surface slope. The slope is very steep in boulder reach from Barrage site to Patharwa head, about 4'-5' / mile and after that the river enters alluvial reach and slope is 1.6'/mile.

A river model representing 27 miles reach of river from 1 mile upstream of barrage site to 2 miles D/S of tail of Chitauni bund was constructed. It was moulded to post 1965 flood survey. The scales adopted were on basis of past experience.

Horizontal scale $= \frac{1}{300}$

Vertical scale $= \frac{1}{40}$

Vertical Exeggeration = 7.5

Discharge scale $= \frac{1}{76,000}$

Since the river reach does not show defined meanders, the discharge scale ratio $Q_r = L_r \; D_r^{3/2}$. The bed material in the Boulder reach upto Patharwa head was moulded in shingle of size $\frac{1}{8}'' - \frac{3}{8}''$, the alluvial reach was laid with Ranipur sand of grain size 0.36 m.m corresponding to river sand.

The verification of the model was accepted when gauges at various cross sections tallied within 1' of prototype gauges. In order to reproduce the curvature of flow, some islands which spilled at about 4 lacs cfs had to be raised and turfed. Adequate bed movement was noticeable in the model. Since the data of sediment transport at various discharges was not known, arbitrarily sand was injected in the model at Genughat at the rate of 1 cuft/ cusec / hr. for discharges above 33% of the dominant discharge. The entire modified hydrograph was run for 8 hours. In this period about 32 gauges were run each for 15 minutes. The highest (peak) discharge was run for only 7½ minutes. After the running was over, the bed contours were surveyed and contour plotted at 5 ft interval. The determination of surface flow lines and bed flow lines was done by running the hydrograph over the model on the following day. From the hydrograph it is visible that time scale adopted in this case is $T_r = \frac{8}{31 \times 24} = \frac{1}{96}$.

On the basis of past experience with river models, only

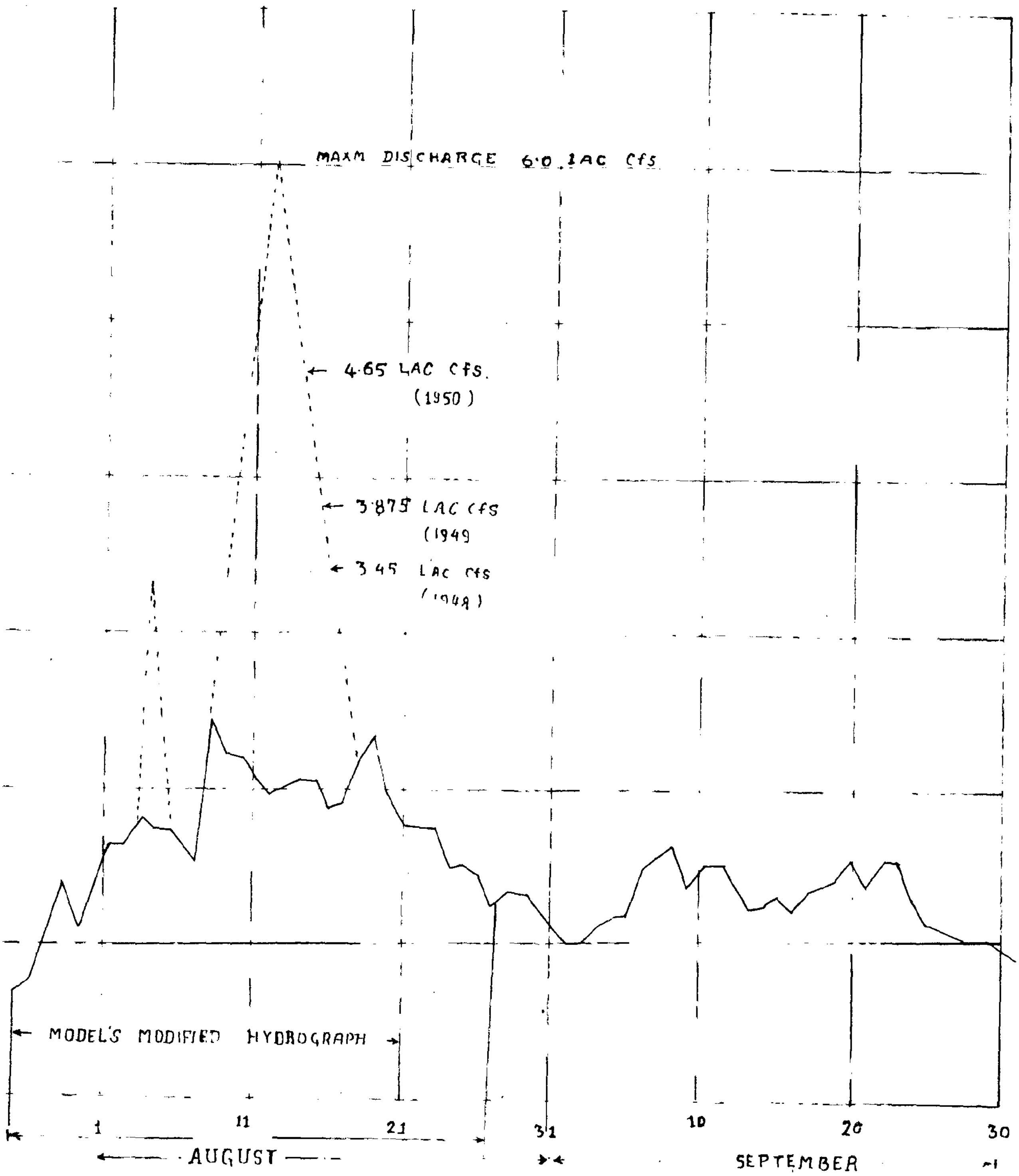


Fig 2. GANDAK RIVER TRAINING

1960 MODIFIED FLOOD HYDROGRAPH

SCALES :- HOR · 1" = 10 DAYS
VER · 1" = 1 LAC Cfs

five experiments were considered necessary for obtaining satisfactory solution of the problem and recommendations were made by the research station accordingly.

Reach to be protected	Location of Spurs	Length of Spur
(1) For Bund 'B'	(i) At M 3-5-150	450'
	(ii) M 3-6-640	250'
	(iii) M 4-0-520	575'
(2) For W.Gandak Canal between M 23-27.4	(i) M 24-3-50	3725'
	(ii) M 25-1-270	6700'
	(iii) M 25-7-0	10100'
(3) For lands below M 10 of Chitauni Bund and behind Railway Embankment	(i) At 15450' From Chitauni Railway Station	7700'
	(ii) At 10100' From Chitauni Railway Station	4450'
	(iii) At 5080' From Chitauni Railway Station	3740'
	(iv) At 620' From Chitauni Railway Station	2750'

In this, river training at a band has been done generally by two spurs, one located at the exit of the curve and the other at the entrance point. At times a short spur in the middle has been provided for river training in a ~~flood~~ flat bend.

<u>Comments</u> - It is observed that at Irrigation Research Institute Roorkee the vertical exeggeration adopted in case of this model is comparatively greater. From Lacey's criteria V.E. should be about 6.6 as against 7.5 adopted. At Poona for Hasdeo river model where same length scale ratio was adopted V.E. = 5 was adopted by chosing $L_r = 1/300$ and $Y_r = 1/60$. In both the cases the dominant discharge is of the same order. The advantage of high exeggeration however is that the model discharge is reduced significantly as would be evident from the Table given below by Lacey. Also the depth in model is adequate for measurement and observation of bed movement. But with high V.E. velocity distribution is greatly altered and great care is required while interpreting the results from model study.

TABLE OF DISCHARGE SCALE RATIO AND VERTICAL EXEGGERATION

Qr	V.E.	Qr	V.E.
$\frac{1}{1000}$	3.162	$\frac{1}{50,000}$	6.070
$\frac{1}{2000}$	3.550	$\frac{1}{100,000}$	6.813
$\frac{1}{5000}$	4.135	$\frac{1}{200,000}$	7.639
$\frac{1}{10000}$	4.641	$\frac{1}{500,000}$	8.909
$\frac{1}{15000}$	5.170	$\frac{1}{1000,000}$	10.000
$\frac{1}{20000}$	5.210	$\frac{1}{2000,000}$	11.225

For a dominant discharge of 6 lacs cfs wikh V.E. = 5.210 the model discharge Q_m = 30 cfs but if V.E. = 6.07 is adopted then the model discharge reduces to 12 cfs only. Also if the prototype river has side slope $\frac{6H}{1V}$ the side slope in the model would be $\frac{1H}{1V}$ Usually model discharges of the order of 10-15 cusecs are safely manageable by the staff but too small discharges have low transporting power with regard to bed movement. Very high discharges need great precaution in handling and more number of persons are required.

(C) Siting of a road bridge across Ganga river near Patna[11]

5.3 The problem consists of choosing the best location for the Bridge across Ganga out of four alternative sites by means of Model Studies.

From the History of the project investigation of the River courses in past, 2nd and 4th sites were better in the sense that the 2nd site provides a natural control point at 0.4 M_L upstream of the site at Kasmar and the 4th site proposed by Shri R.S.Chaturvedi,Retd. Head of the Civil Engineering Department of the University of Roorkee, lies in the nodal zone where all courses join and there is no problem of river shifting away from the Bridge site. Although the site is below the junction of Gandak river which emerges from fixed and rigid boundaries, the entrance curvature effects below the confluence provide a favourable approach for the bridge.

For the 2nd site the discharges of Ganga, Sone and Gandak were taken as 17.17 lacs, 2.12 lacs and 1.96 lacs cusecs and for the 4th site the discharges were taken as 17.2 lacs, 2.8 lacs and 8.5 lacs cusecs for model studies.

The model was laid representing a reach of Ganga for 23 miles upstream of the site and 12 miles downstream. The tributaries Sone and Gandak were represented upto one mile upstream of their confluence with Ganga. The model was laid to the post 1962 flood survey and the scales adopted for the Model were

Horizontal scale	$\frac{1}{500}$
Vertical scale	$\frac{1}{50}$
Vertical exeggeration	10
Discharge scale	$\frac{1}{177,000}$
Time scale	$\frac{1}{192}$.

The model was prepared and adjusted by making local changes till the verification was complete. The model was run with modified hydrograph for 2nd site with maximum discharge of 20 lacs cusecs adopting 7000 ft waterway and the surface flow lines were obtained, also the velocity distribution across the Bridge location was found out. This showed that although the waterway is adequate, the velocities along the Patna Bank were higher by 2'/ sec which is undesirable because of the clayey bank formation. Also there was tendency of island formation in the middle of waterway.

For the 4th site proposed by Professor R.S.Chaturvedi different waterways were tried from 7000ft to 10000ft and using the maximum flood discharge of 25 lacs cusecs. From the findings of the different experiments it was revealed, that the waterway of 7000 ft was economical as-wel-as suitable from considerations of surface flow lines. Also there were no high velocities close to Patna bank. Although there was some scour in a few bridge spans, these were sufficiently away from the Patna bank. The bed levels near Patna bank remained unaltered, also bed levels near left guide bund revealed positive lowering of the bed. In order to get the adverse effect of the river loop, the model was moulded to flood survey of 1845 & 1881 and the whole series of experiments repeated.

The various experiments gave results in favour of selection of the 4th site which has been recommended due to its following merits.

(1) Patna bank is not eroded and remains intact

(2) The velocities along Patna bank are not high to cause any erosion of appreciable nature

(3) The site is located in the nodal reach after the confluence of Gandak with Ganga, which helps in the canalising of the flow through Bridge spans and there is no likely hood of adverse loop being formed behind the guide banks

(4) A 700 ft long guide bund is recommended so that approach to embankment is safe from adverse loop in Gandak or Ganga

<u>Comments</u> - The study is highly instructive as it takes into account the flow of tributary streams and their confluence with the main river. Model verification in such intricate cases of confluence is a long and irksome job and many adjustments are required to be made for reproducing the flow characteristics of prototype. Field authorities propose several changes in site of bridge due to considerations of other obligatory points which causes material variation of flood data and the research studies have to be prolonged.

In this model a very high vertical exeggeration has been has been adopted (V.E. = 10). From consideration of Lacey's criteria V.E. = 7.6 was sufficient for this study. In Kosi and Ganga river models where same length was adopted V.E. = 7.1 was provided at C.W. P.R.S. Poona by using vertical scale of 1/70. It is felt that such a high exeggeration is fraught with great changes in patterns of velocity distribution. Recourse should have been made to tilting of the model and using bed material of finer size to overcome the need of high vertical exeggeration.

(D) Training of Rapti river for protection of Molony bund [11]

5.4 Molony Bund has been constructed along the left bank of river Rapti, which is a meandering river. The bund also follows the zig-zag alignment according to the curvature of the river. 19 miles length of this bund protects Gorakhpur town. The bund is under heavy attack in reaches (1) M 5-4 to M 7-0, (2) M 10-5 to M 11-4 and (3) M 14-5 to M 15-2 because it is situated in meander belt.

The catchment area of Rapti at the upper end of the reach in question is 6796 sq.miles. The catchment of Rohini a tributary is 968 sq.miles. The flood discharge from Dicken's formula is 5.5 lacs cfs. Assuming that absorption in the flood ~~plane~~ plain as 3.22 lac cfs the balance flood discharge is 2.28 lacs cusecs. The gauge discharge data for a site at Birdghat are available for years 1959 & 1960 also H.F.L. attained during the years from 1961 to 1964 are known. Maximum discharge of 1.34 lacs cfs was experienced in 1962 and H.F.L. was 248.52. This data was used to extrapolate flood levels at the high flood of 2.28 lacs cfs and the H.F.L. comes to R.L. 251.0 . The water surface slope of the river is 0.4 ft/mile. Thus flood levels at representative cross sections were adopted for verification of the model. The river carries enormous silt load of its own which is further augmented due to tributary streams bringing additional silt load on account of their steep bed grade. The meander belt of river extends 7 - 8 miles in width at the upper end and about 3-4 miles at lower end. The river loops near Molony bund are dangerous as they have arc/chord ratio of 1.32, 1.9 and 5.0 which threatens a cut-off in high floods.

In order to find out the suitable river training measures, a mobile bed model of the River Rapti representing 20 miles reach was constructed. The following scales were adopted looking to the meandering characteristics of the river.

Horizontal scale	=	$\frac{1}{150}$
Vertical scale	=	$\frac{1}{30}$
Vertical exeggeration	=	5
Discharge scale	=	$\frac{1}{22500}$
Grain size scale	=	$\frac{0.18}{0.07}$

The model was laid according to post 1963 flood survey using by-pass sand of 0.18 m.m. dia since the prototype sand was very fine and could not be available in bulk quantity for model studies within economic leads. When hydrograph for 1964 with necessary modifications was run, it was found that banks spilled only at discharge of 70,000 cusecs and curvature of flow was not reproduced. The model was then adjusted by turfing the bank slopes and slightly raising the banks to prevent spilling upto a discharge of 1 lacs cfs. With these adjustments the gauges tallied within 1 ft of the prototype gauges. But with this modification also, the bed movement was not adequate, hence tilting of the model became necessary and the scales were revised as below :

Horizontal scale	=	$\frac{1}{150}$
Vertical scale	=	$\frac{1}{25}$
Vertical exeggeration	=	6
Slope exeggeration	=	8
Discharge scale	=	$\frac{1}{18,750}$
Time scale	=	$\frac{1}{144}$

The model was remoulded to 1964 post flood survey and it was run with discharges obtained from revised discharge scale. Sand was injected at the rate of 1 cu.ft/cfs/hr of model discharge to simulate bed movement arbitrarily. It was found that there was adequate bed movement and gauges tallied within one ft. of the prototype gauges. The curvature of flow was also reproduced as severe action took place in reaches between M 5-4 to M 7-0, M 11-0 to M 11-9 and M 14-3 to M 15-2. This was a sufficient and satisfactory verification of the model.

Various alternatives were tried for adequate protection of the river at the three loops. At each bend according to the curvature, two or three spurs of sufficient length were provided till the flow lines showed that the river flow was diverted away from the Molony bund. After the model was run for 9 hours, the River bed was surveyed and scour at the nose of spurs was measured and bed contours plotted. In all the cases the scour holes pointed the flow direction away from the bund.

On the basis of experimental investigations the research station gave the following recommendations

Curved reach	Details of Spur				
	Location	Shank Length	Kinked Nose	Angle	Posit[illegible]
1st curve (3.5°) in M 5-7	M 5-5-375	525'	100'	170°	At entrance
	M 6-3-100	840'	200'	160°	At ex[illegible]
2nd curve (7°) in M 10-9	M 11-3-510	900'	200'	140°	At ex[illegible]
3rd curve (11°) in M 15-2	M 15-0-150	600'	200'	170°	At ex[illegible]

Comments - This study was very interesting and instructive because the tilting was introduced in model. But as tilting introduces many other ill effects, it would have been better if crushed coal could be used for moulding the bed of this model, so that bed movement could could have been satisfactory even without tilting (V.E. = S.E)

In this case also higher vertical exeggerations has been adopted finally (V.E. = 6). Since the verification of the model was adequate and satisfactory, this slight increase in V.E. from 5 to 6 dose not matter. Dr. Joglekar[5] has rightly mentioned that the aim in river models should be to reproduce known accurrances of the prototype in the model by varying the discharge, sand charge etc. rather than trying to impose the theoritically worked out scale ratios on the model. In this study if the prototype data of silt load for various discharges were known then the corresponding amount of silt injection at appropriate points in the model would have given still better results.

(E) Training of Ganga River near Rishikesh[12] for water supply of Anti-Biotic Plant of All India Pharmaceuticals Limited

5.4 The River Ganga flows in boulder stage near Rishikesh and bed material consists of an admixture of sand to large boulders about 1-2 cft in size. From the point of intake at E, the river changes from an incised hill stream to a wide river with braided pattern. Since it has a tendency to meander, it needs to be trained so that it hugs to the right bank and the intake always gets water.

The flood discharge at Rishikesh is of the order of 4 lacs cfs. At high flows the water surface slope is .00240 and at low stages the slope is .00156. At different discharges the bed material moved has different sediment size and different rate of sediment transport. No prototype data regarding the sediment transport is available.

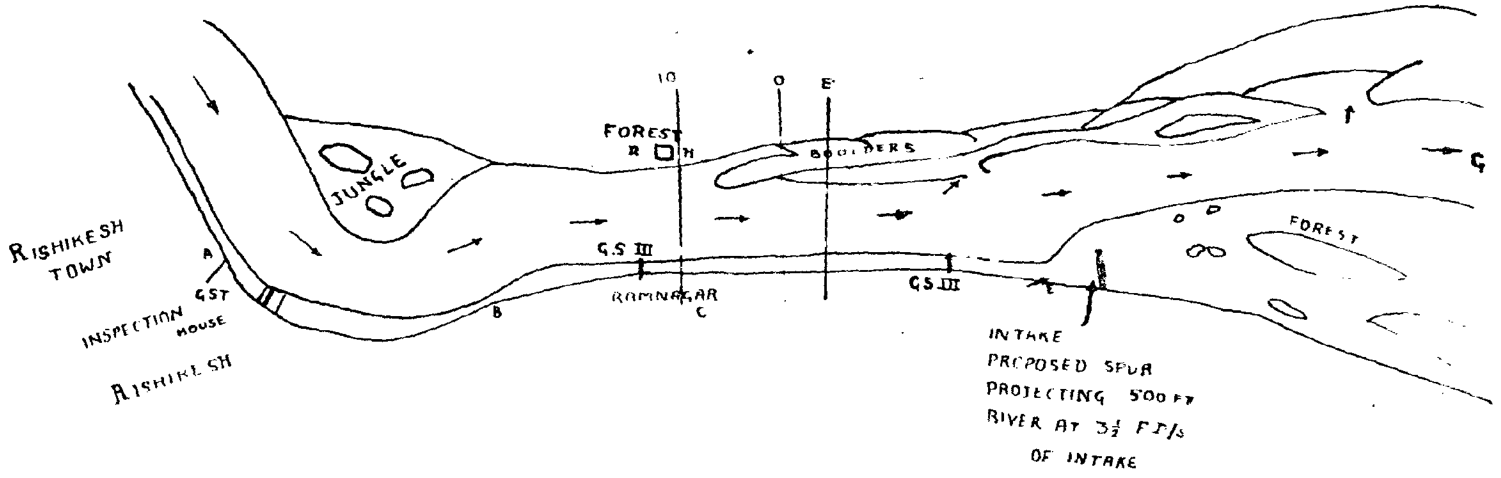
A mobile bed model for a reach of 4 miles of river was made from P.W.D. Inspection House at Rishikesh to gauge No.4. Since no prototype bed samples were available, the sediment size of prototype was determined from Lacey's formula

$$f = 192\, R^{2/3}\, S^{1/2} = 1.7 \sqrt{d}$$

This gives $$d = \frac{192^2\,(14.5)^{4/3} \cdot (.0024)}{1.7^2} = 23 \text{ M.M}$$

Also from combined Manning's and Strickler's equation we get d= 21 m.m

PLATE 3.



PLAN OF GANGA RIVER

Fig 3. TRAINING OF GANGA RIVER FOR INTAKE OF ANTI BIOTIC PLANT AT RISHIKESH

This has been obtained by putting $n = \frac{d^{1/6}}{29}$ in the equation

$$Q = \frac{1.49\ A\ R^{2/3}\ S^{1/2}}{n}$$

Thus it has been assumed that the prototype sediment was 24 m.m. in diameter, however, it was observed that huge boulders moved during high stage of the river.

The different scales of the model were fixed as under keeping in mind the space and discharge limitations

Horizontal scale	=	$\frac{1}{200}$
Vertical scale	=	$\frac{1}{40}$
Vertical exeggeration	=	5
Discharge scale		$\frac{1}{40,000}$
Sediment scale	=	$\frac{1}{4}$
Scale ratio of bed stress	=	$\frac{1}{8}$

Since no hydrograph was available for running in the model only five discharges were run corresponding to prototype discharges of 34,000 cfs, 57,500 cfs, 109,500 cfs, 162,500 cfs and 242,000 cfs. Four cross section were considered for taking the water surface slope and for finding out average area and wetted perimeter. For each discharge, the rate of bed movement was determined by collecting the bed material in samplers for 5 to 15 minutes and bed load transport rate was determined. Material was then injected at the same rate while running the model for verification.

Since all observations were made in the model with the use of scale ratios, the various prototype quantities were evaluated. The prototype and model gauges tallied closely and also the wetted perimeter obtained from Lacey's formula tallied with observed values. Q Vs R plot for model as-wel-as prototype was also coincident for higher discharges. The bed load rates tallied closely with that obtained from Einstein's equations. Thus the verification of the model was satisfactory and model was taken as proved.

The model experiments for river training considered the effects of location of one spur located on left bank at about 1.5 Km upstream of intake or alternatively the effect of a single spur on the right bank at about 0.8 Km downstream of intake. The various trials revealed that a 500 ft long spur inclined at 12° upstream from the normal to the bank gave most favourable results. This caused gradual shoaling in the channel EF and the Channel EG became active drawing about 75 % of the total discharge. This tendency would rule out the possibility of river shifting away from the right bank and leaving the intake high and dry.

Comments - The model study is very instructive as it has considered for the first time the simulation of bed load movement in the boulder reach of the river. In this, Einstein's equations of bed load movement have been used and the bed load transport ratio has been used in a quantitative manner. It is felt that if instead of running five arbitrary flood discharges in the model, a regular flood hydrograph would have been run then better results could have been achieved. For the model hydrograph, the flood history at a neighbouring site for bridge or barrage could have been used.

(F) Critical review of scales adopted for river models studied

5.6 The scale ratios fixed for model studies have been compared with the scale ratios obtained by Einstein's and Ning Chien's equations the comparison has been shown in table No. 7 . It has already been mentioned earlier that the different approaches for fixation of scales for model are only tentative and are modified till the model is proved by verification tests.

(From the case studies, it becomes evident that the degree of distortion adopted is greater for large prototypes viz. V.E. = 10 for Ganga river Bridge near Patna and V.E. = 5 for Ganga river at

Rishikesh . Also it would be noticeable that different length ratios have been selected for the models and L_r varies from 1/150 to 1/500 The depth scale ratio varies from 1/25 to 1/50. In all the cases studied the model material had the same specific gravity and the sand scale varies from 1/4 to 2.6. If light weight materials could be used such as crushed coal, then with lesser degree of distortion the same amount of bed movement could have been obtained.

As the prototype size increases, a smaller length scale becomes necessary because river characteristics change with the size. Depth scale fluctuation is not large because certain minimum depth of about 6 inches to 18 inches is required for observing the bed movement. A smaller depth scale would give very shallow depths and their would be shallow shore line effect causing unduly high resistence.

A distorted model needs much more exhaustive survey and field data for model verification than a geometrically similar model. In this several adjustments are required for similarity of current directions and bed movements, but the smallest feasible geometrically similar model would be prohibitively costly for construction and model studies.

(G) Undesirable effects produced with large distortion in river models

5.7 K.D.Nichols of U.S. Experimental Station Vicksburg had analysed different models and studied the effects of various degree of exeggeration. The range of length scale varied from 1/50 to 1/2000 and the depth scale varied from 1/50 to 1/200. He has found that because of distortion, sides play an important part in velocity distribution. When the velocity distribution changes, the energy distribution is also altered and tractive forces also changed due to distortion. All these have direct effect on the scour pattern which is also distorted. The discrepancy becomes more with larger discharges. As a result of distortion, excessive scour is produced at bends, noses of spurs, and in narrow sections and at junction of streams and at the head of islands. The side slopes becomes steeper in model and the banks need to be moulded as rigid because the material cannot stand at the steep angle which is greater than the natural angle of repose of the material. In such cases the scour of the banks cannot be correctly predicted. The curvature of the stream lines along and around structures such as guide bunds etc. is markedly different from prototype.

The primary aim of a distorted model should be to achieve hydraulic similarity in a specific region, even though overall similarity is not attained. The distortion should be kept as low as possible and vertical exeggeration should not be more than 6.

Other effects of distortion are

(i) Excessive Bar formation in wide reach

(ii) Displacement downstream and straightening of crossing

(iii) Eddy formation at point of sudden widening

(iv) Improper sand deposits in bends

(v) Improper flow distribution around an island due to one channel becoming active and the other deteriorating

Inspite of the numerous limitations of distorted river models outlined above, very satisfactory results have been obtained with such models within reasonable limits of distortion., and by judiciously interpreting the results of models.

TABLE 5

MODEL OBSERVATIONS

Run No	Q Cfs	A Sft	P ft	R ft	S	U ft/sec	q_b lb/sft	q_b lb/sec
1	1.50	0.65	4.0	0.162	0.00788	2.30	0.0028	0.0114
2	2.07	0.76	4.29	0.178	0.00860	2.72	0.00488	0.0209
3	2.80	0.89	4.86	0.184	0.01040	3.15	0.0117	0.057
4	4.00	1.13	5.66	0.210	0.01088	3.55	0.0565	0.315
5	7.26	1.82	6.50	0.280	0.01083	4.00	0.183	1.07

TABLE 6

VALUES OBTAINED FROM EINSTEIN'S EQUATIONS ETC.

Run No.	n	Lacey's P	ψ	ϕ	S_p	η	U_r	Q_r	Q_p (cfs)	P	Q_t	Q_f from model lbs/sec	Prototype Gauges from Model	Prototype Gauges from G/D Curves
1	.017	3.27	25.6	.00084	.00156	1.00	4.81	38,480	57,500	2.50	34,200	34,200	324.66	325.25
2	.016	3.85	21.1	.00146	.00184	1.08	5.01	40,080	34,000	2.60	45,200	45,200	325.25	325.70
3	.016	4.47	16.9	.00350	.03212	1.02	4.88	39,046	109,500	2.45	36,800	36,800	325.75	326.10
4	.015	5.34	14.1	.0169	.00241	1.11	5.07	40,560	162,500	1.67	49,800	49,800	326.55	326.70
5	.017	7.20	10.0	.0545	.00241	1.11	5.07	40,560	242,000	2.67	49,800	49,800	328.00	327.80

CRITICAL REVIEW OF SCALES ADOPTED FOR RIVER MODELS IN CASES STUDIED

S.No.	River	Scale Ratios & Distortion Adopted in Model						Scale Ratios from Einstein's Eqns*					Descripti of River Reach
		L_r	h_r	Q_r	D_r	V.E.	S.E.	h_r	S_r	D_r	V.E.	$(\rho_s - \rho_f)$	
1.	Gandak River for B Bund	$\frac{1}{300}$	$\frac{1}{40}$	$\frac{1}{76,000}$	1	7.5	7.5	$\frac{1}{52}$	5.5	$\frac{1}{3.14}$	5.78	31.0	River is i boulder stage upto Patharwa & then in alluvial stage.
2.	Ganga River near Patna	$\frac{1}{500}$	$\frac{1}{50}$	$\frac{1}{177,000}$	1	10	10	$\frac{1}{78}$	6.5	$\frac{1}{3.46}$	6.4	41.0	River in alluvial stage
3.	Rapti River for Milini Bund	$\frac{1}{150}$	$\frac{1}{25}$	$\frac{1}{18,750}$	2.6	6	8	$\frac{1}{33}$	4.5	$\frac{1}{2.72}$	4.54	20.4	River in alluvial stage carr ing very fine silt.
4.	Ganga River at Rishikesh	$\frac{1}{200}$	$\frac{1}{40}$	$\frac{1}{40,000}$**	$\frac{1}{4}$***	5	5	$\frac{1}{40}$	4.9	$\frac{1}{2.88}$	5	23.9	River is i boulder stage for most of t portion except bey offtake.

Note - * The ratios of Einstein's equations have been reversed for sake of comparison.
** The discharge scale was variable from 1/38480 to 1/40560.
*** Graded Material was used. This relation is approximate as prototype bed material size is not exactly known and indirectly determined.

CHAPTER VI

FIELD DATA REQUIRED FOR CONSTRUCTING A MOVABLE-BED RIVER MODEL

(A) GENERAL

(B) DETAILED PLAN OF THE RIVER COURSE

(C) LONGITUDINAL SECTION OF THE RIVER

(D) CROSS SECTIONS OF THE RIVER

(E) FLOOD DATA

(F) SILT AND SEDIMENT DATA

(G) INFORMATION OF BED AND BANK MATERIALS

(H) DETAILS OF PERMANENT STRUCTURES

(I) PAST HISTORY OF THE RIVER COURSE

(J) OTHER HYDROLOGICAL DATA

CHAPTER VI - FIELD DATA REQUIRED FOR CONSTRUCTING A MOVABLE BED RIVER MODEL [13]

(A) General

6.1 In order to conduct model studies pertaining to a river problem, affecting a certain reach of the river, it is of prime importance to collect preliminary field data regarding the river cross sections, gauge and discharge observations, historical data of previous floods and extent of damage and rectification measures adopted. This enables laying down an accurate model, which would simulate the conditions of flow through the river. The detailed survey data should include the following informations.

(B) Detailed plan of the river course

6.2 The survey data should start from a reach where the entrance conditions are stable. Ordinarily, the plan should include survey for two meander lengths up stream and one meander length down stream of the reach under investigation. The base line should be laid as far as possible in the centre of the river bed and its length and bearings should be accurately measured by prismatic compass. The high flood level marks should be shown on both banks. The base line should be tied down (for its subsequent location) by reference pillars fixed at known distances on left and right side of the base line preferably above H.F.L. The area needing protection should be clearly indicated on the plan and all properties and fields situated on both banks shown. Aerial survey plan be supplied if available.

(C) Longitudinal section of the river

6.3 The longitudinal section of the river taken along the base line should furnish levels at intervals of 100 ft and at every change in direction of base line. The observed high flood level should be marked on the longitudinal section giving the corresponding flood levels at each chain as far as possible. For a very long reach to be represented, the levels along the base line be furnished at intervals of 500 ft.

(D) Cross sections of the river

6.4 The cross section of the river should be taken at every chain and should be normal to the base line. Where oblique sections are taken, the angle between the base line and cross section line should be indicated. For large rivers, cross sections should be spaced at 1/4 to 1/2 the river width. Extra cross sections should however be taken at bends and confluence with tributaries. The cross section survey should furnish levels of points about 3' to 5' above the H.F.L.

(E) Flood Data

6.5 As far as possible, high flood levels with dates of occurances should be marked on all cross sections and the corresponding flood discharge indicated. If, however, the river has not been gauged in past for its discharges, the discharge should be approximately worked out by assuming suitable value of friction factor 'n' in Manning's formula. The need for correct evaluation of discharge passing a certain cross section cannot be over imphasised. No sooner a problem of river control or training is referred for model studies, gauging of prototype discharges should be started a-t a suitable site on systematic lines. Gauges should be installed at the point of commencement of the surveyed reach and about 1/3 mile below the last cross section surveyed for the river studies. At the site of investigation, two gauges should be installed on both banks and these gauges should be read at an interval of 3 hours during floods and once in a day at a predecided hour during non monsoon period. For flashy rivers, observations of gauges during the nights are very essential during flood season and special staff should be appointed for the same. Gauges should also be fixed above the junctions of the tributaries to assess their contribution of discharge s to the main river.

6.6 For establishing a gauge and discharge observation site on the river, a stable reach of river with almost permanent cross section (such as on rocky out crop sufficiently up stream of a steep drop in river) should be chosen. The river cross section should be surveyed every month and after the passage of a sizeable flood. Record of special features during flood such as Strong eddy formation, bank erosion, land slips etc. should be kept and extra-ordinary erosion due to scour at various places should be noted, if bed is exposed after flood has passed away.

(F) Silt and sediment data

6.7 It is essential that data regarding suspended silt charge and bed load during a flood be collected. During non monsoon months, fort-nightly silt and sediment charge should be determined by using bottle type samplers and bed load samplers. The percentage of different grades of materials should be determined by approved methods viz mechanical analysis and Hydrometer tests etc.

(G) Information of bed and bank materials

6.8 The nature of river bed (whether it consists of sand, erodible soil or otherwise rocky inerodible bed) should be clearly indicated. If the bed shows varied features, there extent should be clearly indicated in each cross section as-well as in the plan. Some trial pits should be taken in river bed to reveal the bed-formations, as this would be useful in designing the protective measures later on. Wherever exposed rock is found in river bed, the nature and type of rock should be indicated (such as sheet rock, fractured rock, faulted rock zone etc).

6.9 Similarly the type of material found in the banks should be precisely described such as highly erodible, semi-erodible, scour resistent. Sometimes due to river action, the banks reveal the profile of rock or soil formations in their natural sequence. This information should be clearly marked on the cross sections. Trial pits should be taken to clearly show the subsurface features when the information is not apparently visible.

(H) Details of permanent structures

6.10 Detailed information must be collected regarding the permanent structures constructed across the river. For cause ways, Road Bridges, Railway bridges, etc; the number of spans, waterway provided and highest water mark touched on these structures should be submitted alongwith their designs.

(I) Past history of the river

6.11 A detailed history of the river course should be collected from local enquiry, such as -

(i) information regarding highest flood levels observed at 5-6 locations along the stretch of river under study

(ii) changes in river course (shift of main current from one bank to other) and protective measures adopted by local people on small scale or by Government bodies on a large scale

(iii) whether the training and control measures adopted improved the conditions or further aggravated the situation or their ill-effects were transferred to some other locations.

(J) Other Hydrological data

6.12 This includes the catchment characteristics of the river which have an important bearing on surface run off, sub-surface flow and their contribution to total silt charge and discharge of the river. Information regarding artificial exploitation of water of the tributaries by different types of works should also be collected. Any commitments of tail water uses downstream in an integrated development of river or due to interstate sharing of waters should also be mentioned while providing the details information regarding the river data.

6.13 For important schemes, photographs of the river behaviour during the floods are very important. In case of erosion problems, the stage at which erosion is most pronounced should be noted. Also, lines of flow during specific flood stages (low,medium and high stages) should be photographed for the prototype. Plans of previous river surveys indicating the yearly changes in the course of river, should also be provided. All levels of the surveys should almost invariably be connected to G.T.S. bench marks.

CHAPTER VII

CONSTRUCTION OF A RIVER MODEL AND VERIFICATION OF THE MODEL

(A) ANALYSIS OF FIELD DATA

(B) CONSTRUCTION OF A MOBILE-BED RIVER MODEL

(C) PRELIMINARY CHECK-UP BEFORE RUNNING THE MODEL

(D) RUNNING OF THE MODEL AND VERIFICATION OF THE RIVER MODEL

(E) DISCREPANCY IN VERIFICATION, ITS CAUSES AND REMIDIES

CHAPTER VII - CONSTRUCTION OF A RIVER MODEL AND VERIFICATION OF MODEL

(A) Analysis of Field data

7.1 After the field data has been received, it is properly scrutinised and processed for its direct use in the model studies. A hydrograph for the floods that occurred at the prototype is prepared and approximate time scale is chosen to run the whole hydrograph in about 8-10 hours. The other scales of the river model are fixed from considerations mentioned in chapter IV. When all the features of the model are finalised, the construction of model is started.

(B) Construction of a Mobile bed river model (14)

7.2 A river model can be constructed in open field, where the soil is not susceptible to heavy swelling or shrinkage, due to rains and exposure and to sun. Loamy soils of Uttar Pradesh and Punjab are good and model can be moulded on them directly, after initial scraping of the soil. In black cotton soil tracts, such as in Madhya Pradesh and Andhra Pradesh, the river model needs to be moulded in a water tight trough so that there is no loss of water through cracks which develop in the soil.

7.3 A river model is moulded with the help of templates hung from carefully levelled upper edges of the side walls of trough. At times, wooden rails are fixed on posts on both sides of river model. The top of rails are at same elevation to form datum for vertical measurements. The inside walls of the trough are treated with bitumen or marine glue for water tightness. Over the bitumen, calico cloth is spread on the bed and side walls and hot iron press is passed over so that the bitumen impreguates the cloth. Three coats of bitumen and varnish are applied on calico. In India the Water tight trough is usually made with brick masonry in cement mortar of suitable thickness. It is plastered with rick cement mortar both inside and outside and the fillets are provided at junctions of widewalls with bed. The trough can also be made of welded or bolted steel plates clamped to angles spaced suitable or a pre-fabricated steel flume is used. A gentle slope is provided to the concrete floor of the trough to reduce the thickness of the bed material.

7.4 Templates for giving the bed profile are cut from good quality ply wood or sheet metal. The lower template is used for giving the bed profile up to inerodible bed and the upper template gives sufficient margin for moveable bed material to be placed between the lower and the upper templates. The thickness of the mobile bed to be provided should be corresponding to maximum scour observed in the river bed any where or 1½ times the scour depth obtained from Lacey's formula. The space between the adjecent bed templates is filled up with brick bats, quarry waste or clinker, gravel etc. leaving ½" thickness at top which is finished to profile with lean cement mortar, which is sprinkled with dry cement to give a cement crust $\frac{1''}{32}$ - $\frac{1''}{16}$ thick after sprinkling water on it.

7.5 At times, when river bed indicates infinite depth of alluvium, the moulding of river bed is done only with the help of one (upper) template which is cut to give the bed profile with the help of vertical and horizontal controls. These templates are out from sheet metal or masonite sheets. Ply wood is unsuitable, as the various plys peel off due to alternate wetting and drying which softens the binding glues. The templates are usually spaced not less than 6" nor more than 2' apart in the model. The bed can be moulded with 1:1 mixture of sand and clay up to 6" or 9" below the final bed profile. The upper layer is moulded with the mobile bed material of suitable grade. In order to represent correct entrance and exit conditions, the starting cross section and end cross sections are moulded rigidly with brick in mortar. The essential points for construction are, that the model should be flexible enough to incorporate any changes and modifications without requiring much rebuilding.

7.6 When the model is laid directly on the ground itself, then the loose top soil is removed and the tight soil excavated to give the required cross sections and bed slope etc. Extra excavation is required to keep minimum depth of movable bed material. The final bed profile is obtained by fixing pegs with their tops at elevations corresponding to levels at the points in prototype.

7.7 The banks are moulded rigidly or with the erodible material as in the prototype. When these are much steeper than the angle of repose, local adjustment is made while casting with mobile material or bank represented with Lumnite cement of weak strength.

7.8 Arrangements should be made for supplying, regulating and measuring exact quantity of water over a calibrated notch and waves should be damped by suitable stilling arrangements in the forebay before water enters the model. Also at the exit, tail water control arrangements should be provided to get proper depth on down stream side. Usually a Rehbock's weir or standard V-notch is used for discharge measurement. The head over the notch should be measured at a point situated sufficiently upstream of the notch proper. This eliminates the effect due to velocity of approach and effect of draw down in head just up stream of sharp crested weir. Also the frictional resistance of side walls is minimum. The jet of water falling over the gauging weir should be properly aerated by means of aeration pipes. The tail water control gates of different patterns can be adopted viz. sliding gates, venetian type controls or an adjustable tilting gate.

(C) Preliminary check up before running

7.9 After a model isfully ready for first running, once again a thorough check should be made to see that there are no possibilities of leaks and model structure is safe for running. Minor alternations which appear necessary should be implemented. All the measuring instruments should be checked to ensure that they are in good working order.

(D) Running of the Model, verification of model

7.10 Water should be gently led into the model so that it does not create any disturbance in bed and is gradually soaked up in the bed and sides. After this initial wetting of the bed, the accurate model discharge is passed in the model and proper tail water level is maintained. It should be observed whether the bed load movement and curvature of flow (indicated by attack at corresponding points) are simulated by model. This procedure is known as " Proving " of the model or " verification " of the model. Before a model can be relied to predict some future occurances, it should faithfully reproduce the events that have occurred at the prototype in past (for which data is available). Thus " verification " is an intricate cut and try process of adjusting hydraulic forces in the absence of complete dynamic similarity.

7.11 It is essential that the flow in the model at all times is turbulent viz. the Reynolds Number should be more that 4000. The U.S. Experiment Station at Vicksbur has laid down the criterion for turbulence by the following emperical formula. According to this the product of model velocity (in ft/ sec) and depth (in feet) should be more than 0.04 ($VD > 0.04$) This should be verified by actually observing the velocity of flow and depth of flow in the model.

7.12 A model is considered as proved if

(i) the corresponding stages give corresponding floods

(ii) the bed movement starts at corresponding discharges in model and prototype

(iii) the current directions and velocity distributions and reproduced and the flow in model remains turbulent for all studies

(E) Discrepancy in verification, causes and remedy

7.13 If some descripancy is found in the model verification the lacuna should be traced out. There might be some gross inconsistency in representing the data of the prototype in the model or information supplied by field staff might be wrong. Model may need redesigning by change of depth scale, discharge scale or slope scale. At times it may be required to change the size and specific gravity of the bed material to simulate bed movement. When the prototype data is wanting in respects of discharges (as regards its accuracy) or flood levels, simulation of curvature of flow and bed movements are sufficient indications to accept the fact that the model is proved. Results obtained from such models need to be interpretted with great caution (erring only on the safe side). At a later stage when complete and correct prototype data is obtained, the model should again be verified at the earliest opportunity so that the earlier recommendation given can be quickly modified by the Research Station before it is too late for the field authorises to incorporate the necessary changes in the prototype.

CHAPTER VIII

MODEL AND PROTOTYPE CONFORMITY

(A) GENERAL

(B) QUALITATIVE AND QUANTITATIVE CONFORMITY

(C) TYPICAL EXAMPLES OF GOOD MODEL-PROTOTYPE CONFORMITY IN HYDRAULIC STRUCTURES

(D) EXAMPLES OF CONFORMITY IN RIVER PROBLEMS

(E) CONCLUSION

CHAPTER VIII - MODEL AND PROTOTYPE CONFORMITY

(A) General

8.1 It is of considerable interest and importance to compare the behaviour of prototype with respect to the model indications. This comparision indicates as to what degree the model should be relied for quantitative results, even if the laws of similitude have been rigidly adhered to. Specially for the Hydraulic structures, comparision of the co-efficient of discharge, pressure distribution on the boundary surfaces, water surface profiles and erosion of river beds are of great value. These studies result in more efficient and economic designs of structure in future.

(B) Qualitative and Quantitative conformity[15]

8.2 There are two types of comparision possible. These are qualitative and quantitative. The former is easiest and most frequently made, this however has a limited value as direct transferrence to prototype requires judgement and extra safety-factor. A quantitative comparision is most valuable, as direct transfe-rence ~~form~~ from them is possible for evaluating the prototype dimensions.

(C) Typical examples of good Model-prototype conformity

8.3 In the following few paragraphs certain typical examples of Model and Prototype conformity are reproduced. These are for works constructed in India and in other foreign countries.

(i) Spillway coefficient for the Norris dam

The Norris dam across Tennessee river has a spillway about 200' high and has 3 spans of 100' each. 14' high drum gates have been provided for flood moderation. Eight conduits provided in the dam, have a discharging capacity of 37,000 cusecs. A 1:72 scale model was prepared to study the co-efficient of discharge over the spillway with drum gates. With different elevations co-efficient of discharge varied in model due to shape of drum gate. A calibration curve and equation of flow were formulated by the Bureau, giving value of 'C' in the model. It was found from actual observations of a wide range of discharges over the prototype, that the co-efficient of discharge given by model studies was 3.9% higher than that obtained on the prototype. The range of heads and discharges covered in the studies extended from 1.05' - 9.84' and from 1150 cusecs - 30,600 cusecs. As regards quantitative comparision, this is an excellent case of model prototype confirmity. If more model tests were conducted in the range of low discharges (which occur more often in prototype) still better results could be reasonably expected.

(ii) The Ghataprabha canal Head sluice -

This structure was remodelled for higher capacity on the basis of model studies. The co-efficient of discharge in submerged condition under 12.5' head was found to be 1.3 compared to 1.51 obtained from model studies.

(iii) Volute syphons of Hirebhasagar dam -

Detailed observations were made on syphon No.10 of this dam. The prototype performance revealed that there was high degree of confirmity between the model and prototype as regards, pressure distribution discharge and geometry of flow. Limitations were however observed in respect of air entrainment and cavitation. Even for discharges, there was close confirmity for non-cavitation and full bore working of the syphon, but for partial running of sypho however, the discharge predicted by model was only approximate.

(iv) The outlets of Grand-Coulee dam -

In this dam sixty out lets of 102 in. dia were provided in 3 tiers to release a total discharge of 225,000 cusecs. Each outlet had a bell-mouth entrance transition followed by straight conduit which terminated in to an elbow and come on the down stream surface of dam. Emergency and service gates have been provided for each outlet and these are supposed to run full to avoid vibration. A 1 in 17 geometrically similar model was constructed of the outlet and piezometers were installed for studying the pressure variation on boundary surfaces with the help of water and mercury manometers. Arrangements to make similar observations were made in the prototype as well. It was noted that pressure drop in the model of the bell mouth agreed with the prototype within 5%,but in the elbow and cone, the model pressures were 30% greater than in prototype. As regards the discharges, the model predicted discharges which were on average 8% lower than those actually observed in the prototype. Thus the quantitative comparision in model and prototype has been reasonably good. The slight error in pressure distribution is rightly attributable to inability to test the models at correct range of Reynold's numbers.

(v) The Break-water of Ferkings-stadt Harbour (16) -

This break-water has been constructed to protect the harbour against the high sea waves from North sea. The break water is a rubble mound structure. The stability of this break water was tested in the River and Harbour research laboratory at Technical University of Norway. On a geometrically similar model to a scale 1/40 the breach in breakwater was simulated after ascertaining the hind cast of wave heights which damaged the breakwater. This was done with the help of synoptic weather charts and refraction analysis The damage occured due to 8.75 m. high waves in front of the breakwater. The wave period was 10 seconds in prototype. As a result of high model prototype conformity, heavier cover blocks of 25-30 tons on pervious core were suggested.

(vi) Wheelar Dam lock -

Under the Tennessee Valley development, many dams were constructed across the Tennessee river and some of the dams viz. Pickwick, Wheeler and Guntersville dams were provided with locks. Hydraulic model studies were conducted on Geometrically similar models to predict -

1) Filling and Emptying time of locks
2) Velocities through the ports at intake, manifold and discharge sections
3) Co-efficient of discharge of the entire hydraulic system(lock).

Accurate measurements were made on the prototype after completion, regarding the same aspects as considered in the model. There was very high degree of conformity between the model and prototype. The model predicted filling time of 7 Min. 12 sec. where as the prototype lock filled in 7 Min. 5 sec. Emptying time from model test was 8 Min. 2 sec., where as the prototype emptied in in 7 Min- 22 secs. The filling and emptying co-efficients for Wheeler lock also agreed very closely. This indicates that the theory of similitude on which the hydraulic models are designed and operated is correct and trustworthy and geometrically similar models can solve the difficult problems of design and operation of intricate new structures.

(vii) Under-sluices of Sarda Barrage -

In 1964 ft long Sarda barrage, there are 34 bays of 50' width seperated by 8' thick piers. There are four undersluice bays on the right and their cill level is 2 ft lower than the rest of the bays. Although the barrage was designed for a maximum flood discharge of 6 lakh cusecs, in 1958 two under sluice bays were damaged due to a sustained high flood of 3.7 lakh cusecs. The problem of remodelling of these bays was studied on a part length geometrically similar model of scale 1:24 representing only two bays. The profile of weir and floor elevations of downstream and energy dissipation arrangements were suggested. The proposals formulated from model studies were implemented by project authorities. In the succeding years, observations on the prototype revealed, that water surface profiles were fairly comparable with those given during model studies for flood intensities of 200 and 300 cusecs / ft run.

(viii) Vaitarna Dam spillway -

Model studies on this spillway were carried out at Poona and it included testing of four ski jump buckets at different elevations with intermediate divide walls. The adequacy of heights of down stream training walls and the discharging capacity of the spillway were observed on a 1/48 scale model. When the prototype came in operation in 1954, the observations revealed that the central bucket had the same effect as predicted by model. There was no jet-action for low discharges but only a roller was formed. This bucket however works as ski-jump bucket for medium and maximum discharges. Also on the downstream side of the central span, same nature of piling up of material was observed near the toe of the prototype, as was depicted by model for similar running conditions. Thus there was good conformity between model and prototype.

(ix) D.V.C.Dam Spillways and studies of Energy dissipation -

The Damodar Valley Corporation has provided at Maithon, Panchet Hill and Tilaiya dams the inverted buckets in the spillways. The energy dissipation for these structures was tested in scale models and scour patterns were predicted. Very close conformity has been obtained for Maithon dam, where model predicted maximum scour of 30' depth at about 150' down-stream from the end of bucket, where as in prototype 25' deep scour has occurred at 200' from bucket lips after 10 years of operation.

The Konar dam was provided with the conventional type of stilling basin 268' long and 360' wide with 10' high end si

The performance of this basin has been quite satisfactory and there has been no bed scour in the channel downstream. However, there has been erosion at left and right flank for 100 ft on either sides and this extends for 400 ft down stream.

(x) Spillway of Gandhisagar dam -

In the same way, for Gandhisagar dam across Chambal river also, the depth and localisation of scour has been judged correctly on the basis of model studies.

(D) Examples of confórmity in river problems -

8.4 In the same manner qualitative comparision between model results and prototype behaviour has been carried out for river and harbour problems in which distorted models with mobile bed are used for flood control, river training or navigation facility. Some examples of success-ful river training works are ennumerated below.

(i) Training of Ganges river for protection of Buxar town (17) -

The Buxar town is situated on the concave bend of rive Ganges and as such the river has been hugging the bank and there by causing erosion and endangering the town. A scheme of flood protection was envisaged which included construction of 10 nos. of spurs. This was carried out in view of impending danger of flood disaster before 1957 monsoon. In order to judge the adequacy of the spurs constructed, a mobile bed model was constructed to the following scales.

Vertical scale	1 in 60
Horizontal	1 in 330

The river reach represented in model included 1½ miles length upstream of confluence with a tributory and 1/2 miles downstream of that point. The bed was represented with fine sand and banks were made of sand and gravel according to details of survey.

The model was proved for three high stages and discharges, observed in the prototype during the following monsoon season and hence was conclusively ready to predict river behaviour during higher floods. Higher discharges were run in model corresponding to the flood of 6.5 and 4.5 lakh cuseds. During model running, a small quantity of silt was also injected. The deepest river course and bed scour and silting of banks by the side of spurs proved that the arrangements made were successful. The siltation observed in the vicinity of spurs 1 to 4 compared fairly well in model and prototype

(ii) Training of River Danube near Bratislava,Czechoslovakia -

This is a case of medium water training for navigation. The river is characterised by branching of the main channel for about 100 Kms and all branches rejoining the main channel at lower end. Due to heavy bed load movement, there is considerable change of cross section and discharge in different branches. On one of the main branches training measures were required to provide a deep channel to handle the international navigation traffic. For solving this problem, a vertically exeggerated model of the river Danube(showing the main channel and the mouth of the branch) was prepared. The horizontal and vertical scales adopted in the model were 1 in 300 and 1 in 100 respectively. The model represented 4.5 k.m. long reach and mobile bed was reproduced by coal of grain size of 1.15 m.m. of sp. gravity 1.30. The model was verified as regards bed load movements by circulation method of sediment feeding at specific places and collecting the same in the sediment tanks. The time scale for sediment movement obtained from verification tests was $\frac{1}{900}$ - $\frac{1}{1300}$ for several tests. After this a hydrograph for for about 1056 days was run in the model adopting $ts = \frac{1}{1200}$ and the bed contours were plotted. This tallied very well with the bed contours in the prototype. There-after suitable river training measures were tried in order to determine the best and economic proposals which would give a stable deep channel of requisite depth suitable for navigation. For all these tests the hydrograph for 1056 days was adopted in natural sequence. The proposals included modifications of some existing groynes, construction of new ones and dredging at specific places and blocking a certain portion of branch to give the desired effects. The recommendations based on findings of model studies were implemented in the prototype and after 3 years of running the bed contours formed in the branch and main channel showed very good qualitative correspondence and to a considerable degree quantitative correspondence as-wel-as bed levels between model and prototype did not vary more than 1 meter in elevation.

(iii) Training of River Rupnarain near Kolaghat Railway station -

The river Rupnarain is a tidal river flowing across the old B.N.Railway about 85 miles from Calcutta. The river was cutting it's left bank progressively since 1938 when the left bank loop was 2900 ft away at the nearest point. By 1945 the erosion had reduced this distance to only 1500 ft and there was grave danger of the high approach embankment to the bridge at Kolaghat being breached. This would have resulted in virtual cut in the supply line to the west by the river out flanking the bridge. The Railway officials were proposing short Denehy's (T Headed) groins for protecting the left bank. The problems was referred to C.W.P.R.S.Poona to halt the progressive erosion by suggesting suitable training measures.

The station conducted model experiments on a rigid bed model and advanced two alternatives.

(i) A long curved head upstream of the Bridge

(ii) A repelling groyne 1400 ft long on left bank pointing 60° up stream.

The second alternative was accepted by the Railways and the repelling groyne was constructed in 1944. The object of this spur was to cause afflux on the upstream and deposit silt both on upstream and downstream of the groyne. The results expected were the diversion of flow to the right bank and reduction in the velocity of flow along left flank. This object was realised in the prototype as substantial shoaling has occurred (12-18 ft deep extending up to 5000 ft near the left bank) where 15 feet deep channel existed previously.

(iv) River training at Sarda barrage

The Sarda barrage at Banbassa was constructed in 1928 across Sarda river (in boulder stage). The barrage is 1964 ft long and designed for maximum flood discharge of 6,00,000 cusecs. A Wooded island splits the flow of main river in two channels upstream of the barrage. Before the construction of the barrage, the Right channel was effective and carried greater part of flow,but on comple tion of the barrage the left channel became active. In order to reduce the attack on left flank, in 1928 Nepal spur was constructed, one mile upstream of barrage, pointing upstream. Later in 1929 another hockey shaped spur pointing upstream was constructed mid-way between the Nepal spur and barrage. These measures created further complication of cross-flow upstream of barrage during high floods. The problem was referred to Research (C.W.P.R.S.Poona)Station which suggested 840 ft long Banbassa spur from the right afflux bund. This provided sufficient protection to right afflux bund. In 1958 floods, a powerful current was noticed upstream of Banbassa spur and to protect its harmful effects Solani spur 2950' long was proposed (and constructed before 1964). In 1952 however,the nose of Nepal spur was broken off, which was replaced by a straight extension of 1850 ft as decided from model studies. Since then,there is no damage apprehended at the barrage and thus model studies have helped in river training at the headworks of Sarda canals.

(v) Protection of Dibrugarh town on the banks of Bramhaputra river

The river Bramhaputra shows braided pattern of its numerous channels before it debouches into the plains of Assam. The river is 7 to 9 miles wide at places and there is shoaling and island formation due to high intensity of silt and sediment charge Due to high seismic activity in the Assam plains, the bed of the river rose by about 10 ft due to heavy land slides in the Himalaya region in 1950. In 1954, a severe flood washed away about 1/7th of the Dibrugarh town. In order to protect the 4 mile long reach, model studies were started. The model clearly depicted that for achieving desired results, spurs pointing upstream should be adopted. This was quite contrary to earlier design proposal. Thus models saved huge investment on a wrong proposal. Based on the model studies, five stone spurs and fifteen permeable timber spurs were provided for training the river. The spurs withstood the attack of 1955 floods well, however the launching aprons slumped into river, which were made good by dumping more stone crates. Thus the Dibrugarh town was saved from the danger of being washed away. The location and sizes of the spurs and permeable groynes are show in the accompanying sketch. Fig 8

(E) CONCLUSION

8.5 From the foregoing examples it is evident that a well designed and constructed model provides very useful results regarding the prototype behaviour. However due to scanty data of prototype observations, which present their peculiar difficulties, information regarding this important aspect of studies is very muc lacking. The reason for this is due to lack of interest in making observations, when once the prototype starts functioning satisfactorily. The high velocities of flow through prototype, the cop discharge volumes let down through the structures, the high range of pressures and vibration effects etc. pose very difficult problems and very costly and robust, yet sufficiently sensitive, instruments are required to be used for prototype measurements. Model studies not only assist in evolving an efficient, economic design but experiments with models also reveal the best mode of operation of structures. They also aid in remodelling of an existing structure for improving its capacity or its working.

8.6 Usually, the extreme conditions of flow are tested in the model which occur once in a while after many years of the operation of prototype. Thus, the opportunity of comparision between model and prototype working is denied. Actually the models should be experimented for a large number of lower discharges, which are frequently encountered and their safety in the extreme conditions of operation should be ensured.

8.7 Not only good conformity between model and prototype is of significance, but also, the anomalies and difference between model and prototype are of great importance, as they help in a better understanding of similarity-limits and scale-effects, which cannot be reproduced in models. Usually, it is human tendency, that man marks when he hits the target, but fails to mark the target is missed. Thus, whenever any discrepancy is found in conformity further systematic investigations should be made to detect the flaws. The indirect advantage of nonconformity is that, it may at times save us wasteful expenditure on not only abnormally costly, but in-effective plans.

8.8 Thus, great value should be attached to model studies of difficult hydraulic problems as is done in other progressive countries of the world viz. United States of America and Russia. Money spent on the research work should be considered as insurance charges against heavy outlays on gigantic structures, that might fail due to lack of knowledge of subtle principles of Hydrodynamics.

PLATE 4.

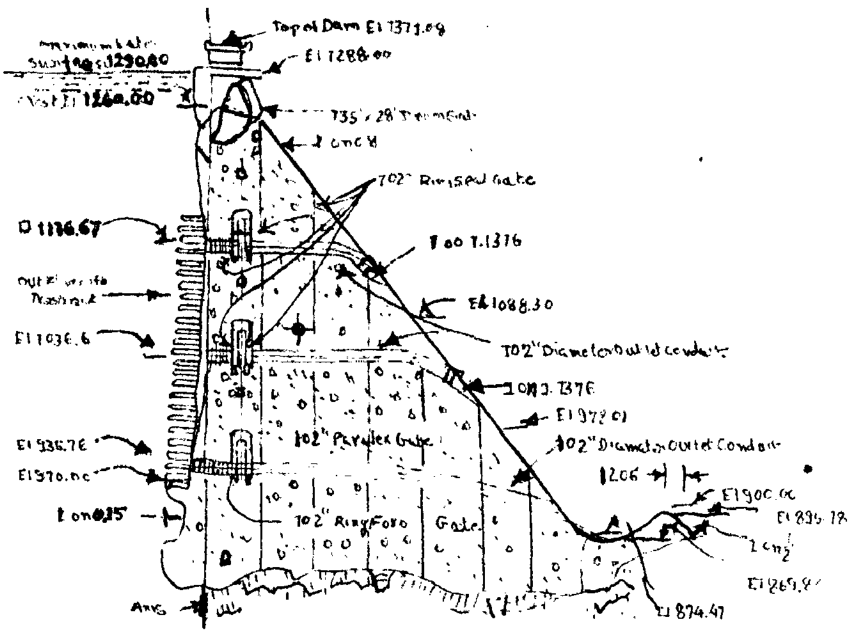


FIG. 4 SPILLWAY SECTION GRAND COULEE DAM

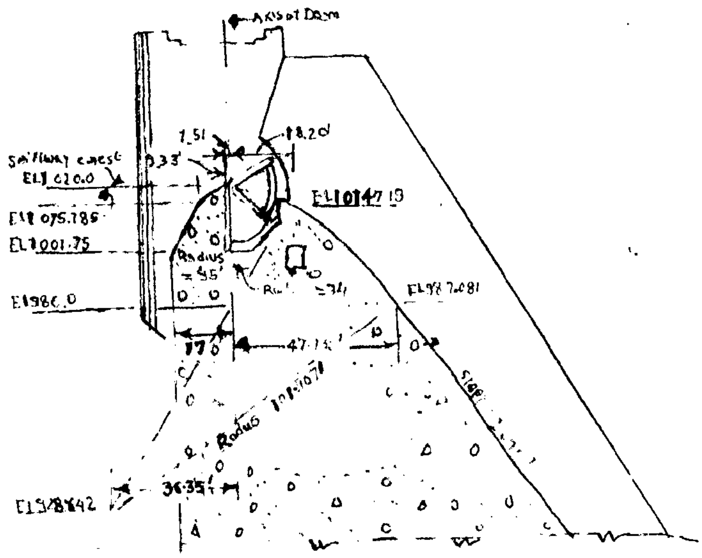


FIG 5 DIMENSIONS OF SPILLWAY CREST
NORRIS DAM

Fig:- 6 CROSS-SECTION OF BREAKWATER AT FERKING-STADT PLATE

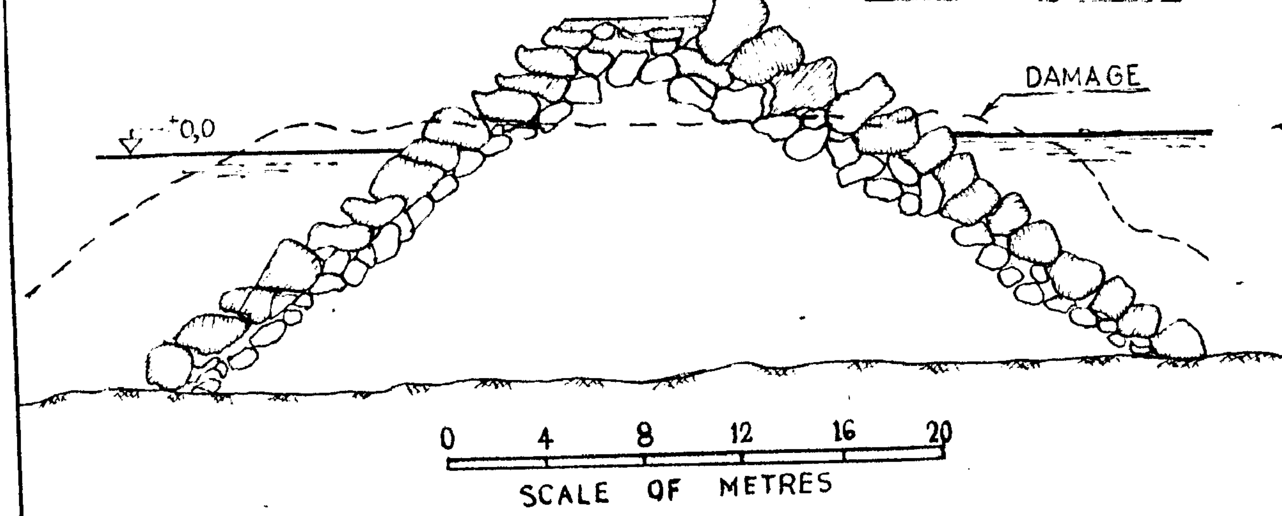
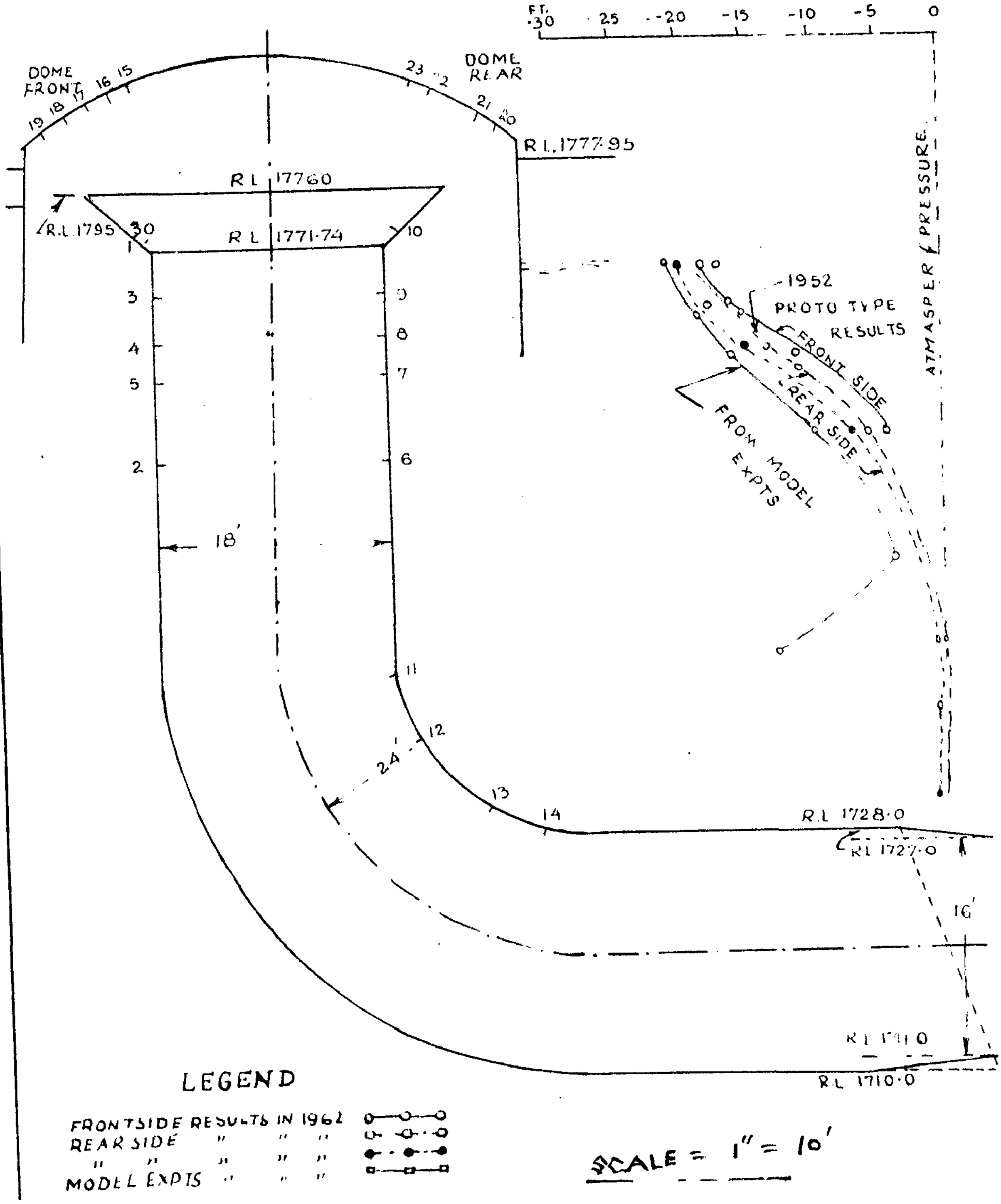


FIG. 7. VOLUTE- SYPHON MODEL.



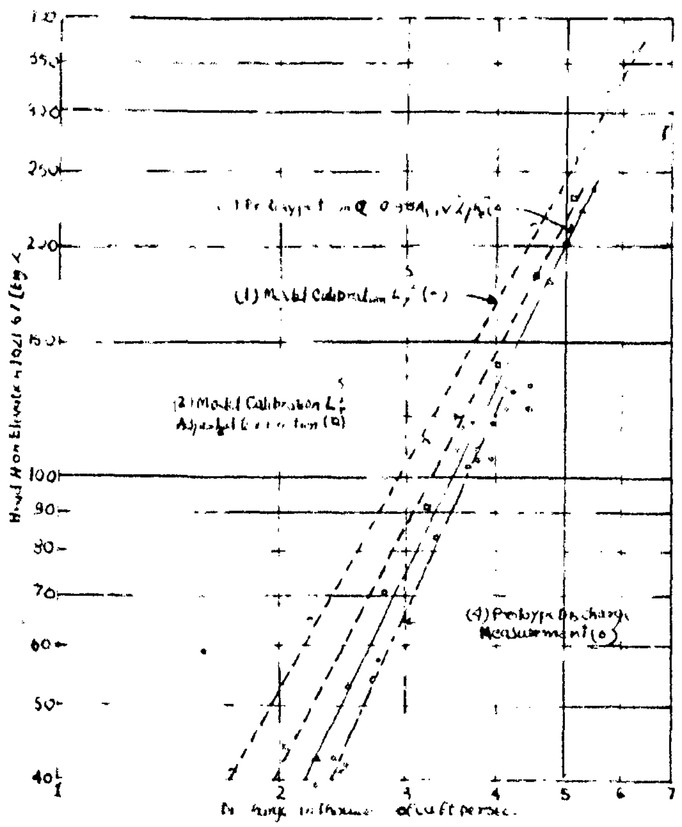


FIG 8 COMPARISON OF DISCHARGES

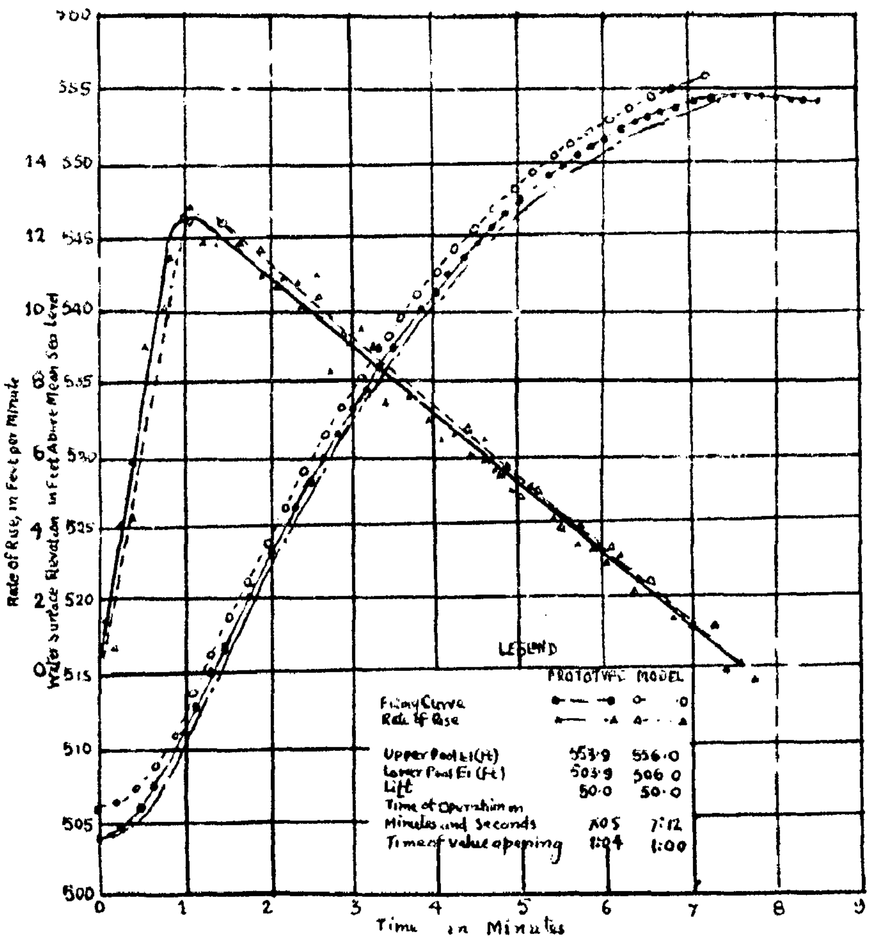


FIG 9 FILLING CHARACTERISTICS WHEELER LOCK

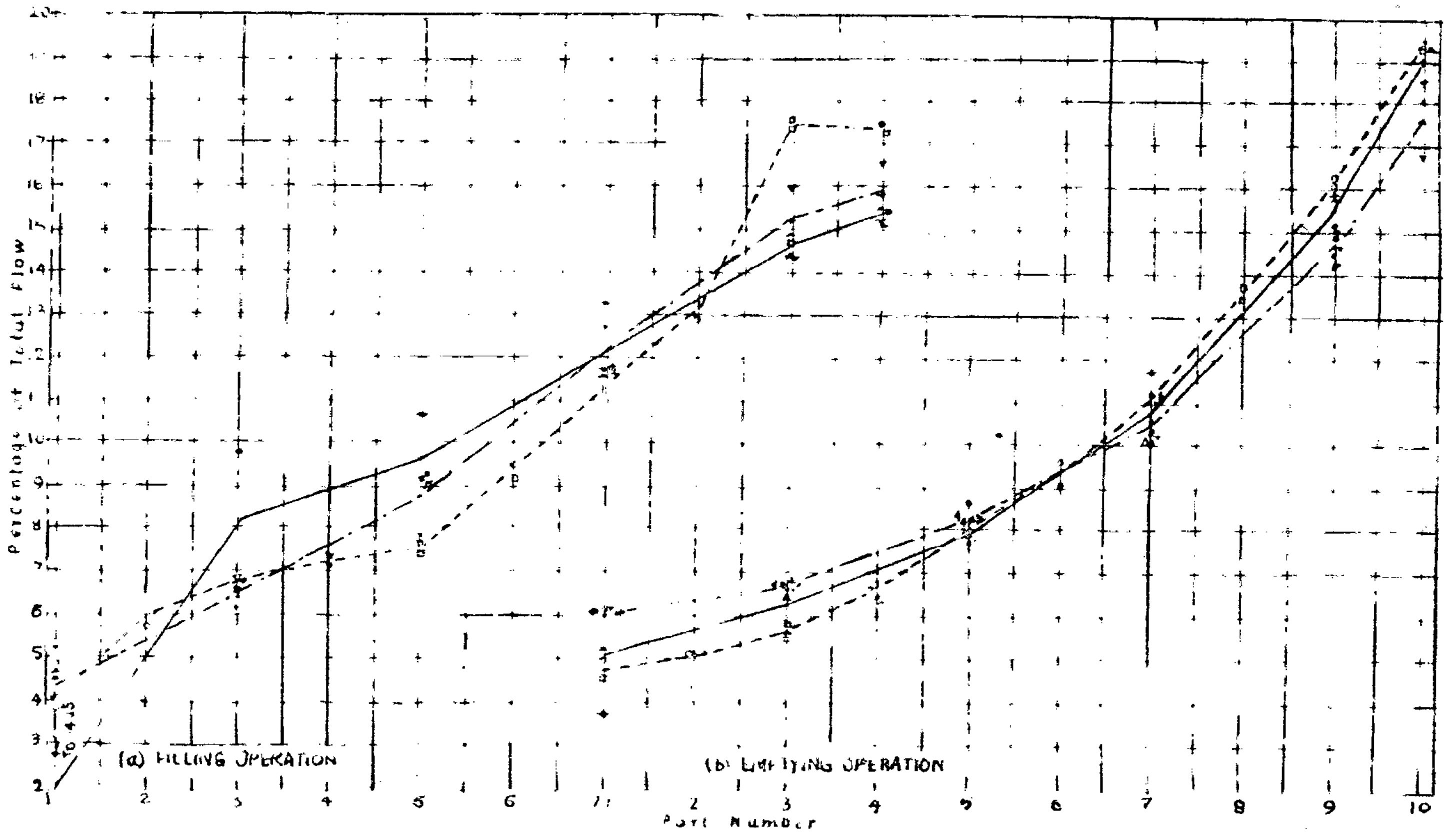


FIG 70 DISTRIBUTION OF FLOW LOCK-CHAMBER PORTS WHEELER LOCK (RIVER WALL)

Description	Average of Three Heads[a]			Average of Six Heads[b]						Average of Two Heads[d]	
	◆	▲	▼	⊠	△	●	◀	▶	▼	□	○
Head in feet	48	45	40	35	30	23	20	15	10	35	24
Discharge in cubic feet per Second											
(a) Filling operation			2,430	2,180	2,110	1,920	1,755	1,540	1,260	2,370	1,960
(b) Emptying operation			2,390	2,240	2,040	1,810	1,690	1,440[c]	1,180	2,150	1,760

[a] Discharge ... flow into and out of lock chamber through ports in river wall [c] Prototype
[b] 16 ft head [d] Model

MODEL-PROTOTYPE CONFORMITY

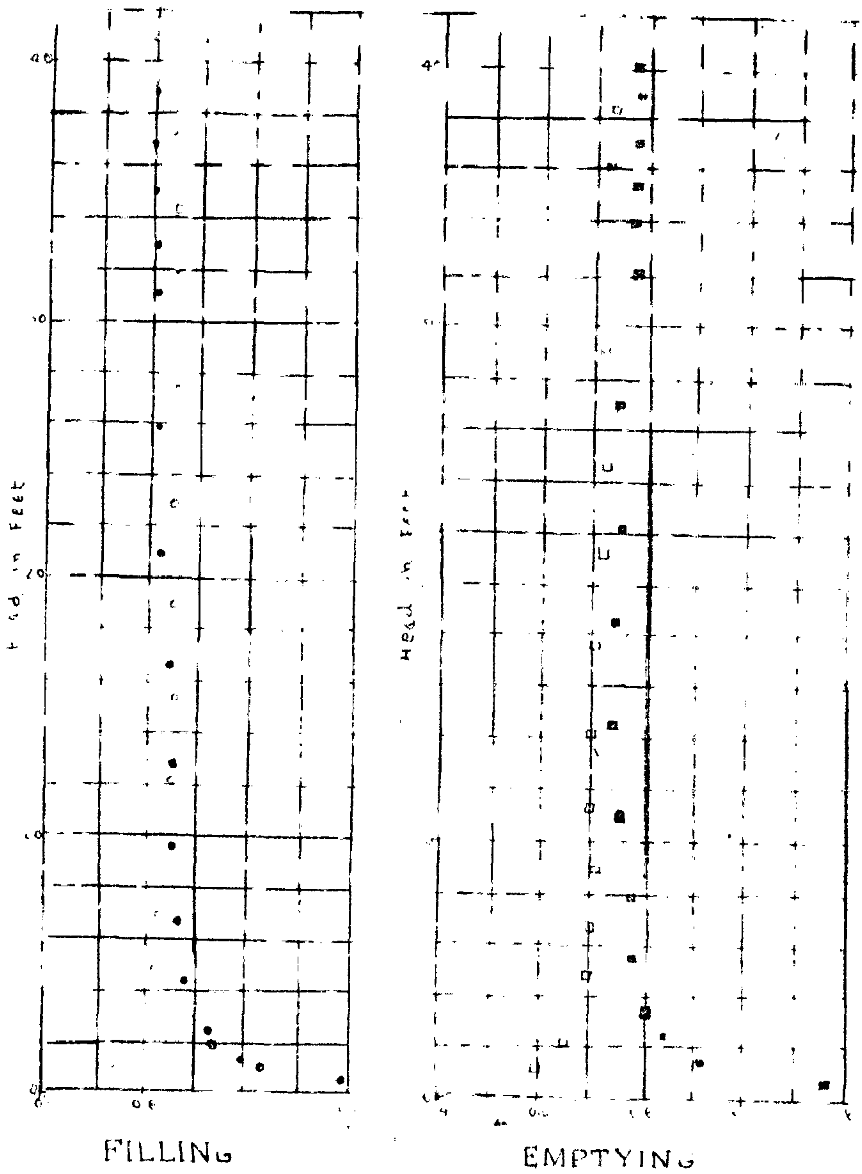


FIG 11 LOCK COEFFICIENT OF WHEELER - DAM

	Legend			
[illegible]	•	o	■	□
F[illegible] [illegible]	Prototype	Model		
Emptying operation			Prototype	Model
Average coefficient C (Eq 4)*	0.91	0.87	0.73	[illegible]

* For heads greater than [illegible]

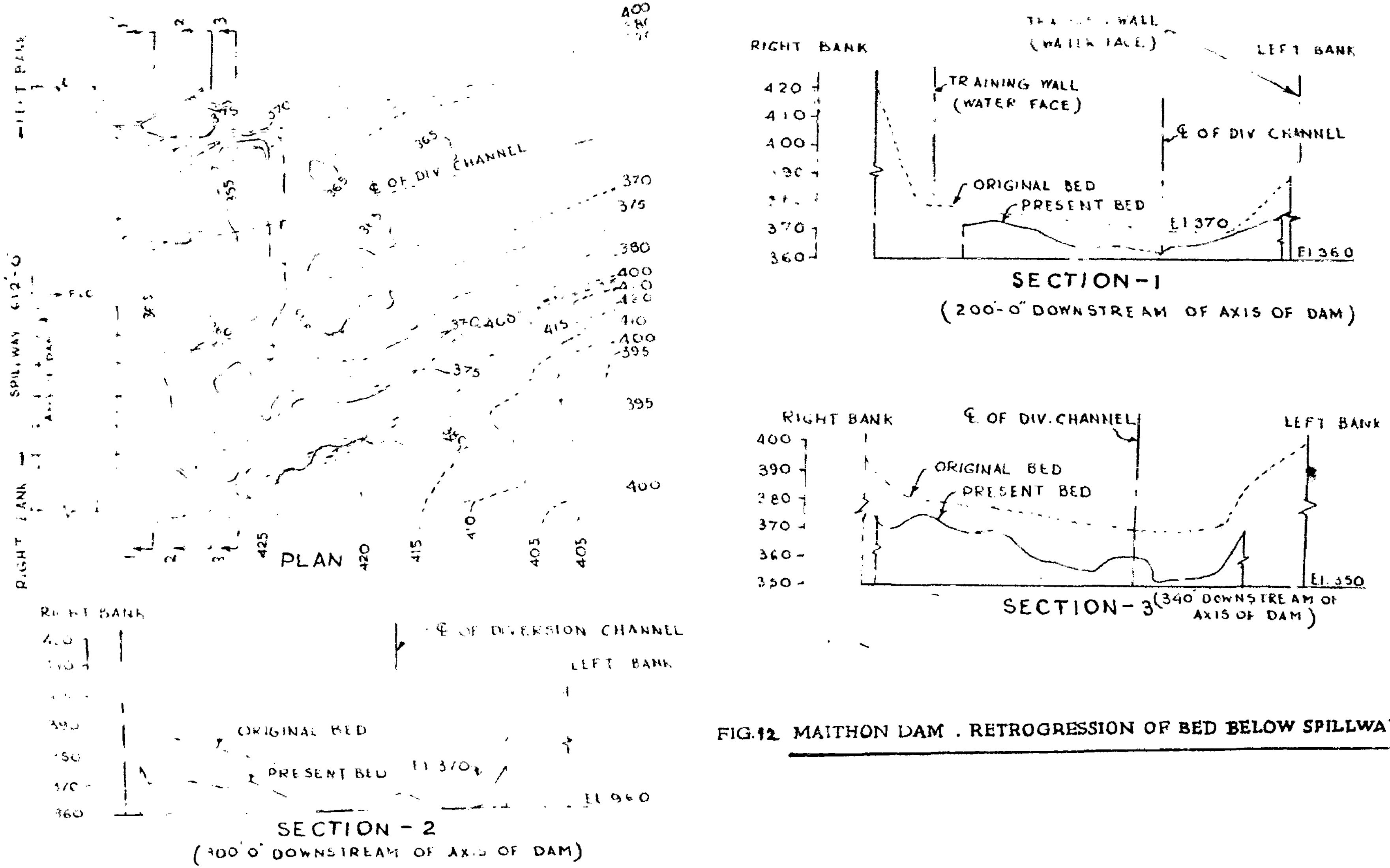


FIG.12 MAITHON DAM . RETROGRESSION OF BED BELOW SPILLWAY

Levels were taken during May 1963. Dotted contours show spillway channel bed as existed during April 1955. Firm contours show present

PLATE 10

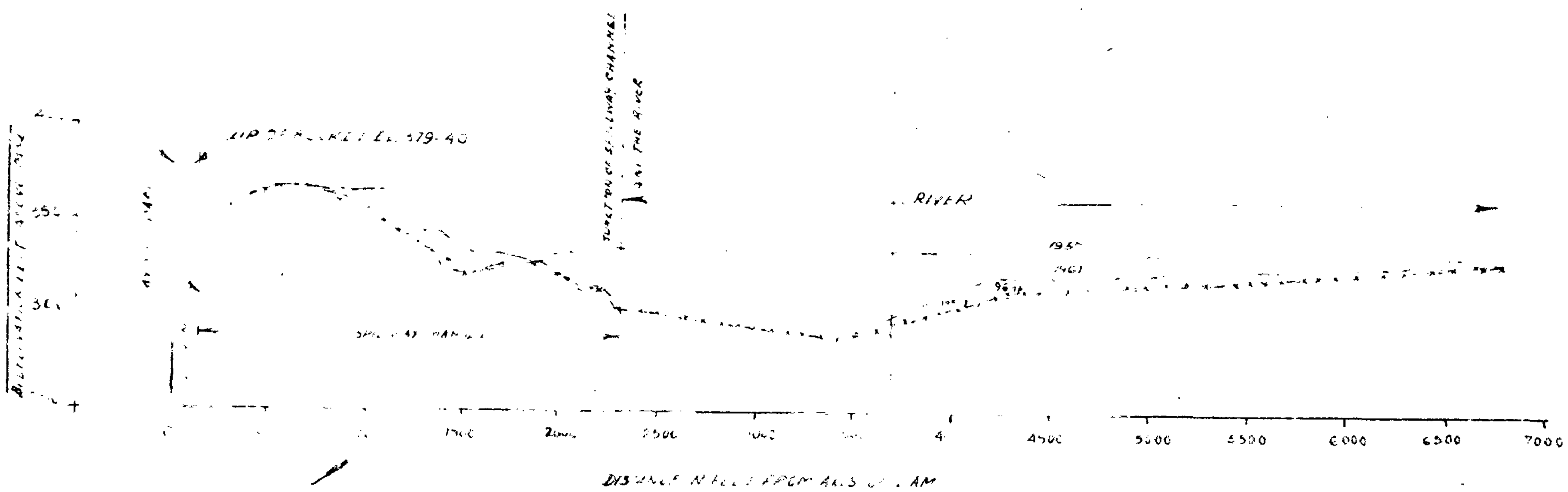


FIG. 13 MAITHON DAM BE[illegible]OFILE ALONG SPILLWAY CHANNEL

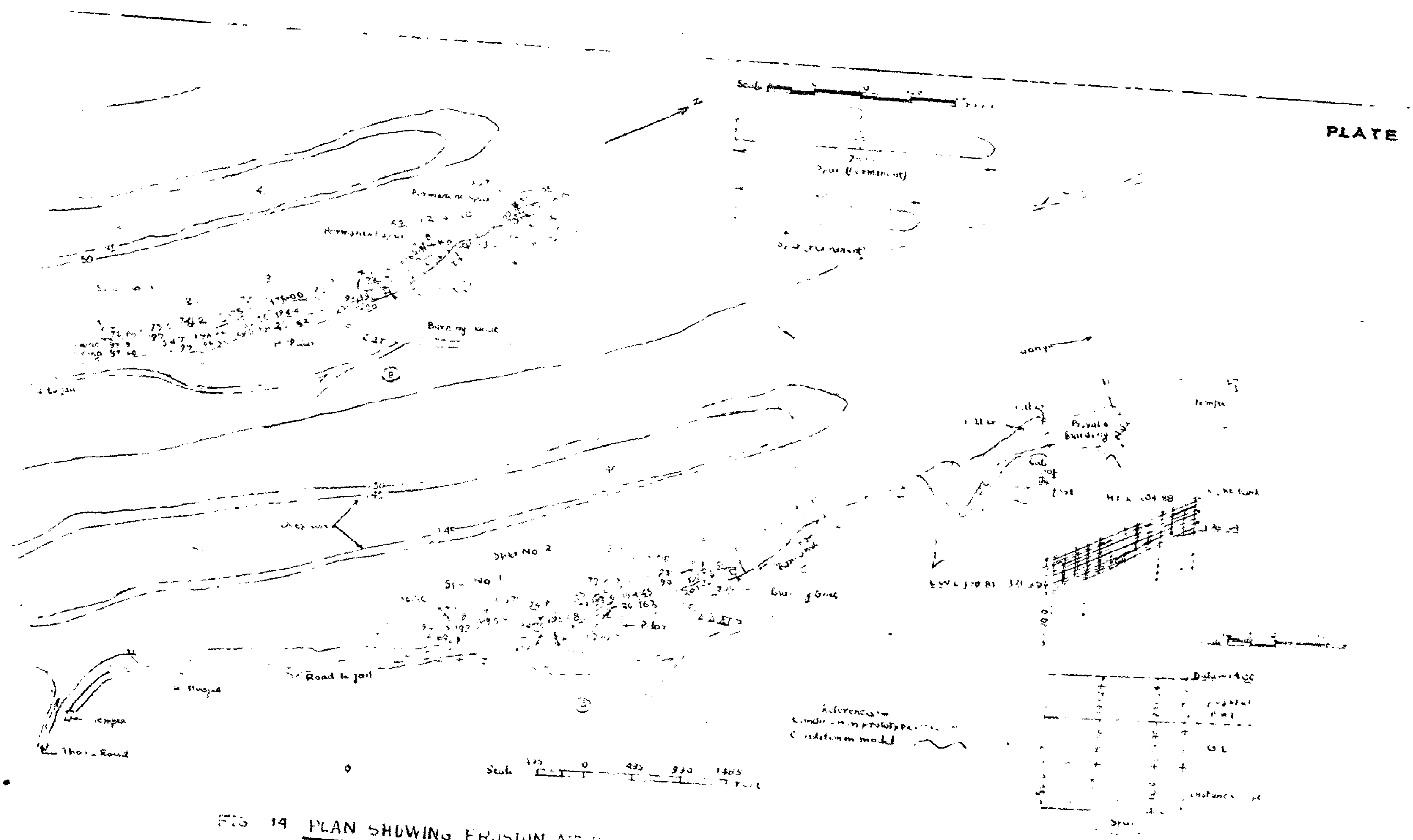
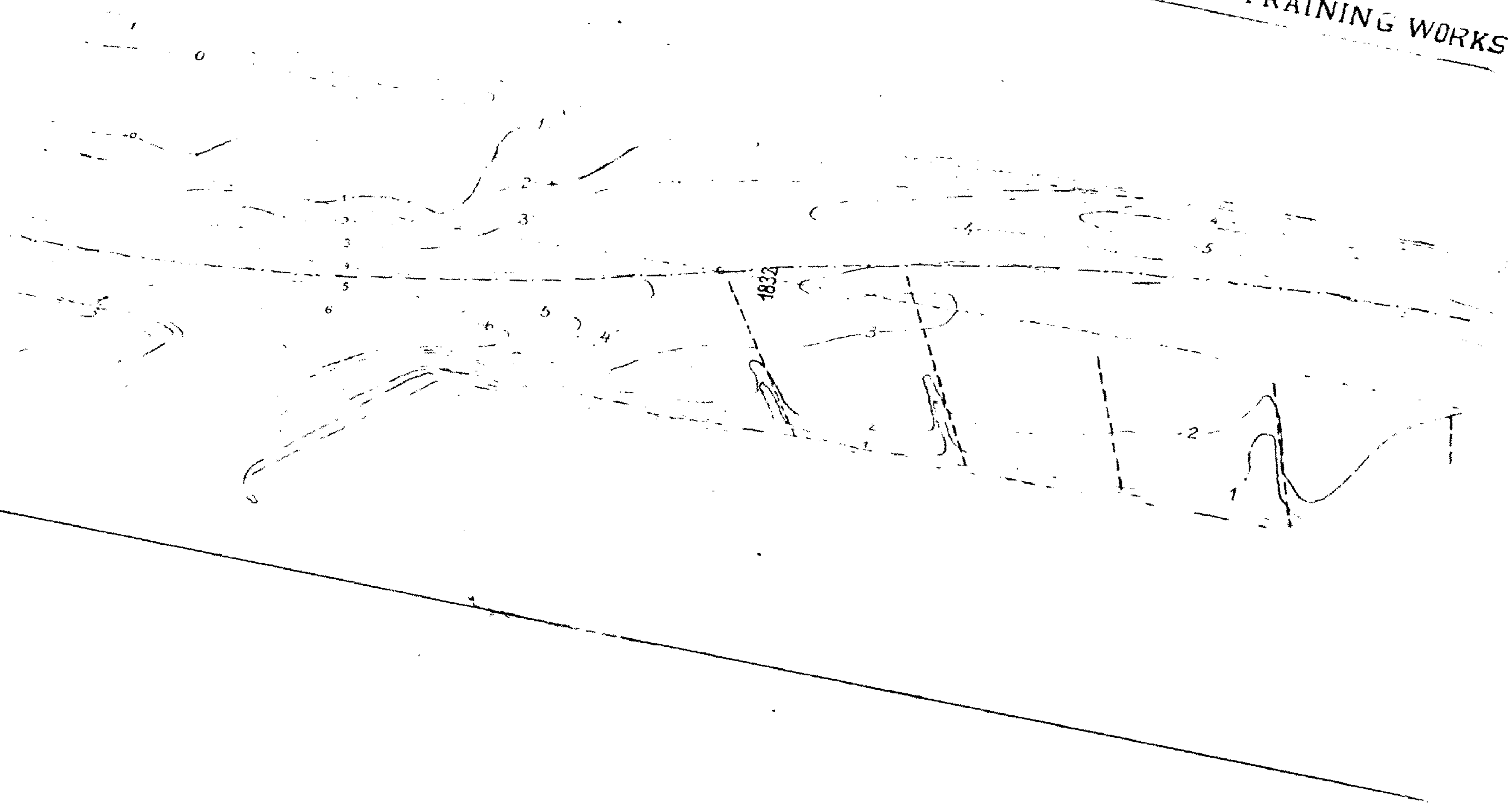
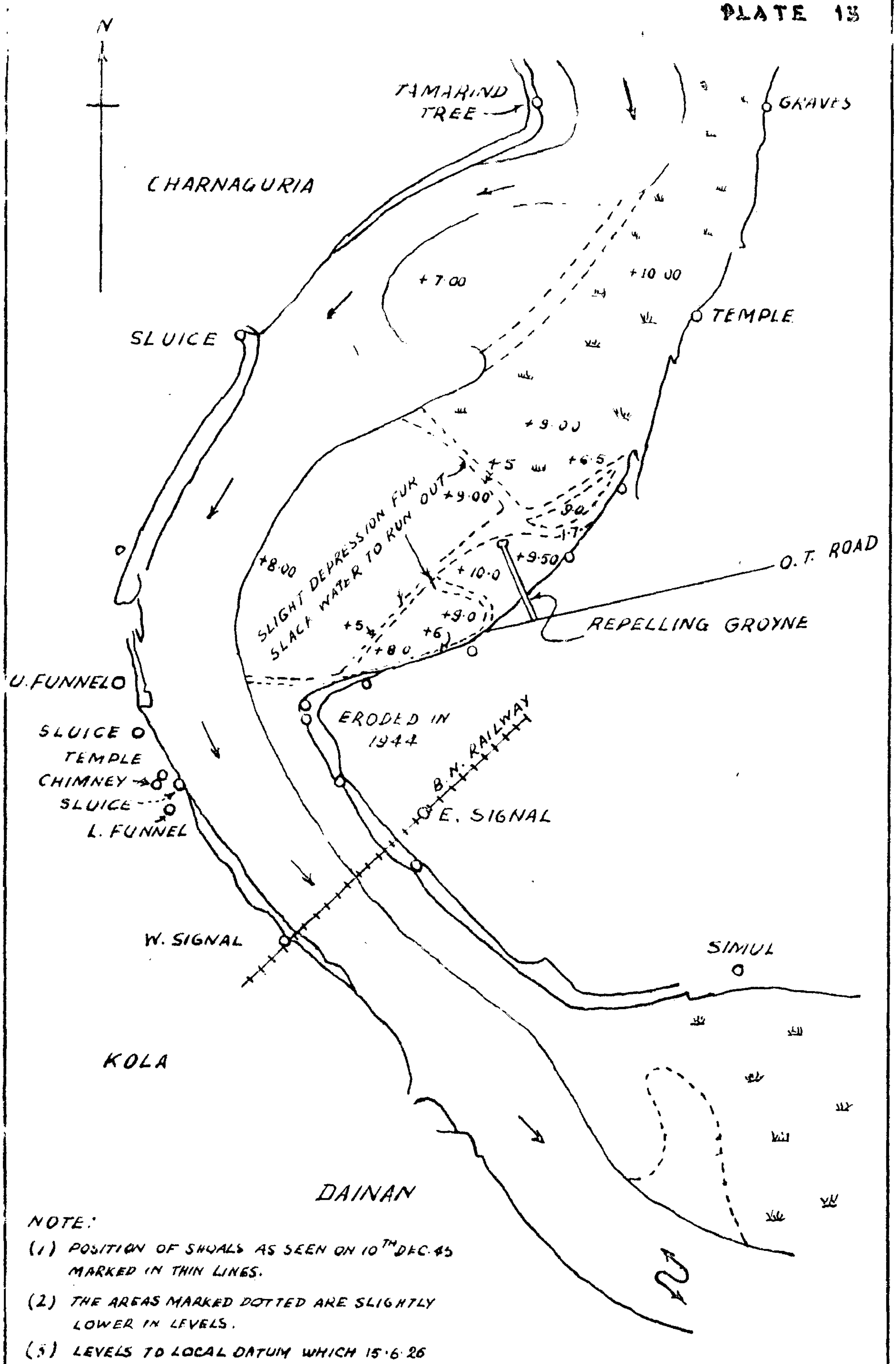


FIG 14 PLAN SHOWING EROSION AT BUXAR FROM RIVER GANGA

FIG 15 BED CONTOURS OF DANUBE RIVER TWO YEARS AFTER CONSTRUCTION OF TRAINING WORKS





NOTE:

(1) POSITION OF SHOALS AS SEEN ON 10TH DEC. 45 MARKED IN THIN LINES.

(2) THE AREAS MARKED DOTTED ARE SLIGHTLY LOWER IN LEVELS.

(3) LEVELS TO LOCAL DATUM WHICH 15.6.26 ABOVE K.O.D.S.

FIG. 16

RUPNARAIN RIVER

CONSTRUCTION OF SPUR

PLATE 14

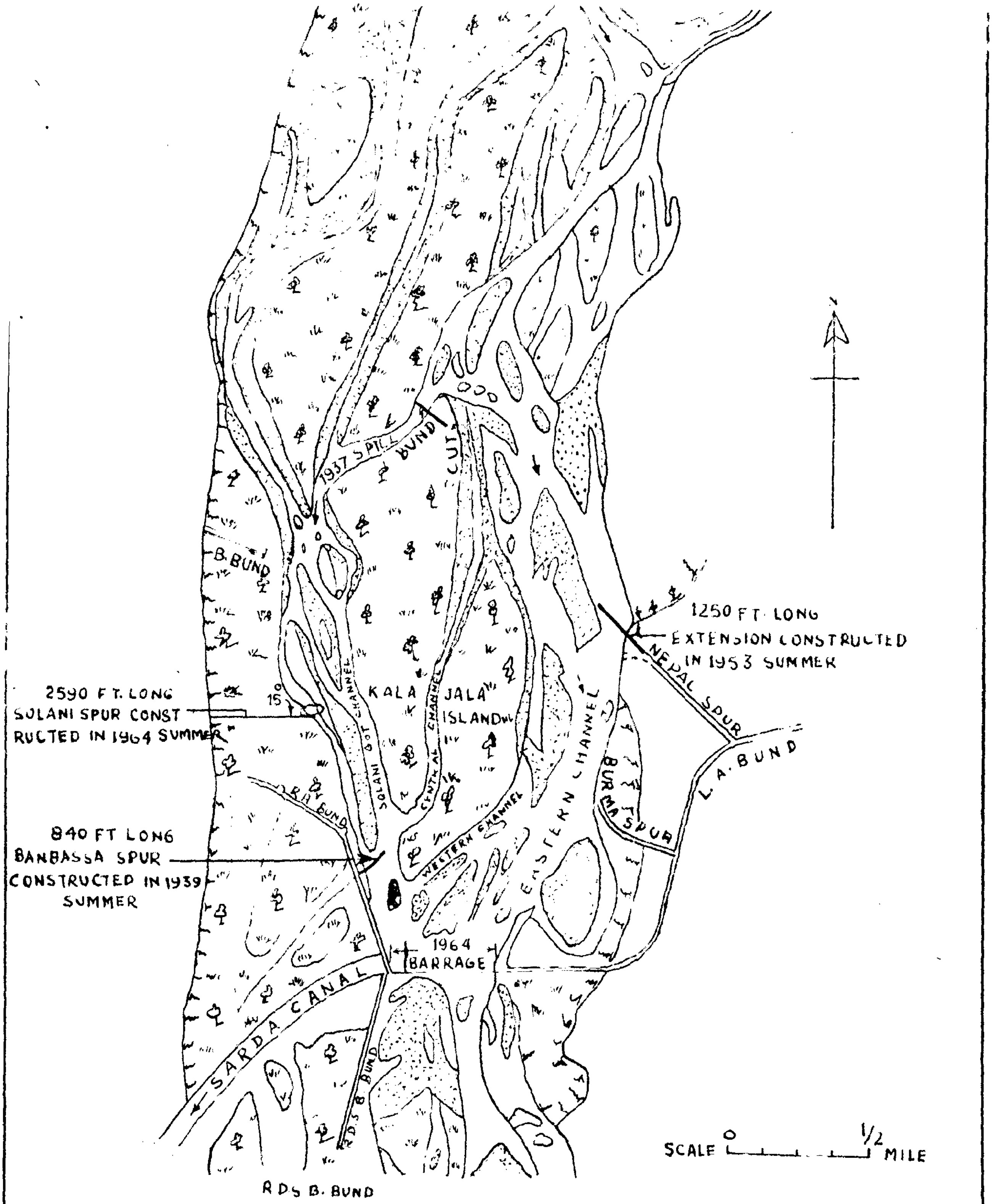


FIG.- 17

SARDA RIVER AT BANBASSA (U.P.)

AFTER 1938 FLOODS

CHAPTER IX

JUSTIFICATION FOR HYDRAULIC MODEL STUDIES AND SCOPE FOR FUTURE DEVELOPMENTS

(A) HYDRAULIC MODEL AS A POWERFUL TOOL OF RESEARCH WORKERS

(B) UTILITY OF A GEOMETRICALLY SIMILAR MODEL

(C) ROLE OF DISTORTED RIVER MODELS

(D) ESSENTIAL REQUIREMENTS OF MODEL STUDIES

(E) SCOPE FOR FUTURE DEVELOPMENTS

CHAPTER IX - JUSTIFICATION FOR HYDRAULIC MODEL STUDIES AND SCOPE FOR FUTURE DEVELOPMENTS.

(A) Hydraulic model as a powerful tool of Research workers

9.1 Hydraulic models are necessary adjuncts for solving intricate problems connected with design, construction and operation of Irrigation, Navigation, Embankment and Drainage structures and also for Training and Control of rivers, which are not amenable to mathematical solution.

(B) Utility of a Geometrically Similar model

9.2 Very great reputation has been built up for such models, which give quick results which can be directly transposed for prediction of the prototype performance. Studies on these models at moderate costs have resulted in evolving efficient profiles of spillways, energy dissipation devices etc. and great economies have been achieved in the projects costs.

(C) Role of distorted river models

9.3 For problems connected with irrigation, navigation and flood control, the distorted river model with rigid or mobile bed serves as very useful aid. In spite of certain limitations which are inherent with such a model, many successful river improvement schemes have been carried out in India and in other countries with the aid of these models.

(D) Essential requirements of model studies

9.4 For successful model studies, great care, ingenuity, patience and ability for interpreting model results are required for the research worker, because hydraulic models are not computors or electronic brains, which when fed with the design data would yiel correct solutions. These are no substitutes to human reasoning , nevertheless, they provide important information to an experienced and well trained research worker.

(E) Scope for future development

9.5 There is great scope for filling the gaps, in knowledge of the science of fluvial Hydraulics and efforts of many research workers require coordination all over the world for understanding the intricacies. Stabilisation of alluvial channels, sediment transport, unsteady non-uniform flow phenomenon, tidal phenomenon are some of the aspects which need thorough study by a consortium of research workers. Actually, there is need for recognising hydraulic nationality or internationality for solving the complex hydraulic problems which pose a challenge to mankind.

APPENDIX

EXAMPLE - 1

A river at a certain reach has flood discharge of 4 lacs cusecs and resistance law applicable to model and prototype is of the form $\frac{V}{V_*} = 8.1 \left(\frac{R}{K_s}\right)^{1/6}$. If space available in hydraulic laboratory introduces limitations to adopt $L_r = 250$. determine the depth scale, sediment scale, slope scale, velocity scale, time scale and determine the density of model material. Assume river slope = .0024.

The resistance law shows that C = 8.1 & m = 1/6. Assuming Δ values = 1 and $\eta r = 1$ We have from Table 2 for L_r given by Einstein.

(1) $h_r = L_r^{\frac{m+1}{4m+1}} = L_r^{0.7} = 200^{0.7} = 41$

Adopt $h_r = 40$ so that V.E. $= \frac{200}{40} = 5$

This value of vertical exeggeration is not too high

(2) Again $D_r = L_r^{\frac{(2m-1)}{2(4m+1)}} = 200^{-1/5} = \frac{1}{2.83}$

This shows that model material should be 2.83 times coarser than prototype material.

(3) $S_r = L_r^{\frac{-3m}{4m+1}} = L_r^{-0.3} = 200^{-0.3} = \frac{1}{4.9}$

or $S_m = S_p \times 4.9 = .0024 \times 4.9 = 0.01176$

(4) $V_r = L_r^{\frac{m+1}{2(4m+1)}} = L_r^{0.35} = 200^{0.35} = 6.4$

(5) $t_{1r} = Lr^{\frac{7m+1}{2(4m+1)}} = 200^{0.65} = 31$

(6) $t_{2r} = L_r^{\frac{5m+2}{4m+1}} = 200^{1.7} = 8000$

(7) $(\rho s - \rho f)_r = L_r^{\frac{3(1-2m)}{2(4m+1)}} = L_r^{0.6} = 200^{0.6} = 23.9$

Taking the specific gravity of prototype material as 2.65 $\left(\frac{2.65-1}{X-1}\right) = 23.9$ or X = 1.069 sediment density of model.

Thus all the scale ratios have been fixed from the Einstein's Equations.

EXAMPLE - 2

Deduce the scale ratios from Komura's equations for a wide and shallow river with following hydraulic particulars.

(1) Maximum flood discharge	=	10,000 Cfs
(2) Bed width of river	=	280 ft
(3) Maximum depth during flood	=	12 ft
(4) Bed slope of river	=	0.3 ft/ mile
(5) Average size of sand in river	=	16 m.m.
(6) Model sand size available	=	4 m.m.

The model has to be laid in 80 ft long tray to represent 1 mile reach of the river.

$$L_r = \frac{5280}{80} = 63.5$$

$$d_r = \frac{16}{4} = 4$$

Assuming $\alpha = 2$, $\beta = 1 + \left(2 \times 2 \times \frac{12}{280} \right) = 1.16$

Taking the channel has rectangular cross section ,

$$Q_r = L_r^{17/8} \cdot d_r^{3/8} \cdot \beta^{-3/2} = 63.5^{17/8} \cdot 4^{3/8} \cdot 1.16^{-3/2} = 81$$

$$D_r = L_r^{3/4} \cdot d_r^{1/4} \cdot \beta^{-1} = 63.5^{3/4} \cdot 4^{1/4} \cdot 1.16^{-1} = 32$$

Thus the vertical exeggeration is $\frac{63.5}{32.2} = 1.98 = 2$ approximatel

$$S_r = L_r^{-1/4} \cdot D_r^{-1/4} \cdot S^{-1} = 63.5^{-1/4} \cdot 32.2^{-1/4} \cdot 0.3^{-1} = .4$$

$$U_r = L_r^{3/8} \cdot d_r^{3/8} \cdot \beta^{-1/2} = 63.5^{3/8} \cdot 4^{3/8} \cdot 1.16^{-1/2} = 7$$

Thus all the scale ratios have been fixed.

Solving the above example by Lacey's formula *

$$Q_r = L_r^{2} = 63.5^{2} = 4050$$

$$D_r = L_r^{2/3} = 63.5^{2/3} = 16 \quad . \quad V.E. = \frac{63.5}{16} = 4 \text{ appro}$$

$$S_r \doteqdot f_r^{5/3} \cdot Q_r^{-1/6} = 2^{5/3} \cdot 4050^{-1/6} = 0.82$$

$$V_r = D_r^{1/2} = 16^{1/2} = 4 .$$

Thus all the scale ratios have been obtained.

* REVERSE NOTATION OF SCALE RATIO ADOPTED FOR COMPARISION

EXAMPLE 3

A river is 4000 m long, 600 m wide and has mean depth of 6 m , its mean velocity is 1.2 m per sec. If the model and prototype have bottom roughness and grain size as given below, evaluate the length scale, velocity scale ratio and the amount of tilting required in the model.

		Prototype	Model
Bottom roughness	k	0.2m	0.02m
	d_m	$0.5.\ 10^{-3}$ m	$0.2.\ 10^{-3}$ m
Bottom material	d_{90}	1.10^{-3} m	$0.5.\ 10^{-3}$ m
	s	2600 Kg/m^3	1300 Kg/m^3
	f	1000 Kg/m^3	1000 Kg/m^3

$$C_{prot} = 18 \log 12 \frac{6}{0.2} = 46\ m^{1/2}/sec$$

$$C_{mod} = 18 \log 12 \frac{0.12}{0.02} = 33\ m^{1/2}/sec$$

$$C_r^2 = \left(\frac{46}{33}\right)^2 = 1.95 \quad \text{or} \quad L_r/h_r = \frac{L_r}{50} \quad \text{or} \quad L_r = 100 \text{ approx}$$

The width of the model is 6 m.

Next $$\mu_p = \frac{46}{18 \log 12. \frac{6}{1x10^{-3}}} = 0.39$$

$$\mu_m = \frac{33}{18 \log 12 \frac{0.12}{0.5x10^{-3}}} = 0.37$$

or $$\mu_v = \frac{0.39}{0.37} = 1.05$$

Since $V = C \sqrt{Y.I}$; $V^2 = C^2.Y.I$ or $\frac{V_r^2}{C_r^2} = Y_r \cdot I_r$

Since $(\text{———})_r = 1$; $(\quad)_r = Y_r \cdot I_r$

$$\frac{1.6}{0.3} \times \frac{0.5}{0.2} = 13.3 = \frac{1.05.\ V_r^2}{C_r^2} = \frac{1.05\ V_r^2}{1.95} \quad \text{or} \quad V_r^2 = 25$$

Thus $V_r = 5$

$$I_p = \frac{V_p^2}{C_p^2\ Y_p} = \frac{1.2^2}{46^2 x6} = 1.14 \times 10^{-4}$$

Slope of water level $= I_r = \frac{Z_r}{L_r}$ and Bed slope $= I_r = \frac{Y_r}{L_r}$

Tilting required $= I_p \left(\frac{L_r}{Z_r} - \frac{L_r}{Y_r} \right) = 1.14 \times 10^{-4} \left(\frac{100}{25} - \frac{100}{50} \right) = 2.28x1$

Since $Z_r = \frac{L_r \times V_r^2}{C_r^2\ Y_r} = \frac{100 \times 25}{1.95 \times 50} = 25.$

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