K. R. SHARMA

IRRIGATION ENGINEERING

BY

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FOREWORD

By

A.M.R. Montagu

I. S. E., C.I.E., M.I.C.E., A.C.G.I.

Chief Engineer, Irrigation works, Punjab.

I look upon it as a great privilege to write the foreword to Mr. Sharma's comprehensive work on Irrigation. I have known Mr. Sharma and his work for a number of years and have always been impressed by his keen interest in the physical foundation of irrigation practice. But he does not confine himself merely to principles of hydraulics involved in the art and practice of irrigation. He has made a prolonged study of all matters that can be said to touch upon irrigation in general. Even the revenue side has received his careful and zealous attention.

In many respects, Punjab irrigation leads the world. Without irrigation, the Punjab would still consist of a few strips of land contiguous to the rivers, upon which the anxious cultivator would sow a crop uncertain whether or not it would reach maturity. Today, by virtue of its canal systems, the Punjab is the granary of India. Valuable crops are now grown in areas which were originally arid waste whereon nothing was found but thorn and the camel which throve upon it. During the last few years, no less than 14 million acres of crops have been matured by irrigation water from the fifteen separate canal system of the province.

The actual distribution of the available water between the canal systems has necessitated the construction of feeder canals which convey surplus waters of one river to a river in deficit. Distribution among the individual channels of a canal system is a relatively simple matter. Far more difficult is the distribution among the individual cultivators. In the Punjab, the greatest attention is concentrated upon this distribution. Where this distribution is faulty, cultivators promptly bring facts to notice. The individual irrigation officer is concerned not only with distribution by existing known methods, but in many cases utilizes his scanty leisure in endeavours to secure fresh forms of outlets which will still further improve and stabilize the distribution to the cultivator's watercourses.

Among these energetic and enquiring gentlemen, Mr. Sharma occupies a high place. His work on outlets is well known. His activities in other directions have not perhaps received all the attention they deserve. This book is a compendium of all knowledge available to him on the subject and will unquestionably prove of the greatest value not only to the practising Irrigation Enginear but also to the student who is preparing himself to serve in that great service, the Irrigation Branch of the Public Works Department, Punjab.

It is but natural that many of the views put forward by the author are coloured by his Punjab experience. Readers foregin to the Punjab may conclude that undue weight is laid upon the Punjab practice and Punjab views. The answer to any such implied criticism lies in the fact that the Punjab is very confident of its capacity to design, construct and administer irrigation systems of the largest size. In many ways, such as distribution of water already mentioned, the design of major works on the shifting sands of rivers, the economical design and maintenance of relatively minor stauctures, the Punjab Engineer may justifiably hold that he is to the fore front of modren practice and knowledge. In other directions, the Punjab Engineer is first to recognize that he can learn not only from other countries of the world, but also from other provinces in India. It is probable that the construction of high dams will soon occupy the attention of the Punjab. Experience in this field is negligible and we turn to other countries, notably the United States of America for instruction and guidance in this field. The Punjab is behind other countries in the use of machinery, but it is probable that post war developments will impose an advance in this direction also.

In many ways, the advance of knowledge in our relatively restricted field is phenomenal. : example, in the matter of pumping water from the sub-soil water resevoir by means of tube-ls, it is possible that the practice described in Mr. Sharma's book may have advanced by the e these volumes are in the hands of the public, but for many years to come the student of gation engineering may turn with confidence to these pages for a useful and comprehensive de to irrigation practice, with special reference to the Punjab.

Lahore,
15th January 1945.

A.M.R. Montagu.

PREFACE TO 1st EDITION

The science of Irrigation Engineering in India has developed by leaps and bounds during the last two decades. The Irrigation Works of India especially of the Punjab, occupy a prominent place in the world, both from the point of view of the academic interest as a scisnce and of the successful results of large irrigation engineering projects. (Since the epic books 'Irrigation Work' by Bligh, 1907, and Irrigation Pocket Book, by Buckley, 1911, were written, the development of the Science in India is confiend to the technical papers contributed by the eminent Engineers available in the Proceedings of the various Engineering Societies, such as the Punjab Engineering Congress, Lahore, the Institution of Engineers India, Central Board of Irrigation, Bombay Engineering Congress, the Departmentat Technical Papers of P.W.D. Punjab and Bombay and the Research Publications of the Punjab Irrigation Institute and Central Hydro-Dynamical Research station Poona, Bombay.)

The purpose of this book is to present the science and the practice of Irrigation Engineering in a concise form comprising practically all the modern developments. The book is essentially meant to be used as a Text Book for the students preparing for the Engineering Degree Examinations of the various universities in India and other competitive examinations of the Central Government. Selected examination questions usually set in Degree Examinations of the varions universities in India are given at the end of each Chapter. Some examples of the typical designs of Irrigation Works have been worked out for the guidance of the students. Since Irrigation Engineering (distinct from Hydraulics) is not a Subject taught in the British or other European universities, no text books dealing with the subject are available. This book is intended therefore to meet a long standing need of the student community in India.

The book also deals with the actual practice of science in the field and its meant to be used as a reference book. I have, therefore, attempted to cover the requirements of the students in the examination questions (covering the syllabus of the various universities in India and the competitive examination of the Fedral Public Services Commission). In the Chapters which are considered beyond the scope of the students no questions are given at the end.

The subject of Irrigation Engineering has developed mathematically so much that it goes beyond the capacity of an average student or a practical Engineer to cram up all the formulae. In an accompanying volume III, the diagrames generally used in irrigation practice are given which can be used by the students and practical Engineers for solving the problems relating to the design of Irrigation Works.

The book is devided into six parts and forty four chapters. The first part, comprising two chapters deals with Lift Irrigation the second comprising 20 chapter deals with Flow Irrigation, the third comprising six chapters deals with Tank Irrigation, the fourth comprising four chapters deals with Drainage Engineering, the fifth comprising six chapters deals with Ground Water Engineering (water-logging) and the last part camprising six chapters deals with general information usually required in engineering practice. The fourth and fifth parts comprise the subject which is usually defined as Hydrology in American practice.

Obviously the whole of such a comprehensive book cannot be original. Detailed references to the publications consulted have been mentiond in the text. The list of authors is so large that it is not possible to acknowledge gratefully the help and the use of their work by naming them individually. Similarly the refences to the proceedings of various Engineering Societies, referred to in the text are gratefully acknowledged.

The help rendered in compiling, editing and improving this book by the following Irrigation Engineers of the Punjab is thankfully acknowledged : (1) Rai Bahadur B. N. Singh I. S. E., retired Chief Engineer, Punjab Irrigation. (2) Rai Bahadur D. K. Khanna I. S. E. Superintending Engineer (now Chief Engineer Irrigation Works). (3) Rai Bahadur B. L. Uppal I. S. E., retired Superintending Engineer, Punjab Irrigation. (4) Rai Bahadur Kanwar Sain I.S.E., Superintending Engineer (5) Rai Bahadur B. K. Kapur, Director Irrigation Research, Lahore.

(6) Rai Bahadur Hakim Rai, I.S.E., Superintending Engineer (7) Jatinder Singh, Assistant Engineer and (8) K.S. Pathak, Assistant Engineer.

The author is extremely grateful to the Honourable Minister of Public Works Department (now premier of the Punjab) Malik Khizar Hayat Khan Tiwana and the Honourable Member of Revenue in charge of Irrigation Department, the late Sardar Bahadur Sir Sundar Singh Majithia, for selecting the author to do spade-work of Civil Engineer teaching in the Punjab as the first professor and the head of the Civil Engineering Department at the Maclagan Engineering College, Lahore. The author availed of this opportunity to study the, subject as a whole and to compile his lectures in the form of this Book. He is also grateful for the encouragement received from the Honorable Minister of Revenue, Rao Bahadur Chaudhri Sir Chhotu Ram.

The author's thanks are also due to A. M. R. Montagu Esquire. C. I. E. Chief Engineer, Irrigation, for the trouble of going through the whole Book and then writing the foreword extremely useful and valuable suggestions to improve the Book were kindly supplied by him.

Last but not the least, the Author is specialy indebted to E. S. Crump, I.S.E. ; C.I.E. Retired Superintending Engineer under whose guidance, the author carried out experimental research work for more than 5 years.

Amritsar.
27th May 1944.

K.R. Sharma

Preface to 2nd Edition.

Part III of this book dealing with the Storages and Dams (Tank Irrigation) has been revised and enlarged. It comprises now of six chapters instead of four in the 1st edition.

Design diagrams are bound in volume III. These are intended to be used by the students in the class room and could be issued to the students in the Examination Hall for the solution of the problems dealing with practical designs.

The help rendered by Dr. J.K. Malohtra, mathematical officer Irrigation Research Institute East Punjab in editing the 2nd edtion of this book is gratefully acknowledged.

Amritsar.
1st. Decemper, 1948.

K.R. SHARMA

Preface to 3rd Edition

According to the requirements of the times, Part III, Volume II of this book, deeling with the Storages and Dams (Tank Irrigation), has been thoroughly revised and improved. Examination questions have been made up-to-date. They appear at the end of each chapter.

The help rendered by Shri K S. Pathak, Executive Engineer, Irrigation and Shri R.C. Sharma, Executive Engineer, Buildings and Roads, in the revision and improvement of this edition is gratefully acknowledged.

Jullundur City
15th October, 1959

PUBLISHERS

NOTATIONS USED IN THE BOOK

(A) Hydraulics.

Steady flow is that state of flow in a stream where the discharge across any defined section of the stream remains constant in respect of time.

Uniform flow. Uniform flow is steady flow in a stream when the depth does not vary with constant discharge.

Non uniform flow. When depth varies in steady flow in a stream with constant discharge, it is non-uniform flow.

Normal flow. Is that state of steady flow of a stream, where the fall of water surface corresponds to the consumption of energy by friction. It is stable. The principal symbols are subscribed with the letter 'n' to indicate this condition. (This is also called **Neutral flow**).

Regime flow. Is that state of stream, flowing in self borne alluvium, where there is neither silt nor scour. Regime flow also postulates normal flow as a preliminary condition. The principal symbols are subscribed with the letter 'r' to indicate this condition.

Critical flow. When water flows according to Bernouli's Theorem there is a particular condition of the flow, which is of special interest and is known as the critical condition of flow. This condition of flow is more simply called **Critical flow.** The depth and velocity are also called critical. Mathematically it is defined to be the condition of the flow when the change of velocity head is just sufficient to produce the change in elevation of the water surface. It has certain properties for which the student should refer to Part VI Chapter IV paragraph 4. The principal symbols are subscribed with letter 'c' to indicate the condition.

Constants.

C	Arbitrary or experimental co-efficients.
f_l	Lacey's "silt factor".
g	The gravity constant.
K	Theoretical constant in the "free fall" discharge formula ; $K=\sqrt{g}\,(\frac{2}{3})^{3/2}=3{\cdot}0888$.
N	The co-efficient of rugosity of a channel.
N_a	Lacey's co-efficient of Roughness in a channel.
w	The weight of 1 cubic ft. of water=62·5 lbs.
W	Total weight.

Discharges.

Q	The discharge in cubic feet per second (cusecs) of a channel or work.
q	The discharge in cusecs per foot width of a channel or work.

Energy.

E	The rotal energy expressed in feet head of water above a fixed datum. When plotted, this depicts the "total energy line".
F_1	The 'energy of flow' expressed in feet head of water above the bed. When the total energy line has been plotted the "energy of flow" is depicted by the intercept between the bed line and the total energy line. (In hydraulic books, the term specific energy of flow is used instead).
E_c	Energy of flow in critical conditions of flow.
h	The head or energy required to produce a valocity, $h=\frac{V^2}{2g}$
H	Depth on crest including velocity of approach head=$G+h_a$.
H_a	Afflux head
h_a	The head or energy equivalent to a velocity of approach V_a.
H_w	The available working head, *i. e.* the difference in total energy levels between two sections.
h_m	The minimum working head required between two points.

H_l	The head lost between two points, due to all causes except destruction in a standing wave.
H_L	The head or energy destroyed in a Hydraulic jump.

(Please substitute H_L for H_l where it has been used for the Head or Energy destroyed in a Hydraulic jump.)

Velocities.

V	The mean velocity in feet per second of a stream at any section.
V_a	The mean "velocity of approach"
V_n	The mean velocity of normal (Neutral) flow corresponding to D_n.
V_r	The mean velocity of regime flow corresponding to D_r.
V_c	The velocity of flow, at a section with critical flow.
V_o	Kennedy's standard silt-charge velocity. $V_o = 0{\cdot}84\ D^{0\cdot64}$.
x	$= \frac{V}{V_o} =$ critical velocity ratio (Kennedy's)

Dimensions.

L	The length of a channel or work measured along the line parallel to the direction of flow between two points.
L_t	The length of parallel sides of the throat of a weir, flume, etc.
B	The length of a channel bed transverse to the direction of flow. *i.e.*, the width.
B_t	The width of the throat of a weir, flume etc.
D	The depth below the surface of a stream bed at a stated point.
D_n	The normal depth of a stream corresponding to V_n.
D_r	The regime depth of a stream corresponding to V_r.
D_c	Critical depth corresponding to V_c.
A	The cross sectional area of a stream at a stated point.
P_w	The wetted perimeter of a channel.
B_s	The top or surface width of stream.
R	The hydraulic mean depth of a stream $= \frac{A}{P_w}$

Miscellaneous.

S	The actual slope of the total energy line at any given point.
S_w	The actual slope of the water surface at any given point.
S_b	The actual slope of the bed of a channel at any given point.
S_n	The slope of the total energy line in the case of normal flow.
S_r	The slope of the total energy line in the case of regime flow.
f	Co-efficient of friction.
s	The side of a trapezoidal channel.
G	The gauge reading. Zero of gauge must be specified.
P	The total pressure on a cross section of a stream.
p	The intensity of pressure at a stated point.
r	The instantaneous radius of curvature at a point in general, and for thin streams...........of the bed ; exactly...............the mean radius of curvature of of all the filaments in a stream.

(B) Hydrology.

Areas in square miles :—

A	Total area of a catchment.
A_e	Effective area of a catchment.
A_o	That area for which dispersion is unity or that area which can be wholly covered by a storm with unvariable intensity of storm.
A_s	Area covered by a particular storm.
A_h	Area between two isohyetals.
A_g	Area of influence assigned to a rain gauge station.

Rainfall or precipitation in inches :—

P	Meam annual total rainfall over a catchment,
P_a	Total annual rainfall over a catchment for any year.
P_m	Total monthly rainfall over a catchment for any month.
P_t	Total rainfall during an interval 't'.
P_s	Total rainfall of a particular storm as recorded at a rain-gauge.
P_h	Mean annual precipitation between two isohyetals.

Run-off (for volumetric studies) in inches :—

R	Mean annual total run-off from a catchment.
R_a	Total annual run off from a catchment for any year.
R_m	Total monthly run-off from a catchment for any month.
R_s	Total run-off of a particular storm.

Discharge for (intensity studies) in cusecs :—

Q	Maximum discharge from a catchment.
Q_s	Maximum discharge from a catchment on account of a particular storm.
Q_o	Maximum discharge from a catchment A_o.
q	Discharge per square mile of catchment.

Time in hours :—

T	Inlet time.
t	An interval of time.
s_t	Duration of storm.

Temperature in degrees (Fahrenheit) :—

θ	Temperature in degrees Fahrenheit.
θ_a	Mean annual temperature over a catchment.
θ_m	Mean monthly temperature over a catchment.

Miscellaneous :—

L	Distance in feet of watershed from the stream along the line of flow.
B	Absorption into the soil in inches of depth per hour.
E	Evaporation in inches of depth per hour.
D	Total loss in inches per hour $=E+B$.
I	Intensity of rainfall.
I_m	Maximum intensity of rainfall.
S	Maximum slope of catchment from watershed to the drainage.
F	Reduction in inches due to rain initially held by trees, crops and undergrowth.

To determine mean annual rainfall of a catchment :—

Exmapels.

(*a*) Isohyetal method :—

$$P=\frac{\Sigma P_h.A_h}{\Sigma A_h}=\frac{\Sigma P_h.A_h}{A}$$

(*b*) Weightage method :—

Extra notation.

P_g Precipitaion at any rain gauge station.

$$P=\frac{\Sigma P_g.A_g}{\Sigma A_g}=\frac{\Sigma P_g.A_g}{A}$$

(*c*) The straight average method (for a large plain area)

$$P=\frac{A_1P_1+A_2P_2+\quad.\quad.\quad.\quad.\quad.\quad A_nP_n}{A_1+A_2+\quad.\quad.\quad.\quad.\quad.\quad A_n}$$

Note :—The total area is divided into n sub-divisions, A_1. A_2.................... A_n and P_1. P_2....................P_n represent the average rainfall for each sub-division.

Run-off formulae (volume)

(a) Vermeule formula ;—$=RP-(11+0{\cdot}29P)(0{\cdot}35\theta-0{\cdot}65)$

(b) Khosla's formula :—$R=P-\frac{\theta}{2}+C$

Extra notation

C = A constant which allows for catchment characteristics humidity, glacier contribution etc., but not for absorption, evaporation and transpiration covered by the temprature factor $\frac{\theta}{2}$

Maximum discharge from a catchment (intensity) :—

(a) Inglis formula :—

$Q=\frac{7000A}{\sqrt{A+4}}$ for fan-shaped catchments, and

$Q=7000\sqrt{A}-240(A-100)$ for an elongoted catchment

(b) Khungar and Gulhati's formula :—

$$Q=645A_o\times\frac{Z_{max}-F}{T}\left(\frac{A_e}{A_o}\right)^m$$

m — Index of dispersion.

Z_{max} — Maximum height of a theortical hydrograph for a rainfall of maximum possible intensity in the catchment.

(C) Flow of water through sub-soil under weirs and dams.

Symbols :

H — Head in feet or difference in water levels upstream and downstream of a work (Percolation head).

P — Pressure head in feet in a pressure observation pipe measured above the downstream water level.

P_c — Pressure head at point C.

G — Gradient, or rate of change of head.

G_e — Exit gradient.

t — Temprature in degrees Fahrenheit.

ρ — Density of fluid.

μ — Viscosity$=\frac{{\cdot}00003716}{{\cdot}4712+{\cdot}01435t+{\cdot}000682\,t^2}$

η — Kinematic viscosity$=\frac{\mu}{\rho}$

Q — The discharge in cusecs of a channel or work.

q — The discharge in cusecs per foot of width of a channel or work.

ϕ — Pressure head expressed as a percentage of the total head$=\frac{P}{H}\times100$.

Determination of exit gradient :—

(a) *Floor with pile at dawnstream end* :—

Extra notation.

d — Depth of pile line below floor surface.

b — length of impervious floor.

α $= \frac{b}{d}$

λ $= \frac{1+\sqrt{1+\alpha^2}}{2}$

G_e $=$ Exit gradient $= \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$

(*b*) *Floor with pile at downstream end with step* :—

d_1 Depth of pile line below upstream impervious floor.

d_2 Depth of pile line below downstream pervious floor.

b_1 Length of impervious floor upstream.

G_e $= \frac{H}{(d_1-d_2)\sqrt{\lambda}} \times \frac{K}{1-K}$: where K is given by the equation.

$$\frac{\sqrt{1-K^2}}{K} \text{Cos}^{-1} K = \frac{\pi d_2}{d_1-d_2}$$

Pressures at different points on pile line or depressed floor :—

d_1 Depth of pile line below usptream floor.

d_2 Depth of pile line below downstream floor.

b_1 Length of impervious floor upstream.

b_2 Length of impervious floor downstream.

b b_1+b_2

ϕ_e ϕ at junction of upstream floor and pile line.

ϕ_d ϕ at bottom of pile line.

ϕ_c ϕ at junction of downstream floor and pile line.

Mutual interference of piles :—

$$C = 19 \frac{d_2}{\sqrt{b'}} \cdot \frac{d_1+d_2}{b}$$

C the correction to be applied as percentage of head.

b′ the distance between the two piles.

d_2 the depth of line whose influence has to be determind on the neighbouring pile of depth d_1

d_1 depth of pile on which the effect of pile d_2 is sought to be determined.

b total floor length.

(D) Dimensional Formulae.

Name of quantity.	Symbol.	Common English Unit	Dimensional formula in terms of mass, length and time.
1	2	3	4
Geometrical quantities.			
Length (any linear dimension)	L	ft.	L
Depth	D	ft.	L
Head	H	ft.	L
Diameter	d	ft.	L
Radius	r	ft.	L
Hydraulic mean depth	R	ft.	L
Wetted Perimeter	P_w	ft.	L
Area	A	$(ft.)^2$	L^2
Volume	V	$(ft.)^3$	L^3
Kinematical quantities.			
Time	T	second	T
Angular velocity	ω	Radians/Sec.	1/T
Angular acceleration	α	Radians/Sec.^{2}	$1/T^2$
Velocity	V	ft./sec.	L/T
Acceleration due to gravity	g	ft./sec.^{2}	L/T^2
Lacey's silt factor	f_l	ft./sec.^{2}	L/T^2
Kinematic velocity	$v=\frac{\mu}{\rho}$	ft.^{2}/sec.	L^2/T
Discharge	Q	ft.^{3}/sec.	L^3/T
Dynamical quantities.			
Mass	M	Mass	M
Surface tension	v	lb./ft.	M/T^2
Viscosity	μ	lb. × sec./ft.^{2}	M/LT
Pressure (unit stress)	p	lb./ft.^{2}	M/LT^2
Modulus of elasticity	e	lb./ft.^{2}	M/LT^2
Shearing modules	e_s	lb./ft.^{2}	M/LT^2
Density	ρ	Mass/ft.^{3}	M/L^3
Momentum	m	Mass × ft./sec.	ML/T
Force	F	lb	ML/T^2
Weight	W	lb	ML/T^2
Torque	P_q	ft. × lb.	ML^2/T^2
Energy	E	ft. × lb.	ML^2/T^2
Work	E_w	ft. × lb.	ML^2/T^2

(E) Greek Alphabets.

α	Alpha	λ	Lambada	υ	Upsilon
β	Beta	μ	Meu	ξ	Xi
γ	Gamma	ν	Neu	η	Eta
δ	Delta	o	Omicron	ζ	Zeta
ϵ	Epsilon	π	Pi	χ	Chi(ki)
θ	Theta	ρ	Rho	ω	Omega
ι	Iota	Σ (σ)	Sigma	ϕ	Phi
κ	Kappa	τ	Tau	ψ	Psi

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    (*c*) Note dated 5-6-39, 15-12-39 on Hydrodynamic Soil Pressure observation at Lahore.
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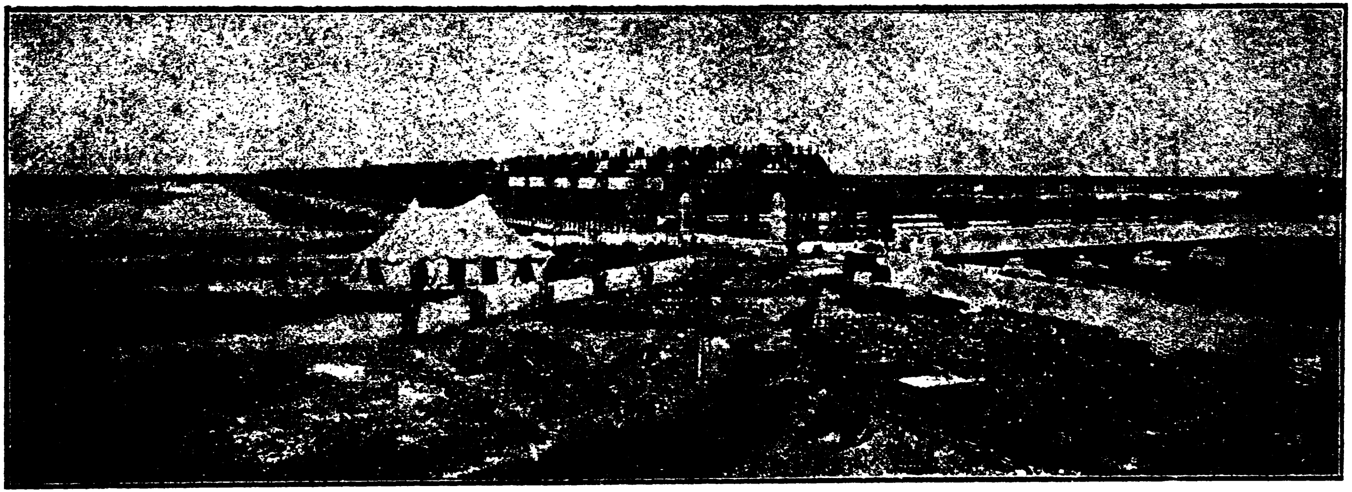
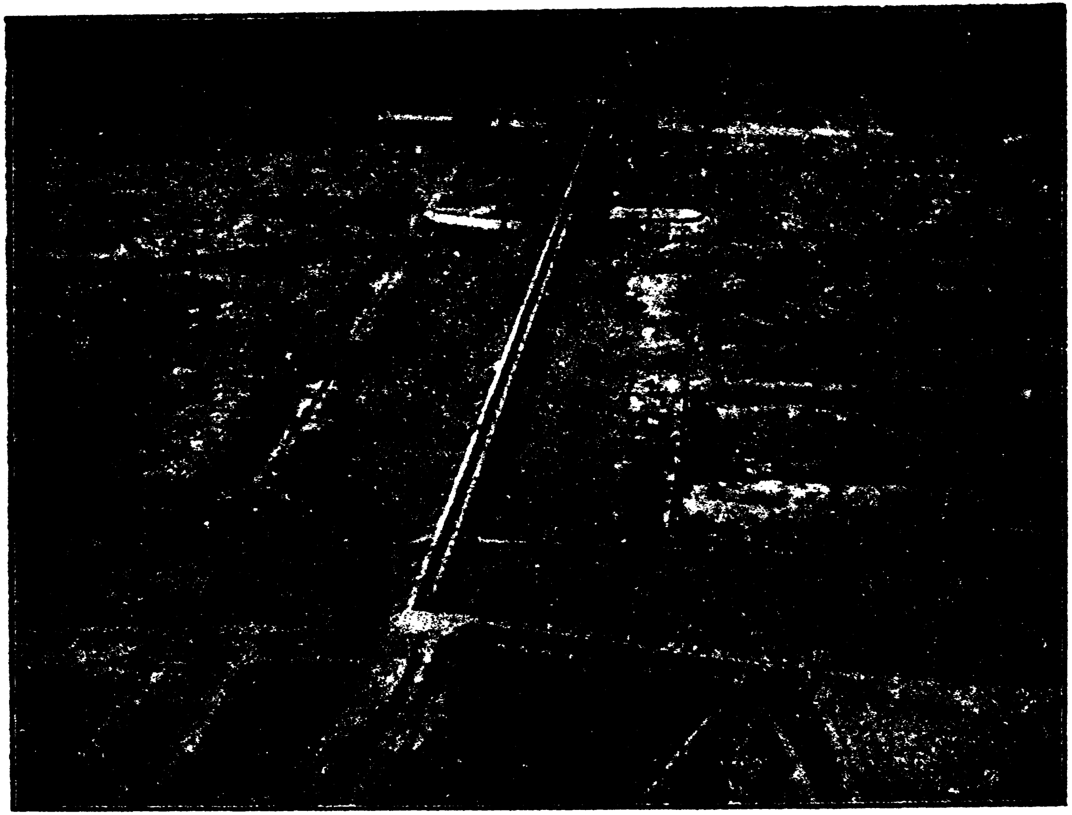
    (*k*) Note dated 26-3-40 on "Hydrodynamic Soil Pressure observations at Sheikhupura site.
34. "Division of the Punjab" by an engineer, June 1947.



Emerson Barrage at Trimmu. Fig. 3 (k), Page 119.



Aerial View of Punjnad Head Works. Fig. 3 (j), Page 119.

IRRIGATION ENGINEERING.

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Volume III

IRRIGATION ENGINEERING

Introduction

IRRIGATION is the art and practice of causing water to flow and spread over land with the object of nourshing the growth of plants. Plants may be agricultural crops, fruit trees in gardens, or trees in forest. From the point of view of art, it is a branch of Engineering and from that of actual practice it is one of Agriculture.

Plants require for their growth proper weather conditions of heat, light and moisture. In irrigating countries situated chiefly in tropical and sub-tropical regions, the first two are abundant and do not need to be supplemented. The last named is frequently deficient there and is the only one which can be supplied artificially on a large scale, and this is done by irrigation. In hot climates, even when the total rainfall is abundant, irrigation is required for certain crops such as rice on account of the uncertainty of rainfall and the need of constant supply of water. In countries with scanty rainfall, irrigation is an absolute necessity for the growth of agricultural crops and for producing water supplies for other necessities of life such as water for drinking and domestic purposes. Irrigation supplements rainfall to procure, all amenities of life to the human beings.

Irrigation may be natural or artificial. Natural irrigation occurs where rain falls, or where a river floods over the lands in its vicinity known as "*Sailab*" in Hindustani. Sometimes both of these natural systems of irrigation are controlled by the construction of suitable ***bunds*** around fields to retain rain water and by making embankments along rivers to regulate rivers spills.

Artificial irrigation is the artificial application of water to lands for agricultural purposes. The science of artificial irrigation may take any of the following forms and may involve the construction and maintenance of works pertaining to them :—

(*a*) LIFT IRRIGATION—Water is lifted from open wells, tube-wells, natural drains or canals and used for irrigation.

(*b*) CANAL IRRIGATION—Artificial channels taking off from the rivers are constructed. Water is then used for irrigation by flow from the distributing channels through outlets. In the case of permanent canals the river at the canal off-take is controlled by constructing Head Works. In the case of inundation canals there are no permanent Head Works.

(*c*) TANK IRRIGATION—Dams are constructed at suitable sites in the hills, where rainfall is abundant, for storage of water in reservoirs and then water is let off from them in channels which are used for irrigation of crops in the plains.

(*d*) DRAINAGE ENGINEERING—Drainage Engineering includes Hydrology and deals with the science of measuring and analysing the flow of precipitation of water (rainfall) evaporation and analogous phenomenan to provide the correct basis for estimation of the supplies available for irrigation purposes from the natural drains such as rivers. This further deals with the artificial channels known as surface drains or sub-surface drains, which are designed and constructed to drain off surplus rain water. The water from such drains is sometimes used for irrigation purposes, by lift irrigation.

(e) GROUND WATER ENGINEERING—This deals with the science of water occuring below the surface of the earth, its distribution and movement. This further deals with the dynamics of water-table movements as influenced by rainfall, irrigation and other factors. Water percolates to the underground reservoir from the canals, from other subsidiary irrigation channels, and from the irrigation fields. Irrigation results in excessive additions of water to the water-table which gradually rises under ground. The lands are said to be waterlogged when water-table rises so near the natural surface as to diminish the produce of the crops. The works required to reduce and prevent waterlogging, necessarly pertain to that branch of engineering which deals with irrigation.

The object of this book is essentially to deal with artificial irrigation involving works of the five types as enumerated above. This book deals with the technique of the construction and maintenance of works relating to all of these five branches of irrigation engineering with a view to equip the student with sufficient practical and theoretical aspect of the problems relating to them. A large number of solved examples of the design of irrigation works are given. Selected examination questions are given at the end of chapters, important for the students preparing for the Degree Examinations of the various Engineering Universities in India.

Italy may well be considered as the parent country in the science of distribution of water. It was the first country to take up large schemes involving large irrigation works. The Italian storage reservoirs and aqeducts have served as a guide to irrigation engineers in other countries. Indian Irrigation no doubt followed the foot steps of Italian practice. The first large canal was constructed more than a century ago in the United Provinces in India. The Ganges Canal is still a great engineering success, and was followed by large canals in the Punjab. The Punjab canals are first-class examples of the development of the science of irrigation engineering. The revenue return from the Punjab canals is the highest and the efficiency of their working surpasses all irrigation works in the world. It is a well known fact that the Egyptian irrigation system is founded on the Indian system. The science of Irrigation Engineering independently developed at great strides in the United States of America. Large irrigation schemes both in the forms of tank and canal irrigation have been successfully constructed there. The American practice undoubtedly now leads the world in the technique of the design and construction of high dams.

PART I

LIFT IRRIGATION

CHAPTER I

Irrigation From Open Wells

1. Definition.

Wells are merely holes in the ground to tap the underground water. They may be entirely unprotected, partially protected or entirely protected by means of lining of brick masonary. They are called open wells as distinguished from the tube-wells.

2. Source of Water Supply.

The source of water supply for open wells is the sheet of water found under the ground. This underground reservoir of water commonly called the water-table is fed by rainfall which, soaked into the soil, permeates under the ground. Water is held there by the presence of the impervious stratum below this sheet of water. The retentive substratum is nothing but the consolidated sodium clay hardened by the heat in the interior of the earth globe which increases towards the centre.

The laws relating to the flow of the sub soil ground water are dealt with at length in part V of this volume. There is only slight lateral flow in the sub soil reservoir towards the main drainages, *i. e.*, rivers; and as the country slopes from the foot of hills to the confluence of the rivers, the main underground flow occurs along the slope of the country.

The underground water is available in three forms :—

(*a*) Rock hollows.
(*b*) Artesian basins.
(*c*) Ground reservoir

Rock hollows filled with water are found in the hills, they are generally small and cannot be used for irrigation. The artesian basins are found near the foot of the hills. The rainfall permeating in the hills reappears on the natural surface on account of the high pressure gradient and the presence of the impervious soil consisting of coarse sand and gravel available near the foot of the hills. In some cases artesian wells and springs can be used for irrigation. Some springs found in the Himalayas have great medicinal value on account of the dissolved salts in water. In the plains, the ground reservoir is the chief source of water supply. Wells are dug down into the water bearing strata and water is lifted and used for irrigation.

3. Location of Well.

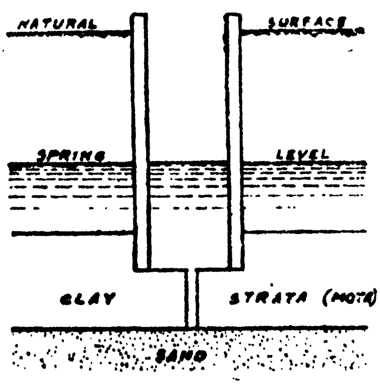


Fig. 1.

Suitable site for an irrigation well is that where a well can be founded in clay strata (called *mota* or *kar*) below sub soil waters level and the clay strata can be pierced to ensure permanent supply as shown in Fig. 1. In a sandy soil, a bigger well will be required for the same discharge to ensure no disturbance of the soil behind the lining or steining of a well, as the rate of supply depends on the velocity and the area of percolation. The soil movement is governed by the critical or optimum velocity of percolation.

4. Discharge of Irrigation Wells.

An irrigation well should be sunk about 20 to 25 feet below the spring level so that in years of draught there is a margin of about 10 feet leaving about 10 to 15 feet infilteration head.

The average discharge from an irrigation well, water being lifted by Persian Wheels, has been found to be 0·137 cusec (page 253 Farm Accounts 1933-34 Punjab). The discharge is more than this in the case of *mot* or *charus*. The discharge requirement of a well is never going to be more than 0·15 cusec.

The discharge formula to estimate the discharge from an open well may be used in the following form :—

$Q=\frac{4}{3}\,\sigma\, AH$ where Q = Discharge in cusecs.

A = Area of the well in sq. ft.

H = Infilteration Head or Depression Head (Percolation Head)

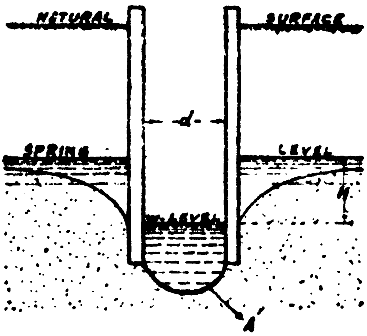


Fig. 2.

σ = Percolation intensity co-efficient in cusecs per unit area per unit head.

The constant $\frac{4}{3}$ represents ratio of the actual area A′ of the safe cavity formed to the area A of the well as shown in Fig. 2. The usual value of σ for the Punjab soil of the water bearing strata is $\cdot 75\times10^{-4}$ which gives a corresponding value of the Transmission constant of about ·00023 in the Darcy formula of viscous flow through the soil.

EXAMPLE.

Find the diameter of a well to give a discharge of 0·15 cs. with an infilteration head of 12·0 feet.

Let percolation intensity co-efficient per sq. foot $=\cdot 75\times10^{-4}$

Let the diameter of the well = d feet.

$$\text{Area of the well}=\frac{\pi d^2}{4}\text{ sq. ft.}$$

$$Q=1\cdot330\times\sigma\, AH=1\cdot330\times\cdot 75\times10^{-4}\times\frac{\pi d^2}{4}\times12=\cdot 15\text{ cusec}$$

$$\because\; d^2=\frac{\cdot 15}{\cdot 75\times10^{-4}\times12}=163$$

$$\therefore\; d=12\cdot 8\text{ say }13\text{ ft.}$$

For an average sand of 16/1000 inch average diameter in the Punjab, the optimum velocity of ·005 ft./Sec. has been found to move the finest soil particles in the experiments carried out in the Irrigation Research Institute, Lahore (vide page 228, Paper No. 248 Punjab Engineering Congress).

$$\text{The actual velocity of flow in the above quoted example}=\frac{Q}{A}=\frac{\cdot 15}{1\cdot 33\times\frac{\pi}{4}\times13^2}=\cdot 0009$$

ft./Sec. which allows approximately a factor of safety of 5. A factor of safety of at least 3 should always be allowed to provide for the years of draught when the well has to be worked under the worst conditions.

5· Suitability of well water for crops.

All sub soil waters are not suitable for irrigation purposes. Water of the well should be tested to examine the amount of salt contained in solution. The irrigation Research Institute, Lahore has developed a technique in evolving an expression to which the name of Salt Index is given. This is a measure of the quality of irrigation water in relation to the soils. The sodium salts present in the irrigation water cause base reaction to take place in the soil with its consequent deterioration. If calcium salts are present in the irrigation water, the reaction is opposed and delayed some what.

Salt Index=(Total Na—24·5)—(Total Ca—Ca in $CaCO_3$) 4·85. When the expression is negative, the water is suitable for irrigation of the crops, and when positive it is unsuitable. The brackish water as found in the Western Punjab is unsuitable for irrigation, while the sweet water of the Jullundur, Hoshiarpur, Amritsar, Gurdaspur and Sialkot districts is very good for irrigation.

6. Classes of wells.

(*a*) Impervious lined wells.

These are the only types of wells used for irrigation purposes. They are usually of type shown in Fig. 2. The design and the process of sinking such wells is described in detail in the subsequent paragraphs. When the bottom of the well is in sandy soil, it is usual to provide inverted filters at its bottom so that a dangerous cavity is not formed during the years of draught. The inverted filter simply consists of a layer of coarse sand at the bottom, a layer of fine gravel in the middle and a layer of *bajri* of about $\frac{3}{8}''$ to $\frac{1}{2}''$ size at the top. These layers are about one foot thick each.

(*b*) Pervious lined Wells.

A hole is dug in the ground to the water-table and dry brick or stone lining is then laid on a well curb up to a height of 8 or 10 feet. The soil inside is scooped and well can be sunk easily 8 to 10 feet below the spring level. While sinking is done, the sand will no doubt flow out from the joints. This will be very much reduced, if brick ballast of size $\frac{3}{4}''$ to $1''$ is filled around the lining, so that it may sink along side the lining. Such wells are economical in the case of small depths. And when the discharge required is not much, water percolates from the sides also. They are quite successful when the arrangement for lifting water is *Rati* or *Denkli*.

(*c*) Kacha Wells.

This supposes that clay stratum is available from the natural surface down to a few feet below the spring level and that the soil is strong enough to stand without the help of the lining. Such wells are used for low lifts up to 10 to 12 ft. by means of *Rati* or *Denkli*.

7. Lined or Pacca Wells.

All open wells used for irrigation are circular. The thickness of the lining or steining is usually 2 bricks *i.e.*, 1·5 ft. for depths up to 40 or 50 ft. For depths up to one hundred feet, the thickness of lining should be 2½ bricks, *i. e.*, 1·9 ft. Open wells for depths exceeding 100 ft. are never built, because lifting of water becomes very uneconomical. The lining is built in lime mortar of proportion 1 : 2 (one lime and 2 *surkhi* or sand) or cement mortar of

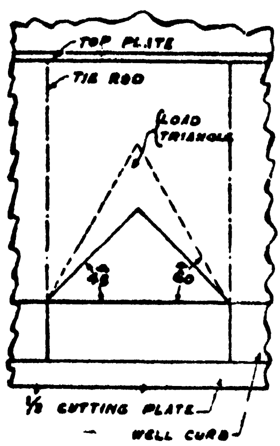


Fig. 3 c.

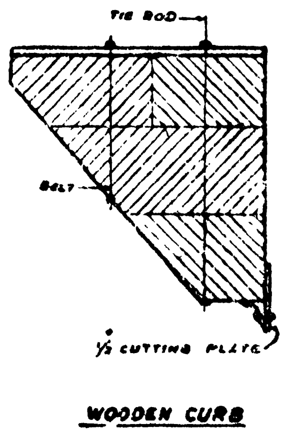


Fig. 3 a.

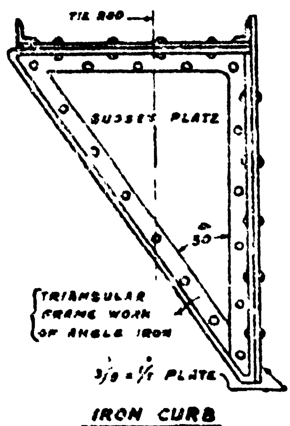


Fig. 3 b.

proportion one cement and 4 sand. When a large number of wells are required to be built, it is advisable to get special bricks burnt which conform to the circular shape of the well. Ordinary flat bricks need a lot of labour to dress them to the circular shape, and also require an excessive quantity of mortar to fill the joints. When special bricks are used, 25 cft. mortar will do for 100 cft. masonry, but for the ordinary bricks 33 cft. will be needed. The lining is built on a well curb. The well curb may be of wood, iron or reinforced concrete. Sections of suitable designs for wooden and iron well curbs are sketched in Fig. 3a, 3b.

The wooden well curb is made of hard wood as *Kikar*, *Shisham* or *Sal*. The iron curb is made of 6 to 8 triangular frames of angle iron covered by $\frac{3}{8}''$ to $\frac{1}{2}''$ plate all round. There are usually 6 or 8 tie rods of $\frac{5}{8}''$ to $\frac{3}{4}''$ diameter. The load of lining on the curb is shown in Fig. 3 (c).

The well curb is a continuous bent beam in between the tie rods supporting the triangular wedge of masonry of isosceles shape with angle 45° when the masonry is in cement mortar, and with angle 60° when the masonry is in mud or lime mortar. The lengths of the rods should be at least double the height of the weight triangle of masonry. The tie rods are anchored at the top in a circular plate iron $\frac{3}{8}''$ to $\frac{1}{4}''$ thick and about 6″ wide.

The section of the wooden well curb can be designed according to the formula :—

$$M=\frac{d^2}{2\sqrt{2}} \text{ or } d^2=M2\sqrt{2}$$

where d = depth in inches

M = bending moment in inch tons.

It is supposed in deriving this formula that

the safe tensile stress in wood is $\frac{1}{2}$ ton/sq. inch and that the section is triangular with top width 12″ R. C. section of a curb can easily be calculated considering it to be a beam of triangular section carrying a wedge load in between the rods as shown in Fig. 3. (c).

8. Construction and sinking of wells.

(*a*) A circular pit is dug in the ground up to the water-table or up to the bottom of the clay strata, which ever is higher. The pit should be sufficient in circumference to allow a space of at least 2 ft. around the lining. In clay the sides of the pit can usually stand vertical.

(*b*) The well curb is laid when soft soil or water-table is reached.

(*c*) Masonry of the required thickness is, then, built on the well curb to a height of about 8 to 10 ft. above the ground level.

(*d*) Then a temporary platform is made on the top of lining to support the weight of gunny bags filled with earth to facilitate its sinking. The load is placed on the outer sides of the platform leaving a clear space in the middle sufficient for lifting the material dug by means of a pulley arrangement laid over the platform.

(*e*) The soil within and below the curb is gradually scooped by means of *Kassies* to fill a big bucket which is pulled out by the pulley arrangement. The loaded curb thus sinks gradually.

(*f*) It is necessary to see that the curb with lining sinks vertically. To watch this 4 plumb lines are hung at equal intervals on the outer circumference of the steining. If the steining gets out of plumb, the workmen are instructed to cut the obstruction under the curb on the side which lags behind in sinking. The odstruction may be in the form of roots of a tree or pieces of stone. If there is no such obstruction, it is then a case of the unequal soil friction against the lining. The weights on the platform are then shifted to the side which is sinking slowly. If unequal loading cannot cope with unequal skin friction, then the soil behind the lining should be wetted by using water jet under pressure.

(*g*) The digging of the inside soil by men is usually possible up to five or six feet below the spring level by continuously bailing out water. When the ingress of water is more than the removal, then digging by means of *kassies* cannot proceed. Further digging is done by a *jham* which is a large *kassi*. A diver dives and fills the *jham* with soil, which is lifted by the pulley arrangement. This is no doubt a slow process. Wells have heen successfully sunk from 20 to 25 ft. below the spring level by this method. The modern dredgers like the bull's dredger clench the soil by weight of the blades and when the chains are pulled, the blades automatically close with the soil filled in the dredger. The digging of soil under water is very much faciltitated by this method.

(*h*) When the curb with steining has thus been sunk to the required level, the space around the lining is filled with good clay earth consolidating it by ramming and watering in successive layers of about a foot depth.

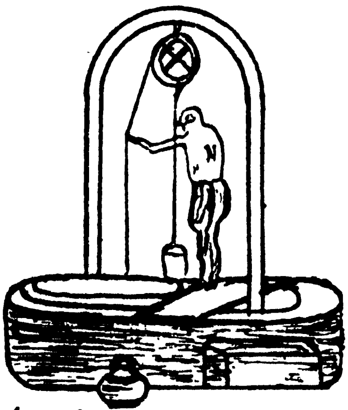
(*i*) Finally the top parapets are built on the top of the steining to suit the water lifting arrangement which is usually a Persian Wheel.

9. Methods of Raising Water.

The following water lifts are more or less in general use in India :—

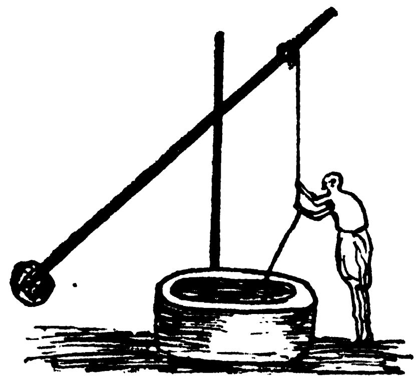
1. *Rati* or pulley block arrangement for small garden plots.
2. *Denkli* or lever for small gardens and vegetable plots.
3. *Charus* or *mot* or leather bag for agricultural crops.
4. Persian Wheel for agricultural crops.

These arrangements are shown in Fig. 4 (a to d).



LIFT IRRIGATION - THE RATI

Fig. 4 a.



THE DENKLI

Fig. 4 b.

LIFT IRRIGATION - THE CHURUS

Fig. 4 c.

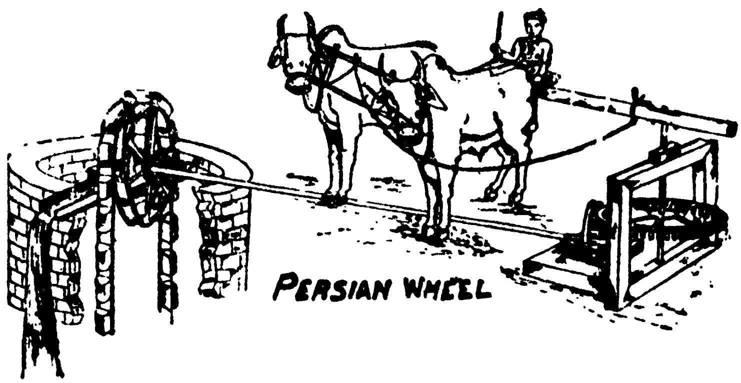


Fig. 4 d.

In very few cases, the water lifting arrangements have been modernised in the form of chain pumps, ordinary suction pumps worked by bullocks, and modern pumps worked by the electric motors or the oil engines. The efficiency of different water lifting arrangements was tested in experiments by Captain Clibborn given in Roorkee Treatise on Irrigation Works Vol: 1, reproduced below :—

No.	Class of lift	Lift in feet	Labour	Foot tons per hour per head
1	Rati	17	Man	32·26
2	Denkli	15·4	,,	21·99
3	Chain Pump	15·0	,,	30·55
4	Charus	32·0	Cattle	58·8
5	Persian Wheel	30·0	,,	57·1

A ton of water raised one foot ton.

10. Duty of well water.

The duty of a well cannot be defined in terms of acres per cusec discharge because there is no fixed and continuous discharge.

Area to which a well gives protection in the case of draught is defined as the duty of the well. It is thus the average area irrigated from a well per annum.

The experiments carried out in the United Provinces by Mr. Anthony gave the following figures :—

No.	Class of lift	Lift	Duty in acres per annum
1.	Charus with cattle	25	3·91
2.	Charus with men	28	2 76
3.	Denkli	15	91
4.	Rati	18	·8

This is a very poor show, but in the Punjab, the usual duty of a well worked by *charus* or Persian Wheel is 25 to 30 acres. Usually there are three shareholders having an area of eight to ten acres each. The higher duty for the Punjab wells is due to the following causes :—

(*i*) In the Punjab the wells are generally fitted with Persian wheels which are suitable for lifts of 20′ to 50′.

(*ii*) The cattle and men in the Punjab are hardy and, therefore, capable of putting in strenuous effort.

(*iii*) Holdings is the Punjab are small and, therefore, water is used economically.

11. Delta of Well Irrigation.

Delta is defined to be depth of water required in feet to mature a crop. The usual depth of first watering form a well is three inches in a ploughed field and the subsequent waterings are of 2" depth each. The average depth of watering from statement No. 1. Farm Account 1933-34. Punjab Agriculture Publication No. 46, works out to be ·193 ft. or 2·33 inches. Probable deltas for various crops in the Central Punjab with average rainfall of 20" per annum (15" in monsoon months of July and August) is given in the table below from personal observations :—

No.	Crop	No. of waterings.	Delta in feet.
1.	Wheat	4	·80
2.	Cotton	5 to 6	1·0
3.	Vegetables	6	1·2
4.	Potatoes	8	1·6
5.	Fodder crops Chari, Maize and Senji	3	·6
6.	Shaftal Berseem	6	1·2
7.	Sugarcane	8 to 10	1·8
8.	Maize	5 to 6	1·0

A comparison of well delta with the canal irrigation figures in Part II Chapter II will show that about $\frac{2}{3}$ water is used in well irrigation as compared with canal irrigation. The yield of crops from well irrigation is usually somewhat better than that from canal irrigated crops on account of timely waterings and better manuring facilities.

The reasons for low delta in the case of well irrigation are :—

(*i*) The cultivator is aware of cost and the labour employed by him to lift water and, therefore, he uses it very economically by resorting to small *kiareis* in his fields.

(*ii*) Well water-courses are usually well maintained and there is no wastage from breaches and overflows.

(*iii*) Well water-courses are always kept clear of all jungle and grass and are often well consolidated and plastered with mud to reduce absorption losses.

(*iv*) Water is lifted according to the requirements of crops at the will of the *zamindar*. He has not to wait for his turn as in the case of canal irrigation.

12. Cost of well Irrigation.

The cost of well irrigation from Persian wheel is worked out on page 253, Farm Account 1933-34 Publication No. 46 of the Punjab Agriculture Department.

Cost of well	Rs. 774/-
Area cropped	17·4 acres
Average number of irrigators	4
Time taken to irrigate one acre	2·84 days

COST PER ACRE

	Rs.	a.	p.	
Overhead charges ...	11	13	0	
Bullock labour ...	11	4	0	
Manual labour ...	11	12	0	
Total Rs. ...	34	13	0	per acre.

Cost excluding manual labour=Rs. 23/1 say Rs. 23/-

The manual labour does not cost a cultivator anything as he or his children usually supply the requisite labour. The overhead charges for the cost of a well etc., are also very excessive. Bullocks will also be used for ploughing and cartage and other miscellaneous work. The well irrigation is definitely many times more expensive than the canal irrigation.

The Irrigation Research Institute, Lahore, has also worked out the expenditure and income of one family in Jullundur District possessing ten acres of land on a well with one third share in the well. The gross receipts in a year work out to Rs. 741/- while his actual expenditure is Rs. 383/- for paying land revenue, casual labour, bullocks, share of well and Persian Wheel, seed and *kamin* and miscellaneous expenditure. His net income is about Rs. 358/- per annum.

13. Well Irrigation Versus Canal Irrigation.

Well irrigation has got a definite advantage over canal irrigation, because a cultivator can raise water when it pleases and suits him.

In certain tracts, well water is more beneficial to the crops as in the Bist Doab in the Punjab.

The well irrigation should be encouraged parallel to the canal irrigation as it is an effective anti-waterlogging measure and serves to keep the water-table low. Well irrigated tracts should be debarred from canal irrigation when new irrigation is extended to the tract, so that wells may not become extinct. Canal water should be given to about 50% intensity of the irrigable area so that the well irrigation should also develop.

The canal irrigation in the Punjab has a definite advantage over the well irrigation in the manuring value of the canal water carrying fine silt and clay in suspension. All canals take off from alluvial rivers.

14. Examination Questions.

1. (*a*) Define duty of a well.
   (*b*) What factors govern the delta irrigation from open wells ?
   (*c*) Why is delta for open well irrigation the lowest as compared to the delta for tube-well or canal irrigation ? (*P U. 1941*)
2. Describe the various appliances used in the Punjab for lifting water from open wells and which is the best in your openion ?
3. How will you classify the soil in the Punjab with respects to the clay contents ? Mention the crops which will flourish in each type of the soil.
4. Describe the process of sinking an open well and the precautions that you will take so that the steining of the well does not get out of plump while sinking.
5. (*a*) How would you judge the suitability of well water for agricultural purposes ?
   (*b*) What do you understand by (*i*) Salt Index (*ii*) Salinity of water ?
6. (*a*) What is the average discharge from an open well in the Punjab ?
   (*b*) How much below the water-table, an open well meant for irrigation should be sunk in the Punjab conditions ?
   (*c*) What factors determine the diameter of an open well meant for irrigation ?
7. What are the different types of wells used for irrigation and what are the average discharges for each type used. Why is delta for well irrigation lower than for canal irrigation. (*P U. 1954*)
8. Define "Duty". Explain "Base of Duty". How can "duty" under an existing system of irrigation be improved ? (*P.U. 1949*)
9. What is a suitable site for location of a well ? Find the diameter of an open well located in Punjab to give a discharge of 0·18 cusec with an infilteration head of 15ft. Co-efficient of permeability $\cdot 75 \times 10^{-4}$ ft. per second. Explain circumstances under which a well settles down or is liable to crack. What is the approximate factor of safety against settlement in the above example ? (*P.U. 1958*)

PART I

LIFT IRRIGATION

CHAPTER II

Tube-Well Irrigation

1. Tube-Wells.

Open wells are only suitatable for small discharges up to 0·15 cusec. It is not economical to lift a small supply by means of mechanical appliances ~~such as electric~~ motors or oil engins. Tube-wells, therefore replace the open wells in the case of large irrigation schemes. A tube-well is simply a pipe sunk into the ground fitted at bottom with a filter usualy called a strainer. Straners from 100″ to 150″ length are located in the water bearing strata. An average discharge from a tube-well used for irrigation is about 1·5 cusecs. The under ground reservior is the source of water supply as in the case of the open ~~well.~~

2. Water-Bearing and Non-Water-Bearing Formations.

Beds of sand and gravel —Beds of sand and gravel are very porous, as much as 30 per cent to 40 per cent of the volume of some being open space ; so that saturated layers of sand and gravel penetrated by wells yield copious supplies. In general water from sands and gravels in the sub-soil gets filtered naturally and is pure. The sand and gravel strata in the indo-Gangetic plain are the chief sources of tube-well water in North India. In some cases we get dry granite sand under pressure but without water. This sand contains no clay and when a tube-well bore reaches to this stratum the loose sand rushes inside the bore. In such a case an inexperienced borer is likely to be misled by the coarseness of the sand and may think that he has tapped a water-bearing sand ; but he should be very careful to examine the nature of such strata, and a bailer test star will quickly show whether the sand stratum is aquiferous or not.

Pure clay.—Pure clay is nearly impervious to water. Water is sometimes reported in clay but as a rule it comes from more or less sandy layers within a clay bed. In some places, sand that approaches clay in fineness and is mistaken for clay yields much water. Clay is however of great importance as a confining layer to porous sand. When it is neccessary to obtain water from a clayey stratum, through open wells, they should be as large as practicable and deep enough to provide ample storage, as clay yields only a small quantity of water and that too very slowly.

Till.—Till is a heterogenous mixture of clay, sand, gravel and boulders. In texture it ranges from very pervious to impervious varieties according to the predominance of sand or clay. In few places is it definitely bedded. Water generally occurs in it in minute, more or less tubular channels but occasionally is distributed through interstratified sandy beds. In finer, more loamy phases, the supply is not abundant, but coarser portions furnish water more plentifully. In the aggregate "till" yields a large quantity of water, and where sufficiently thick, forms a convenient and accessible source but because so irregularly disposed, the success of wells varies greatly. "Till" is widely distributed in the underground strata in Baroda State.

Sand-stone.—Sand-stone is, of solid rocks, the best water-bearer. Water from sand-stone is of better average quality than that from any other material except sand and gravel. In Baroda State porous sand-stone in thin layers is occasionally met with but the quantity of water in it is small.

Conglomerate.—Conglomerate is a mass of sand, gravel, etc., embedded in a matrix of finer material. Conglomerate may yield some water if it contains a large proportion of porous sand, but its absorptive capacity is not so great as that of sand-stone. It is widely distributed in thin layers in Baroda State.

Quartzite :—Quartzite is a metamorphosed sand-stone, the space between grains being filled with hard siliceous matter. Because of this filling of pores, there is little chance for water to get stored in such strata.

Shale :—Shale is a kind of clayey stone splitting readily into thin plates. This is a poor water-bearer, but may yield some from bedding joint or cleavage planes, and other crevices.

Lime-stone :—In lime-stone water occurs mainly in channals and caverns formed by solvent action along joint or bedding planes. These are very irrigularly distributed and location can seldom be determined by examining the surface. The water in lime-stone beds is hard.

Granite and gneiss :—Granite and gneiss are of dense compositions and have very small pore spaces and so are not water-bearers.

Trap :—Trap is impervious rock but water gets collected in it through fissures and is held in these or in the crevices of the stratified trap. We get water in trap if there are fissures in it. This is found in Amreli Baroda and Navsari Divisions of Baroda State.

3. Object of Tube-Well Irrigation.

The object of installing tube-wells is likely to be three fold in the Punjab :—

(*a*) All available supplies of the Punjab rivers have already been tapped by the existing canal system as limited by river the discharges at the time of sowing of *rabi* and *kharif* crops. There are still large tracts of land in the Punjab where irrigation water is badly needed. The canal supplies can no doubt be supplemented by resorting to tank irrigation as dealt in Part III, but ground water is an other valuable and readily availalbe source of water supply which can be, easily tapped to increase the irrigated areas in the Punjab.

(*b*) The tube-well irrigation can readily supply the load necessary for the full development of the Mandi Hydro-Electric Scheme which has already been executed but has not yet developed its full load.

(*c*) In the waterlogged areas in the Punjab, the resort to the new tube-well irrigation or the replacement of the canal irrigation will relieve waterlogging trouble to a great extent and will also reduce the expences on the anti-waterlogging measures.

4. History of the Development of the Technique.

No large scale schemes for tube-well irrigation have so for been carried out in the Punjab Some of the attempts made in the Punjab and the United Provinces are given below.

(*a*) Amritsar scheme of 1910.
It was intended to lower the water-table near Amritsar. This led to the introduction of Ashford strainer and the Washable Leggett strainer. The scheme failed and was closed in 1936, because the discharges of the wells fell down, and because they were too few to have any effect on the water-table.

(*b*) Mr. T. A. Brownlie, the Agricultural Engineer to the Government of the Punjab devised strainers knows as "Gonvoluted" and "Compslip," the latter is supposed to remove the trouble of clogging for ever.

(*c*) Qadian Irrigation Scheme of 1934 of Mr. P. D. Tandan.
Three tube-wells were sunk and one of which failed subsequently, but two of them are doing successful irrigation with the promised discharge.

(*d*) Karol Area Tube-Well Project.
The scheme was prepared by Mr. A. M. R. Montagu and executed by Messrs. H. L. Vadhera and A. R. Talwar. The results have been published in paper No. 248, Punjab Engineering Congress 1941. The scheme consists of 31 tube-wells to irrigate finally 17389 acres. 20 tube-wells are at presnt installed and are working. It is very successfull so far.

(*e*) Ganges Canal Hydro-Electric Scheme was conceived and carried out by Sir William Stampe K. C. I. E., Chief Engineer, United Provinces, India. The energy has been developed from the low head falls on the Ganges Canal. The complete scheme envisages the generation of 30,000 Kilowatts of which more than 24,000 are from canal falls. The bulk of the power is used to energise 1500 state operated tube-wells with an average discharge of 1·5 causes each. It is meant to irrigate 1300 square miles area. Tube-wells are expected to run for about 3000 hours per annum. This scheme has proved a financial success and the returns are very promising. This is the biggest undertaking in India for Irrigation from tube-wells.

5. Boring of tube-wells (*An extract is given from Bib. 28*).

Choice of site for boring.

(*A*) **Water divining.**—Considerable difficulty often arises in determining a suitable site for a boring. But for locating springs or flowing water near the surface there are often other indications which may prove valuable.

Water divining has recently received much attention and there is abundant proof available to show that the art of "dowsing" is a fairly reliable means of detecting the presence of flowing water near ground level. Many theories have been formulated to explain the phenomenon but no proof is yet available. (Bib. 1.)

There are indications which may lead to the discovery of springs or water near ground level, in cases where nothing would appear, to those unaccustomed to observations of natural phenomena, to induce a belief in their existence. The following are some of the simplest. (Bib. 1.)

In the early part of the year, if the grass assumes a brighter colour in one particular part of a field than in the remainder, or, when the latter is ploughed, if a part is darker (or damper) than the rest, it may be suspected that water will be found beneath it.

In summer, the gnats hover in a column, and remain always at a certain height above the ground over the spots where springs are concealed. In all seasons of the year, more dense vapours arise from those portions of the earth's surface which, owing to the presence of sub-terranean springs are more damp especially in the morning or in the evening.

In selecting sites for wells to tap the upper layer of sub-soil water it is also necessary to study the plant life in the locality. It is common knowledge that certain trees thrive well where the sub-soil water is more plentiful or nearer to ground level.

These rules apply generally to springs near the surface. Where the source is lower these are rarely sufficient. It is claimed that with the aid of a "divining rod" or electrical instruments springs several hundered feet below ground level can be located fairly accurately. For further information on this subject a reference may be made to "The Physics of the Divining Rod," by Maby and Franklin, published by Bell and Sons Ltd., London, S. W. 2. But experience with these instruments has not been very encouraging and the only safe guide for correctly determining deep sub-soil aquifers is a trial boring.

(*B*) **Boring for Wells.**—The diameter of boring tubes is measured on the outside as against ordinary pipes which are measured on the inside, because in the former case we are concerned with the size of the bore made. This point should be taken note of when ordering boring tubes from foreign countries. In practice both the methods of measurement are being used by the borers almost indiscriminately ; and in India generally inside diameter is measured, for the sizes of boring tools strainers, pipes, etc., have to be adapted to the inside diameter of the boring tubes. A safe course is to mention the outside diameter as also the thickness of the boring tubes when placing an order for these.

Another point, about which there is a good deal of confusion is regarding the nomenclature of casing pipes, blind pipes, blank pipes, plain pipes, solid pipes, etc., and these words are used indiscriminately and in different senses by different borers.

The generally accepted practice which is adopted in this book is as below.

All pipes and tubes used for boring have been described as "boring pipes" and "boring tubes," or "casing pipes" and "casing tubes."

All pipes used for the tube-well itself (whether between strainer as in a strainer well or in continuation with a slotted or perforated pipe as in the case of slotted tube-wells and perforated pipe wells, or in cavity wells where "boring pipes" are not used) are described as "plain pipes" or merely as "tubes."

The small length of plain pipe (generally 4 to 5 feet) used at the bottom end of a strainer or slotted tube-well and having a cap or bail-plug fixed to it is described as "blind pipe."

In the case of compound wells the larger diameter plain pipe on top, in which the bore-hole pump bowls are housed is sometimes described as "housing pipe."

The use of the words "blank pipes" and "solid pipes" has been done away with as it is confusing.

Though boring requires skill and care, yet in principle it is extremely simple. The operation consists, as its name implies, in boring a hole of a diameter varying according to circumstances, and in a vertical direction. Many systems have been and are now followed in carrying out this kind of work.

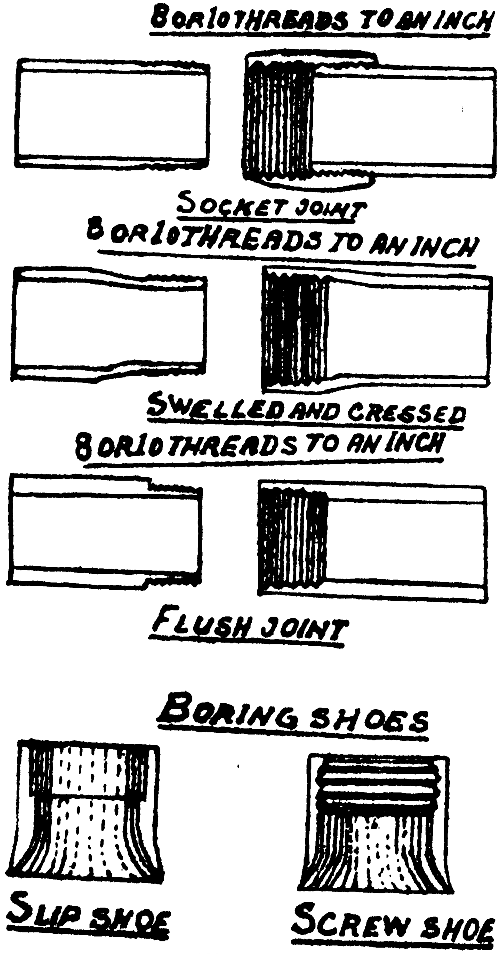


Fig. 1.

(*C*) **Boring System.**—Boring system may be divided into three main classes, *viz.*, percussion boring, by which a hole is made into the ground by reducing the strata to powder, water jet boring which makes a bore by washing out the strata, and rotary boring by which a hole is drilled into the strata and a solid core obtained. The percussion system is suitable for ordinary sand or gravel strata, while hard tenacious clay is better penetrated with a water-jet plant and the rotary system of boring with core drills is adapted in the case of firm and hard strata. The driven tube is the simplest form of bore hole. This method is rarely employed for larger wells.

(*i*) (*a*) **Percussion rope boring.**—The old system of well boring is the original Chinese rope boring system which is described in the beginning of this book and is of great antiquity. The present method of rope boring is described below :

A pit is dug at the site where boring is to be done, about 6 to 8 feet in diameter and about 15 to 20 feet in depth, and the boring tube is lowered into it with a cutter shoe fitted to the bottom of it. These cutter shoes can be "slip-shoes" or "screw-shoes" and are of slightly bigger diameter than the boring tube itself so that they may provide a clear passage for boring tubes as they are sunk. These shoes are made of tempered steel Fig. 1.

The slip shoe as its name implies is slipped on to the boring tube when strating boring and left at the bottom of the bore when the tube is withdrawn. The slip-shoe is of a larger diameter than the screw-shoe for the same diameter of tube. This shoe is useful in stiff clay, rock soil etc., and for deep bores as it clears a slightly larger hole than the screw-shoe and thus the friction on the outer surface of the boring tube is reduced both in sinking and in withdrawal for the clearance created by the larger shoe remains open to some extent while the tube is being withdrawn. On the other hand the screw-shoe is screwed to the bottom end on the bore tube and comes out with the tube when it is withdrawn. In this case as the screw-shoe has to be withdrawn with the boring tube there is no extra clearance left while withdrawing. Therefore a screw-shoe is chiefly used for sandy soils.

Screw-shoes are generally good for the average size tube-wells in India and with experience this practice has been established here. There should be female threads to the shoes to receive the screwed end of the boring tube which should fit in butt to butt with the shoe. In the case of very deep borings, or borings in clayey strata, the bore is started with a larger diameter boring tube and carried down till the friction on its outer surface makes it difficult to carry on the boring any further without damage to the tubes. A slightly smaller size boring tube is then lowered inside the larger size tube. The boring is next continued with the smaller size tube which is free from any frictional resistance to the bottom of the larger size tube. If necessary a still smaller size of boring tube is lowered inside the inner tube when the friction on the latter also becomes excessive after carrying on the boring lower down. Thus the diameter of the bore-hole is successively reduced as the conditions demand. All the tubes should however extend right up to ground level or above the bed level of the pit in which the bore is made, so that they may all be extracted with jacks when required.

This system of lowering boring tubes one inside the other to distribute and overcome frictional resistance in the sub-soil strata is known as "telescopic boring." Care has to be taken to select the size of the various boring tubes in such a way that the inside diameter of the inner-most tube, reaching the bottom of the bore, is sufficiently large to allow the lowering of tube-well pipes and strainers inside the finished bore.

The boring tubes are steel tubes with the following types of joints :—

(1) Flush joints.

(2) Swelled and cressed joints.

(3) Socketed joints.

These are shown in Fig. 1. The flush joint is the best for ease in lowering and withdrawal but these tubes have to be thicker than socketed pipes or swelled and cressed pipes to have sufficient strength at the joints.

After being lowered in the pit as described above, the boring tubes are clamped in position with wooden clamps, and partly filled with water. Water is necessary for boring operations and must be poured from the top of casing pipe until water is struck in the bore-hole itself. The actual boring is done with a tool known as a "sludger" or "sand pump". This implement is a steel pipe of about 1/4" thickness, varying in length and diameter according to the size of well to be bored. It is about 2" less in diameter than the casing pipe and its length varies from 8 feet to 12 feet for borings from 8 inches to 16 inches in diameter. A cutting shoe of "hard steel" or tungsten steel, is riveted to the sand sludger at its lower end. The cutting shoe is tapered out slightly at the bottom to give it a clearance in cutting the material at the bottom of the bore. A flap or ball valve is fitted inside the sludger pipe just above the cutting shoe.

The sludger is suspended by a wire rope from a pully on a tripod. The legs of the tripod are to be buried about two feet under ground to prevent their sliping or tilting. The leg opposite the carb winch has to be securely buried and anchored so that the tripod will not turn over when subject to a pull from the winch. The sludger comes centrally over the boring tube and can work smoothly inside it. When the sludger is worked vertically up and down by manual labour, or an engine with cranking arrangement, etc., a circular motion is also automatically imparted to it by the torsion in the wire rope. On the downward stroke of the sludger the flap valve is forced open and the loose material pounded at the bottom of the bore-hole enters into the sludger pipe. As the up-stroke begins the flap valve closes thus retaining the loose material inside the sludger pipe. At the same time the up-stroke creates a momentary vaccum at the bottom of the bore as the sand sludger is lifted upwards. Owing to this vaccum or suction the pounded loose material in the bore is sucked up and gets better mixed with the water at the bottom of the bore. The loose material thus pulled up and remaining suspended in the water enters into the sludger pump on the next downward stroke of the sludger and is retained by the valve. At the end of the downward stroke the cutting shoe cuts some more of the material at the bottom of the bore which is pulled up and enters the sludger in subsequent storkes. After about 30 to 40 strokes the sludger is taken out and the loose material retained inside it is emptied out. This material is carefully examined as it is a sample of the stratum at the bottom of the boring where the sludger has been working at that time. A change in the charcteristics of this material is an indication that the stratum is changing and a careful record of the nature and depth of these strata is kept. The samples of the excavated material are also preserved.

The boring tube is clamped with sleepers at some convenient point above ground level and over the clamps a platform is made which can be loaded with sufficient weight to overcome the surface adhesion on the boring pipe and force it down as the sludger excavates a hole below it. Thus boring progresses and one casing pipe after another is serewed on and lowered till the required depth of bore is reached.

(*b*) **Ancher-bolt method of loading boring pipes.**—A convenient and more efficient method of applying load to the boring pipes is the "Anchor-bolt" method.

The anchor-bolts consist of two mild steet rods 1½ inches or more in diameter and about 16 feet in length. Both these rods are threaded with square threads for about 2½ feet length at one end and about 1 foot at the other end. If these rods are not available in one length two rods 8 feet in length, each may be used to form one anchor bolt by coupling the two pieces together tightly with a solid coupling. To each of these rods, on the end having threads for about one foot length only, are bolted together two rail pieces, R. S. Joists or channels about 6′×4″ in section and about 4 feet in length to form the anchor. For this purpose holes about 1½ inches in diameter are drilled in the centres of these pieces and the anchor bolts are passed through these and the two pieces bolted to the rod.

A pit about 5 feet in diameter and 7 feet in depth is dug at the site where a boring is to be made, and the boring pipe with the cutting shoe at its bottom end is placed vertically in the centre of this pit. The two anchor-bolts with the channel anchors fitted at their bottom ends are placed on either side of the boring pipe and kept about 4 inches away from it. The channel anchors are placed in a cross form. The pit is then filled with earth and well rammed taking care that the boring pipe and the anchor-bolts remain in their correct positon.

A wooden clamp is made from two pieces of sleepers 10″×10″ by 5 feet long. Each of these two sleepers has a semicircular hole in the centre so that when clamped together they form a circular hole slightly larger in diameter than the outside diameter of the boring pipes. On either side of this hole and about 4″ away from it the clamp has two smaller holes about 1½ inches in diameter into which the anchor-bolts are loosly fitted. These two wooden sleepers are clamped together with four mild steel bolts 1½ inches diameter and obout 2½ feet long.

This loose clamp is fitted across the boring pipe and the top of the anchor bolts and the two halves are clamped together with the four-tightening bolts described above. The loose clamp is held in position and prevented from moving up or slipping down by nuts and washers screwed into the top threads of the anchor-bolts on both sides of clamp.

A second wooden clamp similar to the loose clamp described above but with the central hole tightly fitting to the outside of the boring pipe is also required. When clamped together its central hole must grip the boring pipe tightly. It is fixed to the boring pipe below loose clamp at such a point that the vertical distance between the two clamps is just sufficient to put in two jacks with their caps fully screwed in. On top of these caps iron plates are placed to act as bearing surfaces when the jacks are worked.

This second clamp is fixed to the boring pipe as described above and the jacks (20 to 40 tons capacity each) are put in position. The sludger pump is now worked and the jacks are put into operation by unscrewing them. This exerts a pressure on the loose clamp. But as it can not move up the pressure reacts on the bottom tight clamp itself. The bottom clamp is thus pushed down and drags the boring pipe with it. When the jacks have completed their travel they are screwed in and taken out, and the verticility of the boring pipe checked and set right if necessary. The tight clamp is loosened and raised up, and again fixed below the top loose clamp at a point just low enough to accommodete the screwed-in jacks. The process is repeated till the boring pipe is driven in. Both the clamps are then taken out and a new boring pipe screwed to the bottom pipe and the operations repeated till the required depth of boring is reached.

Another convenient method of applying load to the boring-tube is to use a weighting frame made up of duplicate R. S. Joists assembled around the tube and resting on the ground. A pair of winches are bolted to the joists of the weighting frame and a pair of pulley sheaves to a clamp fastened on the boring-tube at convenient point above ground level. A $\frac{3}{4}''$ diameter rope with one end tied to the weighting frame passes over these pulleys and is wound over the winch drums. As the winches are operated the load of the weighting frame and winches comes on to the clamp on the casing pipe which is thus pulled down and enters the bore made by the sludger. But the weighting frame is sufficiently loaded so that it is never lifted off the ground and always rests on it. Thus there is no risk of its slipping down and injuring the workmen operating it. This arrangement is also used for extracting boring tubes. For this purpose the clamp with pulley sheaves is fastened to the boring tube a little below ground level. The rest of the operation remains the same as before and the tube gets pulled up.

The pipe with the weight on it, supported over a clamped platform, is, at times, rotated with manual labour to help its sinking down. In this case it is necessary to watch the movement of the pipe going into the bore. If the pipe goes in all of a sudden, *i.e.*, if it slips, it will take along with it the surrounding earth which will jam the pipe, and it will be difficult to move it either up or down. Hence it is necessary that the pipe should be held back with a rope and a crab and only allowed to go in slowly.

A careful record of the boring progress and the strata samples met with as also of the morning and evening spring water levels inside the bore is to be maintained. When a hard stratum is met with, the sludger is replaced with different tools such as chisels, underreamers, auger, spring rimor, etc., to pierce through it.

(*c*) **"Morning", "Evening" and "Working" water level.** (Indications they give with a "bailor-test" regarding the water contents of a stratum.)

The excavated material coming to the surface with the sludger will indicate what type of stratum is being passed through at the bottom of the bore. The existence of water-bearing strata is indicated by the presence of fine, medium or coarse sand, gravel, boulder, or round-edged *kankar* etc. While clayey strata are generally non-water bearing. An indication of the pressure at which water is flowing in a particular stratum is given by the level to which water rises in the boring-tube when it is passing through total stratum. It is clear that when the bore passes through a non-water bearing stratum the water level inside the boring-tube will be very low for there is no water coming into it from the non-water bearing stratum at the bottom of the bore. But as soon as a water bearing stratum, is reached water will rush into the tube, The pressure of water in the water bearing stratum as indicated by the level to which water rises in the boring-tube should be carefully observed both in the morning, before starting work and also during the progress of the work in the day time. The former gives the static pressure of water in the bore provided no boring has been done at night and the water has had time to attain its natural level, and the latter indicates the working pressure. The working pressure is generally lower than the static pressure, for as the sludger is taken out to the surface to empty it of the excavated material some of the water also comes with it and thus throughout the day water is being bailed out from the bore. Hence the working and evening water levels are effected by this contniuous bailing out of water. The working water level also gives an indication of the flow of water in the stratum and if it is not appreciably lower than the static water level it may be safely assumed that the flow of water in the stratum is satisfactory and that it will give a good yield. A record of morning water levels observed before starting work and of evening water levels taken after stopping work should be carefully maintained. As a matter of fact it is desirable to conduct a "bailer test" as each individual water-bearing stratum is met with while boring, so that some information regarding the probable yield from the stratum may be had by observing the difference between the static water level and the lowered water level in the stratum after a "bailer test".

In carrying out this test the sludger with its flap valve is used as a bucket to bail out water. If the level of water inside the tube-well remains steady even after bailing out water, for say a couple of hours (approximately 80 bails of 6″ to 8″ diameter and 8 feet height in the case of 8″ to 10″ diameter borings), the stratum may be considered satisfactory for location of strainers, or for further development and gauging in the case of a cavity well.

(*ii*) **Rod boring.** Another system of boring is the rod boring system. This is similar to rope boring except that rods extend from ground level to the boring tool instead of a rope. The rod boring system is used for very deep borings or borings in very hard strata. The rods are of ash wood with the metal male and female screws spliced on to their ends, or steel rods with screws (in length from 3 feet to 20 feet and from 1 inch to 3 inches thick). Boring-rigs are constructed either of wood or wrought iron tubes, with a head-pulley-block over which a wire rope is passed to suport the boring rods and tools. The other end of the rope passes to the windlass in the hand-power-rigs or to the winch or winding drum in a steam driven plant.

In order to obviate the jarring effect of the great length of rods, particularly in very deep borings, a trip link is introduced 20 to 30 feet above the tool as it is dangerous to exercise a percussive action of such power which will expose the lower rods, to the danger of breaking. The method usually adopted is to employ a form of sliding joint, on one of the rods, 20 or 30 feet above the tool. This is based on an invention of Oeuyenhausen. It consists of a slide joint of two parts which are able to slide upon one another for a distance of about one foot and so arranged that during the descent one becomes detached from the other. The upper part is balanced by a counter-weight suspended to a lever, and the lower one only allowed to act by percussion. Certain adaptations of this joint in the form of jar-bars and other sliding joints are now generally used.

Square rods with taper joints are sometimes used for shallow borings. In this method earth auger, cross chisels, flat chisels, bull-nose auger, etc., are used as boring tools. The square rods are rotated with drill, and the auger is allowed to penetrate in the earth about one foot and then withdrawn. The earth is removed from the auger and the clear tool is again inserted into the bore. The above process is repeated till the boring is made to the required depth. In Gujrat bores up to 150 feet are sunk with this method and with manual labour. For shallow borings this method is generally more expeditious than rope-boring.

(*iii*) **Boring by Hydralic Wash or Water-Jet-boring.** At places where sufficient water is available for working a water-jet plant, the hydraulic washing system is a very convenient method of boring through hard and tenactious soluble clays in which progress of boring with ordinary percussion boring sets would be very slow. Hollow boring rods or pipes, with a swivel-head at the top and connected to a chisel at their bottom end, are used and a rotary motion is given to these by rotating the water-tubes from ground level. A slow rotary motion is given to the boring-pipes also to expedite their sinking. For 8 to 12 inchs diameter borings going to depths of 3 to 4 hundred feet, ordinary W.I. pipes about 4 inches in diameter are used as hallow boring rods. These are suspended from a pulley in the derrick of the boring ring with an adjustable length of wire rope, and pass inside the boring-tube. At its bottom end the water-tube is fitted with a chisel having two nozzle orifices, one on either side of the chisel. The nozzles are kept at about 9′ above the bottom of the bore. A suitable connetion with an electric or oil-engine power pump is made (at the upper end of the pipe) by means of a flaxible hose pipe, and water is pumped into the tube. The capacity of the pump should not ordinarily be less than 15,000 gallons per hour and it should be capable of working up to heads of 200 feet. The water which is pumped through the tube to the bottom of the bore-hole shoots out through the nozzles and rises in the annular space between the water-tube and the casing pipe carrying with it any comminuted material and overflows into the settling tanks at ground level. The velocity of water in this annular space should be sufficient to carry all the comminuted material with it. This velocity can be regulated by controlling the discharge from the pump. After settlement and clarification in the settling tanks this water can be used again for boring purposes.

Although this method has proved efficient it is not very widely used owing to the difficulty of obtaining sufficient water to ensure a constant supply at full pressure at the bottom of the bore. Where it can be used there is a great economy in time as the tool needs to be raised less frequently. In addition a clean working face for the chisel is maintained at the bottom of the bore-hole. Care should, however, be taken to record changes in strata very vigilantly when working with this method.

As recording of morning, evening and working water levels would not be practicable with water-jet boring, a combination of water-jet boring and rope boring methods has been adopted in many cases. The boring is started with the the water-jet system, but as soon as a water-bearing stratum is reached the water-jet plant is put aside and the rope boring system is adopted. This enables accurate records of the various water levels to be kept as also of the depth and sample of the stratum. When the water-bearing stratum has been crossed the rope boring plant is removed and the water-jet plant is used again. A combination of these two methods gives expeditous results and incorporates the advantages of both the systems.

(*iv*) **Rotary Boring.** The rotary system was originally devised to drill wells through unconsolidated rock, such as beds of clay, sand, etc. Where the material to be drilled through is soft, the rotary system is used to far greater advantage than the standard percussion system. Here speed is a great factor, as much as 1,000 feet having been drilled by this system in 24 hours (Bib. 3).

The practical operation consists of rapidily rotating a column of pipe, at the lower end of which is a cutting bit. In the bit are holes through which water, with clay in suspension, is sent under pressure. The fluid mixes with the cuttings and carries them up to the surface in the annular space between the drill pipe and the bore hole. The clay that is held in suspension in the water "muds up" the walls of the hole, prevents caving, and causes the pipe to turn more rapidly as there is practically no friction between the boring pipe and the sides of the bore hole. The muddy fluid, because of its greater density, also brings to the surface cuttings which could not be lifted by water alone.

A derrick like that of the standard rigs is used, but machinery and tools are unlike those of a percussion outfit. A pump with proper pipe connections is provided for feeding a constant stream of the mud flush to the tools when drilling. Usually a second pump is kept ready for use as a stand by. Thin mud or slush plays an important part in drilling. A slush pit, an essential accessory is usually dug near the derrick about 40 feet long, 15 feet wide, and 3 to 4 feet deep. A ditch where sand may settle out of the mud is cut from the well circuitously to the slush pit from which hose or pipes lead to the pumps.

A machine called the rotary, is used for rotary boring. This rotates the drill pipes and at the same time permits their being lowered in the bore as required. The chief parts of a rotary are a rigid base and a turn-table rotated by a system of gears actuated by means of a chain drive sprocked. The drill pipe is held by a cable, which passes over the crown pulley attached to the boring rig and around the drum shaft of the draw works. This drill pipe is clamped by a block to the turn-table and forced to rotate with it. Modern rotaries are more elaborate and are fited with double turn-tables, special clamps drive rings, etc., having interchangeable bushings for holding casing and turning drill pipe.

A constant stream of mud slush or water must be circulated through the hole while drilling. This is supplied by pumps which force the water through a hose pipe and a rotary swivel connection and then cown the drill pipe.

Wrought iron or steel pipe is used for drilling purposes. This comes in random lengths of about 20 feet, it is about $\frac{1}{4}$th to $\frac{1}{2}$ an inch thick and from $2\frac{1}{2}$ to 6 inches in diameter. Special rotary tool joints are used to connect two pieces of pipes, in place of ordinary couplings, for greater strength and safety.

When the bore has been drilled to the required depth casing of the proper size is inserted to support its walls.

(*v*) **Core drills.** The rotary system of boring with core drills is hardly ever used when boring for water in sand, gravel and clay as sufficient water is usually found at depths which do not justify the use of this machinery. But this system is largely used for oil borings where great depts are attained for rock boring. In rock, sand-stone, trap, slate, granite, etc., the rotary core drill with shot bits is found most efficient.

The ordinary core drill consists of a tool called a "crown" which is a short piece of cast steel tube, into one end of which a number of "black diamonds" are fixed circumferently. Black diamonds are an amorphous variety of diamond usually called carbons and are only valuable on account of their extreme hardness. Chilled steel shots are now generally used in place of diamonds, The upper end of this crown is screwed to steel pipes of the rotary drill. Machinery on the ground surface causes the pipes and the crown to rotate as explained in the case of rotary boring and the crown cuts through the strata causing a "core" to rise up the hollow tube into a core barrel. Lead balls or some gravel about ¼" to ⅜" in diameter are then injected with water in the bore so that these balls or gravel may act as a wedge between the core and the crown. If the tool is now rotated from top the core gets sheard off at the bottom and breaks. The tool is then raised and taken out and broken core comes out along with it. The crown, when it has been taken out, is given strokes with a copper hammer and this makes the core drop out from it. From time to time the core is broken off, the tools raised, and the core extracted. Water is pumped down the hollow rod as in hydraulic washing and rotary boring and enters into the bore hole acting as a lubricant to the drill and carries off loose debris when returning to ground surface through the annular space between the drill and the bore hole.

(*vi*) **California or stove-pipe method of boring**—This is a method of boring used for wells of 6 to 30 inches in diameter. The casing consists of short sheet-iron cylinders, forced down by large hydraulic jacks and perforated in place by a special tool. Materials within the casing are excavated by a sand bucket (sludger pump).

This method is chiefly used far unconsolidated alluvial deposits and is known as California method, as it was originally developed there. It is also called stove pipe method on account of its special casing. The casing is made of lap riveted or welded cylinders of sheet-iron or steel usually of 10 to 14 gauge 24 inches long and 6 to 30 inches in diameter. Two sizes are used one of which just fits within the other, so that the joints of one may be adjusted to fall midway between joints of the other. Sections are added one at a time as sinking proceeds, each 2 feet section adding 1 foot to the length of the casing. Outer and inner sections are united simply by denting with a pick ; and the casing is watertight. Casing may be started from a properly recessed steel drive-shoe and drive pipe. Casing is usually sunk by two or more hydraulic jacks burried in the ground and pulling at a casing yoke placed on top of the casing pipe.

A sand pump is used for boring as in the case of percussion rope boring described before. A starta chart is carefully kept, as usual and after the well has been sunk to the required depth a cutting knife is lowered inside the casing and vertical slits cut in the casing in selected water-bearing strata. Revolving cutter used is such that it punches five holes at each revolution of the wheel. Vertical slits are of a form and size that do not clog readily. For best results in fine material, a natural strainer is formed surrounding the casing by removing the finer materials adjacent to the pipe by pumping the well as low as possible for several days.

It is seldom necessary to reperforate a well casing. The yield of an old well may often be increased by using compressed air for back-blowing or by heavy pumping, thus loosening the surrounding sand and gravel. (Bib. 2)

6. Tube-well pipe. (*An extract is taken from Bib 28.*)

(*a*) **Threads on boring-pipe.**—From American practice it has been found that the 8 threadspipe (*i.e.*, 8 threads to 1") lacks strength, owing to the deeper cut required for the thread. It was found that the pipe often broke in the thread when subjected to strain. Besides the 10 thread pipe proved more consistently watertight and less liable to admit particles of sand and grit, which would bind the thread and destroy it in unscrewing. The Amercians strongly recommend use of 10 thread pipe (Bib 33).

It is however experienced that when casing pipes with 10 threads to an inch are jacked out, during the process of extraction form a bore, they at times slip from the joints specially if the threads are partly worn out. Casing pipes with 8 threads to an inch are better in this respect because their threads are cut deeper and therefore they do not slip so easily. But greater depth of threads necessitates that the thickness of pipes should be correspondingly greater so that they may not lack in strength and may not part (break) at the joints. Generally speaking pipes with 10 threads to an inch are preferable where the thickness of pipes is less than 1/4th of an inch, but with thicknesses between 1/4th and 3/8th of an inch pipes with 8 threads to an inch are preferred especially when used for boring purposes.

(*b*) **Collar-bond pipe.** Boring pipes if socketed sometimes become "collar-bound" in a hole. In such an event the pipe can be worked for about the length of a single joint, but will not pull above a certain point. This is caused by mud loading heavily upon the collars and by a mud-laden collar, usually the bottom one, coming in contact with a tight place in the hole. The condition may be aggravated by the sand filtering in and finding a resting place upon the mud-laden collar. (That is why it is a advisable to have boring pipes with flush joints, fig. 1, where the collar at the joints are dispensed with). In such cases, it is necessary to keep the pipe in constant movement untill it will pull freely, pass the point of obstruction and pass easily down through the same place ; otherwise such a condition is sure to cause the pipe to 'freeze' rapidly. The expression "frozen pipe" is applied to boring tubes rendered immovable in a bore by mud, sand, limestone cuttings, and other gritty substances, settling around the outside of the tube.

When a pipe is collar-bond too powerful an effort to pull it through the tight place in the hole will often cause it to become lodged so that it will nither pull nor fall back to the bottom, a common occurrence in the Vijapur area of the Baroda State. This condition should, if possible, be obviated by the exercise of patience in keeping the pipe in constant movement. Usually a collar-bond pipe, if worked patiently up and down the bore, the collar being pulled gently into the tight place and then lowered before the pipe gets lodged, will clear itself sufficiently of the sand and mud gathered upon the collars to permit free movement. Unless great care is exercised in controlling the pipe, it may be injured in dropping, because a pipe thus lodged drops with a tremendous force under the impetus of its own weight when once started. Therefore a free fall of even a few inches may result in serious damage if the pipe is allowed to strike the bottom.

Sometime dynamite is used to give jerks to the pipe and thus to loosen a jammed pipe from the surrounding earth. The vibrations taking place at the time of exploding the dynamite shake the whole length of the boring-pipe and also give vibrations to the surrounding soil. Due to these vibrations the soil round the pipe which has struck to its sides is likely to get loose and the boring-pipes become free. This process was tried in Padra (Baroda State) boring for the extraction of the jammed casing pipes with considerable success.

(*c*) **Bad pipes i. e., pipes in a state of partial collapse, or split up while boring.** A pipe remaining in a bore for some time will frequently "go bad". This expression refers to a pipe that has become dented from a stone bruise, etc., and is in a state of partial collapse from outside wall pressure or is perhaps split and partly flattened. The pulling of such a pipe for the purpose of replacing the damaged joints is often inadvisable because of the danger of breaking the pipe at the "bad place" and leaving the lower part of the string of pipes in the bore. Such a pipe is often frozen, and attempts to free it by applying too much force would cause the weakened joints to part.

"Bad pipe" is usually first detected by the bailed or sludger hesitating in its drop as it is let down the bore. This hesitation if recurring continuously at the same place, leads the borer to suspect that the pipe is giving way at that place.

One of the remedies for a bad pipe is "swaging". The 'swage' is a heavy piece of steel, oval in shape, with a small groove cut into the steel as a watercourse. At its greatest diameter, the swage should be a fraction of an inch smaller than the diameter of the boring pipe through which it has to pass. Thus a boring pipe with a 10 inch inside diameter should permit the entrance of a swage 9⅞ inches in diameter. The common method is to drive a swage of a small diameter through the bad place first. This swage is then withdrawn and a swage of a larger diameter is used. This procedure is repeated untill a swage of maximum permissible diameter is driven through the impaired pipe. The swage is run below the stem with long stroke jars and tools working immediately above the swage. The operation of driving the swage down through the bad pipe and jarring it back through the same place should be repeated until the swage will pull freely through the impaired pipe. It can then be safely assumed that the pipe has been extended to its normal size. If a swage has been driven through a spilt joint of casing, and cavings are falling into the pipe through the split portion great care should be taken to prevent the cavings from wedging the swage in the pipe, the swage being so nearly flush with the pipe, that any hard obstacles may cause it to wedge.

7. Sand blowing during boring, i.e., Sand rushing into the boring pipe during boring.
(An extract from Bib. 28.)

While boring is in progress there may be encountered a stratum of fine sand, or sand and clay etc., which rushes up the boring pipe to a considerable length and thus hampers the progress of further boring. This sand is generally very fine (with or without clay) and dry. It would appear that this particular sand stratum is sandwiched between two strata and is under considerable pressure. It may also be that it is in an arched condition, all the while, between the two strata above and below it. When this stratum is struck during boring its stability is disturbed and the boring pipe provides an opening for the release of its pressure. The sudden release of pressure causes the sand to below up inside the boring pipe to considerable heights. In one well in Okha, Baroda State, sand was blowing up everyday from a depth of 680 feet below ground level to 140 feet *i.e.*, to a height of 540 feet.

The difficulty about sand blowing is that when the bore is very nearly cleared of all the sand and the bottom end of the boring tube is reached, the stability of the sand underneath is again disturbed and it blows into the pipe up to the old level and during this blowing up the drillers tools etc., may get struck up or jammed or dammaged in the rushing sand ; and it provides a difficult problem to deal with.

The simplest method is to release the clamp which is holding the boring pipe so that the pipe may sink down under its own weight whenever sufficient sand has been cleared from inside the bore. All the while the sand is being cleared out, the boring pipe should be kept full of water right to the top, which will keep the sand under pressure and prevent it from rushing up, specially when the pipe is being cleared near its bottom. As the boring pipe is unclamped it will sink down by its own weight when the sand is cleared below the cutter shoe. In some cases it may become necessary to extend the boring-tube above ground level by adding another piece on top of the tube, and filling it with water to increase the head of water over the rushing sand. This process of watering, cleaning and sinking is followed until the stratum has been crossed by the boring-pipe.

If the thickness of the stratum is great and the pressure of sand high it may not be possible to pierce through the stratum in one day. In such cases even though the pipe is kept full of water during the night it may not be able to prevent the sand from rushing in during the closure hours. The well at Okha referred to above is an example of such sand blowing. Every morning the driller had to spend about 5 hours in clearing the sand before the could commence further sinking of the boring-pipe.

The principle followed in the above method is to line the bore while at the same time taking out the sand, which is prevented from rushing in by keeping it under pressure of water. It is obvious that if this pressure of water on the sand could be maintained there would be a lesser chance of the sand pushing its way through and blowing in even during the closure hours. This is achieved by the use of "aquajel." "Aquajel" is a powder of the colour of cement and heavier than clay. When mixed with water it forms a jelly which is very tenacious and which does not mix with sand. So when the trouble of sand blowing is encountered 'aquajel' is added from the top of the boring pipe. It being heavier than water, sinks to the bottom where the boring tool is working. By the action of any solid boring tool (not a sludger which will tend to collect the 'aquajel' inside it and bring it up with it when the sludger is lifted up) the 'aquajel' will get mixed with water and a jelly will be formed which will stick to the sides of the bore as a plaster and will bind the sand and keep it in place near the bottom of the boring tube. The boring operation can then be stopped, and the layer of 'aquajel' will form a sort of bulb at the bottom of the boring-tube and prevent sand form blowing in during closure hours.

If the rushing sand is coarse it indicates good water bearing stratum provided its hydrostatic pressure is heigh. When such a stratum is struck water will rush in the boring tube and along with it may bring in sand also. But this sand does not rise so high as the dry fine sand referred to above. It is easier to penetrate through such water bearing stratum and a sand pump suffices for this purpose as the sand does not rush in so often or as high.

8. Fishing tools. (*An extract is taken from Bib. 28.*)

When sludger or other tools remain inside a boring pipe due to the snapping of a wire rope there are different types of fishing tools which can be used to fish out the sludger etc., from the bore. For a detailed description of these, "Oil, Gas and Water Wells" by Lucy Manufacturing Corporation, New York, or other books on this subject may be referred to (Bib. 6).

The handbook on oil wells, published by the Gil Well Supply Co., Pittsburg, U.S.A. and also the handbook published by the National Tools Supply Corporation, Ohio, U.S.A. are good reference books on this subject.

A borer should keep a log book in which he should keep measurements of length and diameter of each tool, their thickness at various points, length of threaded parts, number of threads per inch in the various tools, etc. These constitute important information for a borer, for in case of an accident he will have full information to prepare fishing tools, of the correct size easily to enable him to fish out the broken parts of tools left inside a bore.

9. Types of tube-wells. (*An extract from Bib. 28.*)

Tube-wells generally comprise the following types :—

(1) Strainer wells (including radial wells).
(2) Cavity tube-wells.
(3) Slotted-tube-wells (including shrouded wells).
(4) Perforated pipe tube-wells.
(5) Artesian tube-wells.

In a tube-well we can draw water not only from the water bearing stratum nearest to the surface of earth but also from one or more of the water bearing strata lower down, and this makes it possible to draw out a larger quantity of water from a tube-well as compared to an open well. The water bearing stratum nearest the ground suface is not, however, tapped in the case of larger tube-wells in order to avoid interference with the supplies of the adjacent open wells and prevent their getting depleted.

Open wells generally get their supplies from the first water-bearing stratum. When we start boring from ground level we reach the uppermost water bearing stratum in which water stands as we call "spring water level" or "static water level" and it is this stratum which feeds most of the open wells. If we continue boring we pass through impermeable strata of clay, conglomerate or stone alternating with other water-bearing strata lower down. In the case of tube-wells we tap one or more of these lower water-bearing strata.

Radial wells and ground water collecters. In some cases it is found that vertical strainer wells need replacement very frequently owing to rapid incrustation of the strainers. It is generally believed that the incrustation occurs from the liberation of dissolved carbon dioxide as a result of the release of pressure or change of temperature due to the tapping of underground water from aquifers at different depths. The loss of CO_2 decreases the solvent power of water and causes precipitation of the minerals on the vertical well strainers.

As an improvement, radial wells, with radially driven strainers, have been constructed. These strainers are driven radially in a horizontal plane in the same water-bearing stratum and are thus at all times under the same water pressure and temperature.

A shaft of suitable diameter (12 feet or more) is sunk down to the water-bearing stratum to be tapped.

The shaft is constructed of reinforced sections (air-applied concerete and sheet steel sections) each about 10 feet long. At the bottom there are treble-walled steel sections comprising the cutting shoe, the intermediate and port hole sections. These are all welded together with the annular space reinforced and filled with concrete.

Through the port hole section the slotted screen pipes about 10 inches in diameter (with a pointed driving end are driven to about 200 feet length horizontally with specially constructed sliding hyraulic packer. The screen pipes are made from $\frac{3}{8}''$ copper bearing steel plate punched with $1\frac{1}{8}'' \times \frac{1}{4}''$ slots.

There are about 36 screens driven radially from one shaft and kept just above the clay bed underneath. (For London Water-works Installation, these screens were kept one foot above the clay bed.) The open ends of these screen pipes project inside the central shaft and are fitted with sluice valves and vertical back-wash pipes which extend to ground level, and can be operated from there. All these screens are back-washed and properly developed. The sand thus taken out averages about 3 cft. per lineal foot of screen pipe projected.

The water from all these screen tubes flows under gravity into the central shaft and is pumped out with deep well vertical motor driven pumps.

The yield from one such radial well is of the order of 7·003 g.p.m. with a drawdown of about 7 feet.

Cavity wells are tube-wells, which, being without strainers, draw their supplies from one acquifer or water-bearing stratum only.

10. Suitability of the Tract for Tube-Well Irrigation.

(a) Geological strata.

Below the soil crust in the ground water reservoir, there should be available 2 coarse sand stratum from 100″ to 150″ deep to locate the strainer of a tube-well. Geological conditions in the Punjab are such that unfortunately such a water bearing strata is not available everywhere. Trial bores are absolutely essential to ensure suitable strata for every individual tube-well. Water bearing strata are often separated by clay or *kankar* bed layers which are only partially pervious and yield muddy water. The trial bores indicate the position of such layers. If the strainers have to pass through such layers, the portions should be closed by putting blind pipes.

The Geological conditions in Northern India :—

"There are three crests of the crustal warpings or underground rock ridges sweeping across the Northern India as shown in the map of the Punjab Plate I. Vol. III. One major and a number of minor crustal warps, run transversely across the three main crests. These ridges of rock, varying in depths from at least 2,000 feet to surface outcrops, have an important bearing on the flow of sub-soil water."

"The middle one of the three main crustal warps, is of vital importance to the Punjab and United Provinces. It runs from Sargodha, through to Delhi and thence to Allahabad, Bhagalpur and Shilong. The major transverse crustal warp appears to run from the head waters of the Jumna river, via Delhi and Ahmadabad to Bombay. The northern portion of this warp follows, very approximately, the boundry line of the Punjab and the United Provinces. The rocky floor, of which these crustal warps are an integral part. is covered by alluvial deposites of the Indo-Gangetic plain. The entire stretch from Karachi to Calcutta is of the same geological type, but within this type there is marked difference of detail. In that area of the United Provinces covered by the upper waters of the Jumna and the Ganges, the alluvial sub-soil is lenticular in formation. Lenticles of clay are contained in a martix of sand. East of the line Lucknow-Cawnpore, the clay becomes predominent and the characteristic tends to reverse. In the Punjab, west of the major transverse warping below the Jumna, already referred to, the characteristic changes sharply. The clay appears in beds of layers, of varying thickness and very varying extent. In some instances the sub-oil water appears under pressure between two clay beds and acquires artesian properties. In the United Provinces the general direction of the sub-soil flow is that of the Jumna and Ganges rivers, in an easterly direction roughly parallel to the main crustal warps. So far as these crustal warps are concerned therefore, the flow is unimpeded in the United Provinces though obstruction is probable in Bengal, between Jal Paiguri and the Ganges river. But in the Punjab; the direction of the sub-soil flow is roughly normal to the main crustal warps and is impeded by the second warp on the Sargodha—Delhi line."

(b) Soil.

The soil survey should be carried out by taking samples for every 100 acres of the proposed area for tube-well irrigation. The pH value should be from 7 to 9. The significance of the so called pH value explained in Part II Chapter II.

(*c*) **Suitability of the Pumped Water.**

It is determined by working out the Salt Index as explained in the previous chapter on well irrigation. The Salt Index should be negative for suitability of the water for irrigation purposes.

(*d*) **Source of Water Supply.**

The source of water supply should be such that the yield of the tube-well does not deteriorate in course of time. The yield is affected in two ways; firstly by the choking of the strainer and secondly by insufficient inflow to replace the depletion when the soil is not sufficiently pervious. The first factor can generally be controlled by an engineer by a suitable design of the strainer, but the second one is often beyond the control of the engineer due to the irregular presence of the clay lenticles which are only partially pervious.

11. Tube-well Chaks.

(*a*) **Size of Chak.**

The size of *chak* depends on the discharge of the tube-well and the delta or the depth of the water required by the crops which will be raised. There is yet very little information on the delta performance of the tube-well irrigation in the Punjab. This cannot be compared to the United Provinces irrigation from the tube-wells because the average rainfall of the tracts irrigated in that province is from 30" to 40" per annum, while even half of that is not available in the Punjab plains. In the Punjab canal irrigation is fully developed and the delta statistics of this are available. The average figures of delta for the canal irrigation are given in Part II Chapter II. The delta performance of tube-well irrigation is likely to be about three fourth of canal irrigated depths of waterings to mature a crop. The Karol tube-well irrigation scheme has been based on an average *Rabi* delta of 1·1′ and an average *Kharif* delta of 1·7′, while the adjoining Shalamar Disty., of the Upper Bari Doab Canal has an average of 1·57 ft. in *Rabi* and 2·57 ft. in *Kharif*. Assuming that the pump will work for 5000 hours a year, it will be able to deliver annually about 420 foot acres per cusec of the pump supply. Allowing the depths of watering as stated above and assuming the discharge of a tube-well to be 1·5 cusecs, the *Rabi* area is likely to be 190 acres and *Kharif* area 120 acres. With irrigation intensity of 75% an average *chak* works out to be 424 acres.

(*b*) **Suitable discharge for a tube-well.**

Excessive discharge for an individual tube-well results in wastage from breaches on water-courses and excessive depths in the fields. Too little discharge needs a extra number of wells and results in wastage in the field by taking relatively longer time to fill a field. A discharge of 1·5 cusecs is considered to be suitable for a tube-well *chak*. Depth of first watering is likely to be 4 inches and that of the subsequent waterings 3", which are very nearly the same as for the canal flow irrigation, but the major economy of water is likely to result from lining of the water-courses, and from division of the fields into small *kiaries*, because in this case a cultivator has to pay for the energry or the volume of water used.

(*c*) **Location of Tube-wells and Water-courses.**

A tube-well should be located near the centre of the area to be irrigated, so that the radiating water-courses are not more than one mile in length in any direction. It should be situated at the highest place in the *chak* so that the water-courses do not run in embankment. The depressions or drains should from the boundary of the chak. Main water-courses should be aligned in such a way that no field is more than half a mile from the Government water-course.

In designing water-courses the usual practice is to provide a field command of ·2 ft., a slope of 1 in 3000 in the subsidiary (*zamindari*) water-courses and a slope of 1 in 5000 in main (Government water-courses).

It is desirable to line the main water-courses to save absorption or percolation losses. Tiles one and a half inches thick laid in cement mortar 1:4 will serve as a useful non-erodable lining while the sodium carbonate lining is liable to be easily damaged and subject to cracking on drying.

12. Selection of pump Sets.

An extract is given from the Punjab Engineering Congress (1941) paper No. 248 by Messrs. H. L. Vadhera and A. R. Talwar.

"Selection of a suitable type of pump is of primary importance in any tube-well scheme. Before making the final choice, different kinds of pumps and headgears installed in the United Provinces and elsewhere were examined."

"Centrifugal pumps with horizontal spindles are usually installed at the normal lower level of the sub-soil water-table to ensure constant priming. Due to the occasional rise of wate-table during the monsoon season, when the demand is slack, the installation is in danger of being drowned by the rise of the seepage water in the well. Actually, a number of motors are said to have been ruined in the United Provinces in this manner."

"The difficulty has been overcome by using centrifugal pumps with vertical spindles and driven through vertical shafts from vertical spindle motors installed 8 to 10 feet above the pump. The only drawback with this type of pump as offered by the tendering firms was that its discharge varied more rapidly with a variation in the total pumping head than in the case of other types of pumps, otherwise it is the most suitable type and has been proposed to be used on the Karol Tube-Well Scheme."

"The propeller type of pumps gives a fairly constant discharge against all heads, but its efficiency is comparatively lower than that of the centrifugal pump. The bore-hole type of pump was also examined, but on account of its lower efficiency it has not been adopted for the Karol Scheme. The use of a bore-hole pump is obligatory when the suction head exceeds 22-24 feet and sump wells cannot be constructed to place pumps of other types at a lower level due to large changes in water-table."

The vertical spindle certrifugel pumps which have been used were specially manufactured by the Harland Engineering Co. Ltd., and are known as the "Spiroglide pumps".

The following special features of the Spiroglide pump set are noteworthy:—

Pump and Motor.—The pump is provided with cutless rubber bearing instead of the standard ball and roller bearings. This arrangement renders the bearings immune against the effects of dampness or flooding of the well. It also means that the pumps require no periodical lubrication, one gland being the only part requiring occasional attention.

The headgear is extended downwards and provided at its lower extremity with a steady bearing. The unsupported length of vertical shafting is thereby considerably reduced, giving additional rigidity and freedom from vibration.

This feature avoids the necessity of the provision of an intermediate steady bearing for vertical shafting carried on a girder spanning the pump chamber, such bearing being prone to get out of alignment in course of time due to the settlement of masonary walls of the chamber, which alter the position of the girder supporting the bearing, thereby causing vibration in the vertical shafting.

The weight of the vertical shaft and pump motor is carried on a thrust bearing located in an accessible position in the headgear, where also is the means of adjusting the position of the shaft and the impeller in the pump casing.

The vertical shaft between pump and motor is of an abnormal strength, being of 3½" diameter. The reason for using vertical shafting of such a large diameter is to ensure that the first critical speed of the shafting is beyond the full load speed of the pump sets, so that when the pump is being started, when running and when being shut down, the shaft does not pass through a critical speed which would cause vibrations.

The motor-supporting headgear is of massive construction and has a large area base, machined on the under-side for supporting on chamber. After erection, dowe pins area filleted through the headgear base into the girders so that should the neccessity arise for the headgear to be renewed there is no difficulty in re-erecting the pump set in correct alignment, the girders being levelled when first installed, so that no packing strips or shins are used between the headgear and the girders.

13. Selection of Strainers.

A few types of strainers usually used in Northern India are described next page :—

(*a*) **Cook Strainer.**

This is manufactured in America and consists of a solid drawn brass tube slotted with wedge shaped horizontal slots. The slots are cut with a circular cutting tool from inside the tube, to various gauges to suit the coarseness of the sand, the usual gauges in the Central Punjab varying between 6/1000" and 16/1000". The strainer lengths are generally jointed together by means of screwed collars of brass.

(*b*) **Tej Strainer.**

This is manufactured by the Reliable Water Supply Service of India and consists of a brass tube constructed of a brass sheet bent round to form the tube, the vertical joint being brazed. The slots are wedge shaped and cut before the sheet is bent to various gauges to suit the coarseness of the sand. The strainer is made in 8' lengths generally and from 3" dia. upwards, the lengths being joined together by means of screwed collars of brass.

This strainer is similar to the Cook strainer, except that it is not made from a solid drawn tube. It is niether so robust as the Cook, nor are the slots cut so accurately. It possesses the advantage of being considerably cheaper and is easily obtainable, being locally manufactured.

(*c*) **Layne and Bowler Strainers.**

They are iron slotted strainers. The strainers are made in America and consist of wedge shaped steel wire wound to a suitable pitch round a slotted or perforated steel or wrought iron pipe the lengths being jointed together by screwed collars. These are heavy and robust and can stand rough usage, but having to be imported from America, it takes a considerable time to get delivery.

(*d*) **Ashford Strainer.**

The strainer is generally made in 8 feet lengths, the lengths being jointed by half rings bound with wire and soldered over. This strainer has to be very carefully handled lest the wires are broken or displaced.

(*e*) **Brownlie Convoluted Strainer.**

The strainer consists of a polygonal convoluted steel plate round which a copper mesh strainer consisting of heavy parallel copper wires woven with copper ribbon is placed.

(*f*) **Leggett Strainer.**

This is new strainer provided with cleaning device in the shape of cutters which can be turned in the slits and it is claimed, by this means, that the clogging of the strainer by the deposition of solid matter on the outside of the strainer and in the slits, can be prevented. The cutters are operated from the surface. The strainer is said to be somewhat expensive.

(*g*) **Phoenix strainer.**

It is calcium plated and is supposed to be free from the danger of choking and corrosion caused by the chemical action.

All brass Tej strainers appear to the best because brass is not readily acted upon by water and is easily procurable locally. Coir and *Munj* rope strainers have been used in the Punjab with success for small supply tube-wells meant usually for domestic purposes. Hard wood slotted strainers are at present in an experimental stage for use on tube-wells in the Irrigation Scheme in the Punjab.

14. Choking of strainers.

The strainers get choked up usually in two ways ; (*a*) chemically (*b*) mechanically.

(*a*) **Chemical Action.**

The chemical action may deteiorate a strainer in two ways, firstly by choking and secondly by corrosion. If calcium bicarbonate be present in water to the extant 15 parts per 10^5 the reduction of pressure due to pumping releases carbon dioxide and causes calcium carbonate to be precipitated on the strainer. The effect is the cumulative. In course of time yield begins to fall on account of choking. The corrosion of the strainer metal results in the complete collapse and the subsequent choking of the portions below it. The mild steel and cast iron are attacked by the sodium salt. Zinc is particularly susceptible to sodium carbonate and aluminium is even more so. Copper is attacked by sodium carbonate and sodium chloride. Brass is not readily attacked by salts present in the soil while calcium in the form of plating is non-corrodible. The sub-soil water in the western Punjab is usually saline and unfit for drinking purposes when salinity is more than 15 parts in 10^6, and not suitable for agricultural purposes when salinity is more than 60 parts in 10^6. The chemical choking by deposition of carbonates is very much reduced by providing a large slit area or low velocity of inflow which means less depression head and the reduced liberation of CO_2.

(*b*) **Mechanical Choking.**

It is simply blocking of slits with the material such as fine sand. This can be guarded against by providing suitable slits expanding inwards. The surest remedy to remove this trouble is that the velocity of inflow should be lower than the optimum velocity which can be experimently found capable disturbing the material. The proper screening or shrouding of the strainer with coarse material will remove this trouble to a large extent.

The pulsating action of the centrifugal pumps tends to break the adherence of the sand particles in the slits by arching action and is also useful to retard the deposition of the carbonates.

15. Size of Tube-well Pipe and Strainer Length.

The minimum diameter of the suction pipe is fixed after the considration of the maximum permissible velocity through the pipe. The frictional losses vary as the square of the velocity and directly as the wetted area. A 10" diameter is considered suitable for suction pipes for a discharge of 1·5 cusecs ; this gives a velocity of 2·76 feet per second, which is less than 3·0 feet per second, the standard permissible velocity in water supply schemes.

The factors affecting the design of a tube-well strainer are : (*a*) Transmission constant of the soil, (*b*) Depression head, (*c*) Length and diameter of the strainer and (*d*) Shrouding. The transmission constant of the soil varies considerably from stratum to stratum.

In water bearing sand, consisting of particles, say 16/1000" mean diameter, a mean velocity of 0·005 foot per second has been found to move only the finest particles of a negligible diameter. This would limit the discharge to ·005 cusec per sq. foot of the strainer surface. Large diameter strainers are, however, very expensive and, therefore, to keep down the cost shrouding is resorted to. From practical experience a 10" diameter strainer of 128 feet total length is found sufficient to give a discharge of 1·5 cusecs with a maximum depression head of 12·0 feet. The diameter of the strainer should, however, be not less than that required for the optimum velocity.

The indraw into the strainer is not uniform throughout the length of the strainer and the gross discharge does not vary directly with the length of the strainer and, therefore, it would be uneconomical to have a uniform diameter of the strainer throughout its length. It is desirable to vary the diameter of the strainer retaining the optimum velocity of three feet per second in the pipe as far as possible.

Suitable length of strainer of varying diameters from 4 inches to 10 inches may be used. A typical location chart is shown in fig. 1.

The saving in cost consequent on using the smallar diameter strainer is considerable for instance :—

The cost of strainer and plain pipe for one tube-well of uniform diameter of 10 inches throughout its length is Rs. 4,575/- compared to Rs. 3,485/- when strainers and pipes of varying sizes from 4 in. to 10 in. are used for the same tube-well giving a net saving of about 25 per cent in the cost of strainers and plain pipes.

There is, however, one objection to the use of the small diameter strainers. The strainers of 4 inches to 6 inches diameters, especially the Tej brass strainers which have been used on the Karol Tube-Well scheme, reqire extreme care in their handling when lowering the same and also during the process of shrouding. Great care has to be exercised to see that the bottom of the lowest strainer is not allowed to touch the ground and to take the weight of the strainers and plain pipes while being lowered into the boring. It is a matter of simple calculation that a 4 inches diameter strainer will not bear the superimposed weight of strainers and rising pipes.

In view of the extreme care required in handling the delicate smaller diameter strainers of 4 in. to 6 in. diameters, their use is discouraged and only strainers of 7 in. to 10 in. diameters are used.

16. Different operations of sinking tube-wells.

Extract from Paper No. 248 Punjab Engineering Congress, 1941.

"**Boring.** Boring is done by the percusssion system. To accomodate 10 in. rising pipes and strainers and allow for 4 in. thick shrouding 18 in. casing pipe is sunk by means of a sludger. The sludger with a steel cutting shoe and a non-returning valve is either worked by power plant or manual labour depending upon the hardness of the strata.

Strata Samples. Soil samples are taken at every 10 feet depth and also at every change of strata. They are preserved in a compartmented box and are also sent to the Irrigation Research Institute for the determination of transmission constants and measurement of percentages of clay and *kankar*.

Water Samples. Water samples of each water bearing stratum are taken in a properly cleaned and steamed Winchester quartz bottle and sent to the Irrigation Research Institute for the determination of the pH value, conductivity, salt index, quantity of salt present in water, as also the total solids, in order to find out whether water is suitable for irrigation or not and its probable effect on the life of the strainer.

Water levels. – During the process of boring, observations of water levels in the bore are recorded regularly twice a day before starting the work and after finishing it for the day. The difference in the water levels observed on the close of the day's work and the next morning before starting the work gives an indication of the recuperation of water levels in the bore and the nature of the strata at the bottom of the casing pipes.

Location of strainer and shrouding. After the completion of a bore, a location chart of strainers and rising pipes is designed quided by (*a*) the transmission constants of the soil strata, (*b*) the analysis of samples of water, and (*c*) the water levels in the bore.

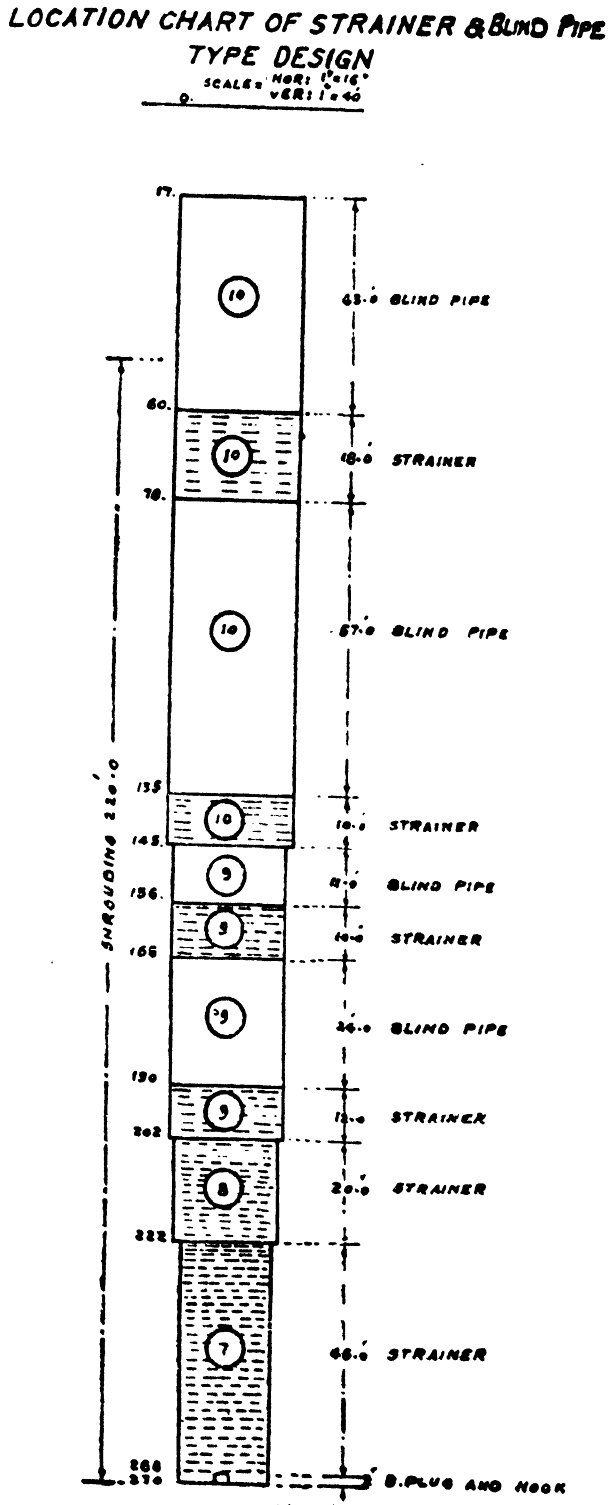


Fig. 1.

After the strainer has been lowered in the bore according to the location chart, the process of shrouding and extraction of the casing pipes is started simultaneously. Shrouding consists of ¼ in. Pathankot *bajri* retained on 1/10 in. mesh. An accurate record of the material used in shrouding is maintained showing its calculated quantity as also that actually used. In certain cases, specially at the junction of sandy and clayey strata, big cavities are formed which require considerable quantities of the shrouding material to fill them. During the location of the strainer, great care has to be taken so that the strainer and rising pipes are always suspended and are not allowed to rest on the bottom of the bore as otherwise the strainer at the bottom is subjected to unsafe loads, and is liable to be crushed.

To ensure that the strainer has been located and the shrouding has been done without any damage to the strainer, it is necessary to take soundings inside the tube-well at frequent intervals. A history sheet for each tube-well is maintained at site, giving complete information regading each bore.

17. Eccentricity in the bore.

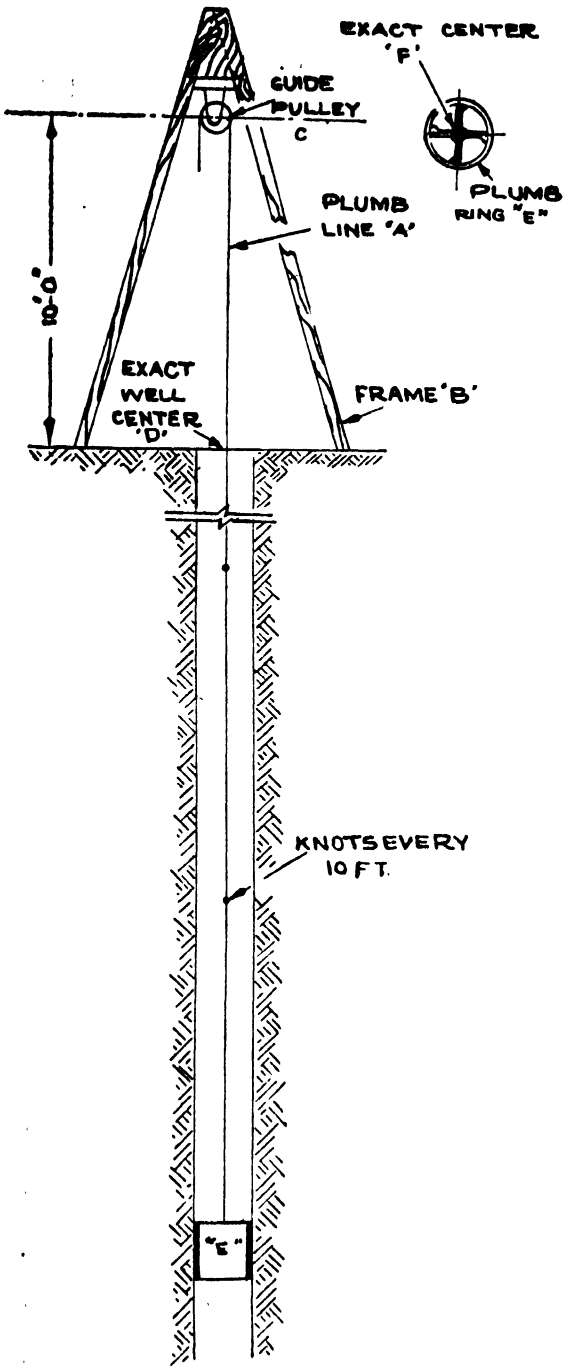
In all cases where borehole pumps are installed, whether the wells are compound wells or otherwise, it is of great importance that the finished well and the bore itself should be as nearly vertical (*i. e.* in plumb), as possible. Borehole pumps generally revolve with a speed between 1,500 and 3,000 revolutions per minute, and it can easily be imagined how damaging the effects of eccentricity can be in such cases on all the moving parts of the pumps and motor. The stress and strain on all bearings, shafts, bushings, etc., are so greatly increased due to the eccentricity that all the care and patience exercised in securing a truly vertical bore is amply rewarded by the ease and smoothness attained in the installation and working of the pump later on.

In spite of exercising reasonable care in boring the finished well is likely to get slightly out of the plumb partly due to some eccentricity in the bore itself and partly due to variation in the verticality of the top pipe when earth is filled round it after extraction of the boring tube, *e.g.*, when bell-sockets are used for a compound well.

The inside diameter of the top casing pipe is always kept slightly larger than the outside diameter of the pump bowls, so that there is a certain gap between the outer face of the pump bowls and the inner surface of the casing pipe. So long as the eccentricity does not exceed the clearance allowed in this gap, advantage can be taken of this in reducing the eccentricity. The eccentricity will be neutralised if the pump is installed not centrally in the casing pipe, but in such a way that one edge of the lowest pump bowl is almost touching the casing pipe on the side to which it is inclined. But if the eccentricity exceeds the clearance allowed by this annular gap (*i. e.*, the difference between the inside diameter of the casing pipe and the outside diameter of the pump bowls) it cannot be eliminated without setting right the eccentric casing pipe.

Measurement of eccentricity- A simple method for measuring eccentricity in a bore is by the use of a reel-like instrument which can easily be made by joining together two circular discs of the same diameter by means of a central hollow tube. The tube should have threaded ends with nuts to fit on to these so that discs of different diameters, according to the size of the bore, may be fitted on to the ends of this tube as required. The discs should have holes punched in them so that they may be immersed in water without any obstruction. The discs and tube should be sufficiently heavy to enable them to be lowered in bores full of water without difficulty.

The tube with discs can be suspended by means of a thin strong steel wire or copper wire so that it can be moved vertically up or down in the bore. A tripod having a pulley rigidly fixed to its apex is placed over the tube-well and the disc instrument, fitted with discs about 1/8th inch smaller in size than the inside diameter of the tube-well, is suspended from it with the wire passing over the pulley. The instrument can now be lowered or raised as desired. The tripod is so adjusted that the disc instrument will come centrally over the top of the tube-well when freely suspended. Let us assume that the disc instrument is so hung that its top disc is centrally situated and flush with the top of the bore tube. Let the distance between the point of suspension (*i.e.* the pulley of tripod) and the top of the disc instrument, when its top is flush with the top of the tube-well, be 10 feet. The instrument is then lowered in the bore, say, by 10 feet. This distance can be found out by taking measurements of the length of the suspension wire. Let



METHOD OF PLUMBING A WELL.

us assume that the wire deviates from its central position and the deviation as measured at the top of the tube-well is 1/10th of an inch. Therefore at the top of the upper disc (which is 10 feet inside the tube-well) the deviation will be twice this much *i.e.*, 2/10th (1/5th) of an inch from the vertical line through the pulley. Hence in going down 10 feet it has become eccentric by 1/5th of an inch or the eccentricity is 1/5th of an inch in 10′. (It will be noted that the eccentricity is not 1/10th of an inch in 10 feet, but 1/5th of an inch). The eccentricity at 20 feet, 30 feet etc., can similarly be obserbed.

Setting right eccentric wells. If the eccentricity of the larger diameter casing pipe (in which the borehole pump is to be lowered) of a compound well is found to exceed the clearance allowed by the annular gap between the inside diameter of the casing pipe and the outside diameter of the pump bowls, the casing pipe will have to be brought into plumb before the borehole pump is installed.

A simple method of setting right an eccentric casing pipe is to loosen the earth on the side to which the casing pipe is inclined and then to force the casing pipe back by applying jacks on the other side (opposite to that to which it is inclined). When the casing pipes have been forced into plumb, earth and gravel are filled on the side opposite to that in which the pipe was inclined, thus preventing the casing pipe from resuming its eccentric position.

To set right an eccentric casing pipe, a new bore, say about two thirds the diameter of the casing pipe, is sunk close to the casing pipe on the side towards which the pipe is inclined. The bore is carried down to about 10 feet above the bottom end of the casing pipe (which as explained earlier is generally at about 80 feet to 100 feet below ground level in the case of compound wells.) The point where this new bore is made is kept sufficiently away from the eccentric casing pipe so that on completing the bore its bottom does not come into clash with the bottom end of the casing pipe. The boring tube is then withdrawn with the borehole left intact. Powerful jacks are next applied on the side opposite to that in which the casing pipe is inclined and at a point about 8 feet below the surface of the ground. This enables the jacks to be fixed properly and they get a good seat behind for applying force. As the bottom end of the casing pipe is fixed, and there is a hollow space made by the boring on the side opposite to that from where jacks are applied, the casing pipe can easily be pushed till it is absolutely vertical. When this has been accomplished there will be a hollow space left between the new and the old position of the casing pipe. Earth and gravel are then filled in, this open space and also in any open space left on the opposite side where the new boring was made. The casing pipe is thus fixed in a vertical position and all eccentricity is removed.

The jacks are then removed after making sure that the casing pipe does not tend to spring back to the eccentric position. The well is now ready for installation of a borehole pump.

18. Back washing and back-blowing of strainer wells.

In the case of strainer wells it sometimes happen that the strainers get choked with fine particles of sand and clay or due to bridging (described before) and thus have to be cleaned ; or it is found necessary to draw out larger quantities of sand from the surrounding strata though the strainers, to improve the yield of the well. This is done by resorting to "back-blowing". The process of forcing out water or air under pressure from inside the tube-well, through the strainer slots, into the surrounding strata is called "back-washing" or "back blowing" respectively. In some cases satisfactory results are obtained by these methods and it is advisable to try these for improving the yield from a tube-well whenever it is found necessary.

If sufficient quantity of water is available from a high level storage tank or from a pump which can pump water under pressure into a tube-well, "back-washing" can easily be done with water. This water entering the tube-well under pressure passes out through the strainer slots into the surrounding strata and thus pushes out and carries along with it all the fine particles of sand, clay or dirt and cleanses the strainers substantially. But if the strainers are only partly clogged, the water may pass out of the strainers through the unclogged portions and the whole of the strainer may not get cleansed properly. In such cases a water jet is carried down to a point directly opposite each portion of the clogged strainers by using a small pipe, which can be lowered freely inside it. At the bottom end of this water pipe, there is a foot piece about 4 feet long with perforation on the sides and with its lower end sealed with a cap. The top end of this water pipe is connected to a high level storage tank or the pump. The water from the storage tank or the pump emerging under pressure from these perforations in the foot piece passes through the strainer slots thus washing them throughly and cleansing them. If any of the washed material falls inside the tube-well it can be pumped out from the tube-well along with the water from it.

The other method is to use air under pressure, from an air compressor. The plant consists of an air compressor and air line, foot piece, eduction tube, etc. The air line is a G.I. (galvanized iron) tube from 1 inch to 2 inches in diameter, through which air under pressure is passed from the air compressor to the foot piece. A foot piece is a perforated tube about 4 feet long and of the same diameter as the line with its lower end sealed or plugged with a cap. An eduction pipe is used for discharging water along with any sand, etc., which may have come into the tube-well due to back blowing. The well pipe itself can be used as an eduction pipe but as the washed materials from the strainers, etc., has also to be pumped out along with the, water it is preferable to have a separate eduction pipe about 4 inches in diameter, so that the velocity of water pumped out may be sufficiently great to carry with it all the clogged material etc., from the well.

The tube-well is sealed with a sealing cap at the top and the eduction pipe and line pass through it. There is a sluice valve on the delivery side of the eduction pipe which can be closed when air is to be passed out through the strainers and can be opened when it is desired to pump out water and sand, etc., from the well.

The eduction pipe is lowered into the tube-well till its bottom is about 2 feet above the top of the strainer pipe. The air line is then lowered inside the eduction pipe till its foot-piece comes directly opposite the strainer pipe. The sealing cap between the tube-well and eduction pipe is then closed as also the sluice valve on the eduction pipe. The air coming from the air compressor and passing out of the foot piece has now no passage for escape except through the slots of the strainer pipes, and through these slots it passes out into the surrounding strata. But in doing so as it passes under pressure through the slots, it cleanses them and removes the material which was clogging them. When one portion of the strainers has been cleaned, the education pipe and air line can be lowered to the strainers lower down and gradually all the strainers are cleansed. After cleaning each length of the strainer the sluice-valve on the delivery side of the education pipe is opened and the water containing the washed cloggings etc., is pumped out. Thus gradually all the clogged strainers are cleansed and there is proportionate increase in the discharge from the well.

In the case of compound wells an education pipe of a suitable size, which can easily be lowered inside the main strainer tube, should generally be used for back-washing and back-blowing purposes. But if the diameter of such an education pipe is found to be too small for satisfactory operation the main strainer pipe itself can be utilized as an education pipe. In such cases a loose joint is prepared for extending the length of the main strainer pipe to ground level. For this purpose plain pipes of the same diameter as the strainer tube, have to be lowered from ground level inside the compound well. A rubber ring forming the loose joint is screwed on to the bottom end of these plain pipes and when lowered inside the well rests on top of the main strainer pipe. The bottom portion of this loose joint is tapered so that the main strainer pipe slides into it and abuts against the rubber ring, thus forming a water and airtight joint due to the weight of the pipe on top of it. This pipe is then used as an education pipe and the air line can be lowered inside it and worked as described above. When the strainers have been cleaned, the upper plain pipes with the loose joint can easily be lifted out of the compound well leaving the well clear for installation of a deep well pump as usual.

19. Testing the Tube-Well.

After the extraction of the casing pipes is finished, the tube-well is tested for its discharge. If the yield of the tube-well is satisfactory *i.e.*, it yields a discharge of about 1·5 cusecs for a maximum depression head of 12·0 feet the construction of the masonary part of the works is taken in hand. The arrangement of the stilling chamber and meter flume to measure the tube-well discharge is shown in Fig. 3.

The most interesting feature of the construction of a tube-well is the putting in of a water-tight joint between the top of the rising pipe and the suction sluice-valve. This joint is 7 feet below spring level and it is not possible to fit the suction valve when water is rushing out of the rising pipe under this much head. To overcome this difficulty a mechanical rubber stopper has been devised by Mr. A.M.R. Montagu, Fig. 4.

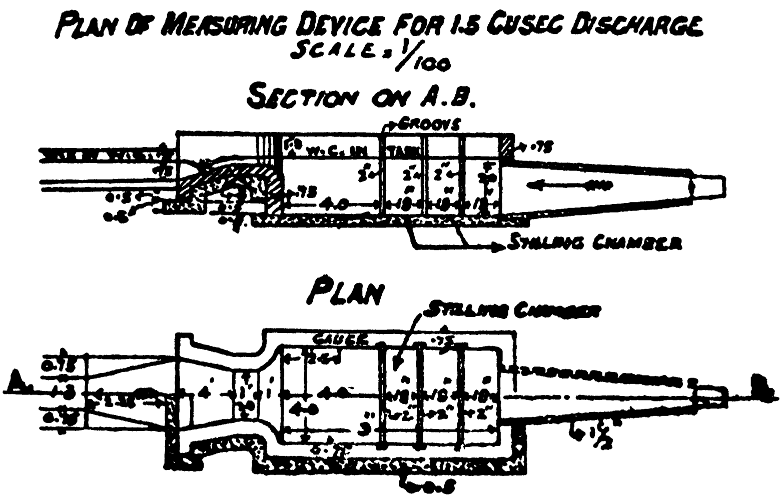


Fig. 3.

The stopper completely staunches the flow of water in the rising pipe. It is then quite easy to put in the pump foundation after the suction sluice-valve has been put in position.

20. Extraction of strainers and pipes in the case of unsuccessful wells.

If, on testing, a tube-well it is found to be unsatisfactory and it becomes necessary to extract the strainers and pipes it can be done by lowering a wire rope with a hook inside the strainer pipe so as to catch the "eye" on the bail plug and then pulling the strainer tube up. As described before, this can easily be accomplished provided the strainers and plain pipes have not been in position for a long time and have thus not been jammed by the surrounding soil. If these have been jammed in position the wire rope method of pulling them out may not be successfull

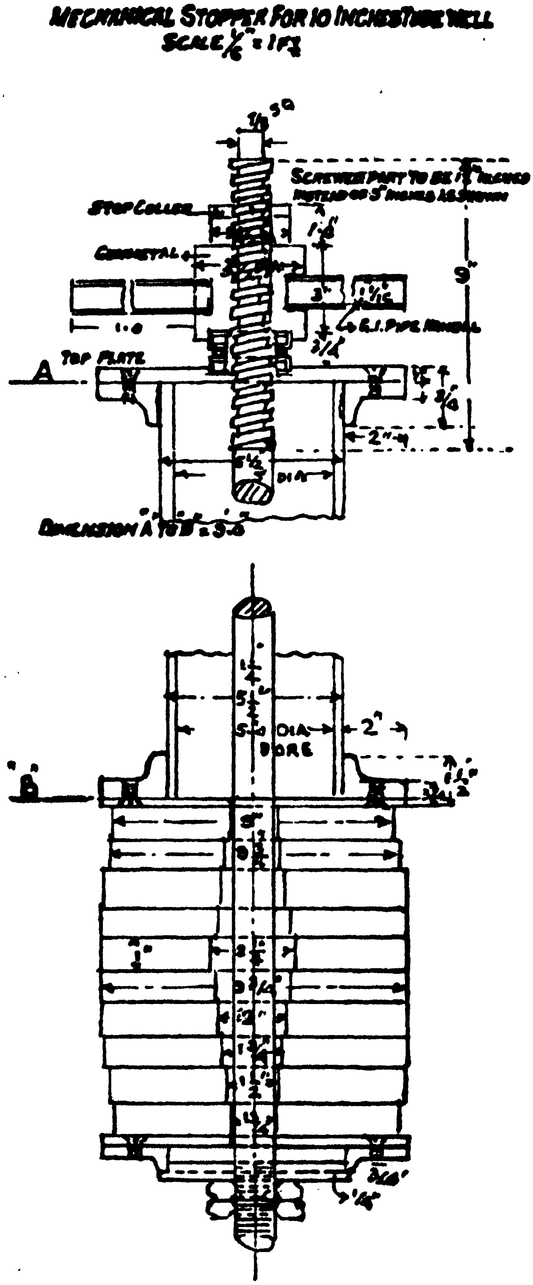


Fig. 4.

In such a case a simple device is to use a tapering, conical, piece of wood like a frustrum of a cone, loosely fitting inside the strainer tube, which can be lowered inside the strainer pipe by means of steel rods. This conical piece of wood is lowered inside the tube-well, with its wider edge at the lower end, to some point opposite a plain pipe. If it is lowered to a point opposite a strainer tube the latter may get damaged when pulling it out. Gravel or stone chips are then poured inside the tube-well and these collect round the conical piece of wood and from a sort of a wedge between the wood and the pipe. When this wooden piece is now pulled up by means of the rods it gets jammed to the pipe because of the wedging effect of the gravel between the pipe and the wooden cone and it tends to pull the pipe also along with it. A clamp is tightly fastened to the steel rods just above the top of the tube-well and rests on it. Another clamp is fastened on the tube-well pipe itself on the outside, holding it tightly, and jacks are placed below this clamp to lift it up. The pull is transmitted from the jacks, through the clamps to the tube-well pipe, and the pipes transmit it to the clamp, on the steel rods resting on top of the pipes. Thus the rod itself is pulled up and transmits the pull to the conical wooden piece inside. The tube-well is in this way pulled up by the jacks directly from outside and by the wooden piece and the wedge of gravel from inside and is gradually extracted out. This is a very convenient method of extracting strainer pipes and can be used where the strainers have been lowered for a long time and have got jammed. The advantage of this method over the hook method is that instead of the tensile force being applied to the hook on the bail plug only, which may get bent or sheard, it is applied to a large surface of the tube-well pipe itself and is therefore more effective.

A special collapsible steel tool is also obtainable which can be lowered inside the tube-well instead of the conical piece of wood. This tool can be expanded out after being lowered inside the strainer pipes thus forming a sort of wedge to grip the pipe just as the gravel wedge holds the pipe in the method described above.

9. Miscellaneous Works.

After the tube-well is installed, the following works are carried out :—

(*a*) **Construction and lining of water course.**

The earthen water courses should at least be run for one crop for consolidation before the lining is done.

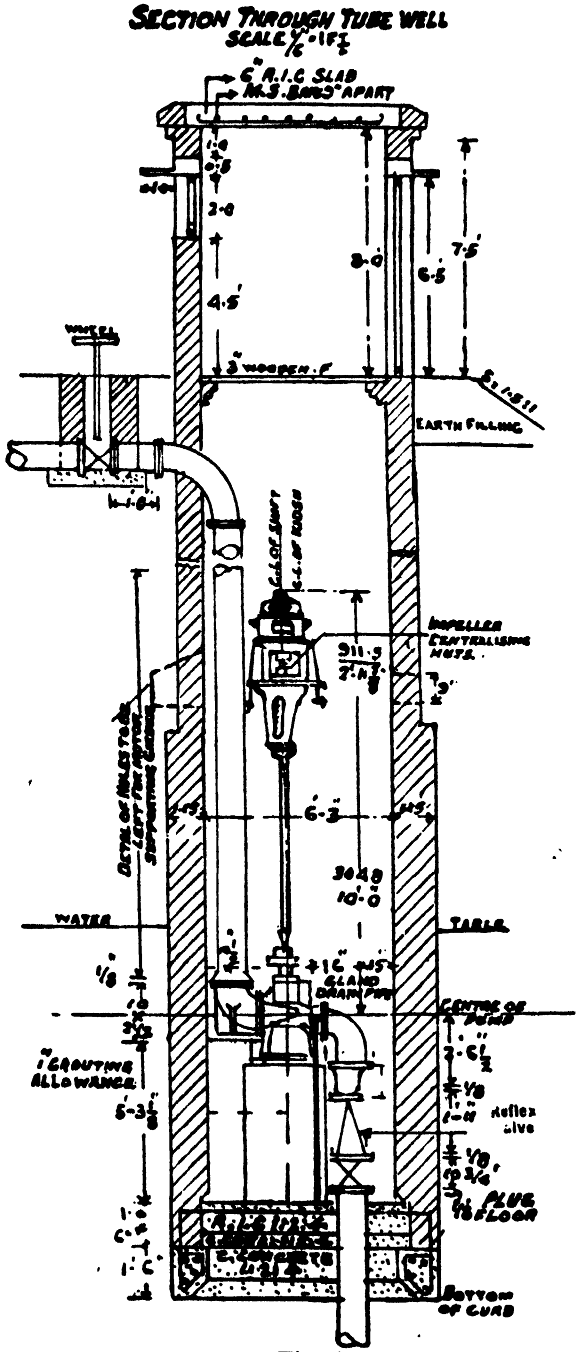


Fig. 5.

(*b*) **Pump Sumps.**

A suitable design is shown in Fig. 5.

(*c*) **Measuring devices.**

It should be in the form of long crested weir to require low working head of the type of meter on canals, (Part II Chapter IX). A stilling device in the form of grids will generally be required after the water leaves the delivery pipe.

(*d*) **Operator's Quarters.**

Two rooms with a verandah, kitchen and courtyard will give enough accommodation.

22. Assessment and Water Rates.

The establishment required for each well consists of one operator and a *beldar*. The operator's duties are as follows ; while *beldar* acts as his assistant :—

(*a*) to start and stop the pump ;

(*b*) to record the time and the meter reading at the beginning and end of each cultivat or's turn ;

(*c*) to record the area irrigated and the crops sown ;

(*d*) to issue cultivator's receipts ;

(*e*) to prepare the daily routine return ;

(*f*) to look after the Government channels.

A *zilladar* is required for about 20 wells to check the revenue work of an operator and to prepare the demand statement. Besides a supervisor is needed for about 20 wells who is supposed to work as a mechanic and an electrician to carry out the necessary repairs.

The Operator maintains a printed and bound log book containing the following information :—

(*a*) Date. (*b*) Time of starting and stopping the motor. (*c*) Meter reading at starting and stopping. (*d*) Units used by each cultivator. (*e*) Value of electricity used. (*f*) Field numbers irrigated. (*g*) Name of crop. (*h*) Area irrigated. (*i*) Gauge in measuring tank. (*j*) Total volume pumped in cubic feet.

In addition to the log book the operator maintains a cultivator's receipt book in triplicate. When a cultivator has finished irrigation, he signs the receipt which shows the time of starting and stopping the pump, the number of units consumed, and the amount assessed. The operator keeps the counterfoil with himself, sends the duplicate to the *zilladar* and hands over the triplicate to the cultivator to enable him to check his water bill at the end of the crop.

On receipt of the cultivator's receipt, the *zilludar* posts the units used in a ledger in which he leaves a page for each cultivator. This forms the basis of the demand statement. The ledger is also kept in duplicate. The duplicate copies are distributed to the cultivators at the end of each crop and take the place of the *parcha* (demand notice), vernacular form No. 8 A used in canal irrigated areas.

Water charges have been fixed per 1,000 cft. of water. Actually, however, the rate is converted into electrical units by actual measurements.

At the beginning of each crop, a complete test of each tube-well is carried out with respect to its discharge, electrical consumption, delivery and suction head, pump efficiencies etc. Electrical consumption per 1,000 cft. of water and the cost of an electrical unit for each tube-well is worked out and intimated to the *zamindar* concerned by posting notices.

The rates per thousand cft. of water are uniform for all the tube-wells, but the rate per unit of electricity varies from well to well, depending on the number of units required to raise the standard volume. The advantage of this system is that the cultivator can read the meter before and after his turn starts and knows exactly how much is to be debited to his account.

Water rates per 1,000 cft. of water are the same as charged in the United Provinces. These are three annas six pies per thousand cft. of water for *kharif* and two annas four pies per thousand cft. of water for *rabi*. These correspond to Rs. 2·12 per watering of four inches in *rabi* and Rs. 3·18 per watering of four inches in *kharif*.

Assuming deltas of 1·8 and 1·1 for *kharif* and *rabi* (vide paragraph 5) the cost of maturing one acre of *kharif* crops works out at Rs. 16·7 and of *rabi* crops at Rs. 6·81.

23. Discharge from a tube-well.

The mathematics to determine the discharge from a tube-well has not yet been fully developed. The appoximate methods to judge the discharge of a tube-well can be applied in two ways. Firstly, very nearly correct estimate of discharge can be made from the actual performance of the existing tube-wells. In the Punjab Engineering Congress paper No. 129 of 1929, Mr. Howel, Superintending Engineer has given actual performance of a large number of tube-wells varying in discharge from 8 to 36 gallons per square foot of the strainer surface per foot depression head. The safe figure for an estimate of a tube-well discharge may be taken as ten gallons per hour per square foot of the strainer area per foot depression head. Secondly an estimate of the discharge in a tube-well can be formed by experimentally determining the safe optimum velocity of say, ·005 ft./sec. as mentioned before in this chapter which will disturb the soil particles irrespective of the depression head. The soil samples used in such experiments should be representative samples of the soil crust in which the strainer is to be put, which is not generally practicable.

The formula of discharge was first developed by Dupuit (1863) and then developed by Kozeny (1927). Reference is invited to pages 255 to 260 of book 'Public Water Supplies' by Turneaure and Russel, New York (1924), Fig. 6.

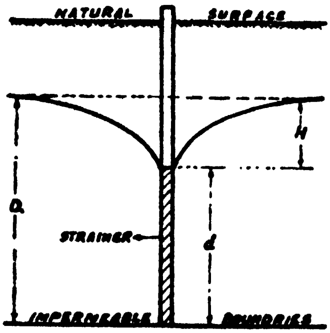


Fig. 6.

Let Q=total discharge of the tube-well.

H=Depression head=D−d. D=Depth of water-table above the impermeable boundary.
d=Depth of water in the tube above impermeable boundary.
r=Radius of the tube.
R=Radius of the influence of the streams radially entering the strainer.
k=Transmission constant of the soil.

The formula of discharge in cusecs

$$Q=\frac{\pi k\,(D^2-d^2)}{\log_e \frac{R}{r}}$$

In this formula k depends on the porosity P and a soil constant say k′ so that $k=k'P$.

The assumptions in the derivation of this formula are :—

(i) Water-table is at rest and is horizontal.

(ii) Bottom of the strainer rests on a horizontal impervious layer.

(iii) The flow is radial and total discharge Q remains constant accross the surface of a series of concentric cylinders.

The formula is not correct because the assumptions are not justified. There are no concentric cylinders of equal discharge entering the strainer. There is no theoretical limit to the value of R, radius of influence. The assumption of the impervious lower boundary is absolutely unjustified.

However tables are given in the above quoted reference to predict the value of R and the likely values of k in different soils based on the observations of the author's.

24. Conditions in the Punjab and the United Provinces compared.

The conditions of tube-well irrigation in the adjoining provinces of the Punjab and the United Provinces, in India are compared below :—

(a) In the tube-well areas of the United Provinces, the geological conditions are such that an immense sub-soil stream flows parallel to the crustal warpings. In the Punjab the flow is transerve to the warpings. Consequently in the United Provinces there is a uniformity of conditions, in respect of depth to water-table below natural surface, which does not exist in the Punjab. Such uniform conditions lend themselves to standardisation of plant and methods which make for maximum economy in sinking and equiping of tube-wells. In the Punjab, the variety of geological conditions necessitates every single site being carefully examined and tested before the tube-well is completed and equipped. Apart from expense, delay in construction is certain to occur.

(b) The nature of the sub-soil, in the tube-well areas of the United Provinces is lenticles of clay in sand. In the Punjab sheets of beds of clay occur irregularly in addition to numerous lenticles both large and small. There are no geological maps which would enable the engineers to avoid such deposits when siting the new wells. Consequently in the United Provinces the entire water-table is interconnected and the whole of the stream contributes to the discharge of the tube-well system. In the Punjab, the irregular geology of the sub-soil results in pockets of water cut off from the main stream. If a pump should accidentally be sited in one of these pockets, the discharge may cease abruptly after a few months' working or may be greatly reduced.

(c) In the tube-well areas of the United Provinces, the soil is light in texture, uniform in quality, free from salt with a pH value aproximating to the neutral. Conditions could hardly be more favourable. In the Punjab, the widest variations of soil occur. Many areas are impregnated with various salts the pH value varies from 8 to 9·2 and over. Unquestionably there are areas suitable for well irrigation. Such areas must be sought for, and carefully tested. Otherwise Government may find itself saddled with tube-well irrigation system in areas, which may deteriorate in course of time.

(d) In the Punjab the water of the sub-soil reservoir is not sweet every where as in the United Provinces. The salinity and the Salt Index of water for every strata where the strainer is laid, must be tested, otherwise the tube-well irrigation : will fail in the Punjab by deteriorating the yeild of the crops, matured with saline water.

(e) In the Punjab, the areas which are available for tube-well irrigation are very much scattered unlike the areas available for the United Provinces Tube-Well Irrigation, because the major portions of the Punjab plains are under canal irrigation. It is only the patches, uncommanded by flow irrigation or those adjoining the *Dhayas* along the rivers, which needs artificial irrigation.

(*f*) In the plains of the Punjab, average rainfall is 5 to 20 inches per annum, while the rainfall of the areas irrigated from tube-wells in the United Provinces is more than 30 inches per annum every where. The number of waterings from tube-wells to suppliment the effect of rainfall in the Punjab will be at least double of those required in the United Provinces. The Punjab cultivator shall have to pay about double the price of water. The average cost of irrigation per acre from tube-wells is Rs. 2-4-0 per watering in *rabi* and Rs. 3-4-0 in *kharif*. If the number of waterings is 4 in *rabi* and 6 in *kharif*, the cultivator will pay double the amount as compared to the canal irrigation.

25. Financial aspects of the tube-well schemes in the Punjab.

The financial aspect of the tube-well irrigation schemes is not at all bright in the province of the Punjab. The type of expensive irrigation cannot be popular because large areas in the Punjab are already being irrigated by flow irrigation from the canals at very cheap rates.

The electric energy has been prepared in the United Provinces at a cost of less than three pice per unit, while the cost of energy per unit from the Mandi Scheme in the Punjab is more than two annas.

The cost of sinking tube-wells must be more in the Punjab than that in the United Provinces, because the water bearing strata are irregular. The lifts are also relatively high.

The life of the tube-wells in the saline waters of the Punjab can hardly be taken as 12 years, while the depreciation charges in the United Provinces are worked on the basis of 17 years life.

Inpite of these difficulties the tube-well irrigation is likely to play a very important part in the future irrigation schemes of the Punjab Province, because there are lots of areas where water can only be supplied by tube-wells, and because it is likely that cheap energy may be available from canal falls by small hydroelectric sehemes or from crude oil engines using country fuel. Moreover, the tube-well irrigation may have to be extended as an anti-waterlogging measure in the waterlogged tracts or as a famine relief measure in certain arid tracts.

26. Canal Versus Tube-Well Irrigation.

A few of the disadvantages of the tube-well irrigation are stated below :—

(*i*) Working expenses are exteremely high, as wells depend on mechanical means of raising the water.

(*ii*) The wells are dependent upon a source of energy. Apart from the question of cost referred to in (*i*) above, a failure of the energy supply is accompanied by a stoppage of all pumps dependent thereon.

(*iii*) A tube-well is liable to progressive deterioration. The strainer is liable to choke and is difficult to clean. Replacement of the well may be necessary after comparatively short period of operation.

(*iv*) Maintenance of a delicate mechanical installation will always present its own diffiiculties.

(*v*) The tube-well water is clear *i.e.*, free from silt. Cousequently weed growth and algae must be expected and their removal catered for. Clear water has not the manuring value which silt in canal water provides.

The advantages of tube-well irrigation are stated below :—

(*i*) Wells may be sunk and equipped as required.

(*ii*) Water can be turned off at any moment, to take advantage of rainfall.

(*iii*) The supplies are likely to be fairly constant excluding always the deterioration of the well.

(*iv*) The loss in transit is much reduced. If water is sufficiently valuable, the lining of water courses is a practical proposition.

(*v*) Wells may be sited to command any desired land, subject always to the technical limitations for this means of irrigation.

(*vi*) Volumetric assessment is possible, in fact obligatory.

(*vii*) Water is likely to be used with maximum efficiency accompanied by benefit to the crops and to the sub-soil water-table.

The canal irrigation is not an unmixed blessing. The supply available for irrigation is limited to the discharges of rivers at the time of sowing and maturing of crops which have already been used up in the Punjab. Flow irrigation from the canals is essencially accompanied by a great loss of water in the distributing channels and the wasteful use in the fields. This wastage brings in turn the curse of waterlogging. The disadvantages of canal irrigation are dealt with in detail in part III Chapter II.

27. Examination Question.

1. (*i*) Discuss the marits of irriagtion by means of tube-wells against irrigation by gravity canals in the western districts of the United Provinces.
   (*ii*) Describe with sketches an electrically worked tube-well installation used for irrigation. Explain an air lift pump for a well lift 20 feet. (*T. C. E. 1935*)
2. (*a*) Name the different types of wells.
   (*b*) Why would you prefer wells over other sources of supply of drinking water ?
   (*c*) Desribe the various methods of boring deep wells. (*P.U.* 1942)
3. (*a*) You are required to place an order for pumping machinery. Please lay down specifications which you, as a Civil Engineer, will communicate to a firm of sppliers. (*P.U. 1942*)
   (*b*) Describe the various types of pumps suitable for use in the tube-well irrigation. Which tvpe do you consider best and why ?
4. (*a*) Why is the cost of the tube-well irrigation more in the Punjab than in the United Provinces ?
   (*b*) Why is delta in the tube-well irrigation lower as compared to the flow irrigation from canals ?
5. What is the life of a tube-well machinery in the Punjab and how will it compare with its life in the United Provinces conditions ?
6. Describe the various types of tube-well strainers used in the Punjab. Which type in your opinion is the best and why ?
7. (*a*) Describe the various factors which contribute to the reduction of yield from tube-wells in the Punjab in course of time.
   (*b*) What precautions and remedies would you suagest to minimise choking of strainers ?
8. (*a*) What is the suitable discharge of a tube-well meant for irrigation ?
   (*b*) What points will you keep in view in designing the water-course system for irrigation from a tube-well ?
9. What are the different types of wells used for irrigation and what are the average discharges for each tvpe used. Why is delta for well irrigation lower than for canal irrigation ? What is the economical size of *chak* for irrigation by tube-well and why ? (*P.U. 1954*)
10. It is proposed to irrigate 2000 acres of land (level tract) by tube-well irrigation. The sub-soil water level in the tract is at elevation 962·50. The drill log of the strata is as follows :— El 990·0 to 970·0 sandy loam ; 970·0 to 960 clay ; 945·0 to 931·0 sand coarse ; 931·0 to 880·0 sand and gravel. Assume usual permeability for the s rata and design the location chart of the strainer and blind pipes. Examination of water between elevation 960·0 and 950·0 reveals positive salt index. How many tube-wells would you use. (*P.U. 1955*)
11. (*a*) What are various methods of artificial irrigation and their relative advantages.
    (*b*) Sketch a typical tube-well installation. (*P.U. 1957*)

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PART II

CANAL IRRIGATION

CHAPTER I

Classification Of Canals

1. Irrigation and Navigation Canals.

Canals are divisible into two main classes *viz.*, irrigation canals and navigation canals.

Examples of irrigation canals of appreciable magnitude are available in India starting with the Ganges canal which was designed and built by Sir Proby Cautley in 1845 and the Madras canals by Sir A. Cotton at about the same time. The finest examples of the irrgation canals are now available in the Punjab as shown in Plate No. 1, Vol. III. Navigation cannot be expected to be successful on irrigation canals, because they have to follow the main water sheds or ridges so as to provide a sufficient head of water to flow over the adjoining land, for irrigation. Moreover, they have to be designed with velocities sufficient to guard against their silting up, consistent with the limitation imposed by the permissible scouring velocity for materials forming the bed and the sides.

An ideal navigation canal on the contrary, should have a very nearly still water channel, so that navigation may be possible in both directions. It should generally follow low country for economical construction. Moreover, a navigation canal should approach conveniently large centres of traffic. Examples of large navigation canals exist in the western countries such as Suez, Panama and Kiel canals.

An attempt was made in the beginning to combine both the functions in the case of Ganges, Western Jumna and Sirhind Canals. In these canals, small country boats were used. The bridges had to be designed high enough to pass the boats underneath and at the falls lock gate arrangement was provided. The velocities in these canals are 3 to 4 ft. per second, and loaded boats cannot be pulled for the up traffic by men or animals. The income from traffic by boats has been too low to justify the cost of the additional works for this purpose.

It is mostly the first class of canals *i. e.*, Irrigation Canals, Which will be dealt with in this volume.

2. Classes of Irrigation Canals.

The irrigation canals are divided into two classes.

(*a*) **Permanent Canals.**

A canal is said to be permanent when its source of supply is sufficiently well assured to warrant the construction of a regular graded channel supplied with masonry works for regulation and distribution. The canals which are provided with permanent canal headworks fall in this category.

The permanent canals may be perennial which receive assured supplies from the rivers througout the year, or may be seasonal such as *kharif* channels in which regular supply is available only in the *kharif* season.

(*b*) **Inundation canals.**

Inundation canals are those which depend for their supply on the periodical rises in water level of the river from which they are taken off. They are not provided with permanent headworks. Water is simply let into them when the river rises through the marginal flood embankments and they are provided with a regulator 3 or 4 miles away from the river. The problems connected with their working are complex and peculiar and are dealt with in Chapter XVIII Part II.

3. Financial classification of canals.

The financial classification of Irrigation Canals is two-fold :—

(*a*) **Productive works.**

When the forecast of the income usually prepared at the time of the preparation of the project of an irrigation scheme shows that the income from the proposed canal will exceed the yearly charges of maintences by a sum equal to at least 4 per cent of the capital invested, the work will rank as a productive work. The percentage of the net return is fixed by the Government from time to time according to the prevalent rate of interest in the market. A certain number of years is allowed for the development of irrigation after construction of the canal, during which the interest charges accumulate as a simple debt to be paid off as the revenue gradually increase. On the expiry of the term, usually ten years, the net revenue receipts should have cleared off all the interset accumulated during the construction and development, after paying for the running charges.

(*b*) **Protective works.**

Protective public works are defined to be those, which although not directly remunerative to an extent which would justify their retention in the class of productive works are calculated to guard against a probable future expenditure for relief of the population. They usually take the form of famine relief works. The construction of an otherwise expensive irrigation scheme may be started to employ the population during the famines. The lining of the canals though not directly a remunerative work is a very useful protective work to reduce the future expenditure on the anti-waterlogging measure and to save the deterioration of lands by waterlogging.

4. Receipt form Irrigation Canals.

The receipts which make up the income of a canal fall in two categories (*a*) Direct receipts and (*b*) Indirect receipts.

Direct receipts comprise, firstly, the income of the water rates fixed for the different crops irrigated ; secondly, the receipts from plantations, thirdly, the income from water-power such as mills or hydroelectric power; and fourthly the miscellaneous receipts from water used in bulk for filling tanks, for building houses, for watering road-side trees or for consolidation of roads.

Indirect receipts are the receipts creditable to the canal department due to the increase in land revenue by canal irrigation to *barani* or *chahi* lands. The difference between the usual *barani* or *chahi* land revenue rate and the revenue rate of the irrigated land is the indirect income due to the construction of the canals. In the case of crown waste land lying uncultivated, whole of the land-revenue comprises the indirect income of the canals. The interest on the sale of crown waste land put under irrigation is also credited under this head.

5. Parts of a canal.

An irrigation canal is divided into the following parts :—

(*a*) **Headworks.**

The headworks comprise all the works necessary to dam, to control the river and to regulate supply into the canal.

(*b*) **Main Canal.**

Usually in the head reaches, the canal is in cutting and below the natural surface and is not required to do any irrigation. In this reach it is called the main canal.

(*c*) **Branch Canals.**

When a main canal leaves the high ground and must, therefore, bifurcate into branches covering the whole tract meant to be irrigated, the canals in such portions are called Branch Canals. Very little direct irrigation is done from the Branch Canals.

(*d*) **Distributaries.**

Small channels which take off the branch canals and distribute their supply through outlets into the water-courses are called distributaries.

(*e*) **Minor.**

Sometimes when the country is such that the water-courses will have to be longer than two miles to reach the fields, it is usual to take off small Government channels from distributaries, which are called minors.

(*f*) **Water-courses.**

This is not a Government channel. It is constructed and maintained by the cultivators according to the alignment sanctioned by the canal engineers from an outlet down to the fields of the cultivators. Their design is dealt with in detail in Chapter No. XVII Part II.

6. Punjab Canals.

A list of the Punjab Canals is given in Appendix I at the end of this chapter. The earliest canal constructed was Western Jumna Canal about the year 1817 from the Jumna river, then came Upper Bari Doab Canal in 1850—1859 from the Ravi river and then Sirhind Canal (1872) from Sutlej river. The Lower Chenab Canal was completed in 1902 from Chenab river and the Lower Jhelum Canal in 1905 from the Jhelum River. The Triple Canal Project for the constrution of the Upper Chenab Canal, the Upper Jhelum Canal and the Lower Bari Doab Cannal was commenced in 1915. In this project, the Upper Jhelum and the Upper Chenab canals are mostly feeder canals connecting the Jhelum river with the Chenab river upstream of Khanki, and the Chenab river with the Ravi river up stream of Balloki respectively. The last named 5 canals (serial Nos. 4 to 8 in Appendix 1) are called the five linked canal because the waters of the three rivers mentioned above can be jointly utilised in these five canals according to the seasons requirements.

The canals at serial Nos. 9 to 19, take off the Sutlej-Beas system and were constructed from 1922 to 1928, and are called the Sutlej Valley Project canals. Most of them are *kharif* channels. The Haveli canal was completed in 1939. This is also a feeder canal from Trimmu (Chenab river) to Sidhnai (Ravi river) and its water has been utilised to give perennial irrigation to the old Sidhnai Inundation canal.

The old Inundation canals in Muzaffargarh and Dera Ghazi Khan districts taking off the Indus river were taken over by the Irrigation Department, P.W.D. from the civil administration in 1880 and the Shahpur canals taking of the Jhelum river in 1894. The lower Sohag Canal and the Para Inundation Canals were built in 1882 from the Sutlej river and have subsequently been absorbed in the Sutlej Valley Canals. The Sidhnai inundation canal was opened in 1886 taking off that the Ravi river at Sidhnai and has subsequently been absorbed in Haveli Canal Project. They took water from the rivers in floods, without any permanent head-works.

The project to irrigate the Sind Sagar Doab is under construction by taking off that canal with a discharge off about 6,000 cusecs from the Indus river at Kalabagh.

Lots of areas of Hissar and Gurgaon districts in the Punjab and Bikaner state need protection by canal irrigation and tank irrigation. Projects for constructing dams in the hills are under completion for this purpose.

7. Distribution of river supplies to different canals in the Punjab.

Some canal systems are inter-linked and the supply available in the rivers is not always sufficient to meet the indents of all the canals. Consequently, the supplies of various rivers are distributed among canals according to orders framed from time to time by the Punjab Government. The following may be taken to be the usual procedure : –

(*a*) **Western Jumna Canal.** The supply at the river Jumna at Tajewala is distributed between the Western Jumna Canal in the Punjab and the Eastern Jumna canal of the United Provinces according to regulation rules for the canals.

(*b*) **Sirhind Canal.** The Sirhind canal is entitled to take up to its maximum capacity all the supply available in the river Sutlej at Rupar.

(*c*) **Upper Bari Doab Canal.** The Upper Bari Doab canal is entitled to take up to its authorised capacity, all the supply that reaches Madhopur with the sole exception that during *kharif* from 1st April to 31st September the Kashmir (Basantpur) Canal on the right bank has a prior claim up to a maximum of 120 cusecs.

(*d*) **The Northern (Linked) Canals.** The Upper Jhelum and Lower Jhelum Canals taking off the river Jhelum, the Upper Chenab and Lower Chenab Canals taking off the river Chenab and the Lower Bari Doab taking off the river Ravi are inter-linked and they are entitled to all the water in the rivers Chenab and Jhelum which they can take up to their authorized capacities. When the supplies of the Chenab and the Jhelum rivers drop below the combined capacity of the linked canals, a destribution programme comes into force.

(*e*) **The Sutlej Valley Canals.** The Sutlej Valley Canals are entitled to all the water that comes down the river Beas and any surplus from the river Sutlej over and above the requirements of the Sirhind Canal. The actual distribution of supplies among the three partners. (The Punjab Government, the Bikaner State, and the Bahawalpure State) of the Sutlej Valley Canals is carried out in accordance with the orders issued by the Government of India, based on the Anderson Report of 1935.

(*f*) **The Montgomery Pakpatan link.** Is entitled up to its authorised capacity to all the water available in the river Ravi at Balloki.

(*g*) **The Haveli Canal.** The Rangpar canal, the Thal canal and the Panjnad canal are regulated according to the orders issued by the Government of India, based upon the recommendations of the Andrson Committee.

(*h*) **The Inundation Canals.** Draw up to their authorized capacities all the water that they can tap from the rivers both in *rabi* and *kharif.*

8. Functions of Canals and other Irrigation Work.

(*a*) **Protection.** The most important function of the irrigation works is to protect the area dealt with against serious loss during seasons naturally unfavourable for agricultural operations. They provide protection against famines. There cannot be any famine in canal irrigated areas. The surest remedy against the recurring famines in the Hissar district of the Punjab is the introduction of the canal irrigation.

(*b*) **Improvement of crops.** Another function is the substitution of superior for inferior classes of crops. The natural result of the introduction of a permanent supply of water to a tract formerly dependent on a fluctuating rainfall. Thus, in some Indian districts, we find wheat replacing barely; sugarcane and indigo replacing light millet crops and, as a general rule the cultivation of mixed crops practically ceasing.

(*c*) **Manurial value of canal water.** The rivers of the Punjab are all alluvial. The water of the Punjab canals contains a sufficiently large percentage of fine clay and silt in suspension. The silt in the canal water has a great manurial value. The yield of crops raised from the canal irrigated fields is more than that of the crops irrigated from wells without other manuring.

(*d*) **Addition to the wealth of the country and the Government Revenue.** The irrigation works add to the wealth of the country both directly by the enchanced value and quantity of products, and indirectly by an increase in the value of the land. The increase in wealth of the Punjab province can well be imagined from the figures of the irrigated areas as given in the following table :—

Year	Irrigated area in acres.	Source.
1867-68	1,025,156	Wells and river spills.
1877-78	2,341,103	U.B.D. & W.J. Canals added.
1907-08	6,039,944	L.C.C. & Sirhind Canal added.
1917-18	9,063,901	Jhelum Canals, U.C.C. & L.B.D.C. added.
1937-38	12,800,000	Sutlej Valley Canals added.
1942-43	14,000,000	Haveli and Rangpur Canals added.

The Government revenue in the Punjab, as credited to the canal department, from the realisation of the owners and water-rates, was about 8 crores of rupees before the World War II out of the provincial revenue of about 11 crores of rupees. The expenditure on the maintenance of the canals in the Punjab was only about $1\frac{1}{2}$ crores of rupees annually.

The profits of, some of canals before 1920 after paying for the interest charges on the capital outlay are shown in the following tables :—

Province.	Canal.	Profits expressed as percentage of expenditure.
Punjab.	Western Jumna Canal.	... 12·3%
	Upper Bari-Doab Canal	... 16·5%
	Lower Chenab Canal	... 43·6%
	Sidhnai Canal	... 31·8 to 51·8%
	Sirhind Canal	... 21·7%
	Lower Jhelum Canal	... 24·0% (now 35%)
United Provinces	Upper Ganges Canal	... 10·4%
	Bijnor Canal	... 11·0%
Bombay.	Bombay Canals	... 10·0%

(*e*) **Population.** The absence of famines by the introduction of irrigation results in an increase of manpower. Good and sufficient feeding results in good health of the population. There is also an increase in population. This increase in population is absorbed in additional labour required for agricultural operations and for the maintenance of the irrigation works. The robust manhood of the Punjab has always been the pride of the Indian Army.

(*f*) **Effect on climate and health.** The Irrigation Works such as canals and storage reservoirs are feeders of the sub-soil supply. With the rise of the water-table, the climate becomes damp. Irrigation on large scale affects the climate by making it cooler and damper particularly at night. The average temperature in the Sargodha and Lyallpur districts of the Punjab have dropped by about 10 degrees by the development of irrigation, on account of the increase in the cropped area and the other vegitation in the form of gardens. The dust storms have been reduced by 75 per cent. Damp climate and vegitation naturally increase some diseases such as maleria ; but this is more than counterbalanced by the increased vitality due to better standard of living.

(*g*) **Plantation.** The banks of large canals are generally planted with trees. This is a great advantage to the tracts through which they pass on account of the shade, timber, fuel and fruit that they provide. Trees and shade are a real necessity in very hot climates, and as constant extensions of irrigation tend to break village plantations up into tillage, the maintenance of permanent and well-cared-for plantations on the irrigation works themselves become desirable. Moreover, these plantations, if well managed, become large sources of revenue.

(*h*) **Navigations.** Another important function of a large canal is the facility it offers for navigation. This function is often neglected probably because the engineer is more attracted by the manifest advantages of the result of the attention he pays to extensions and improvements in irrigation. It would be better, however to take a broder view of the situation and endeavour to develop all possible sources of advantage to the people and State, and the utilization of the fine waterways formed by great canals should not be neglected as it has so far been.

(*i*) **Bathing.** The domestic advantages of irrigation works should not be overlooked. The facilities given for bathing and watering cattle must be greatly appreciated by the inhabitants of the towns and villages situated near the banks. In the colony areas of the Punjab, canal water is used for drinking purposes as the ground water is brackish.

(*j*) **Water-Power.** Water-power is frequently made available for use by the construction of canals and tanks, In India up to the present time this source of revenue, has to a large extent been neglected, owing mainly to the fact that the sites suitable for power generation (falls) are frequently far removed from the manufacturing centres. Electrical generation and its transmission offers such a simple solution of this difficulty, that we may look forward to seeing a great advance in a few years time and probably many great cities may soon owe their lighting, ventilation, and commercial prosperity to the same beneficient work that supplies them with food. The Ganges gride system in the United Province in India has proved a great success in the electrification of a large tract of that province.

Considering the above-mentioned advantages of canals, the irrigation is the best cottage industry for the predominently industrial Province of the Punjab.

7. Examination Questions.

(*i*) Explain the following terms :—
(*a*) Water-courses, (*b*) Branch Canal, (*c*) Minors, (*d*) Indirct receipts.

(*ii*) What are productive and protective canal works ?

(*iii*) What are the difficulties in having combined channels for irrigation and navigation ?

(*iv*) What do you understand by permanent and Inundation canals ?

(*v*) Explain briefly the Triple Canal Project and the five linked canals of the punjab.

(*vi*) Elucidate :—"Irrigation is the best cottage industry for a predominently agricultural Province of the Punjab." (*P. U. 1942*)

(*vii*) "The relative place of irrigation and its early development is even higher than industrial development in the economy of India"—this saying may be commented upon in the light of your own views. (*P.U. 1956*)

APPENDIX 1

CANAL IRRIGATION IN THE PUNJAB

PUNJAB CANALS

Serial No.	Name of Canal.	Discharge.	Gross area.	Culturable comanded area.	permissible area.	Actual irri-gated area, 1942	Kharif Rabi Ratio	Delta Kharif	Delta Rabi
1	2	3	4	5	6	7	8	9	10
1	Western Jumna Canal (W.J.C.) Tejawala	8000	2725881	2278008	890541	1101106	1 : 1	2·1	1·0
2	Sirhind Canal (S.C.) British Area 5257 (Rupar)	9040K 8330R	2424776	2026205	773072	1395622	1 : 1	2·0	1·2
3	Upper Bari Doab Canal [U.B.D.C.] Madhopur,	6900K 5770R	1564135	1445707	1081465	1268971	1 : 1	2·6	·8
4	Lower Bari Doab Canal [Balloki] L.B.D.C.	7000K 6550R	1801410	1438166	939415	1249779	4 : 5	3·3	1·5
5	Upper Chenab Canal [Marala] U.C.C. Used in Irrigation	13084F 5180	1536852	1445000	566860	704643	1 : 1	3·4	1.3
6	Lower Chenab canal [Khanki] L.C.C.	11032K 9956R	3772715	2873269	1873867	2472500	1 : 2	2·7	1·3
7	Upper Jhelum Canal Mangla Head (U.J.C.) Used in Irrigation	8783F 1847K 1242R	538117	502735	303666	339557	1 : 1	2·9	1·3
8	Lower Jhelum Canal [Rasul] L.J.C.	4099	1340645	1239597	756390	936934	1 : 2	2·33	1·34
9	Pakpattan Canal [Sulemanke]	6094+735 [M.P. Link]	1392766	1271122	57%	729000	1:1·5	3·3	1·6
10	Dipalpur Canal [Ferozepur]	6950K	1044060	978823	50%	486622	1 : 1	3·9	·3
11	Eastern Canal [Ferozpur]	3320K	395039	347965	46%	160000	1 : 1	5·4	·3
12	Gang [Bikaner Canal]	2720		Irrigates Bikaner State.					
13	Eastern Sadiqia Canal [Sulemanke]	4917		Irrigates Bahawalpur State.					
14	Fordwah Canal [Sulemanke]	3366		do					
15	Mailsi Canal [Islam]	4883K	752964	688770	54%	373400	1 : 1	4·3	·2
16	Bahawal Canal [Islam]	3000		Irrigates Bahawalpur State.					
17	Qaimpur Canal [Islam]			do					
18	Abasia Canal [Ialam]	230		do					
19	Panjnad Canal [Panjnad]	7770		do.					
20	Haveli Canal [1939] [Trimmu]	5249K 2750R	1116096	1004463	736276	692000 (1941)	—	3·9	1·3
21	Rangpur Canal (Trimmu)	2710K	318425	306124	183674	107303 (1941)			
22	Shahpur Inundation Canals (5 Nos.) (Jhelum river)	2010 Max: 1295 ord:	128000	108000	66170	81864		...	
23	Muzaffargarh Inundation Canals (Indus river) 7 Nos.	9405 Max: 6270 ord:	660367	595000	358165	359455		...	
24	Indus Inundation canals. Dera Ghazi Khan (9 Nos.) Indus River.	10955Max 7350 ord:	702294	646946	291123	331917		...	
25	Ghaggar Canals 3 Nos. Inundation.	1530	131228	107596	64143	44407			
26	Swarasti Canals 2 Nos. (Inundation.)	597	188761	154880	77440	33020		...	

Note :—K=Kharif R=Rabi   Max:=Maximum Ord:=Ordinary   F=Fullsupply

PART II
CANAL IRRIGATION
CHAPTER II
Principal Crops And Assessment

1. Introductory.

The primary object of all types of irrigation whether lift irrigation, canal irrigation or tank irrigation is to supply water to mature agricultural crops. It is, therefore, considered desirable to acquaint the students with the principal crops sown and matured in the Punjab. The procedure of their assessment is also described briefly. The major portion of the revenue of the Punjab is derived from the assessment of the irrigated crops. The economical design, construction and the maintenance of an irrigation project is a means to an end which can be judged in its efficiency from the quality and the quantity of the agricultural crops produced and the revenue which they bring to the state. The subject matter of this chapter is no doubt the domain of an agricultural engineer, but as the efficiency of an irrigation engineer can best be judged from the revenue return from an irrigation scheme, it is, therefore, essential that he in incharge of the assessment of the revenues accruing from irrgation enterprize and is also equipped with the requisite knowledge about the agricultural crops.

2. Soil conditions in the Punjab.

(*a*) The quality of crops grown depends upon the texture of the soil. The texture of soil is determined by its clay contents. An average soil in the Punjab contains clay from 12 to 15 percent. The soil containing less than 2 per cent clay is useless for crops other than *Barani* grams.

Classification of soils.

Class.	Clay content%	Suitable crops,
Heavy soil.	40	Sugarcane, rice, cotton and wheat (Produce of the last two below normal).
Normal soil	10—20	Cotton, wheat, maize, vegetable, oil seeds, fodder crops. All give the best product.
Light soil	2—8	Wheat, Gram, fodder crops etc. Yield of wheat below normal.

(*b*) The chemical characteristic of the soil is very important to determine the suitability of the crop sown in a soil. The important chemical characteristics are :—

(*i*) Salt content.

(*ii*) pH value which defines the degree of alkalinity or acidity.

The principal salts found in the Punjab soils are sodium chloride, sodium sulphate and sodium carbonate. The yeild of the major crops such as wheat, rice etc., is not affected up to ·18 percent of the total salts. After which the yield drops and at about ·25 percent of the salt, soil becomes absolutely infertile. The average salt content in the Punjab soil is ·05 to ·15 percent increasing towards the westernly direction.

The pH figures give an indication of the alkalinity or the acidity of soil on a logrithmic scale of the degree of ionisation. The scale ranges from 0 to 14 and the figure 7 indicates neutral soil in the sense of chemical reaction. Below 7 the soil is acidic and above 7 alkaline. In the Punjab the pH value of soil is everywhere more than 7. It varies from 7 to 11 rising in westrenly direction. The average value in the cultivated areas of Jullundur, Hoshiarpur, Gurdaspur and Sialkote districts is 8·5. It is now very nearly established by the researches in the Irrigation Research

Institute at Lahore that soil with pH value from 7 to 8·5 gives normal yield of all the principal crops sown in the Punjab. The yield declines with pH value from 8·5 to 9 and with pH value equal to 11 the soil is infertile. The soil of Gangetic plain in U. P. is very nearly neutral and there is no trace of sodium salts. The pH value is no where higher than 7.

3. Rainfall.

The crops require fairly well defined quantities of water at definite stages of their growth. Mr. Wilson, the Scientific Research Officer, Lahore, estimated the following total requirements of water for some of the principal crops :—

Crops.	Irrigation in field.
Rice	44″
Sugarcane	40″
Cotton	20″
Wheat	12″
Maize	18″
Fodder	9″

The rainfall will reduce the depth of irrigation required by artificial methods.

(*b*) In the Punjab, the rainfall is very variable. The contours of equal rainfall called Isohyets are shown in the map of the Punjab appended as plate I, Vol. III.

It is evident from this map that in a very narrow strip near the foot of the hills bordering north-east, the rainfall is 35″ to 40″ on the average in a year. In the plains the rainfall is everywhere less than the normal requirements of the depth of waterings required for the principal crops sown in the Punjab. The rainfall connot be turned on and off as required by the crops at the time of sowing and maturity. Moreover in the Punjab, the rainfall is concentrated up to 75 percent in the rainy months of July and August and for the remaining part of the year, the rainfall is very scanty and is very precarious.

It is therefore, that the Punjab needs very badly the help of artificial irrigation which has led to the development of canal irrigation system and which surpasses in its efficiency all such works in the world.

4. Principal crops in the Punjab.

The principal crops sown in the Punjab are given in the Appendix No. I appended to this chapter. The English and the Hindustani names of the crops are given. The time of sowing and harvesting the crops is given in columns No. 4 and 5. The column No. 6 gives value of depth of the canal irrigation required in feet known as Delta. Column No. 7 gives the average produce of the crop in maunds. Column No. 8 gives the approximate water rate charged for the irrigation water. The value of delta varies from canal to canal according to rainfall and the efficiency of the cultivators. Similarly water rate changes from canal to canal according to the soil condition and the anticipated produce. The average produce is very variable according to the nature of the soil from one district to another. The average produce is fixed in the settlement reports of a district according to the nature of the soil. The figures of the columns No. 6 to 8 should be takan merely as a guide to acquaint the student with approximate usual values.

5. Fertilizers and Manures used for agricultural crops.

Introduction.

Of the principal food materials required for growth of plants, by far the most important one in which the Punjab soils are deficient, is nitrogen. The deficiency of nitrogen is the main problem of manuring. Much of the farm-yard manure is burnt as fuel, while a large quantity of combined nitrogen is exported in the form of oil seeds, food and other grains, and animal products such as hides and bones. The only way of stopping the wasteful practice of making manure into cakes and burning it, is to provide an alternative supply of fuel by planting quick growing trees near the villages.

Farm-yard Manure.

Farm-yard manure increases the retentive power of the soil for dissolved substances. It causes the soil to be puffed up and increases the pore space which improves the tilth and general condition of the soil. It increases the water holding capacity of the soil and helps the biological activity going on in the soil.

Farm-yard manure contains about 75—80% water, 0·6—0·7% nitrogen, 0·1—0·3% phosphorus (as P_2O_5) and 0·4 – 0·5% potash (as K_2O). The urine is much richer in nitrogen and potash but contains only traces of phosphorus. A ton of manure contains about 15 lbs. nitrogen, 5 lbs. phosphorus and 10 lbs. of potash. The composition varies with the type of the animal, the quality and the quantity of food, proportion and nature of litter and the stage of decomposition that has taken place in the manure itself. Since the liquid part of the manure is much more valuable as compared with the dung, far more care should be taken to preserve this than is usually the case. The best method of collecting urine is the use of litter. Any waste material – wheat straw, *wheat bhussa, toria* or *sarson pallar*, sugercane thrash, grain *bhussa* and any other vegetable waste can be successfully used as an absorvant of urine. The litter should be spread under the cattle in the evening and may be carried next morning along with the dung into pits, which may be of any suitable length and breadth and only two feet deep. When the pit has been filled, water may be occasionally sprinked over the stuff so that it remains moist but no free water stands. A turning is given to the material after every two sprinklings of water. The stuff becomes ready for carting in about 3 month's time.

As stated in the introduction, quantity of farm-yard manure available for use for crops is inadequate. Something can be done in this direction by the manufacture of composite from any rubbish material on the lines suggested above. Compositing of waste material is now a recognised part of the activity both of the agricultural experiment stations and village improvement associations and we may expect some addition to the manurial resources of the country as a result.

Green Manuring.

Another method by which the deficiency of farm-yard manure can be made good is by green manuring. It provides all plant food ingredients in the soil and supplies a large amount of humus which improves the texture and water holding capacity of the soil. Experiments show that *Gowara* is the best crop for burying in as a green manure under condition in the canal colonies while (*san*) hemp appears to be the most suitable crop in the submountaneous tracts. Green manuring with *Dhaincha* has proved exceeding advantageous under waterlogged and alkaline conditions prevailing in the *Kallar* tracts of the Punjab where rice is mostly grown.

Artificial fertilizers.

Of all the essential elements required by the plants only three *viz.*, nitrogen, phosphorus and potassium are important from manurial point of view, because they are found generally in small quantities in the soil.

(*a*) **Nitrogeneous fertilizers.**

They tend primarily to encourage above ground vegetative growth and impart a green colour to the leaves. They delay maturity of the crops. In cereals plumy condition of the grain is increased while straw gets weakened with excessive applications. Examples are ammonium sulphate, sodium nitrate.

(*b*) **Phosphatic fertilizers.**

They are an essential part in the formation of grain, hasten the maturity of crops, counteract the conditions produced by nitrogen, encourage root development especially of the lateral and fibrous rootlets, strengthen the straw cereals and increase the weight of grain compared with that of straw, increase the quality of grain of cereals and of grass in pastures and increase resistance of plants to diseases. Superphosphate is the common material used as a phosphatic manure.

(*c*) **Potassic fertilizers.**

The presence of available potash has much to do with the general tone and vigour of the plant. By increasing resistance to certain diseases it counteracts the ill effect of too much nitrogen while delaying maturity it works against the early ripening effect of phosphorus. It is essential in formation of plumb and heavy grains. Potassium sulphate is the material used as a manure.

The results of manurial experiments in the various parts of the province with regard to the use of manures have shown :—

(*i*) Farm-yard manure has almost always given the best results. It is the cheapest means of adding organic matter and nitrogen to the soil. Average quantity required per crop is about one to two cart loads per *kanal* ($\frac{1}{8}$ acre).

(*ii*) The application of nitrogeneous fertilizers such as sodium nitrate and ammonium sulphate has given good results. The quantity of manure required per acre per crop is obout one maund. These manures increase the yield of wheat, but the increase hardly covers the cost of manuring. In the case of cotton and sugarcane, these manures have given profitable returns.

(*iii*) The application of phosphatic and potassic fertilizers has almost always resulted in a financial loss. This is due to the fact that the market prices of such fertilizers being high, it is not economical to use them for agriculture.

6. Plant Diseases of the Principal Crops.

(*i*) **Cotton.** Cotton is mostly damaged by --

(*a*) Boll worms.

(*b*) Jassids.

(*c*) Root rot.

(*a*) **Boll worms.**

In the beginning of the cotton season the cater-pillar bores into the top tender portion of the shoots. When flowers, buds and bolls appear, the larvee turn their attention to them and flowers and buds of bolls of the plant are attacked ; observations have shown that up to 75 per cent of the flower buds and 60 per cent of the bolls may be damaged by this pest.

Remedies. There are parasites which feed on the boll worms. This parasite should be encouraged,

In the beginning of the cotton season the attacked shoots should be collected and burnt.

The plants should be shaken thoroughly by dragging a rope over them. The boll worms will drop on the ground. The ground should be immediately watered to drown the boll worms.

After cotton crop is over, cotton sticks should be removed from the fields, cutting them 2" below the ground.

(*b*) **Jassids.**

The adults are reddish in witner and greenish-yellow in summer. The pest attacks cotton in June and remains on the plant up to the November. It sucks the sap from under side of the leaves with the results that the fruit capacity of the cotton plant is very seriously affected. No effective control against this pest is known. Rough hairy leaved types are slightly resistant to the attacks of the jassids.

(*c*) **Root Rot.**

It appears in patches. No effective remedy is known. It is suggested that cotton should not be shown for a year or so in the field in which root rot has appeared.

(*ii*) **Sugarcane.**

There are two main diseases ; red rot and stem borer.

Red rot is an insect disease. The insect bores into the stem. Resistent varieties should be sown. The caterpillar in the case of the stem-borer attacks the central shoot and kills it. This is called the 'dead heart'. These dead hearts should be collected and burnt. The insect also hybernates in stubbles which may also be removed and burnt.

Interculture and irrigation recommended.

Crude oil emulsion placed in irrigation channel is also useful.

(*iii*) **Rice.**

The caterpillar is the most destructive pest of rice. This kills the central growing leaves which dry up, but when plant is attacked at the flowering stage, the ear-head stands bent and is devoid of grains. To control this pest the best remedy is not to provide facilities for the borer to breed. This can be prevented by the absence of the early crop of the rice. Rice stubbles should be ploughed up, collected and burnt.

(*iv*) **Gram.**

Gram suffers from wilt in the month of February. After flowering, the plants dry up. Recent work indicates that salt in the soil accelerates the intensity of the disease. In reclaimed land the attack is less.

The gram cut-worms.

The caterpillars get into the pods and destroy the seed. The damage never is very serious.

(*v*) **Wheat.**

The most common diseases are :—

(*a*) **Ear-cockels or** *Mumni*. This is caused by worms which produce black bolls in place of grain. Seeds free from disease should be sown.

(*b*) **Rust or** *Kungi*. This is a fungus disease which is serious only in cloudy winters. The leaves show yellow or black spots. Rust resistent varities of the seed should be sown.

(*c*) **Smut or** *Kangari*. When this fungus attacks the wheat plant, black powdery substance is produced in place of grain. The seed before sowing should be treated with hot water.

(*d*) **Bunt.** This largly occurs in hill. *Kanals* of grains are destroyed and replaced by ill-smelling blackish powder. Seed from healthy crop should be sown.

7. Economy of Water.

Water is the finished product of the Irrigation Department and has been brought to the fields by incurring a great expenditure in the construction of headworks and the canal system. It is very essential that it should be used very economically. It is both in the interest of the Government and the cultivator to use it carefully and economically and thus obtain more and better crops.

(*a*) The cultivators should be encouraged to line the water-courses. They can save about 20% water and thereby increase the area irrigated by 20%.

(*b*) Wastage should be avoided by adopting *khal kiari* system of irrigation. In this system a field is divided into *kiaris* of one *kanal* and subsidiary water-courses are construted so that water does no flow through the already irrigated portions of the field. When one *kiari* is full, water is closed and led into another one. The arrangement is shown in Fig. 1.

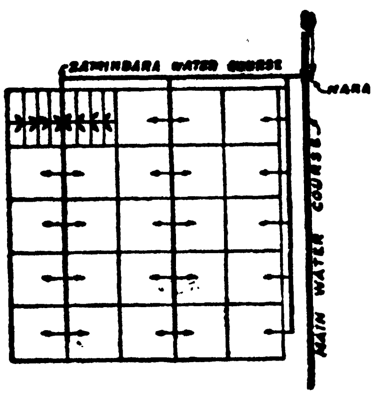


Fig. 1.

There used to be legal binding to enforce this very economical system under the rules of the canal Act No. VIII of 1873 but this has been stopped since 1928. The enfringement of these rules used to be punished by levying additional water-rates.

(*c*) The wastage should be avoided by proper alignment of the water-courses so that there is no heading up in the water-course to command any field.

(*d*) The water-courses should be properly maintain to avoid wastage from breaches in them.

(*e*) Excessive depth of watering should be avoided. It is no use flooding a field with one foot depth of water when depth of 3 inches will do.

(*f*) Economical use of canal water is an effective anti-waterlogging measure in the water-logged areas to reduce the menace of waterlogging. Excessive watering is also injurious to the plants.

8. Sub Irrigation.

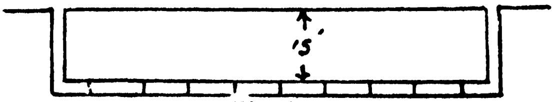


Fig. 2.

(A) Brigadier G. Howison, C.I.E., carried out experiments at Ajmer, India, to grow crops by sub-irrigation with success. The principal is simple. In flow irrigation a field is flooded with 3″ to 4″ depth of water. Most of it is lost in surface evaporation and then in soil evaporation and only a small portion is used for transpiration by the plants by the root action extracting water from the soils. Roots suck water and liberate carbon diaoxide which must escape into the atmosphere. This can only take place when the soil is partially saturated. The complete saturation by flooding is not the necessary feature of the plant life. In this system water is supplied through earthen ware pipe line or porous concrete pipes (mostly through the joints) which are 12″ to 14″ in length. These are laid 15″ to 18″ below ground as shown in Fig. 2.

A pipe line thus laid wets the soil 3 to 4 feet all round in reducing moisture contours. The rise of this moisture is said to be due to capillary action in the soil. Even if the capillary tubes in the soil were not full, this is possible because of the cellular pore space in the soil crust and the affinity of the soil particles for water. The moisture thus drawn up is used by roots of the plants for nourishment. Successful agricultural crops have been raised with out-turn more than normal. The water used is only ¼ to ¾ of that required in flow irrigation.

(B) Precautions and arrangement.

(*i*) The pipe line should be laid in level. There should not be any bend up or down as the pipe line is not closed, that is, water tight. If the ground is very uneven the arrangement should be as shown in Fig. 3.

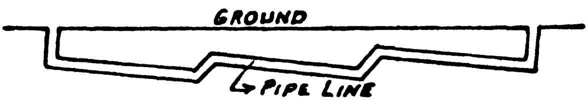


Fig. 3.

(*ii*) 18″ deep trench slightly wider than the external diameter of the pipe be dug and pipe laid by use of spirit level or by letting a little water to flow after laying 8 to 10 feet length.

(*iii*) The lateral spacing of pipe line differs for different crops. Lawn 8 ft. interval. Lucerine 6 ft. interval, and vegetables 3 to 4 ft. apart.

(*iv*) There should be a vent at the end of each pipe line for inspection and to prevent air lock.

(*v*) The crops need irrigation as below in dry weather. Grass and cereals 3 to 4 days. Vegetables and Lucrines 2 to 3 days.

(C) This system of irrigation is only possible when water is available every two or three days but in smaller quantity. The initial expenses for laying pipe lines are very heavy. This system can only suit small gardens under strict supervision. Water saving is no doubt large.

9. Internal distribution of water.

Water is supplied from a permanent masonry structure called outlet or *moga*, the capacity of which is fixed according to a water allowance for a definite area as explained in detail, in Chapter IV Part II. The *chaks* are designed in such a way that the discharge of the outlet is not less than one cusec and more than 2·75 cusecs. Water is conveyed to the fields through channels which are called water-courses or "*khalas*". Main water courses are designed by the Irrigation Department and the work is carried out, the cost being recoverable from the cultivator with the land revenue in small instalments called acreage rate.

All water courses are essentially *zamindari* channels. The cultivators are responsible for their maintenance under provisions of the Canal Act No. VIII of 1873. Usually only one *naka* is given from the main sanctioned water course to every square or holding of a cultivator. The water courses, as shown in Fig. 1, are constructed and maintained by the individual owner of the square or a holding.

(*b*) The distribution of the water by the cultivators amongst themselves is essentially the concern of the *zamindars*. They can take the help of the Canal Department under provisions of Section 68 of the Canal Act No. VIII of 1873 to get share or *wari* of individual shareholder fixed according to the area owned by him in a *chak*. Only Divisional Canal officers are authorised to investigate and pass orders on *warabandi* cases. The turn is usually fixed in 'hours' and sometimes in '*pehrs*' in villages where the cultivators are illiterate. Usually the cultivators maintain a common clock and a gong.

(*c*) The cultivators then follow the turn and time allotted to each shareholder of a water course. Lots of disputes arise in the internal distribution of the canal water but a canal Officer does not interfere except as provided under the Canal Act Section 68. The revenue staff, the *zilladar* and the *patwari*, are supposed to give the necessary advice when required by the *zamindars*. In the cases of *warashikni* (taking water out of turn) only remedy available to the canal department is that the person who disregards an authorised *warabandi* promulgated by the Divisional Canal Officer under Section 68 of the Canal Act, can be charged special rates under Section 31 and 33 for using water in an unauthorised manner. Such cases are instituted by the canal officer on the written application from an agrieved person. It is generally possible to set right irregular working of the *wari* by rendering sympathetic advice to the cultivators.

(*d*) *Nikal* is often the cause of great trouble. The last man receiving *wari* on a water course benefits by the amount of the water contained in the water course when the turn changes to the lands in the beginning of a water course. Sometimes the cultivators who get water at night have a great grievance. The night *wari* can be changed to day *wari* in alternate year.

(*e*) The changes in *chakbandi*, *nakas* and supply of water through the intervening water courses are made under Section 20 of the Canal Act No. VIII of 1873 by the Superintending Engineer and the Divisional Officer. The Divisonal Officers possess final power in the case of water courses and the Superintending Engineer in the case of *chakbandi* changes.

10. Units of measurement of areas.

The land measures are of two kinds, in the Punjab the *bigha* measure and the *kanal* or *ghumao* measure. They are shown in the following table : –

I. (*a*) Ordinary *Bigha* Measure,

Description.	Indian equivalent.	Area in sft.	Area in acres.	Remarks.
1 sq. karam	1 Biswani	22·6875	·000521	
20 Biswanis	1 Biswah	543·75	·010416	
20 Biswahs	1 Bigha	9075·0	·2083=5/24	

I. (*b*) *Shahjahani Bigha* Measure.

Description.	Indian equivalent.	Area in sft.	Area in acres.	Remarks.
1 sq. Gatha	1 Biswani	68·06	·00156	Note—This is used in southern Punjab and in States.
20 Biswanis	1 Biswah	1361·25	·03125	
20 Biswah	1 Bigha	27225·0	·624=5/8	

II. (*a*) *Ghumao* measure ; *Karam*=5·0′

Description.	Indian equivalent.	Area in sft.	Area in English acres.	Remarks.
1 sq. karam	1 sarsahi	25·0	·000574	(a) This is used in Central Punjab.
9 Sarsahis (kan.)	1 marla	225·0	·00516	(b) 1·21 ghumao equal to 1 acre.
20 marlas	1 kanal	4500·0	·1033	
8 kanals	1 ghumao	36000·0	·8264	(c) One karam is two paces of a mam or 3 trots.

II. (*b*) Acre measure ; *Karam*=5½′ (colony area)

Description.	Indian equivalent.	Square feet.	Acre
1 Square karam	1 sarsahi	30·25	·000694
9 Sarsahi	1 marla	272·25	·00625
20 Marlas	1 kanal	54·45	·125
8 kanals	1 acre	43560·0	1·0

The length of *karam* varies in various districts of the Punjab. In the colony areas, a *karam* is 5·5 ft. and one *ghumao* is equal to one English acre. It is said to be a canal *ghumao*.

11. Assessment.

There are three usual methods for the assessment of revenue returns : -

(*a*) Permanent settlement.
(*b*) Fluctuating assessment.
(*c*) Volumetric assessment.

(*a*) **Permanent Settlement.**—This system is in vogue in the areas which are not irrigated from the canal. The land revenue is worked out on the basis of quarter net-assets for the *chahi* and *barani* crops for the various kinds of land such as heavy soil (*rakhar*), normal soil (*rohi*), light soil (*maira*) and sand soil (*retli*). Sometimes these are sub-divided into various categories. The land revenue is payable irrespective of the fact whether a crop is sown or not. There is no remisson for the failed crops unless the calamity is general and wide-spread when a part or whole of the land revenue can be remitted at the discretion of the Government. The land revenue rates are revised at time of the settlement which takes place after every 20 to 25 years.

(*b*) **Fluctuating assessment.**—The Fluctuating assessment is in vogue in the canal irrigated tracts. Only the area sown under crops where the crops attain maturity is charged the land revenue and the water rates. The rate charged for canal water supplied for the purpose of the irrigation to the occupiers of the land is called the occupiers' rate.

In addition to the occupiers' rate, a rate called owner's rate (*khush-hasiati or malikana*) is levied on the owners of the canal-irrigated lands on account of the increase in annual value or produce by the introduction of the canal irrigation. This is not to exceed the land revenue in vogue.

The term water rates includes both the occupirer's and the owner's rates.

Land revenue (*asal* or *mamla*) is worked out on the basis of quarter net-assets as in the case of permanent settlement at the time of periodical settlements.

There are a few other miscellaneous payments which a cultivator is required to pay. They are called cesses. They include *malba* and other local rates. The *malba* is the collection of the village common fund and local rates are imposed by district boards for amenities of life such as roads, schools, hospitals etc.

In the case of fluctuating assessment all of the above-mentioned charges are remitted when a crop fails totally. The remissions for failed crops are given on application (*tanaza*) form. The cultivator intimates to the canal departmant the field number and the crop which has failed. The field is inspected by the canal department official called *zilladar* and an estimate of the likely produce, as compared with the average produce of the locality ; worked out in the settlement reports of the districts ; is made.

(*a*) If the crop is more than 4 annas but less than 8 annas ($\frac{1}{4}$ to $\frac{1}{2}$ produce), then half remission is granted. Water rates and land revenue are remitted by half.

(*b*) If the crop is less than 4 annas (produce less than $\frac{1}{4}$th) full remission is granted.

Remissions recommended by a *zilladar* are checked by the Deputy Collector and Sub-Divisional-Officer according to a fixed percentage.

In the case of a wide-spread calamity such as hailstorm, general crop diseases, or locust attack, remission (*Kharaba*) is sanctioned by the Government, for the whole tract for all fields sown, at its discretion according to the damages caused by such a calamity. The local Goverenment delegates powers to the civil and irrigation officers to grant such remissions generally. Lately half remissions have been abolished in *Tanaza* remissions, but are still in use in the case of *kharaba* remissions (vide-spread calmity).

(*c*) **Volumetric Assessment.** In this case assessment is made on the quantity of water supplied. A gauge is fixed on or adjacent to the outlet from which the discharge is calculated in a simple manner which is understood by the consumer. The gauge is, thus the sole measure of the water supplied. Ten per cent variation in the supply of the outlet is ignored without any compensation to the parties, But if it is less than 10% the supplier makes it good with in 15 days. The rates charged for such assessment are arrived at by calculating the value of cusec per day as given below :—

$$\text{Value of cusec per day} = \frac{T \times 1{\cdot}1}{A \times N}$$

T = Total direct receipt for the crop at the distributary.

A = Average daily discharge for the crop at the distributary head.

$$= \frac{\text{total discharge at head of distributary during the crop}}{183} = \text{Period of the crop}$$

N = 183, and 1·1 a constant.

The values of T and A are the average values of the last three years.

12. Record of Irrigation.

In the fluctuating assessment, the record of irrigation, as it is done, form the basis of the assessment. The correct record of irrigation is. therefore, as important as measurement of the work designed and executed by an engineer, because it forms the basis of the revenue to the Government.

Patwari is the canal official who makes the initial record of the irrigation in a book called *Shudkar Khasra*. Every page has two parts. One is called *Shudkar* where irrigation is recorded giving the field No., the occupier's and owner's name, approximate area of irrigation and the crop sown ; the other is called *Khasra* where the correct area of the field, the crop matured and its class of assessment are entered. The crops totally and partially remitted are noted therein.

From the *Khasra* the entries are transferred to the demand statement (*khatauni*) which gives details of all recoveries due to water rates, land revenue and the cases by *khatas*. *Khata* is the holding of an individual. The assessment work in the irrigated tracts is done by the canal engineers and the demand statement is sent to the Deputy Commissioners of the districts for realization through the *lambardars*. *Lmabardar* is the headman in a village appointed by the Deputy Commissioner. He recovers his remenuration as *pachotra* for the revenue collected by him and paid into the treasury. The *pachotra* is usually 5 per cent of the collections normally and 3 per cent of *abiana* in the canal irrigated tracts.

A canal *Patwari* is supplied with certified corrected copies of the following records by the collector :—

(*a*) *Shajra*—map of a village on cloth showing full fields by numbers.

(*b*) *Khasra Bandobast*—Field book containing record of the correct area of each by numbers.

(*c*) *Jamabandi*—Records of rights of the owners in the various field numbers.

The entries made by the *patwari* in the *shudkar khasra* are checked by the various officials, *zilladar*, Deputy Collector, Sub-Divisional Officer and the Divisional Officer. The checking should be efficient as this record forms the basis of the income coming to the Government. Some of the bad practices of the *Patwaris* are mentioned below :—

(*a*) Not to record a field at all (*b*) to record less area (*c*) to convert the nature of irrigation, *nehri, chahi, barani* (*d*) to omit the name of the crop so that at the end of the crop season he may put the name of the crop having lower charges.

13. Assessment of special charges.

The special charges are leveid in addition to the usual charges under the canal and drainage Act No. VIII of 1873 for the items mentioned below. It is, a misnomer to call them punitive charges or *tawan*.

(*a*) Unauthorised Irrigation "*Najaiz*."

This may take any of the following forms :

(*i*) Use of water through water courses and *nakas* which are not sanctioned.
(*ii*) Cutting a Government Irrigation Channel.
(*iii*) Enlarging or damaging the outlets.
(*iv*) To open the outlets when closed under proper authority in *tatils*.
(*v*) *Warashikni* (taking water out of turn), the powers have now generally been transferred to *panchayat*.
(*vi*) Waste of water "*Abzai*."

(*b*) Water lost from breaches from water courses is charged at special rates as it is the duty of the cultivator to maintain the water courses in good condition.

The special charge levied in each case can be upto six times occupier's rate at the discretion of the Divisional Officer.

14. Miscellaneous receipts.

Miscellaneous revenue falls under the main heads and is charged and collected by the Sub-Divisional Officer according to the rates sanctioned by the Government when the authority to use water has previously been obtained from the Divisional Officer.

1. Mill rents.
2. Fixed contracts for water supplies to Municipalities and other local bodies.
3. Minor items. Water for construction of houses, burning bricks etc.

The usual rates for miscellaneous use of canal water are as below :—

	Rs.			Unit
(*a*) Brick making and pise wall	0	3	0	100 cft. or 100 No.
(*b*) Concrete & masonry	0	2	0	100 cft.
(*c*) Metalling of roads	10	0	0	per mile
(*d*) Consolidation of *kacha* rods	30	0	0	per mile
(*e*) Water supplied in bulk	1	0	0	2500 cft.
(*f*) Local bodies (not commercial purposes)	1	0	0	6000 cft.
(*g*) Watering avenues (*kharif*)	2	8	0	per mile
(*rabi*)	5	0	0	per mile

15. Duty of canal water.

Duty is the measure of the working of a channel. It is defined to be area irrigated a crop, per cusec of the mean discharge. It is got by dividing the irrigated area by the mean discharge utilized in the period in question.

The mean discharge in any month is the aggregate of daily discharges during the month divided by the number of days in a month. The mean discharge of a channel during a crop is the aggregate of the daily discharges throughout the crop, divided by the number of days in the crop.

Kharif crop=April to September=183 days
Rabi crop=October to March=182 days
Delta is the average depth poured on land in feet in a crop or per annum.

$$\text{Delta } \Delta = 2\frac{Q}{A}N \qquad (1)$$

Where Δ=Depth of the water in feet
Q=Mean daily discharge in a crop
A=Area irrigated in acres
N=Number of days in a crop

Delta in feet=Acre foot

Acre foot is the term used in America representing one foot depth of water over one acre, *i.e.*, 43560 cft. of water.

Full supply duty of a channel is the area irrigated divided by the full supply capacity of the channel.

Full supply factor is the authorised or the stipulated area per crop or per annum for one cusec discharge at the outlet head. This used to work out the proposed discharge of the outlet.

Kharif, *rabi* ratio is the ratio of the proposed areas to be irrigated in *kharif* crop and in *rabi* crop. The usual ratio is 1 : 2 *i.e.*, *kharif* area is one half of *rabi* area.

Intensity of irrigation is the percentage of the culturable commanded area which is proposed to be irrigated. Some land should be allowed to have rest during one year. The usual intensity of irrigation in colony area is 50 to 100 per cent and in proprietory villages about 40 to 60 per cent depending on the existing wells in the area.

Efficiency of canal irrigation.

In consequence of the development of the canal irrigation throughout the Punjab province, the water of all the rivers have to be distributed more carefully than in the past. It is very essential that the engineer should watch that the distribution of supply is equivitable to the individual watercourses. The efficiency of the actual performance of the water used from individual outlets and the channels is watched by an engineer by adopting the following methods :—

(*a*) Actual discharge observations of the water course by the use of author's portable tin flumes or cipollete wires.

(*b*) By watching monthly delta calculations for every channel in a delta statement and then final delta for the crop.

(*c*) By ploting the outlet efficiency diagrams. The form and the outlet efficiency diagram is given in Appendix II appended to ; this chapter. No outlets are reduced simply on account of excess in the area irrigated by a water-course because it may be due to good husbandary or timely rainfall at the sowing period.

17. Examination Questions.

1. A distributary is designed for alternate running to irrigate 4800 acres in *rabi* with discharge of 40 cusecs. In a certain year it runs for 60 days in *rabi* with an average discharge of 30 cusecs and irrigates 3000 acres. Work out the channel's (*i*) Full Supply Factor, (*ii*) Duty, (*iii*) Depth of water (Delta) and explain what you understand by each.

2. What do you understand by fluctuating assessment ? How does it differ from permanent assessment ?

3. What measures can an engineer take to ensure economy in the use of canal water ?

4. Explain the following terms :—
(*a*) *Nikal* (*b*) *Khata Khasra* (*c*) *Bandobast*, (*d*) Delta, (*e*) Efficiency diagram, (*f*) *Warashikni*, (*g*) *Tatiling*, (*h*) *Shajra*, (*i*) *Shudkar*, (*j*) *Khatauni*.

5. Describe briefly a few of the common diseases of the following crops and their remedies.
(*a*) Rice (*b*) Sugarcane (*c*) Wheat (*d*) Cotton.

6. Give the times of sowing and maturing and the likely produce in maunds per acre for the followng in the canal irrigated tracts :—
(*a*) Wheat (*b*) Rice (*c*) Cotton (*d*) Maize (*e*) Tobacco (*f*) Barley.

7. What factors govern the suitability of the soil for various crops ?

8. How does an Engineer watch the efficiency of the distributing channels ?

9. Enumerate six principal crops, their seasons of sowing and maturing and their water requirements as found in East Punjab and contingnous areas of Pepsu and Delhi State. (*P U. 1953*)

10. Draw a map of Punjab (India) and Pepsu (rough sketch) and indicate the relative needs for irrigation in the different tracts with reference to the nature of crops grown. Indicate the annual rainfall in the higest and lowest intensity areas. (*P.U 1953*)

Chapter II Appendix No. 1.

Principal crops grown on the canals in the Punjab.

1	2	3	4		5		6	7	8
Serial No.	Name of Crop. English	Name of Crop. Hindustani	Sowing time. From	Sowing time. To	Harvesting time From	Harvesting time To	Delta	Average produce in Mds. per acre.	Water rate in Rs. per acre
			Kharif Crops.						
1	Rice	Jhona. Dhan Chawal Munji	June	July	Sep.	Oct.	2·5	8	6/1
2	Maize	Makki	June	Sep.	Sep.	Nov.	1·5	14	4/-
3	Great Millet	Jowar	June	July	Sep.	Oct.	1·1	...	2/8
4	Spiked Millet	Bajra	July	Aug.	Oct.	Nov.	1·0	8	3/4
5	Millets	Charri	May	July	Aug.	Oct.	1·0	32	2/8
6	...	Moth.	June	July	Oct.	Nov.	1·0	8	2/8
7	Pulses	Mung, Mash	June	July	Oct.	Nov.	1·0	8	3/4
8	Vetch field	' uara	April	June	June	Aug.	1·0	32	3/8
9	Cotton	Rui, Bari, Kapas	April	July	Sep.	Jan.	1·5	6½	5/4
10	False Hemp	Sani	July	...	Oct.	...	1·25	8	4/12
11	Hemp	San	July	...	Oct.	...	1·0	8	4/12
12	Italian Milet	Kangni	March	April	July	Aug.	1·0	32	2/8
13	Indigo	Neel	March	June	Aug.	Dec.	1·5	8	6/4
14	Henna	Mehndi	June	Aug.	Oct.	Nov.	1·5	8	6/4
15	Gingelly or sesame	Till	June	Jully	,,	,,	...	8	4/4
16	Sugarcane	Ganna, ponda Naishkar, kamad	March	...	,,	Feb.	3·0	30	11/-
17	Minor Millet	Madal	May	June	,,	Nov.	...		2/8
18	Chillies	Mirch	Feb.	March	Sep.	,,	2·0	8	6/4
19	Melons	Kachra (phut)	July	...	Oct.	...	·75	8	4/12
20	Water nuts	Singhara	June	July	Oct.	Nov.	...	8	7/8
21	Turmeric	Haldi	March	April	Nov.	Dec.	...	8	6/4
22	Vegetables	Tarkari	Feb.	May	May	,,	...	8	5/8
23	Lucerne	Lusan	Oct.	...	Dec.	June	1·5		2/8
24	Grass	Ghas	May	Aug.	May	Sep.	1·0		2/8
25	Cowpea	Rawan	June	July	Oct.	Nov.		8	2/8
26	Water melon	Tarbuz	July	...	,,	...		8	4/12
27	Raghi or Mandhwa or Mandhal	...	May	June	,,	Nov.			2/8
			Zaid Kharif						
1	Indian Rape	Toria	Sep.	...	Jan.	...	...	8	4/4
2	Turnips	Shalgam	,,	Nov.	Nov.	Feb	1·75	8	5/8
3	Potatoes	Alu	,,	...	Jan.	...	2·0	8	5/8
4	Vegetable	Tarkari	,,	Oct.	Nov.	Fev.	1·75	8	5/8
			Rabi Crops						
1	Wheat	Gehun, Gandam	Oct.	Dec.	April	...	1·25	14	4/4
2	Barely	Jao	Oct.	Nov.	March	April	1·5	8	6/4
3	Mixed grain	Berra	Sep	,,	...	,,	1·25	12½	4/4
4	Oats	Jawi	Oct.	Dec.	Feb.	,,	1·25	11	4/4
5	Gram	Chana, Chhola Nakhud	Sep.	Oct.	April	...	1·0	11	3/4
6	Lentil	Masur	Oct.	Nov.	,,	...	1·0	8	3/4
7	Peas	Matar	,,	...	Nov.	April	1·75	8	5/8
8	Poppy	Post	,,	...	April	...	2·0	8	6/4
9	Kiseen	Saunf	,,	Nov.	March	April	1·75	8	6/4
10	Coriander	Dhania	Sep.	,,	April	May	1·75	6	6/4
11	Saliflower	Kasumba	Oct.	,,	Jan	April	1·75	8	6/4
12	Rape	Sarson	,,	,,	March	...	1·5	8	4/4
13	Linseed Flax.	Alsi	,,	,,	,,	April	1·5	8	4/4
14	Alsi	Alsi	,,	,,	,,	,,	1·5	8	4/4
15	Rocket	Taramira	,,	,,	,,	...	...	8	4/4
16	Carrots	Gajar	Sep	,,	Nov.	Feb.	...	8	5/8
17	Vegetables	Tarkari	Aug.	,,	Oct.	April	1·5	8	5/8
18	Lucerne Grass	Lusrn Ghas	Oct.	...	Dec.	June			2/8F

N.B.—Crops grazed are charged fodder rate (F) and crops used are green manure are not charged at all.

1	2	3	4		5		6	7	8
S. No.	Name of crop. English	Hindustani	Sowing time, From	To	Harvesting time From	To	Delta.	Average produce in Mds. per acre.	Water rate in Rs. per acre.
19	...	Maina	Oct.	Nov.	Feb.	April			2/8F
20	Millet	Senji	,,	...	,,	March			2/8F
21	Person Clover	Shaftal	,,	...	Feb.	May			2/8F
22	Radish	Muli	Sep.	Nov.	Nov.	February		8	5/8
23	Mustard	Rai	Oct.	,,	March	...		8	4/4
24	Fodder	Methra	Sep.	,,	Feb.	April			2/8
25	Egyptian clover	Berseem	,,	Oct.	Nov.	May			2/8
26	Garlic	Lehsan	Oct	...	April	...		8	6/4
27	Cumin	Zira	Sep.	Nov.	April	May		8	6/4
28	Chicory	Kasni	,,	,,	,,	,,		8	6/4
			Zaid Rabi Crops.						
1	Maize	Makki	March	April	May	June	1·0		2/8F
2	Millet	Charri	,,	,,	,,	July			2/8F
3	...	Swank	,,	,,	July	Aug.			2/8F
4	Common Millet	China	,,	,,	May	June			2/8F
5	Tobacco	Tambaku	Feb.	March	June	...	2·0	8	6/4
6	Melons	Kharbuza	,,	,,	May	June	1.0	8	4/12
7	Water Melons	Tarbuz	May	...	June	July	1·0	8	4/12
8	Onions	Piaz	Jan	Feb.	May	June		8	5/8
9	Potatoes	Alu	Feb.	...	,,	...	2·0	8	5/8
10	Vegetables	Tarkari	,,	March	,,	Aug.		8	5/8
11	Cucumber	Khira	,,	,,	,,	June		8	

N.B.—Crops grazed are charged fodder rate (F) and crops used as green manure are not charged at all.

PART II

CANAL IRRIGATION

Chapter III

Hydraulics and control of large rivers

1. Introductory

India possesses a high mountain range called the Himalayas forming the north boundry throughtout its vast sub-continent separating the North Indian plains from Tibet (China), Russia, Caucassia and Afghanistan. Himalayas is a Sanskrit word meaning Home of Snow. In the Himalayas, there is perpetual snow which feeds the great rivers of Northern India by melting. The rivers get their peak discharge due to snow melting in the months of April and May which period synchronizes with the time of maturing of the *rabi* (winter) crops and the sowing of *kharif* (summer) crops. The rainfall in Northern India is concentrated in its intensity and magnitude in the months of July and August to the extent of about three fourth of the annual average rainfall. The rest of the rainfall occurs in the months of December and January when the river supplies fall down due to decrease of the snow melting in the low temperatures during these cold winter months. On account of the heavy rainfall in the months of July and August the rivers of Northern India rise in high floods which are many times more than the normal supplies in them throughout the year. The ratio of the flood discharge is about 50 to 100 times the normal winter supplies of the rivers. All the rivers of Northern India emanating from the Himalayas carry in their waters lot of suspended staff in the form of fine silt and clay, when flowing through the vast plains after leaving the hills. All of them form new lands and erode some while passing through the plains. They are, therefore, called alluvial rivers. The hydraulics of these rivers is very complex and peculiar due to the flood discharges being very high as compared with the normal discharges requiring flood sections vastly out of proportion to the sections required for winter supplies.

2. Major divisions of river channel.

The river channels of the large rivers of the Indo-Gangetic plains are divided into four major divisions.

(*a*) **Mountain stage.**

The river flows through the hilly gorges with rocky bed. Water is clear. The bed slopes are very steep from 1 in 100 to 1 in 500 with occasional rapids and adrupt falls. In hills it is mostly snow water.

(*b*) **Boulder stage.**

When the river leaves the mountains it flows through the sub-mountainous tracts with boulders beds having steep gradiant of the order of 1 in 500 to 1 in 1000. Water is generally clear except in the rainy season. The section is usually well defined. The boulders of the round splunial shape vary in size from adout 1 foot diameter in the hills to about ½″ to 1″ size shingle or *bajri* as the river enters the plains.

(*c*) **Trough stage or rivers in plains.**

The river sections are well defined channels with high banks. Water carries large amount of silt in suspension. The velocities are high enough to erode the bed causing deep scours and to damage the sides causing serious erosions. The proportion of suspended matter in water in the form of clay and silt is usually as high as 1/300 to 1/1000. The bed slope is pretty steep from 1 in 1000 to 1 in 2000.

(*d*) **Deltaic Stage**

The river in this stage is near its outfall into the sea. The river fans out in many tributaries and is subject to spills. Velocity is low. Slope is generally very flat from 1 in 5000 to 1 in 10,000. On account of the low velocity the river bed is generally silting and rising. The water surface is also rising, eventually spilling over and forming new channels. The process of land formations is shown in Fig. 1

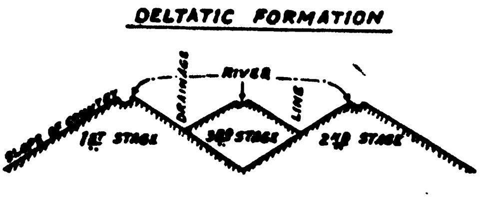


Fig. 1.

The cross section of the country would show the river running at the highest point and the land falling on each side of it at a very gentle slope Fig. 1. In course of time the river would break through one of its banks and assume a new course, when the same process of the gradual rise in its banks would take place. A valley would thus be formed between the two courses, and this itself would eventually be obliterated by the river breaking into it and similarly silting up the area. This action of gradual rise in the country by successive deposits of silt along different courses of the river occurs chiefly in its deltaic portions, but it may also continue higher up when the river breaks through one of its banks and takes an entirely new course. The silts of a very fine, unstable character, and thus the river flows in a channel the bed and sides of which are quite unable to resist any violent attack by the river. Where the river discharge is fairly constant a more or less permanent regime might eventually be established, but this is far from being the case, for during the flood season it may be fifty to one hundred times the minimum winter capacity.

In a maximum flood the width may be 10 to 20 times and the velocity and silt in suspension several times what they are during minimum low water. The great fluctuations in discharge are thus continually changing the conditions, and it is the floods which chiefly affect the regime of a river.

3. Torrents and Streams.

(*a*) The presence of a delta close to its mountain sourse may be held as a feature distinguishing a torrent from a river or stream. The course of a torrent may, like that of a river, be separated into four main divisions, *viz.*, the hill portion debouching in a ravine width, for a short distance below, a deeply cut head ; this rapidly merges into a delta or broad spreading fan of sand and hill debris, tailing as a rule into a swamp or broad marsh formed of the finer clay particles brought down by the torrent. From this marsh small drainages break out in places to unite further down into a defined channel or stream. Fig. 2.

These physical features are marked by one peculiarity, *viz.*, the complete absence of visible water in all the divisions except for a short time during heavy rains. Even the final *nallah* rarely contains any water unless it derives it from some spring unconnected on the surface with the torrent. The floods which pass down these torrents are short lived, though extremely violent, and the slope on the fan generally varies from 25 to 15 feet per mile. Two successive floods rarely or never occupy exactly the same position on the fan for coming down loaded with silt they spread out in different directions, being constantly forced aside by their own deposits. It will be easily understood that the surface

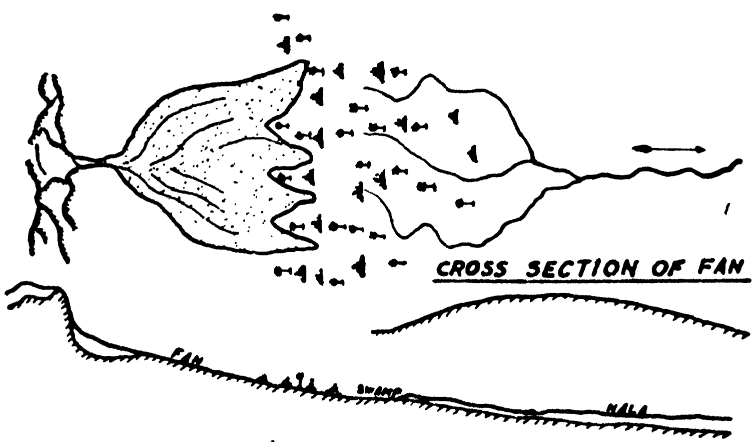


Fig. 2.

of the water in a torrent is not smooth like a river flowing with an established section, but a series of waves one following the other at short intervals. It is now generally admitted that down to a certain limit, the bed in the fan moves with the torrent, and that the surface of the fan after a flood does not by any means represent the actual bed during the flood.

(*b*) **Streams.**

Streams derive their supplies from hill springs, sometimes from lakes which are simply reservoirs filled by rainfall in the last case the supply must be considered as extremely precarious. In the passage from the foot of the hill to the low lands, a considerable proportion of the supply is absorbed in the soil and many of the smaller streams entirely disappear a short distance below their sources. In the larger streams the water thus lost by percolation probably reappears to a certain extent lower down a channel, but the water from the small streams simply goes to feed the great sub-soil reservoir. Canal can be made most profitable from small hill streams and springs with their headworks in the gorges, but the channels have either to be lined with masonry to prevent excessive loss from percolation and erosion of the bed from the great velocity engendered by the high slopes or to be given comparatively flat slopes and numerous falls, including deposits which help to minimise the loss from percolation. Of spring streams there are two distinct varieties *viz.*, the small river with distinctly marked channels emanating from hills, dry or nearly so near the hills, but gaining steady increments to its supply, as it passes through the heavy clay lands below, and the local stream rising from a swamp or nest of dry channels, which gradually accumulates more and more water in its downward course. A river of the first variety has generally a tortuous course, a sandy bed and is subject to severe floods. Its channel sometimes occupies a local watershed, and may indeed be said to be slightly deltaic in its nature, and it differs from the large rivers in not possessing a valley—this renders the floods more violent in their results and causes them frequently to spread over the surface of the country and to eventully find their way into the adjoining rivers on either side, thereby often cutting up the doabs by cross drainages.

4. Molloy's River Regime Theory.

Fig. 3, an plan and longitudinal section illustrates the theory of R. A. Molloy, late Executive Engineer, Punjab Irrigation Department published in Technical Paper No: 118 of Government of India, Central Printing Office, Simla.

(*a*) Taking the plan first it will be seen that the river bends alternately from one bank to another. At the crest of each bend is deep water and in the lengths where river changes over from one side to the other, shallow crossings or bars are formed by deposition of silt in the bed.

(*b*) The bars are produced by the action of the side channels. The side channel has the "off-take" and the outfall as shown in the plan. The subtraction of discharge into side channel at the off-takes results in the reduction of velocity in the main stream which results in the deposit of silt forming 'bars.'

(*c*) Between the side channel and the main river are "islands." These islands are formed when flood subsides and the river divides itself into main and side channels.

(*d*) **Levels of the river.**

The longitudinal section shows the conditions of surface levels. Many gradients are shown by dotted lines and actual ones are shown by firm lines.

(*i*) Taking low water L. Section, it will be seen that owing to obstruction offered by 'bars' actual gradient compared to the mean raised above the bars by an amount indicated by "e" and is depressed by "d" to gain sufficient velocity to pass the bars.

(*ii*) In floods, the conditions shown in the top L. Section are brought about when flood is rising, the bars are silting up and the discharge of the side channel at the off-takes tends to increase. This tendency is resisted in two ways as the floods continue rising.

(*a*) Rise of + k and drop of—h at the off-takes resulting in reduced gradient of the side channel.

(*b*) Back-flow at the outfall.

This process will continue till the flood covering the full section is reached and the conditions sketched in the L. Section Fig. 3 are brought about.

MOLLOY'S RIVER REGIME THEORY

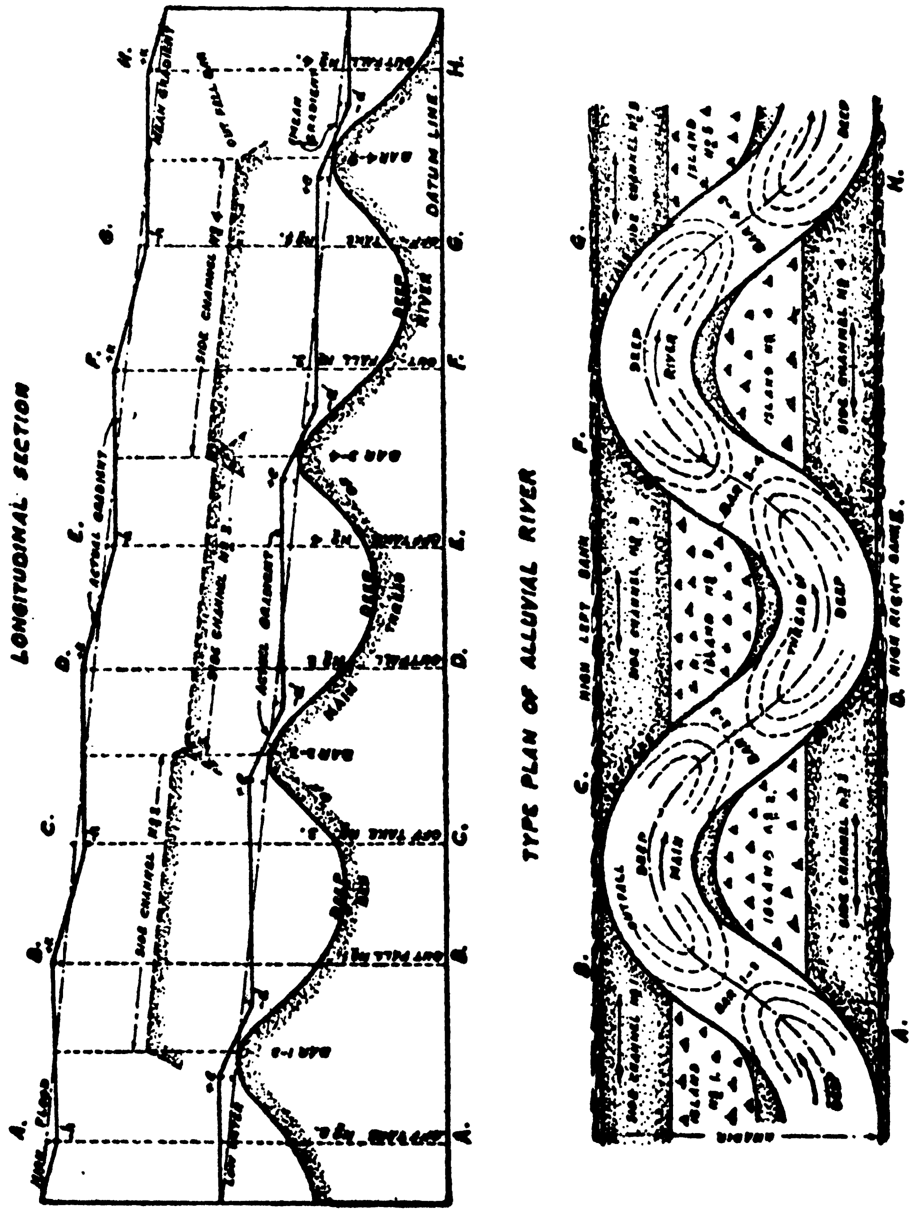


Fig. 3.

(*c*) **Definitions.**

$-h$ = the lowering of surface of the main stream due to discharge having been drawn off by a side channel.

$+k$ = the corresponding rise in surface level at the outfall of a side channel where it rejoins main stream.

$+e$=the heading up above the off-take (and the adjoining bar below it) of a side channel the discharge of which is reduced or stopped altogether.

$-d$=the lowering of water surface at the outfall of a side channel which becomes dry or is closed.

$-l$=the general lowering in levels of the surface and bed caused by a cut-off.

zero=the condition between a general lowering and a general raising.

$+l$=the general raising of level of the surface and the bed due to an incearse of the tortuosity of the river.

General remarks on Molloy's Theory.

Molloy in his theory explained all variations in the behaviour of rivers by change in discharge, but there may be other factors such as change of curvature and change in gradients also contributing to changes in river regime. He states in para 7 Paper 118, Government of India.

"Such inequalities in the range of rise and fall are only to be accounted for by great variation and disturbance of momentum. Usually, the implied postulate of all river formulae is absolutely uniform in condition, section, gradient and velocity. But the actual characteristic of a reach of such a river (an alluvial one) is irregular alterations of all the conditions and consequent changes of momentum. The term channel does not apply to such a river ; it is rather a series of pools, or compartments, of which the sides are the high-level islands, and the bar crossings are moveable, transverse panels very difficult to move."

This description of river by an Engineer who studied the rivers as a life time problems clearly shows that rivers are not channels like the irrigation channels as Mr. Lacey considers them in his theory of channels in alluvium as described in Chapter VII of this part.

5. Other River Regime Theories.

(*a*) **Oldham's River Regime Theory.**

Mr. Oldham following the theory of M. Dausse, French Engineer, enunciates his theory by four principles.

1. Every stream tends to a condition of equilibrium in which the velocity developed is just sufficient to enable the stream to transport its solid burden. If the velocity is in excess, the stream will cut down its bed thus reducing its gradient, the velocity and the silt transporting power. If the velocity is in deficit, the silt will be deposited, thus reducing the section, and increasing the velocity required to transport silt.

2. Every stream is alternately collected into a single well-defined channel called a reach or spread forming a fan, Fig 4. These fans are seen in river bed when it dries up in winter.

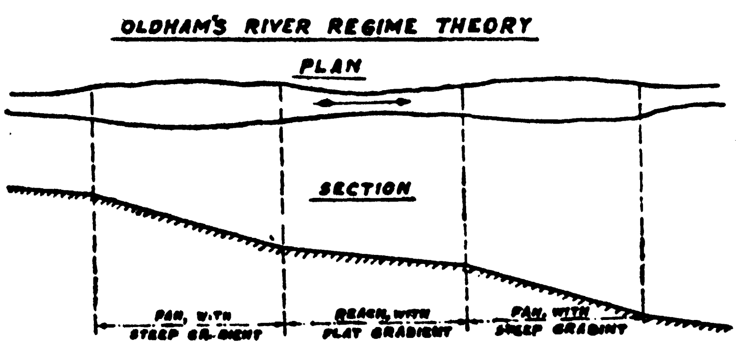


Fig. 4.

3. The gradient of a stream is not a uniform one. The bed will be flatter in the "reaches" and will be steeper in fans relative to the mean gradient. Fans are shallow and thus with reducing velocity.

4. Both reach and fan work gradually upstream. Reach encroaches the upper fan by erosion and is encroached upon by the lower one by silting.

(*b*) Mr. Ellis Tributary Theory (Vol. CXII, The Engineer, December 15,1911) outlines that a river bends to the infalls of the tributaries owing to its bank being there blank, and thus weak, and the tributary forming a channel on that side. To bring the river under control he says some points should be firmly fixed, and no tributary streams should be allowed to enter nor irrigation canals to take-off from the river except at fixed points. He observes that the conditions usually obtained are :—

(*i*) No stream coming in on either side—a straight, well defined channel.

(*ii*) Streams coming in on one side—a well defined channel on the other side only.

(*iii*) Streams coming in on both sides an ill-defined channel and a disposition to form a shoal in midstream.

(*c*) Another theory is that rivers running north or south attack their west bank owing to the rotation of the earth bringing those banks against the current. This assumes that the water of the rivers does not acquire the same rotatory motion as does the lands, which assumption does not appear tenable. The destruction of Dera Ghazi Khan on the west bank of the Indus was said by some to be due to this cause; Mr. Molloy held with much greater reason that the westerly trend of that river there was owing to its deflection by small reclamation banks on the eastern side which were maintained year after year in the interests of cultivation.

6. Meandering and Avulsions of Rivers.

(*a*) An extract from River Training and Control by Sir Francis J. E. Spring C. I. E. Technical Paper No. 153 of 1923 of Government of India is given below which describes aptly the process of meandering of large rivers of the Indo-Gangetic plains.

"The manner in which rivers of Northern India find it necessary to meander more and more nearer they get towards the sea—in other words the lighter and less coherent becomes the sand composing their beds, may be illustrated by a rough measurement of the Indus between Kalabagh, where it first becomes alluvial and the sea. In the following table, column (A) gives rough measurements along the centre line of the river, while column B gives more accurate measurements round the bends. Both columns have been scaled off a 32 miles to an inch map and doubtless if column B had been scalled off a one mile to an inch map the proportionate increase of length due to the river's windings would have been considerably greater than shown here :—

Successive 100 mile lengths.	A Length measured fairly direct	B Length measured round bends.	Percentage of meandering.
1st 100 miles, beginning at the sea	72	100	39
2nd do	75	100	33
3rd do	72	100	39
4th do	69	100	45
5th do	82	100	22
6th do	82	100	22
7th do	93	100	7
8th do	98	100	2
9th 100 miles to near Kalabagh	97	100	3

The meandering of the Ganges are less regular, as may be seen from the following statement, which, however, as showing irregular distances, is perhaps not very convincing :—

Successive Sections.			A Length measured fairly direct.	B Length measured round bends.	Percentage of meandering.
From the sea	to	Sara	185	200	8
,, Sara	,,	Benares	425	540	27
,, Benares	,,	Allahabad	85	130	51
,, Allahabad	,,	Cawnpore	110	135	23
,, Cawnpore	,,	Rajghat	175	195	11
,, Rajghat	,,	Garhmuktesar	37	42	14
,, Garhmuktesar	,,	Balawali	65	73	12

If lines be drawn connecting the extreme limits of the bends of such rivers, it will be seen that while the meanderings of some of them do not extend outside the breath of one mile, those of others, *e. g.*, the Indus at 700 and the Ganges at 200 miles from the sea, extend to a breadth of 10 or 12 miles. Outside these limits are slightly higher lands, but still for the most parts alluvial, into which the river has not as a fact cut. But the fact of the river not having cut further to right and left may generally be ascribed less to the material of the untouched land being of a more holding nature than in the river valley, than to the fact that for each part of each river there exists some relationship between cohesive properties of soil and velocity as qualified by cross sectional area, such that no more than a certain extra percentage of length of channel is needed by the river to prevent its cutting still further into the up lands.

But even when such rivers as are being described have meandered as for as they ever will within the limits of the shallow valleys which they have scooped out for themselves in the alluvium, they are no nearer than ever to the attainment of a state of permanency or equilibrium within those limits. Indeed in the course of years, there is scarcely an acre within the valley limits which will not sooner or later be eroded quite away and in turn reformed. Action of this sort goes on for the most part during the flood times, and much of it beneath the surface of the slit-laden water. What happens is somewhat as follows :—When the flood begins to rise with the melting of the Himalayan snows in June, it finds (*a*) ready prepared for it since the previous year, a more or less deep channel. As it rises higher (*b*) it spreads itself over the low sandy spits and fills short-cut channels. At its half flood stage (*c*) it finds itself topping extensive areas covered with heavy reeds, grass and brushwood. Later again (*d*) if it should happen to rise somewhat higher than that in recent years, it tops cultivated areas and drives the inhabitants of the more or less temporary villages, to whom the cultivation is due, to take shelter on trees, boats or house tops. This last state of things (*d*) is seldom of more than a few days duration, but the conditions described at (*b*) may be of six months and the intermediate stage (*c*) of perhaps three or four weeks duration.

Now, owing to the comparatively brief time during which the flood tops the higher levels where cultivation is being carried on, it seldom has time to do any surface erosion at such places, on the contrary, the tendency is to raise such places by the deposition of silt, owing to the great reduction of velocity due to the lessened depth of water and the restraining influence of the vegetation. Moreover, a skin of vegetation, though of no avail against the edge, caving or erosion, is a very effective preventive of surface erosion. Therefore so far as mere superficial action is concerned, the tendency is for the higher places, below highest flood level to grow higher still to the great profit of the cultivators, to whom a few days of exposure and semi-starvation are of slight importance in comparison with the increased fertility of their lately flooded fields.

But, flowing as it does over a great breadth of the valley, swiftly in the deeper parts where last season's channel is, and more slowly in the shallower parts, the river finds itself taking shallow short-cuts across great bends, at a comparatively low velocity owing to the shallowness, and then cataracting down into the main stream at a very high local velocity at the down-stream end of each short-cut. It often happens, that if the bend is a very big one, the short-cut channel has not time enough to erode its bed and banks to any great extent, and in this case on the fall of the river, the results of the action will be exhibited in the form of an abortive channel, which, had the flood only lasted a few days or hours longer, might have established itself right across the bend. But should the flood stand up long enough for the short-cut channel to erode itself adequately, we find the following state of things, *viz.*, (a) the main river running round a great curve, say 10 miles long, with velocity due to a fall, say, one foot per mile measured round the bend, and with a favourable stream cross section; and (b) a comparatively small side shoot with the same fall, say 10 feet, in perhaps 3 miles instead of ten, but handicapped by its comparatively unfavourable cross section. If the net result of the favourable fall and unfavourable section should be that the velocity in the narrow short-cut is effective in cutting its way so as to rapidly widen and deepen the small side channel or chute, with the consequence that, in a few days or hours - the author has seen such a change occuring on a very large scale in 24 hours—the small side channel will itself have constituted the main river, widening itself by caving, and the long bend will have got silted up. A summary of Mr. R. A. Molloy's account of the causes of cut-offs, in rivers of the Indus class, has already been given in this chapter.

Now this short-cutting action is constantly going on at one or the other part of the river. Sometimes it fails and sometimes it succeeds but whenever it succeeds its effects on the river for many miles up and down-stream may easily be imagined. For miles above and below the recently established short-cut, there follows a period of utter lawlessness ; banks caving, channel silting and new channels forming, until at length, probably with the advent of the cold weather low water conditions there is a truce for a few months. The next rise of flood finds things still a bit chaotic, and velocity in places quite in excess of what the soil can stand up to ; and the result is perhaps two or three more months of lawlessness, untill at length, by increasing its tortuosity, the stretch of 20 to 30 miles of river effected by the one short-cut settles down into a state are armed neutrality readily to exite itself afresh as badly as ever, when the next short-cut happens to succeed in establishing itseslf. The action which the author has attempted to describe is that which Northern Indian river Engineers have been in the habit of calling avulsion ; but the author prefers to use the term CUT-OFF, as expressing better than avulsion the action in question."

7. Extent of meandering of Rivers.

An idea of the extent of meandering of the alluvial rivers can be had from tables in the last paragraph. An attempt has been made in Publication No. 4 (1939-40) of the Central Irrigation and Hydro-dynamic Research Station Poona, India, to work out the formulae connecting the dimensions in a meander belt of a river as sketched below in Fig. 5.

W=width of river.

Q=Actual discharge of the river.

M_b=Meander belt.

M_l=Distance between successive meaders on the axis of the river.

R=Radius of the meandering land.

The incised river data gave
$W=2{\cdot}48\ Q^{1/2}$

Compared with the Lacey relationship
$P=2{\cdot}67\ Q^{1/2}$ for regime channels.

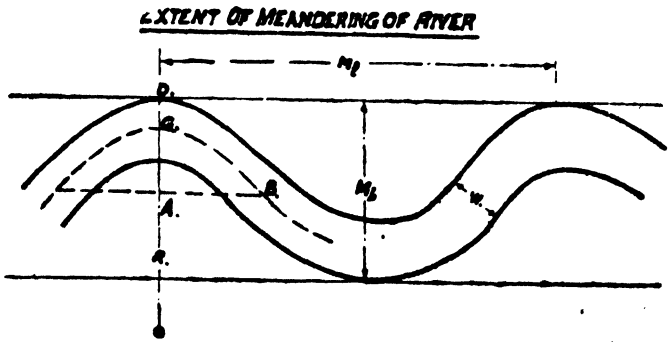


Fig. 5.

The data appears to comfirm that M_b, M_l and W (*i.e.*, P. wetted perimeter) all very roughly as $Q^{\frac{1}{2}}$ (in other words, are scalar) and that the meander lenght is of the order of 10 P or roughly 25 $Q^{\frac{1}{2}}$ to 30 $Q^{\frac{1}{2}}$

Radius of curvature was worked out from the same data as shown below :—

$AB=M_l/4$; $AC=AD—CD=M_b/2—W/2$ and $AB^2=AC\ (2R—AC)$

$$\therefore\ \left(\frac{M_l}{4}\right)^2=\left(\frac{M_b-W}{2}\right)2R-\left(\frac{M_b-W}{2}\right)$$

(1) For incised rivers : (High flood section cut below N.S. and not liable to overflows)
$M_l=25{\cdot}4\sqrt{Q}$; $M_b=56{\cdot}4\sqrt{Q}$; and $W=2{\cdot}48\sqrt{Q}$;
from which $R=14{\cdot}23\sqrt{Q}$; say $14\sqrt{Q}$.

(2) For rivers in flood in plains : (River likely to flood the country on both sides in

high floods.)

$M_l = 29{\cdot}6\sqrt{Q}$; $M_b = 84{\cdot}7\sqrt{Q}$; and $W = 4{\cdot}88\sqrt{Q}$;
from which $R = 20{\cdot}64\sqrt{Q}$; say $20\sqrt{Q}$.

8. River Erosion.

Erosion is defined to be the destructive effect caused by the river's lateral action on riparian lands and works, while scour is that due to the river's vertical action on its bed.

Erosion may be said to be of three kinds :—

(*i*) **Normal.** This is caused by straight, unobstructed currents parallel to the river's bank. Such erosion proceeds gently and regularly, and may deepen the river channel next the bank from 30 to 40 feet below low water level.

(*ii*) **Abnormal.**—It is caused by straight-flowing currents impinging at an angle against the river bank. Such erosion produces caving, or embayment, of the bank, and may be rapid ; it may deepen the river channel next the bank from 60 to 100 feet below low water level.

(*iii*) **Extraordinary.**—This is caused by rotatory swirls set up where the onflowing currents are bounded by dead or non-flowing water. Such erosion is extremly destructive locally, and may deepen the river channel where it occurs from 80 to 120 feet below low water.

The principal factors on which erosion depends are ; the velocity of the river, the duration of its flow, the nature of the material of the bed and banks, the angle which the current makes with the bank and the curvature and general alignment of the bank.

Erosion is due to the local lowering of the surface and the bed. A rise of surface level often reduces erosion, and when this is combined with the reduction of velocity, erosion must stop.

9. River Scour.

Scour is defined to be the destructive effect due to the river's vertical action deepening its bed. Scour may follow an erosion, as a deep channel will be formed at the foot of a caving bank. Scour depends chiefly upon velocity but also greatly on the amount of silt carried by the river, if that is in excess, there will be silting ; if that is in deficit, there will be scour ; and if there is only just the amount of matter suspended in the water which its velocity will enable it to carry, there will be neither raising nor lowering of the bed. Thus scour is worst during a falling river, because the water then contains a small amount of silt, during a rising river the water is charged with silt, and this tends to fill up the hollows in the bed.

Deep channels are likely to become deeper owing to the concentration of flow down them, especially when the water is not fully charged with silt.

When a flood is at its maximum it has already deposited some of its silt upstream and will therefore tend to scour ; thus a river may be deepend, although both the flood level and the velocity have decreased.

An unerodable covering protects the bed from scour, thus a ruined masonry structure dumped stones and refuse, or matted vegetation, will tend to prevent scour under them.

10. River Swirls.

A swirl in a channel is formed by a forward moving current by the side of still water which thus gives the flow a circular motion. An obstruction to the flow in river, such as an artifical spur, will produce dead water and a fast river stream passing near it shall cause formation of swirls both near the upstream and the downstream ends. The obstruction may be in the form of an old masonry bar in the bed, or stem of a tree sticking in bed and causing dead water and the consequent swirls :—

(*i*) The reason why swirls cause deep scour is that they throw off to their outer circumference the silt that was supended in the water or scoured from the bed and fresh silt can enter the swirl area from caving banks.

(ii) Swirls have dead water at their centres which may cause silt to deposit there when they occur near a caving bank, silt falls in on the land side, thus raising the bed there; the deepest water is always on the outside of a swirl.

(iii) The destructiveness of a swirl depends upon the velocity of the current, the area of the still water, the size and nature of the sand of the river bed and the duration of the flow.

(iv) Swirls are the most destructive form of erosion, and protective works should therefore be designed so as to avoid them and certainly not to produce them.

11. Silt in River Water.

There is a vast difference between the sands of various rivers and even between the sands of various localities on the same river. People speak of the sand of the river bed as if all river sands were alike, whereas as a fact, there is as much difference between sand and sand, as there is between, say, pumpkin and a potato, or between road metal and powdered window glass. The shape of the grains of sand has a good deal more to do with their transportability than their size or even their specific gravity.

In the boulder stage of river, the rolling material near the bed may be of size 6" to 12" diameter, and the suspended substance from ⅛" diameter to the road metal size. The suspended substance called *bajri* or grit does actually travel always above the bed like the silt of the canals, or the suspended sand in the trough stage of the river, but it strikes against the bed at some angle and is again shot up for some distance.

Further down along the great rivers of Northern India the rolling substance is grit or *bajri* and the suspended substance is the coarsest sand. In the plains in the trough stage, the rolling substance is coarse sand while the suspended substance it silt and clay.

It is only in the deltaic stage of a river that the bed material and the suspended substance are of the same nature and consist of silt and clay. It is only in this stage that a river could be said to be flowing in self silted aluvium but in this stage it has no definite channel. It is always discarding an old course, forming new land and new channels.

It is usuall to classify both the bed silt and the suspended silt by mechanical analysis using standard sieves. The classification goes according to the mean diameter in millimeters of the particles. The central Board of Irrigation, India (C. B. I.) and the American practice is given below : —

Classification of Mean diameter in M. M.

Class of Silt	Central Board of Irrigation India	American
Coloids	......	Below ·001
Clay	0 to 1/256	0·001 to ·005
Very fine silt	1/256 to 1/128	...
Fine silt	1/128 to 1/64	...
Medium silt	1/64 to 1/32	0·005 to ·05 (silt)
Coarse silt	1/32 to 1/16	...
Very fine sand	1/16 to 1/8	0·05 to ·1
Fine sand	1/8 to 1/6	·1 to ·25
Medium fine sand	1/6 to 1/4	...
Madium sand	1/4 to 1/2	·25 to ·5 (sand)
Coarse sand	1/2 to 1	·5 to 1·0
Very coarse sand	1 to 2	1 to 2
Gravel	2 to 4	...

The specific gravity of sand and grit in a river varies very little from 2·63 to 2·74, but it seems to have no bearing on the subject of sand transportation in the river waters. The angle of repose of the bed sand of the Northern India rivers does not vary much. It is about 31° to 37°, and it does not appear to have any law connecting it with the fineness and the coarseness of the sand. There is also very little variation in the pore space of the silt and sand available in rivers.

It is about 40 and 45% of the volume irrespective of the diameter of the substance. However the coarser and heavier the sand, the smaller the amount of scour and the consequent erosion.

12. Vellocities in the Punjab Rivers.

Velocity of a river varies with the gradiant, the nature of the cross sectional area (H. M. D.) and the character of the bed and banks (N, the coefficient of rugosity). The velocity depends more on the depth than on the gradiant. The prevailing velocities of the winter supplies in rivers may be from 0·1 to 5 feet/second and in floods from 15 to 30 feet per second. As greater scour follows increased velocity, the tendency is for the deeper parts of the section to become deeper still. A river has to adjust its velocity to what its bed and banks can stand by changing its section.

The velocity of a falling river is greater than that of a rising river. In the former case the channel is draining away, while in the latter case the flood as it rises has to fill the channel pools and its reservoir area. When the velocity is retarded, silt is dropped, and when the velocity is increased, silt is picked up. A falling river is most destructive in scouring action.

A deep channel scoured out during the flood season may silt up in the succeeding low-water season, as its section will be too great for the reduced velocity then. In the following flood season, the silting will at first reduce the velocity below what it was in the previous one; but if the floods continue sufficiently long and are sufficiently intense, they will scour away the deposite and re-establish the former deep channel. If, however, for any reason the floods are small and last only for a short time, they may not have sufficient power to scour, and the silting up may remain permanently and thus affect the flood regime of the river.

13. Permanent Reclamation or Protection Works.

(*a*) **Reclamation Works.**

The object of reclamation and protection works is three fold:

(*i*) To remove sinuosity of a river to ensure a well defined channel.

(*ii*) To reclaim land subject to floods for cultivation.

(*iii*) To protect valuable lnd and property on the side of a river which is subject to erosive action.

The river main stream wanders from side to side, and therefore the most important principles for reclamations are :

"Never throttle; always close and work with a full head" (Molloy's).

In other words, the riyer should not have its main channel narrowed.

The closing of scour side channel and the prevention of spill over fore shore should be affected by what has been usually called a "reclamation bund". Reclamation bund unlike the Guide Bunds are temporary works for local use. They also require more repairs.

(*b*) **Design and construction of a Reclamation Bund.**

The highest point of a midstream island and one protected by vegetation should be selected for an abutment into which the bund should be well keyed. The bank of the bund should be continued some distance beyond the length to be protected. Where ridges and cross channels are met, cross spurs 300′ to 500′ long should be constructed on the ridges both upstream and downstream as shown in Fig 6.

The cross spurs serve to produce still water especially upstream to induce silting and to protect the main bund. The best material to construct the bund is pure sand and failing that the soil of the locality. The material should be obtained some distance away from the bund on the upstream side.

The axis of the bund should be preferably at right angles to the river and not less than 60° in any case. A good key trench should be dug 6′ to 8′ wide down to spring level. The bank should be raised 4 ft. above the flood level allowing about 2 ft. free board and 1½′ to 2′ afflux head. The section should be raised according to the required strength. If necessary, an escape channel should be constructed.

To protect the bund from wave wash *pilchhi* mattress or grass fascine should be provided. A roll of 6′ diameter will do.

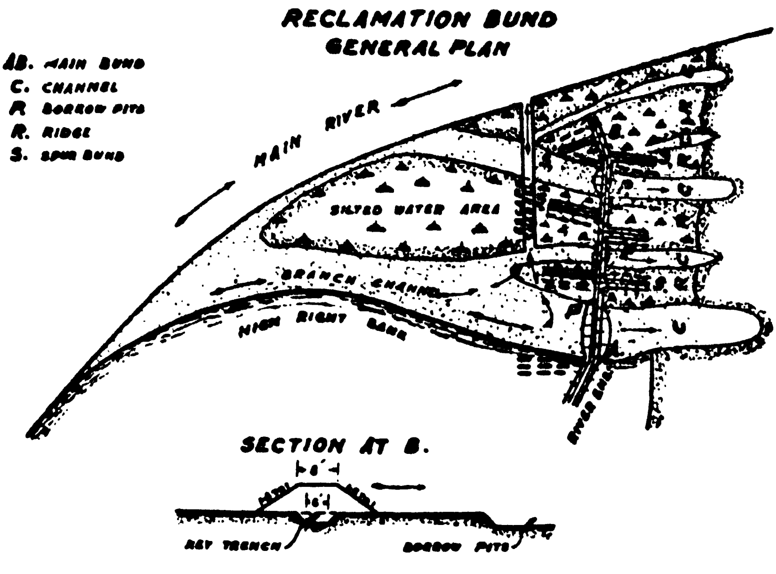


Fig. 6

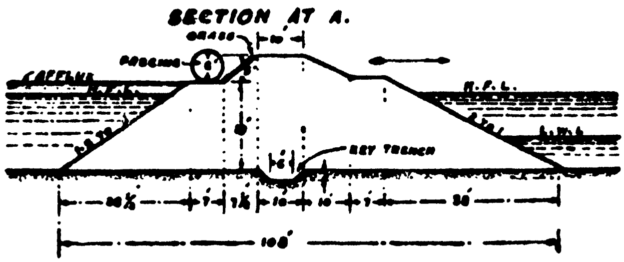


Fig. 6 (a)

The reclamation should be recorded every month and contours plotted to watch effect of Reclamation Bund.

(*c*) **Spurs.**

The earliest form of river control was the construction of spurs projecting from the bank into the current so as to deflect it from the places threatened by the attack.

Where the soil of the bed is firm, this form of protection answers very well. In loose soil there is danger of outflanking.

A spur is essentially a fixed obstruction. It is bound to cause swirls both upstream and downstream near the ends. The swirls will cause deep scour and are very destructive.

The spurs on account of the formation of swirls near their nose are sometimes used to induce a deep river channel on the side provided with spurs at regular intervals.

The protection of banks by spurs is only successful when the extreme most upsteam spur is well-founded in unerodable soil. The spurs yield only temporary relief and it is preferable to resort to the coustruction of guide banks.

(*d*) **Form of Spurs.**

(*i*) An ordinary type of spur consists of branches of trees or jungle and stone in alternate layers so that the spur is pervious as well as capable of sinking unevenly without breaking up. The stone being the permanent part, the more the better.

(*ii*) If stone is not available at the site selected and abutment is formed by digging a trench which is staked and lined with brush wood mats all round. This is filled with bags of earth. The bags are well trodden or founded. The stakes are braced. The construction is extended into the current. The materials are perishable and the life is not long.

(*iii*) Planks are some times driven to form a continuous sheeting, instead of stakes or brush wood.

(*iv*) In Egypt and the Punjab spurs are sometimes made entirely of stone.. Permeable spurs are considered to induce silting up by the slow flow through them. The stone spurs are permeable.

The spurs are usually constructed sloping downstream at an angle of 120°. A spur of length L normal to the bank protects a length of 3L upstream and 4L downstream.

(*e*) **Groynes.**

Spurs of permanent nature are called Groynes. Denehy Groyne as used at Norora weir is a type of permanent spur.

The bed of the spurs is in the form of T, Fig.7 about 400 feet long and is connected to a guidebank, the main flood embankment (Guide Bank) by means of an earthen bund. The Groyne

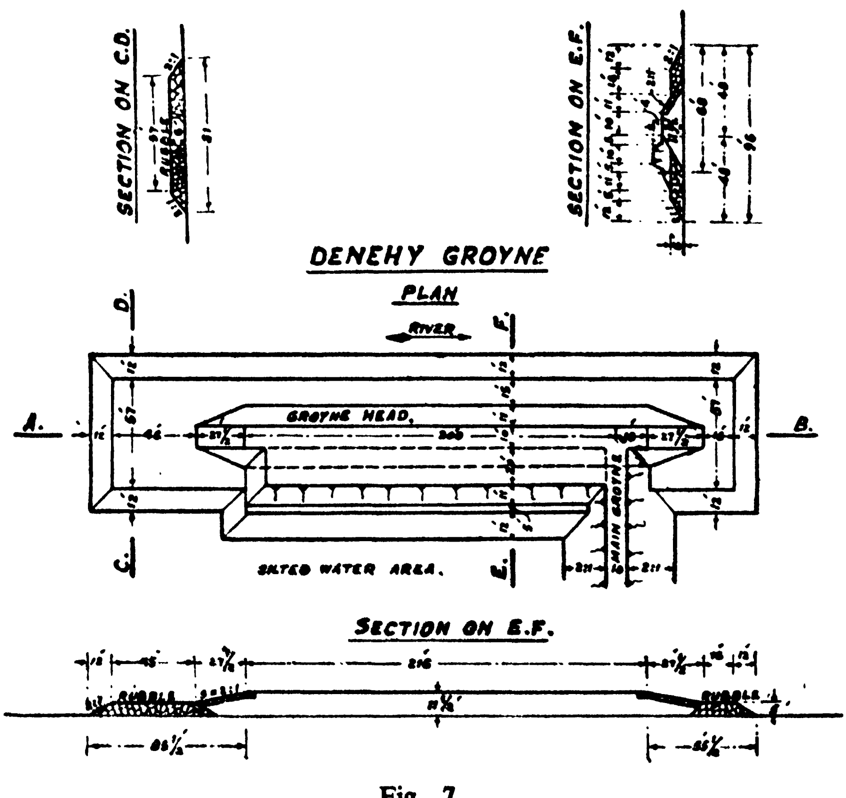


Fig. 7.

is heavily faced with stone and is protected by reserve stone piled on the spur at the T end. The Groyne most upstream should be well protected or finished with an impregnable head.

14 Temporary Reclamation Works

(*a*) **Brownlow's Weeds.**

These consist of series of lines, each fastened to an anchor block upstream in the river bed and supproted downstream by a buoy. Trees and brushwood are attached to the lines, which by deadening the current cause silt to be deposited. A spur of any desired length can be formed by increasing the number of the lines. The success of the weeds depends upon the holding of the anchorages, but when once silting commences, they hold better.

(*b*) The Bengal "*Bandhal*". This is a primitive contrivance for causing minor and side channels in silt carrying rivers to be silted up, so that the main channels may get a concentrated flow down them which will keep them clear and cause them to scour. This results in the reclamation of their foreshores for cultivation. A *bandhal* consists of a line of bamboos upto 50 or 60 feet in length and from 1 to 3 inches in diameter, worked during low water into and across the bed of the channel to be closed and fastened together with a string at the top to form a temporary fence, which is rendered sufficiently impermeable by fixing mat screens to it.

(*c*) **Hurdle Dykes.**

Description of Hurdle Dykes is available in notes on Mississipi river by E. F. Dawson, 1900. The arrangement is shown in Fig. 8.

To prevent the erosion of the river bed from undermining the dyke, a flxible mattress is formed on sloping ways, brought on to a platform carried on special barges, and is lowered on to the bed, weighted with stone. It is manufactured from wattling brush upon poles, spaced about 5 feet apart, and these are lapped together and fastened by spikes and wire to give the required width. The brush wood matteress is spiked to the poles at the edge and elsewhere at intervals to every third pole. The width of the mattress varies from 60 to 120 feet, according to the depth of the water and the liability to scour. The hurdle consists of a row or parallel rows of piles driven through the mattress at about ⅓rd of its width from its upstream edge, either singly or in clumps, each consisting of three or four piles, the tops of which are drawn together by wire rope to form a sort of pyramidal structure, the piles at river bed level being from 8 to 10 feet apart. In the improved from, the curtain, or wattling row, is braced at an angle of about 45° by vertical diagonal braces, which are heeled against a row of clumps and securely fastened at top and bottom to the piles against which they abut. The piles vary in length from 25 to 60 feet, and they are driven into the bed for about 15 feet, with the larger end down, by means of a hydraulic jet and weight of about one ton. They are spaced so as to come to about one pile to the lineal foot of the drift. The tops of the piles are kept about 20 feet above low water, except that of one in each clump of the wattling row, which is 25 feet above that level, in order to catch drift and prevent it from passing the hurdle. The piles of that row are connected along its upstream face by wattling of fine brush or by curtains made of brush; this forms a permeable barrier, through which the slit-laden current passess at a velocity so much reduced that slit is deposited on the river bed upstream and downstream of it. The great advantage is that they can reclaim a fore shore on which permanent reclamation bunds can be designed.

(*d*) **Cross Fixed Spurs.**

They consist of double lines of stakes 3 to 5 feet apart with ropes passing round. Brush wood and twigs are filled between the stakes and are usually loaded with sand bags. They are quite successful in the case of small streams and torrents.

15. Extent of narrowing by construction of Guide Banks.

The great rivers of Northern India, have very high maximum flood discharges. It often happens that in the course of a great river in the plains with surface width in maximum floods from one to 10 miles wide an Engineer is confronted with the problem of constructing a bridge for a road or a weir for a canal. It is not generally possible to find near about the site a rockey gorge section or an increased section in plains. It will not only be expensive to span the whole

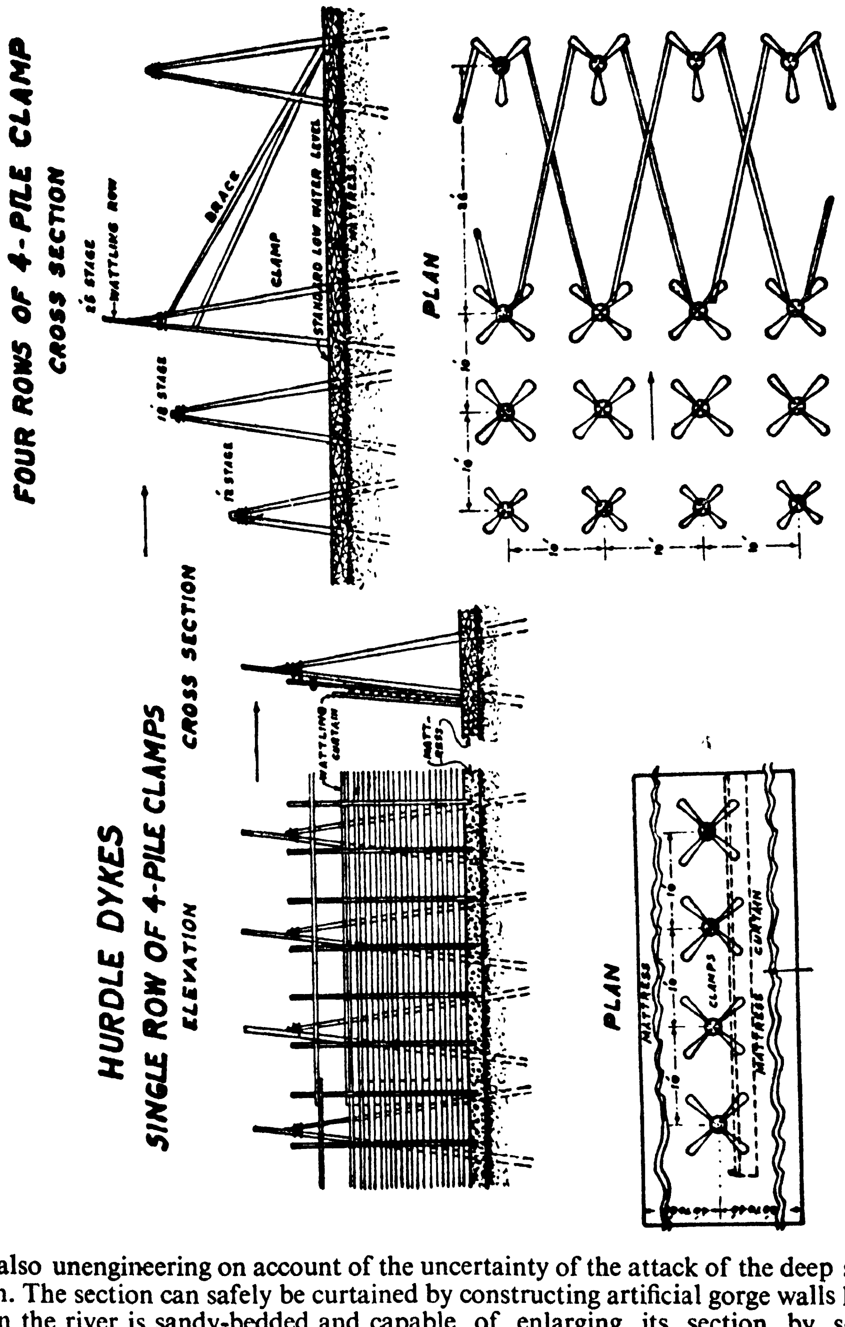


Fig. 8.

width, but also unengineering on account of the uncertainty of the attack of the deep scour in a very wide section. The section can safely be curtained by constructing artificial gorge walls known as guide banks, when the river is sandy-bedded and capable of enlarging its section by scouring its bed without any appreciable afflux.

In the case of bridges such as railway bridges founded on well foundations, the factors governing the extent of narrowing have been summarised by Mr. (late Sir) Spring in 1935 Technical Paper No. 153 on River Training and control as given below :—

"The experienced engineer will without any doubt recognise the wisdome of selecting the place on the river's cross section where the bridge shall be located, so that the area of scouring out of bed section that must be done in the first rise of flood shall be minimum, other things of course being equal. Also he will not fail to observe that, within limits, the greater the narrowing, the deeper may be the scour, and therefore the more expensive will be the stone protection of his

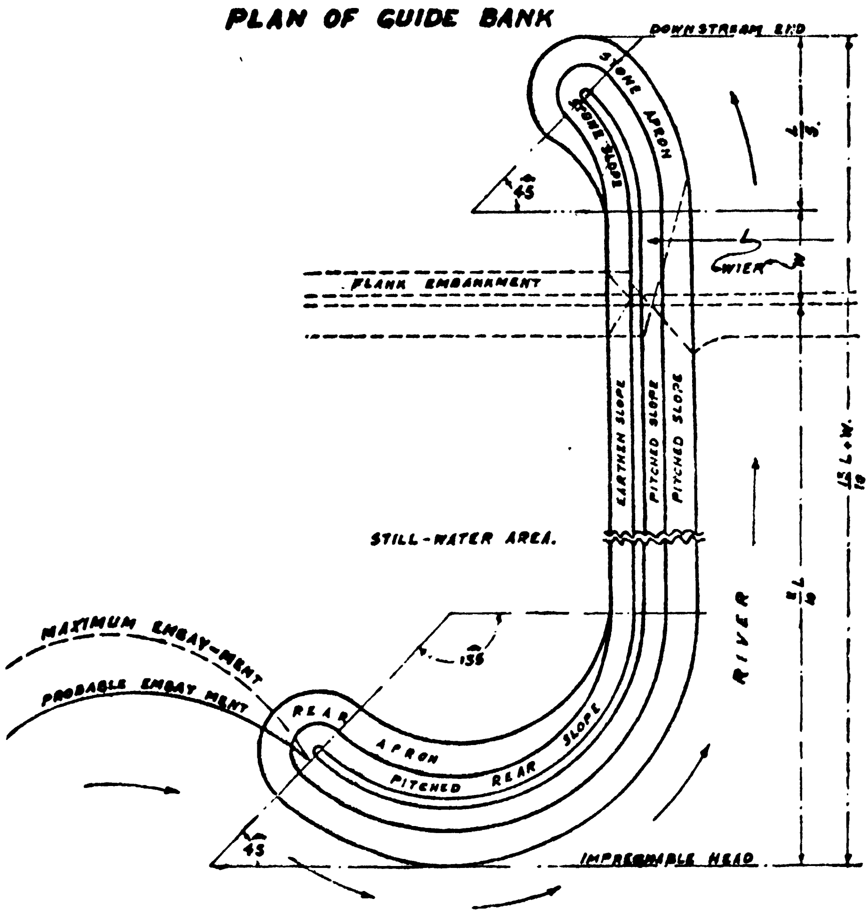


Fig. 9.

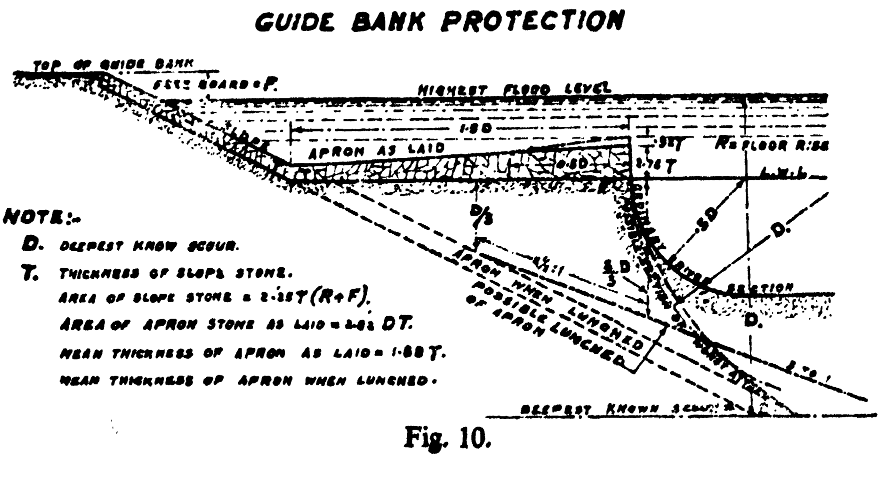


Fig. 10.

guide banks. Thus a practical limit for narrowing is soon reached. The author is inclined to the opinion that in an ordinary sandy bedded river of the class under notice with fall of 18" per mile and less, narrowing may in practice be limited to what will cause an all-over mean scour of from 8 to 16 feet between abutments. The comparison between the area before and the area improved by scour after contraction should be made with regard not to area alone, but to discharge through the old and the new areas, as governed by the velocities got at the varying depths of the successive compartments of their cross sections."

The contraction of the river section is also done in the case of canal weirs by the construction of guide banks, but the problem is much more complicated than that in the case of bridges with open foundations on account of the permissible discharge intensity. the allowable afflux and the required crest level of the weir to maintain a fixed pond level to feed the canals. The mathematics and calculations are given in Chapter XIII of this part to arrive at the permissible contraction at canal weirs.

16. Design of the Guide Banks (Bell's Bunds).

A type plan and cross section of a guide bank is shown in Fig. 9 and 10 respectively.

(*a*) **Layout and alignment of Guide Banks.**

(*i*) Guide banks are constructed in pairs symmetrical in plan.

(*ii*) It is essential that the design should be such that no swirls are produced. The pair of the guide banks should be as smooth as possible. There should be no spurs projecting from the guide bank as the spurs produce swirls.

(*iii*) In order to exercise a gentle entrance for the river between the guide banks, the embankments should be curved inland near the ends. The widening should not be excessive, otherwise it will lead to the formation of islands.

(*iv*) Care should be taken not to select a site for the bund where the river has a tributary or a side channel flowing parallel to the bund. These four fcctors ensure proper alignment of the bund.

(*v*) Sufficient still water area should be provided as in reclamation bund. This can be exercised by providing separate and additional bunds of reclamation type upstream.

(*vi*) The essentials for a guide bank are :—A proper alignment as symmetrical as possible, a sufficient still water area, an ample apron, an impregnable head, and a large reserve of stones for repairs during its early years. Fig. 9 is a typi calplan and Fig. 10 is a typical section of a guide bank. The guide banks are permanent works unlike the reclamation bunds.

(*b*) **Section of guide bank.**

Top of the bank should not be less than 10', and in the railway practice 20' to 30' to allow the working of the material trains. Side slopes should be 2 to 1 and free board 4 to 5 ft. In calculating the height, allowance should be made for heading up, and also for settlement of the banks which may be 10% of the height. The inside slope should be protected with stone pitching, and the outside one by good *pacca* earth; and floating ropes with jungle or mattress may be used to withstand wave wash. The earth for the construction of the bund should always be taken from the apron side and no borrow-pits should be allowed on the back water side as these may induce percolation.

(*c*) **Slope of a guide bank.**

(1) It is essential that the slope should neither be undermined nor slipped. If it fails, a breach would occur as the earthern bank behind cannot resist the river action. Ample stones should be provided in pitching on the slopes and reserve stones should be stacked on the back to replenish the weak points.

(2) Pitching should be atleast 3' higher than High Flood Level.

(3) The pitching stone should be selected having a high Specefic Gravity and weighing from 100 to 160 lbs. In water it looses its weight equal to the wight of the volume of water displaced. The effective weight is, therefore, reduced.

(4) The stone should be laid on 6" deep spawls to protect the bank from wave wash through the interestices in the stones.

(5) As regards thickness of the stone repuired, it would naturally increase with the velocity of the river. Mr. Spring used the following table of thicknesses which apparently includes the layer of spawls :—

Fall per mile of river in inches.	3	9	12	18	24
Sand Classification.	Thickness of stone pitching in inches				
Very coarse	16	19	22	25	28
Coarse	22	25	28	31	34
Medium	28	31	34	37	40
Fine	34	37	40	43	46
Very fine	40	43	46	49	52

(*d*) **Apron of the Guide Bank.**

Apron is the name given to the part of the pitched slope on the river side of the Guide Bank which is laid horizontal at the low water level. In order to make the apron safe and to arrange it efficiently, it is necessary to ascertain the depth of the deepest scour neglecting swirls. The breadth is not to be less than 1½ times the deepest scour, in Fig. 10. The average depth should be 1¼ times in gentle and 1½ times in turbulent river of the slope pitching thickness when launched.

The apron is laid at low water level and it launches by its own weight when the scour starts in floods as shown in Fig. 10. The average slope with a properly laid apron assume, when launched, is 2 to 1.

(*e*) **Impregnable head of the Guide Banks.**

Mr. Spring's design of Impregnable head.

1. The entry should be gentle.
2. It should be curved into reduce embayment.

The proper radius to be allowed depends on the velocity of the river. Mr. Spring gives the following table to design the radii :—

Sand classification	Probable maximum abnormal scour.	Fall per mile ot river in inches				
		3	6	9	12	18
		Radius of upstream Curved End in feet				
Very coarse	Under 20 feet	200	250	300	350	400
,,	Over ,, ,,	250	310	375	440	500
Coarse	Under 30 feet	300	360	425	490	550
,,	Over ,, ,,	350	430	510	590	670
Medium	Under 40 feet	400	425	550	625	700
,,	Over ,, ,,	450	550	650	750	850
Fine	Under 50 feet	500	590	675	760	850
,,	Over ,, ,,	600	725	825	925	1020
Very fine	Under 60 feet	600	700	800	900	1000
,,	Over ,, ,,	800	900	1000	1100	1200

1. The upstream curve should be carried 120° to 140° round the back to secure protection of the inland side.
2. Pitched apron should be carried throughout the curve.
3. The curved portions should be of full height of the bank.
4. The pitching at the impregnable head should be 25% extra.
5. The width of the apron should be double the deepest scour instead of 1½ times.

7. Construction of the guide banks (Bell's Bunds)

The construction of the guide bank should be commenced from the abutment that is the weir, and should proceed regularly upstream. If its completion cannot be achieved in one season, the reduced length commenced should be finished off and should have a temporary impregnable head formed at its upstream end, so that it may hold the river, and not be out flanked by an embayment. In the subsequent season this head should be removed and the guide bank similarly continued or completed. To ensure rapid progress, proper and early supplies of stones should be arranged for, and it will facilitate work if these can be brought to site by trains.

The slope and the apron should be built between templates 100 feet apart. In para 16 (d) the construction of the apron in the dry condition has been described, but this may not always be possible, and part of it may unavoidably have to be formed in deep water. When this is the case, the quantity of stone required for the subaqueous part of the apron should be calculated : this amount should be carefully and regularly dropped into the water from boats and precautions should be taken at first not to extend the apron too far lest it be undermined and the material wasted.

In the most difficult case, a length of the entire guide bank, including the embankment may have to be formed across a deep-water area. If possible, that area should be temporarily reclaimed by silting it up, but if this is nct feasible, the arpon should be formed in advance and gradually built up above the water level, after which the superstructure can be constructed in still water but this involves a very large expenditure of stone.

The cost of guide banks for a Railway Bridge has varied from Rs. 75 to Rs. 180 per foot run. Those for the Sara bridge across the lower Ganges, where the depth was excessive and the sand of the river very fine, were estimated to cost Rs. 475 (£32) per foot run. The cost depends greatly on the price of the stone, and that is due principally to the length of its lead and nature of its haulage. The extent of the sectional area of the protection also affects the cost.

18. (a) **Retired Embankments.**

The conditions of unembanked rivers are unsuited to the cultivation of the adjoining culturable lands on account of the fact that the areas remain under water throughout the flood season *i.e.*, July to September. The river Jhelum floods the lands on the left bank from Rasul to Shahpur upto a distance of 30 miles in high floods exceeding 6 lac cusecs, the maximum being about 9 lacs. This is a typical case were protective embankments would go a great length to save the residential property and the destruction of crops in floods. But a canalised river with closely constructed embankments is prejudicial to the certain reparian rights of the population who derive relatively greater benefits, as compared with the loss in floods, from the bumper crops in the spilled areas for a couple of years following the floods. A system of retired embankments provides a solution of the problem. The embankments should enclose the area which has no other means of irrigation except the river spills and cut off the area which is ensured of irrigation from canals or wells.

(*b*) **Advantages of the retirement of embankments.**

(*i*) The minimum interference with the natural operation of the river in raising the country by silt deposit.

(*ii*) The maintenance of a large river reservoir capacity.

(*iii*) The filling of this reservoir reduces the 'peak' of the high flood.

(*iv*) The emptying of this reservoir will maintain the river at good irrigating level for a longer time.

(*v*) It may be practicable to utilise such a reservoir so as to give temporary storage of weter for canals when the river level falls.

(*vi*) The reduction of the high-flood level will preserve the regime of the river, and thus it will diminish the erosion of good land and the throwing up of sterile sand banks.

(*vii*) The longer life of the embankments will permit of their being constructed carefully and slowly, well in advance of the period when the river will approach them, and thus will give time for their self-consolidation by settlement.

(*c*) **Disadvantages of the retirementof embankments.**

(*i*) **Increased cost.**—Against this must be set the saving of replacement of embankments by retired loops and the loss of crops on eroded and inundated areas.

(*ii*) **Increased length of open canal head.** As the embankment is retired from the river, so will the length of the open canal head between the two be increased. Not being protected from floods, it is particularly exposed to silting.

(*iii*) **Reduction of the spilled area.** Generally it will be possible to grow on the riveraine area post-inundation crops, such as wheat, instead of rice, and where this is not practicable, forests might be planted.

(*iv*) They interfere with the river's natural operations in raising the country generally.

(*v*) They deprive the protected land of much fertilising silt.

RIVER TRAINING WORKS AT KHANKI HEAD-WORKS

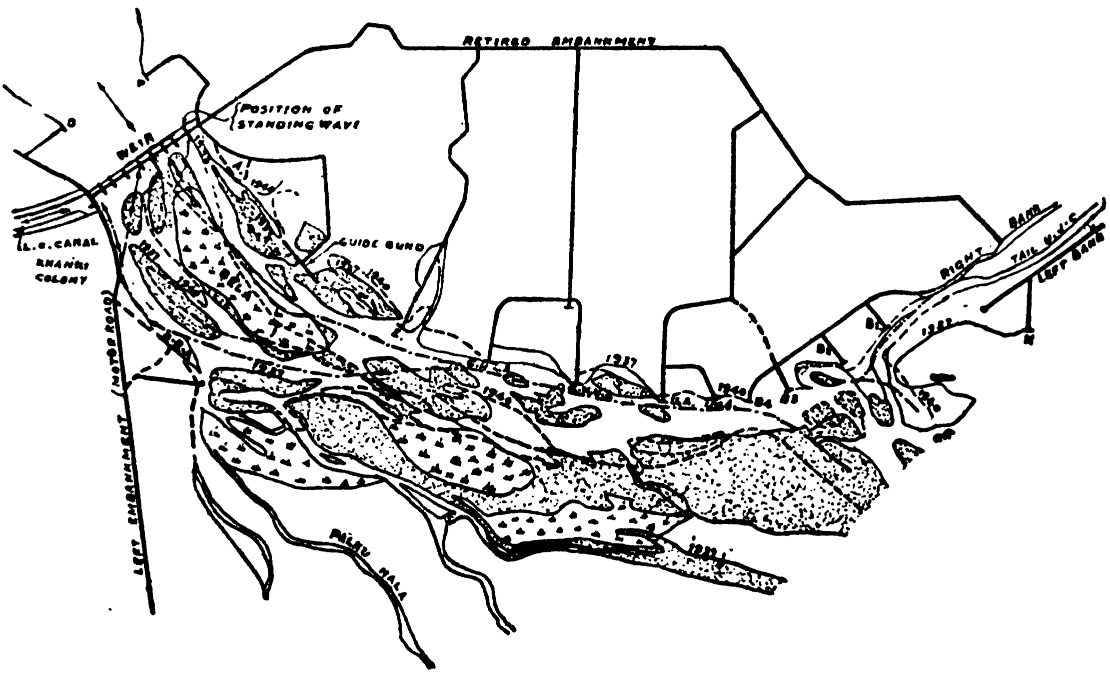


Fig. 11.

(*vi*) By shutting off inundation water they cause the protected land to deteriorate by the rise of subsoil salts.

(*vii*) They raise the river to high flood level.

(*viii*) They cause the river, by changing its course, to erode its cultivable banks and to throw up sterile sand banks.

Of these disadvantages Nos : (*iv*), (*v*) and (*vi*) refer to damage to surface conditions *i.e.*, cultivation etc., and Nos : (*vii*) and (*viii*) affect the natural regime of the river.

(*d*) The advantages of the retirement outweigh the disadvantages, and indicate the desirability of constructing embankments well away from the edge of the river. By adding reclamation bunds to the retired embankments, the latter can be made safer by the silting thus induced, which will tend to produce along them foreshores free from scour channels and more suitable for post-inundation cultivation or forest plantations. If work is carried out on a proper system of reclamation, the embankments can be provided with spurs projecting to the points of the river main channel where it is likely to attack the culturable land. If necessary, the spurs should be ended with Denehy's Groynes.

19. Marginal Bunds and Spurs.

(*a*) The marginal bunds are aligned and, designed like the retired embankment upstream of a canal weir on both sides of the river. Their chief object is to protect the countryside from the river spills which will be caused by the afflux head of 3′ to 5′, usually provided in case of canal weirs and to guard against the outflanking of the weirs. These works are required in addition to the usual guide banks or Bell's Bunds in case of the canal weirs, unlike the road and railway bridges where river bed is capable of being eroded to enlarge the section without any afflux head.

The marginal bunds are usually provided with earthen spurs protected at their ends like the Denehy's Groyne out to the main stream of the river to protect the land enclosed by the marginal bunds and to have a defined and straight flood discharge section upstream of the weirs. Arrangement of the marginal bunds and spurs as provided at Khanki Head works of the Lower Chanab Canal are shown in Fig. 11.

The marginal bunds have a free board of at least 3·0 feet in addition to the afflux head provided in the weir design above the highest flood level. They are projected upstream of weirs upto lengths where the top levels thus calculated coincides with the ground level.

(*b*) **Spurs.** In India, and especially in the Punjab, besides guide banks several other guide works have been constructed in the rivers above barrages. The training works chiefly consist of armoured spurs at suitable points from the river banks or marginal bunds. The types of armoured spurs adopted in the past are : –

1. Hockey spurs.
2. Inverted Hockey spurs.
3. Point spurs.
4. "T" Head spurs.

The type found to be most satisfactory is the "T" Head spur. The spurs are constructed in stone with heavy aprons. Various arrangements of these spurs have been adopted in the past.

Spurs constructed along one bank of the river. In some cases, spurs have only been constructed along one bank. An example of this is the river Chanab above Khanki Head-works. At this place more than a dozen spurs exist along the right bank.

Spurs constructed on both banks of the River. In other cases, spurs on both banks of the river have been constructed. On the river Ravi above Balloki, armoured spurs exist on both the banks thus tightening the river.

On the old weirs of the Punjab, large natural *belas* exist in front of the central bays of the weir. These *belas* extend sometimes for a length of two to three miles, as at Khanki, Rasul, etc. In these cases, the central bays of the weir are masked and their capacity to escape the flood discharge is reduced. From a persual of the above, it will be seen that methods adopted in the past for training the river were not very satisfactory.

(*c*) **The Pitched Islands.** The effect of the pitched island on river training was examined by means of models by Uppal and Mushtaq Ahmad in 1936. In addition to this other methods of river training *i.e.*, armoured spurs coustructed at different places on both banks of the river and on one bank, extension of the guide banks and the sandy *belas* along the guide banks were examined. Results obtained showed that the conditions obtained with the pitched island were the most satisfactory. In 1943, A.M.R. Montagu again revived the idea of the pitched islands. The construction of a pitched island in the river-bed effects a change in the velocity fluctuation and the tractive force. The tractive force near the island considerably increases and causes scour. It draws slowly the main river channel on to itself and holds the river there. The working of the island is satisfactory, especially in the vicinity of control points *i.e.*, weirs and railway bridges. It has proved very useful in actual practice upstream of the Islam weir.

The use of the pitched islands helps in :—(Paper No. 275. The Punjab Engineering Congress, Lahore.)

(*i*) Training the river central above the weir.

(*ii*) Relieving the intensity of the flow on the marginal bunds, the guide banks and outside of river curves.

(*iii*) Training the river in the reach away from control points.

(*iv*) Protecting the valley land against devastation as well as reclaimating it.

(*v*) Improving channels for navigation.

20. Lay out of river embankments.

(*i*) When any new line of flood embankment is under consideration the inhabitants should always be consulted. An embankment may shut off the floods from land which has hitherto benefitted from them and the people may prefer the old arrangements to the new one. A single *rabi* crop in the year (the *rabi* is generally the more valuable crop) with freedom from canal assessment, may suit them the best. Their villages or homesteads are usually placed on high ground or protected by local ring exembankments. There may thus be a temptation for the people to cut embankment, an extremely easy operation because the men who watch it can be evaded. For the above reasons the location of a flood embankment should be very carefully considered, and there should be no delay in providing means of irrigation. If no alternative means of irrigation are provided, there is a great temptation in floods to the population to cut them to spill their lands for *rabi* irrigation.

(*ii*) A flood embankment should not be so near the river as to be in much danger from erosion, but the ground, as already stated, generally falls, in going away from the river, so that when an embankment is set well back it is in lower ground, more expensive and more liable to breach. The most suitable alignment is a matter of judgement and depends largely on where the main stream of the river is at the moment and whether it seems likly to shift.

If the main stream of the river has lately made an inroad and cut away an embankment but has not shifted its course, great caution is needed in fixing the line of the new embankment. In 1882 there was an erosion on the Indus a few miles north of Dera Ghazi Khan, and a flood embankment was cut away. A new embankment was constructed a mile inland. The erosion recurred and the new embankment was cut away within a few months of its completion. Embankments are, where possible, made in straight or properly curved reaches. A flood embankment, at least at its upstream end, terminates in ground which is above flood level.

(*iii*) The top of an embankment is generally 2 or 3 feet above the High Flood Level of the river. It should, of course, be graded parallel to the general H. F. Level but neither the gradient nor the height of the flood is known with accuracy. There is generally a record or mark of some high flood, and this is taken provisionally as the flood level or the level is calculated approximately from the flood readings on the nearest river gauge. If experience shows that the embankment is too low, it is raised.

(*iv*) Naturally a flood embankment shall be economical if the alignment follows the contours of high land but this will usually set the bund very much retired from the river and shall also increase the length. Often the embankment shall have to be looped inward to protect some village or other valuable property.

(*v*) In the neighbourhood of Dera Ghazi Khan it has long been the custom to have a double line of flood embankments in places where the first line is in any sort of danger, either from river erosion or from ordinary breaching owing to low ground or bad soil. This should be accepted as a principal for all localities in which the breaching or destruction of an embankment will cause widespread damage. Even if erosion is foreseen, it may be very rapid and there may not be time to make a second line. It should be there before hand. Sometimes the lines are connected by cross embankments whose function is to localise damage in the event of a breach occuring in the main embankment. Sometimes there is local embankment the function of which is to protect a local area in case the main embankment is breached higher up, for instance the cantonment embankment. The flood embankments at Dera Ghazi Khan are shown in Fig. 12 which gives the general layout of such works.

21. Cross section of marginal bund or retired embankment.

(*a*) **General requirements.** A river embankment should be raised high enough to prevent its being overtopped by an extreme flood, should have a thoroughly consolidated section sufficient to prevent percolation water from causing it to slip, should have a width of base able to support the superstructure, and to prevent creep of water under it with a key trench where necessar to assist in resisting this. It should have its slopes protected from guttering by rain and from erosion by wind and wave-lap. To reduce the cost and to increase safety the highest land practicable should be selected, but care should be taken not to traverse, if possible, ground which is sandy, friable, much cracked, or impregnated with salts, or intersected by numerous channels, or irregular in longitudinal or cross section, as this will lead to differences in settlement. The neighbourhood of villages should be avoided, as their presence increases trespass by traffic.

When an embankment is likely to be breached by erosion of the river, and a retired alignment is decided on, which it may be necessary to extend hereafter, it will be necessary to join the end of the new bank AB to the old line CD by a crossbank. Care should be taken (Fig. 13) that this cross-bank has a downstream direction BD, and not an upstream one. BC for the latter will form what is known as a 'pocket'. When the old bank is breached and flood water is admitted to the new bank, it will in the latter case pond up at B to the level of the river opposite the cut C, but in the former one the water will rise at B only to the level of the river opposite it.

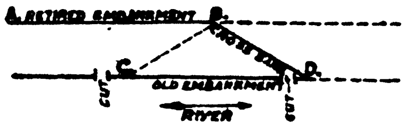


Fig. 13.

(*b*) **Section.**

The proper section depends on the height of the bank, the nature of its material and foundation and the way in which it is to be formed. A typical cross section is given in Fig. 14

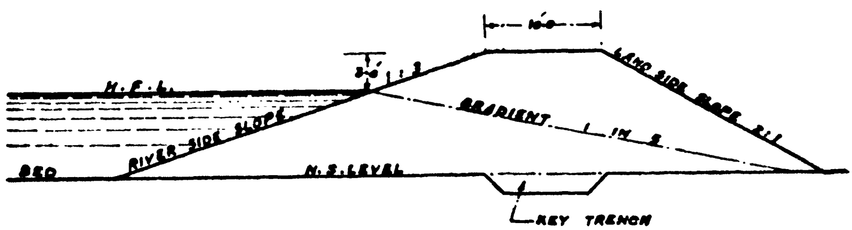


Fig. 14.

If the river water is likely to be on both sides, the bank slopes should be 1 in 3 on both sides. The earth work without clods should be laid in layers of not more than 6″ thickness and consolidated before the next layer is laid. Where the ground is likely to give way under the bank, the base should be widened so as to gain the necessary amount of support. Where wave wash may be expected, the river side slope should be flattened and where the material is unreliable both the slopes should be increased. The top of the bank should be given a slight slope for drainage to the land side, so as to diminish tendency of the river slope to gutter during rainfall, as rain scores will aid wave wash in damaging that slope.

(*c*) **Key Trench.**

Inspection of river banks will show that owing to the absence of pressure and the effects of the weather, a surface soil is generally more porous and fissured than a subsoil of similar material. Moreover when water meets two dissimilar materials, such as made earth and natural soil, it has a tendency to collect at their junction and to endeavour to pass between them. The key trench which is a small middle trench provides the junction which unites the body of the embankment to the subsoil, destroys the continuity of the base plane and secures the thorough examination of the part of the base it occupies, and the removal from it of vegetation, roots and rubbish. A key trench is most important where the ground is porous, sandy or is cracked and traversed by fissures or by water-courses etc. Its section will depend upon the high-flood depth against the embankment, the nature of the sub-soil and the importance of the embankment. Ordinarily its bottom width may vary from 4 to 6 ft. its depth from 3 to 5 feet and its side slopes 1½ to 1 or 1 to 1. A water-tight foundation is essentially necessary for an ordinray hydraulic work.

(*d*) **Slip Trench.**

Wherever there is an abrupt change in the level of the ground, or the top of the embankment under construction, one or more slip trenchers Fig. 15 should be made so as to form junction dowels to prevent the creep of water along the junction plane of the earth works, They may have a bottom width of 3 or 4 feet, a depth of 1½ to 2 feet and side slopes 1 to 1. Their sides and those of the irregularity being dealt with should be sloped so that the embankment during settlement may settle tightly against them. The less the capacity of the soil to withstand percolation, the more numerous and the larger should be the slip of trenches.

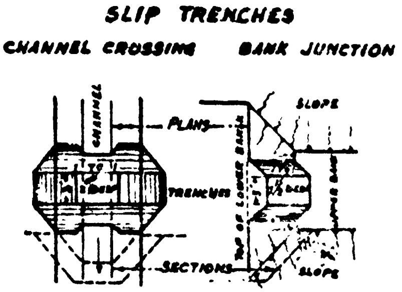


Fig. 15.

(*e*) **Banquette (Berm) or** *Pushta.*

The advantages of berms as shown in Fig. 16 are that they :—

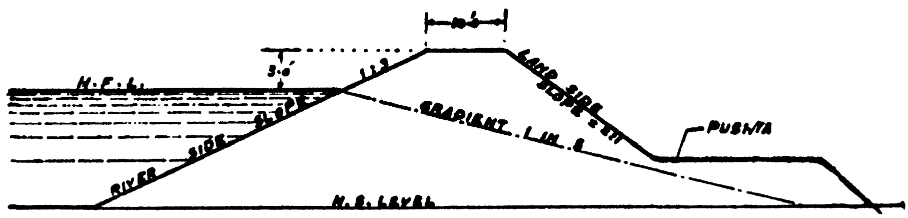


Fig. 16.

(*i*) Cover the hydraulic gradient of the internal percolation line thus render the embankment less liable to slip.

(*ii*) Provide space for a roadway which is better located there than on the top of the embankment.

(*iii*) Provide space for stacking material, or an area from which soil may readily be obtained for the emergent raising of the embankment when necessary during floods.

(*iv*) Furnish a better cross section for subsequent inlargment or raising of the embankment.

22. Examination Questions.

1. Explain with sketches the (1) Denehy's Groyne (2) Bell's Bund. What considerations determine the lengths of both ? (*T.C.E. 1933*)
2. (*a*) Sketch the most suitable type of a *pacca* Groyne.
   (*b*) What points will you consider in designing groyne training works ? (*P.U. 1942*)
3. Describe various types of river training and protection Works. (*P.U. 1942*)
4. (*a*) What are the main features distinguishing a torrent from a river or a stream ?
   (*b*) What are the main division in the course of a river and a torrent ? (*P U. 1941*)
5. Sketch a suitable cross section of a guide bank as used in river training works. Explain the process of launching of aprons in such works ? (*P.U. 1943*)
6. (*a*) Why don't the Railway bridges need a system of protection works embankment in addition to guide banks like the canal head works ?
   (*b*) What points will you keep in view in determining a suitable layout of a system of marginal bunds with spurs ?
7. (*a*) What is the function of retired embankment and what are their advantages and disadvantages ?
   (*b*) Sketch a suitable cross section of an earthen embankment and explain the following :—
   (*i*) Key Trench (*ii*) Slip Trench (*iii*) Berm or *Pushta.*

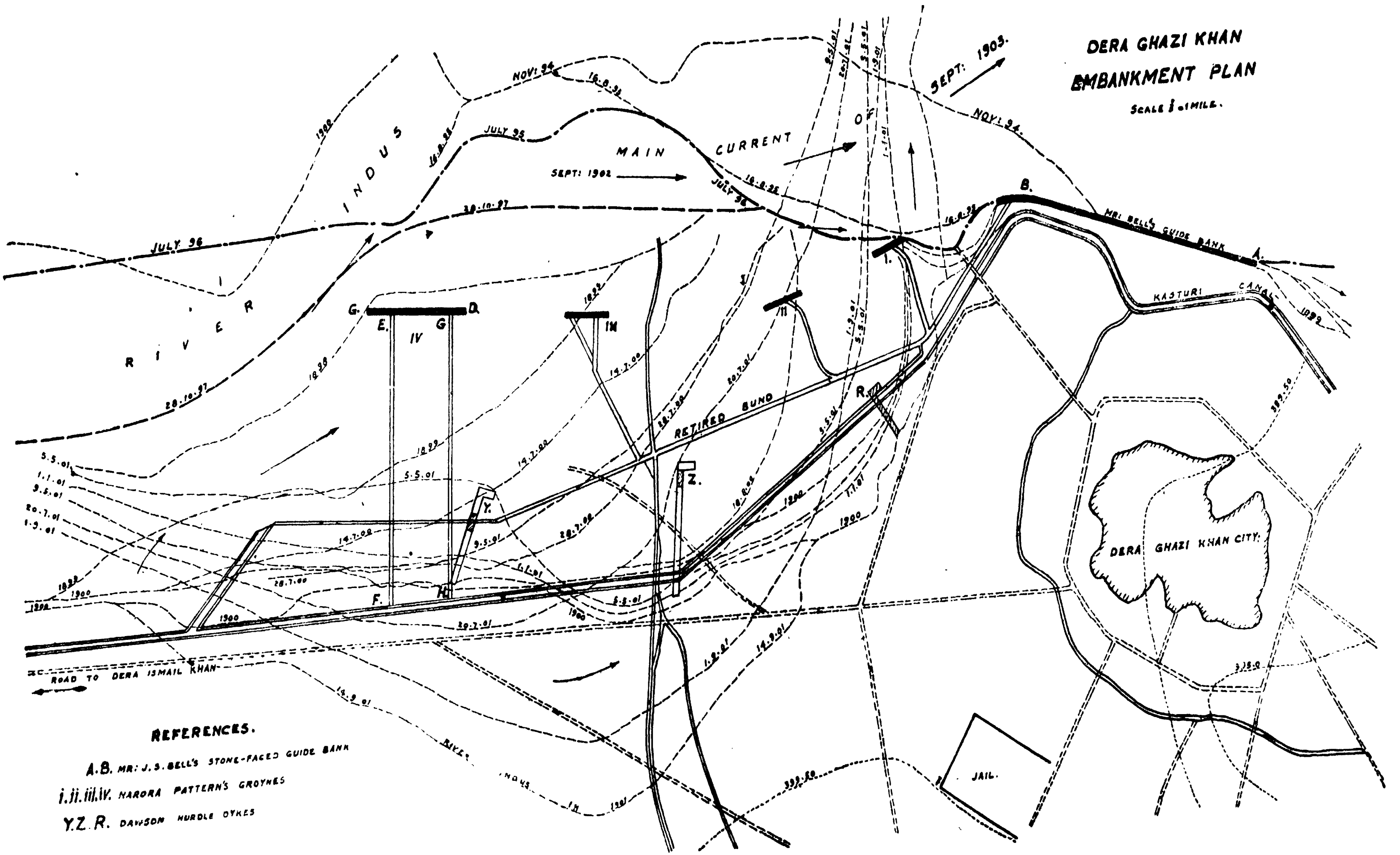

DERA GHAZI KHAN
EMBANKMENT PLAN
SCALE ½ = 1 MILE.
RIVER INDUS
MAIN CURRENT OF
SEPT: 1902
SEPT: 1903.
JULY 96
JULY 95
NOV: 94.
28.10.97
RETIRED BUND
DERA GHAZI KHAN CITY.
KASTURI CANAL
MR: BELL'S GUIDE BANK
JAIL.
ROAD TO DERA ISMAIL KHAN
REFERENCES.
A.B. MR: J. S. BELL'S STONE-FACED GUIDE BANK
i. ii. iii. iv. NARORA PATTERN'S GROYNES
Y. Z. R. DAWSON HURDLE DYKES

Fig. 12

8. Describe with sketches a suitable layout for the design of guide banks and the function of the curved ends up and downstream.

9. (*a*) Explain the following terms in the case of rivers :—
(*i*) Erosion (*ii*) Scour (*iii*) Swirls.
(*b*) Why are swirls most destructive in their action ?

10. Distinguish between scour and erosion. How these occur in large rivers. (*P.U. 1951*)

11. Describe three types of works, which are useful in the training of rivers above the site of a canal headworks. Give comments on the relative cost of maintenance of each type. (*P.U. 1953*)

12. What are the stages of a river. Why does a river meander and what is Molleys's theory on the regime of the rivers ? (*P.U. 1954*)

13. What is Molley's theory on the regime of rivers ?
Describe with sketches a suitable layout for the design of guide banks, (*P.U. 1956*)

14. Describe with sketch the layout of a guide bank and illustrate its design. Explain the process of launching of apron in a guide bank. (*P.U. 1957*)

15. Write short notes with sketches on the following :—
(*a*) Permeable spurs. (*b*) Attractive spurs (*c*) impermeable spurs (*d*) Fending groyne.
(*e*) Falling apron. (*P.U. 1958*)

severely felt and found difficult to cure. If aligned again in deep digging for a great length, the initial cost of construction will be enormous and repairs heavy, as the alignment must be more or less straight the expenditure on the adjustment of drainage, which it cannot curve to avoid, will be no light item. Of the two courses the channel in digging is probably the better in the long run.

3. Lay out of Headworks.

A typical lay out of the canal headworks is shown Fig. 2. and in aerial view of Marala Head works in Fig. 3.

It consists of the following works :—

(*i*) Weir proper divided into bays by divide piers.
(*ii*) Under-sluices.
(*iii*) Canal head regulator.
(*iv*) Fish ladder.
(*v*) Divide wall.
(*vi*) River control works.

The weir should be situated at right angles to the main stream of the river as far as possible. Fig. 2. shows the layout of the headworks at Rasul for the Lower Jhelum Canal. The Rasul weir is 4090 feet long. It is divided into 8 bays of 500 feet width each by the divide piers to avoid cross flow in floods. The fish ladder is situated at the left end of the weir. The divide wall separates the weir from the undersluices.

The undersluices are constructed on the side of the river where the canal takes off through the structure known as the head regulator of the canal. If there are canals taking off from both sides of the river upstream then there shall be two undersluices, one on each side. There shall similarly be two divide walls. The river control works consist of guide banks in continuation of the abutments supplemented by the marginal bunds and spurs to control the river spilling the country-side upstream of the weir on account of the afflux caused by the weirs. The headwork's site is very suitably situated between high ground on both sides and therefore no extensive embankments are shown in Fig. 2. The extensive river control works at Khanki are shown in Fig. 11. in previous Chapter and those at Sulemanki in Fig. 15 of this Chapter.

4. Types of Weirs.

(*a*) Weirs are classified into two classes according to the design of their floors :—

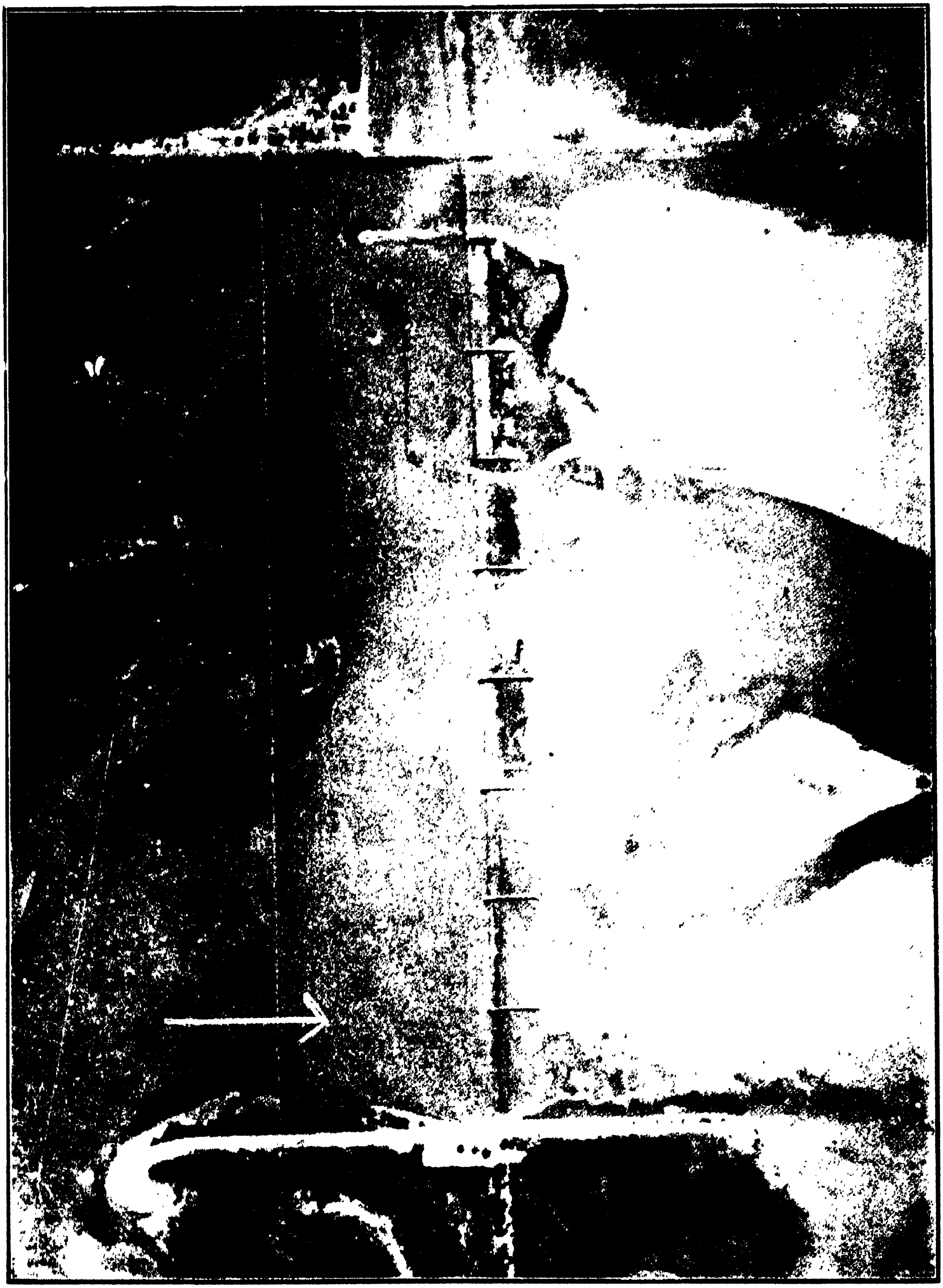
(*i*) Gravity weirs.
(*ii*) Non-gravity weirs.

When the weight of the masonry and the concrete of the floor of the weirs overcomes the upward pressure under the floor caused by the head of water against the weir it is called a gravity weir. In the case of the modern weirs, such as, the Emerson Barrage at Haveli the floor consists of the reinforced concrete slab and its weight is less than the upward pressure. The R. C. floor slab is continuous under the divide weirs, the weight of which keeps the structure safe as a whole against the uplift pressures. Such weirs are called non-gravity weirs.

(*b*) Weirs are classified into two classes according to the control of the surface flow over them.

(*i*) Open weirs or simple weirs.
(*ii*) Barrage.

In the first case, the crest level of the weir will be determined by the permissible afflux during the maximum floods, the discharge per foot run and the pond level. The pond level can be maintained by a permanent masonry crest with its top at pond level as at Rasul or one at a lower level supplemented by the falling shutters as at Khanki or Marala or counter-balanced gates as at Sulemanki and Islam. Permanent raised crest is unsuitable, because it shall cause excessive afflux head in maximum flood and there will be no control of the river in low floods. The choice between a weir with shutters and one with counter-balanced gates is largely a matter of cost and convenience in working. A shuttered weir will be relatively cheaper, but will lack the speed and the effective control possible in the case of a gated weir.



Aerial View of Marala Head Works, October 1936. Fig. 3.

A barrage is a gate controlled weir right across the river with crest at one uniform level. The Sukkur weir in Sind and the Ferozepore, Panjnad and Haveli weirs in the Punjab are examples of this type. A barrage provides a perfect control of the river channel upstream in low floods and affords better facilities of inspection and repairs. A barraged weir should provide efficient control of the channel leading to the undersluices and thereby help in the control of the silt entering the canal.

(*c*) The weirs are classified as below according to the functions they perform (*i*) Storage weirs, (*ii*) Intake or Diversion weirs, (*iii*) Waste weirs. The storage weirs will be dealt with in Chapter III Part III along with the storage reservoirs. It is the second class of weirs which forms the subject matter of this chapter as used for diverting the supplies into the canals. Waste weirs are escape channels for floods which cannot be stored in the reservoirs. These are constructed to ensure the safety of dams and their design is dealt with in Chapter II Part III.

(*d*) **Parts of a weir.**

A typical cross section of a weir (Bligh type) is given in Fig. 4.

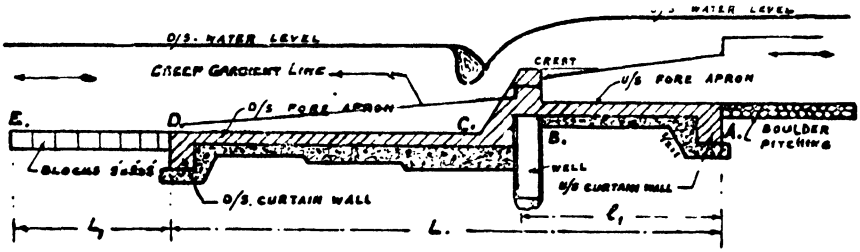


Fig. 4.

A weir consists of the following parts :—

(*i*) Upstream curtain wall.
(*ii*) Fore apron.
(*iii*) Crest.
(*iv*) Downstream apron or floor.
(*v*) Downstream curtain wall.
(*vi*) Riprap or Talus or pervious protection.
(*vii*) Wells under the crest.

(*i*) The upstream curtain wall used to be of shallow depth 6 to 8 feet deep, unlike the modern sheet piles which can be sunk to any depth.

(*ii*) The length of the fore apron is determined according to the empirical formula evolved by Bligh, on page 169 of Design of Irrigation Works.

$W=4.\sqrt{CH}$ where W=length in feet of fore apron

C=creep coefficient, H=head against the weir.

The floor level upstream of the weir is determined from considerations of the permissible scour depth below the maximum flood discharge level up stream of the weir which is the observed flood discharge level at the sight of weir plus the afflux head allowed. The scour depth can be worked according to Kennedy's or Lacey's Theory of design of Irrigation channels as shown in calculations in chapter VI. The thickness of the fore apron is determined from considerations of water tightness because the uplift pressure below is balanced by the weight of water above it. About 1 ft. masonry over 1·0 to 1·5 ft. of concrete will do.

(*iii*) The masonry crest is designed as a wall retaining the water pressure as shown in Fig. 5. The worst conditions for the weir floor are when the water is headed up to the top of the shutters upstream and is at the Talus level downstream.

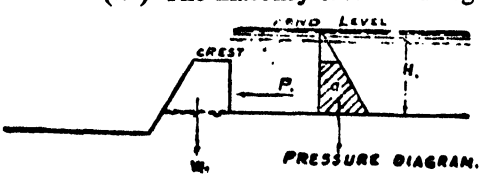


Fig. 5.

The resultant pressure P is equal to the shaded area of the pressure diagram acting horizontally at the centre of gravity of the shaded trapezoid. Similarly, weight W of the crest acts vertically at the centre of gravity of the crest section. The resultant of P and W must pass within the middle third of the base. This determines the width at the base of the crest wall and the top width is fixed from the practical considerations to allow enough spaces for the anchoring and the working of the shutters. The height of the weir crest is determined from considerations of the permissible afflux to pass the maximum flood discharge as shown in the calculations in Chapter V. The required depth on crest for the maximum flood discharge will fix the crest level.

(*iv*) The length of the downstream apron depends upon the total length of the impervious floor as determined by the allowable creep coefficient for the particular soil of the river as given in para 2 Chapter V of this part. The method to draw the creep gradient line shown in dotted is explained therein. The thickness of the downstream apron depends upon the actual pressure acting below it according to the creep gradiant line. It is calculated according to the formula.

$$t=\frac{4}{3}\cdot\frac{H-h}{\rho-1}$$

Where t=thickness of the floor in feet.
H=Total head as shown in Fig. 5.
h=head lost in creep up to the point where thickness is to be determined.
ρ=Specific gravity of the floor material.
$\frac{4}{3}$ represents the factor of safety

ρ is 2·25 for stone masonry and concrete in stone ballast and 2·0 for brick work and concrete with brick ballast.

The floor level downstream of the weir crest used to be kept the same as the upstream floor level (top of the fore apron). It should be kept the lowest as worked from the following three considerations :—

(*a*) Depth required for the flood discharge with the actual surface slope of the river before the weir construction for the intensity of discharge per foot run over the river.

(*b*) From considerations of the depth required to form a hydraulic jump downstream of the weir on the impervious protection.

(*c*) The floor level downstream should be depressed sufficiently below the upstream floor level to allow for the retrogression of the river bed levels which follows the construction of the weirs across alluvial river as explained later.

(*v*) The downstream curtain walls too used to be very shallow, usually 8 to 10 feet deep, because it was not possible to construct them as the spring level in the river-bed could not be lowered very considerably by pumping. In the modern weirs the sheet piles are driven to form a very deep cut off at the end of the impervious protection.

(*vi*) The total length of the pervious protection is calculated according to the formula for

free overfalls, $L_1=10C\sqrt{\frac{H_b\times q}{10\times 75}}$

and for sloping downstream aprons, $L_1=11C\sqrt{\frac{H_b\times q}{10\times 75}}$; (Page 164 of Dams and weirs by W.G. Bligh of 1918.)

Where L_1=length of the talus
C=Creep co-efficient for the river bed soil
q=discharge per foot run
H_b=Hight of the crest wall above bed

The wells under the crest simply serve as nails in the river bed. Their usefulness as destroyers of creep head had never been attained with success because the spaces in between them could not be made water-tight. The usual attempt to drive wooden piles between the wells has not been much successful. It was usual to consider these shallow wells as an additional factor of safety and the creep length required was usually provided, without taking them into consideration.

5. Afflux

Afflux is the rise in the maximun flood level of the river upstream of the weir caused as a result of its construction across the channel. In the beginning, the effect of the afflux in raising the river levels is only felt up to a short distance upstream according to the length of the back water curve but in course of time the river bed rises due to the silting up caused by the additional waterway added to the section upstream of the weir by the afflux height and the effect travels upstream till the river slope upstream of the weir is the same as before its construction. In the design of weirs founded on alluvial sands, the afflux is limited to 3 to 5 feet.

The amount of afflux will determine the top levels of guide banks and their lenghts and top levels and sections of flood protection bunds. It will govern the dynamic action as the greater the afflux or fall of levels from upstream to downstream, the greater will be the action. It will also control the depth and location of the standing wave. By providing a high afflux the width of weir can be narrowed but the cost of the training works will go up and the risk of failure by outflanking will increase. The discharge per foot run, the depth of the scour with the action on the loose protections upstream and downstream, as well as the depth of piles at either end will increase with the afflux.

6. Pond level

The pond level is the water level required in the undersluices pocket upstream of the head regulator of the canal to feed the canal with full supply. The full supply level in the canal at its head depends on the levels of the country which it has to irrigate, and the permissible slope in the canal. The working head at the canal head regulator should be allowed to be about 3 to 4 feet, while the waterway for the head regulator may be designed with less than half the available head. Enough margin should be left in fixing the pond level for the future silting up of the canal bed and the silting up of the river bed downstream of the weir which will cause sluicing difficulties for occasional washing down of the silt in the pocket.

Pond level determines the height of the undersluice gates and the height of the shutters above the permanent masonry crest of the weir.

7. Waterway for weirs.

The waterway for the weirs has in the past been kept arbitrarily limited by the permissible scour depth which would fix the floor level upstream of the weirs. If the waterway is restricted very much, the floor levels will be relatively low necessitating heavy and expensive pumping to lay the upstream and downstream impervious aprons.

(*a*) The scour depth for a discharge per foot run be worked from Kennedy's silt formula

$$V_o = \cdot 84\ D^{\cdot 64}$$

which applies to the Punjab conditions.

It can be modified to suit other grades of silt by introducing another factor known as C. V. R. and denoting it by x.

$$V_o = \cdot 84.\ x.\ D^{\cdot 64}$$

Let the discharge per foot run be q cusecs,

$$\text{Then } q = V_o.\ D = \cdot 84.\ x.\ D^{1\cdot 64}$$

$$\therefore D = 1\cdot 11 \left(\frac{q}{x}\right)^{\cdot 61}$$

and when x is unity

$$D = 1\cdot 11\ q^{\cdot 61}$$

(*b*) Another method to determine the scour depth is that given by Lacey in his paper on "Stable Channels in Allvium" No 4736 (1930) Institute of Civil Engineers, London.

$$V_o = \left(\frac{Q.f_l^2}{3\cdot 8}\right)^{1/6}$$

$$\text{and } R = \cdot 7305\ \frac{V_o^2}{f_l}$$

$$\text{Hence } R = \cdot7305 \left(\frac{Q}{3\cdot8f_l}\right)^{\frac{1}{3}}$$

$$\text{but } P_w = 2\cdot67\ Q^{\frac{1}{2}}$$

$$\therefore \text{ The discharge per foot run or } q = \frac{Q}{P_w} = \frac{Q}{2\cdot67\sqrt{Q}} = \frac{\sqrt{Q}}{2\cdot67}$$

Substituing in terms of q

$$R = \cdot9 \left(\frac{q^2}{f_l}\right)^{\frac{1}{3}}$$

(*c*) In the table given below, the actual waterway provided between the abutments is compared with that worked out according to Lacey's formula $P_w = 2\cdot67\ Q^{\frac{1}{2}}$ in the case of some of the Punjab weirs. Waterway for weirs for the average conditions in the Punjab should be 1·5 times P_w worked out according to the Lacey's formula. The experience has shown that even the restricted weirs like those at Ferozepur and Islam suffer from the same tendency of forming islands upstream of the weirs as in the case of the relatively wider weirs. The restricted waterway increases the scour depth and consequently lowers the apron level. This entails relatively higher expenses of the pumping to lay them.

8. Effect of weirs on the Regime of a river.

(*a*) The river regime is affected by the construction of a weir across its channel in the following ways.

(*i*) The slope of the river upstream of the weir flattens due to the ponding up of supplies.

(*ii*) An increase in tortuosity, as a result of ponding up, as the bulk of silt charge of the river water deposits in the pond, leading to the formation of irrigular shoals.

(*iii*) A progressive degradation or retrogression of bed levels downstream, due to the picking up of bed silt by the relatively silt free water escaping over the weir.

(*iv*) These effects continue for the first few years but later due to continous silting up of the pond and increasing tortuosity, the bed level of the stream will tend to rise as the bed levels at the weir are fixed. This can be explained by the fact that the water will need greater head to overcome the increased distance over which it has to travel. An increase in tortuosity will necessarily enhance the rise of levels due to weir afflux with the result that this rise will be felt higher up the river than would otherwise be the case. As a result of this progressive rise of bed level, there will be tendency on the part of the river to regain its original slope.

(*v*) A stage will come when upstream pond absorbs no further silt burden. Owing to the offtaking canals drawing comparatively silt free water, the excess of silt will go downstream of the weir while the amount of water passing over the weir will be below the normal due to canal withdrawls. The river below the weir, will, thus, have to carry an excessive silt charge with a lower discharge. This will result in progressive silting up downstream, an increase in tortuosity and therefore, a recovery of bed levels downstream.

(*b*) The changes in the regime of a river caused by the construction of a weir have an important bearing on the design of the weirs as outlined below :—

(*i*) **Retrogression of Levels.**

In the first few years following the construction of a weir, the retrogression of bed levels downstream is rapid and progressive. In the case of the Punjab weirs this has ranged between 4 to 7 feet. This lowering of the bed levels in the early stages if not duly allowed for in design may result in a failure like that of the Islam weir in 1929. The retrogression may undermine the stability of a work by an increase in the exit gradient beyond the safe limits. It will increase the destructive action of the standing wave as with the increased fall and decreased depth of downstream water due to the lowering of the water levels at that end, the wave will tend to travel down to the block and loose protection area.

As a result of the retrogression in bed levels, while the low water levels have been found

TABLE 1
Showing waterway, discharge per foot run etc., on various weirs.

No.	RIVER	SUTLEJ				JHELUM	CHENAB				INDUS
	Name of Weir	Rupar	Ferozepore	Sulemanki	Islam	Rasul	Marala	Khanki	Trimmu	Panjnad	Sukhur Barrage
1	Upstream H. F. L. ...	877·00	651·50	572·00	547·00	724·00	814·10	738·00	493·50	341·50	201·00
2	Afflux ...	4·50	3·00	3·00	2·00	0·70	3·00	3·00	...	3·00	...
3	Crest Level { Weir ...	866·50	633·50	560·00	441·00	711·50	802·00	727·00	477·50	325·00	177·00
	Crest Level { Undersluices ...	857·00	633·50	552·00	435 50	701·00	792·00	715·00	472·00	325·00	176·00
4	Width between abutments (B) ...	2663	1956	2223	1621	4000	4475	4414	3022	3400	4725
5	Length of the clear water way { Weir ...	2292	1200	1440	921	4400	4000	3000	2220	2820	3240
	Length of the clear water way { Undersluices ...	240	540	480	480	240	240	960	420	240	720
	Total ...	2532	1740	1920	1401	4240	4240	3960	2640	3060	3960
6	Designed max. flood discharge (Q) ...	315,500	450,000	325,000	275,000	875,000	718,000	750,000	645,000	700,000	1,500,000
7	Max. discharging capacity { Weir ...	251,600	310,000	210,000	218,000	771,000	647,000	435,000	502,000	640 500	1,230,000
	Max. discharging capacity { Undersluices ...	63,900	140,000	115,000	170,000	104,000	89,000	372,000	143,000	59,500	270,000
	Total ...	315,500	450,000	325,000	388,000	875,000	736,000	807,000	845,000	700,000	1,500.000
8	Minimum stable width $=P_w=2{\cdot}67\sqrt{Q}$ ...	1500	1784	1520	1400	2501	2262	2312	2149	2230	3260
9	Ratio $\frac{B}{P_w}$ ...	1.7	1.10	1·47	1·15	1·76	1·98	1·91	1·41	1·56	1·45

to drop from 4 to 7 feet, the maximum flood levels have not been known to have dropped by more than 1 to 1·5 ft. The downstream flood level should be depressed below the upstream river bed by about 4 feet.

(*ii*) **Restoration of the original slope upstream of the weir.**

In the course of time the river upstream will regain its original slope which implies that the affect of afflux due to the construction of the weir will not be confined in magnitude and length to the usually accepted distance as determined by the back water curve (approximately $2.H_a/S$). It will travel very far up and will be felt in full, all that distance. In other words, the entire bed of the river will ultimately rise uniformally throughout the zone of protection and training works. As, however, the full effect will not be felt until after lapse of many years, the free board, may, in the first instance, be reckoned above the H. F. L. as determined by the back water curves.

(*iii*) **Recovery of downstream bed levels.**

The process of recovery of downstream bed levels after the intial retrogression, is slow but steady. It may take 20 to 30 years but the bed levels may rise higher than those before the construction of the weir. At Khanki the rise above the original bed level has been of the order of 2 feet. A rise in the downstream bed levels may lead to the loss of control of the river in respect of silt regulation, making it necessary to raise the weir crest, Khanki and Marala are instances of such raising. The Khanki weir crest was raised by two feet in 1910-11 in bays numbered 4, 5 and 6, and in 1917 in the remaining bays. It was again raised by a further two feet in 1920-22. Marala was raised by two feet in 1925-26 in order to obtain control of the river and to improve silt conditions in the main line. These facts point out the necessity of design for a pond level sufficiently above the full supply level in the canal so as to leave ample margin even after the rise in the downstream bed has taken place.

9. Undersluices.

(*a*) **Object of the undersluices.** The undersluices are required to keep the river under control aiming at the following points :—

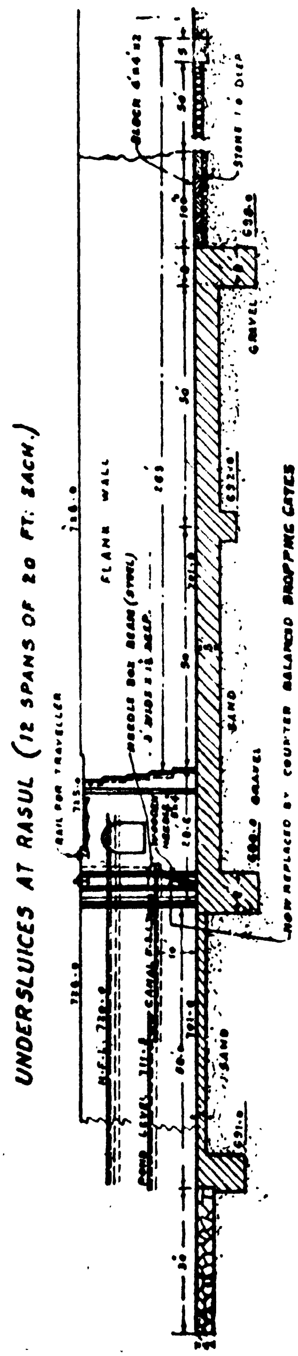


Fig. 6.

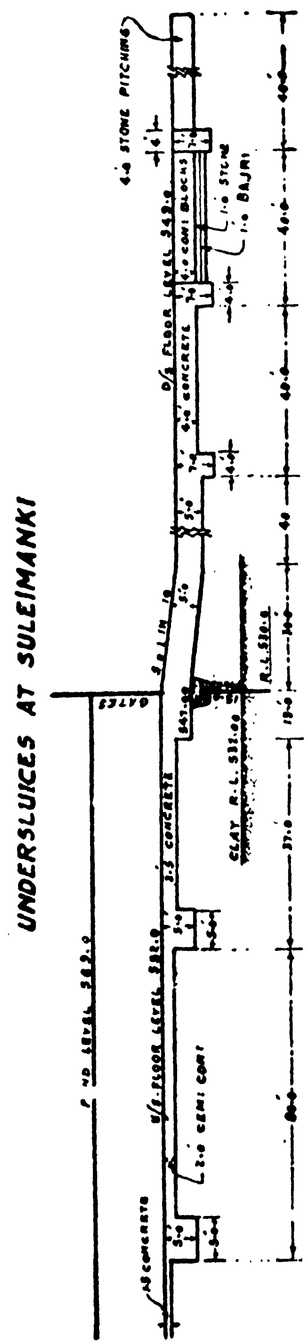


Fig. 7.

(*i*) To scour the silt deposited on the river bed in the pocket upstream of the canal head regulator.
(*ii*) To preserve a clear and defined river channel approaching the regulator.
(*iii*) To facilitate the working of the weir crest when movable in the form of shutters or counterbalanced gates.
(*iv*) To lower the highest flood level.
(*v*) To pass the low floods without dropping the shutters, the raising of which entails good deal of labour and time.
(*vi*) To control the silt entry into the canal.
(*vii*) In the case of sudden floods, the undersluices can be opened to fill the river beds downstream to protect the downstream Talus from the Hydraulic jump action.

(*b*) **The capacity of the undersluices.**

The capacity of the undersluices is fixed on the following considerations :—
(*i*) To ensure scouring capacity, it should be at least be double the canal discharge.
(*ii*) The undersluices should be capable of passing the low flood discharge excepting the three months of the moonsoons without over-topping the shutters.
(*iii*) The undersliuces should be capable of passing the winter freshets under the gates with canal in flow,
(*iv*) At high floods, the undersluices should be capable of passing about 10 to 15% of the maximum flood discharge. This aims at reducing the length of the weirs because long weirs will cause island formation upstream and will also serve to maintain deep water channel towards the canal regulator.

If there are two off-takes, one on either side of the river, there should be two undersluices of such capacities in each case as determined from these considerations.

(*c*) **Design of the undersluices.**

The following points should be kept in view in designing the undersluices :—
(*i*) The crest and the floor level in the pocket upstream should be at the lowest practicable level of the river. It is usually kept at the level of the apron upsteram of the weir crest.
(*ii*) Large spans are preferable. In the case of old weirs the undersluices spans used to be not longer than 20 feet because wider gates were not available and were difficult to work. Now the gate design has been very much improved by the use of counter-balanced weight as in the case of Stony's gate arrangement described later in this chapter. Now spans of 30′ or more are used.
(*iii*) The floor thickness and its length are determined according to the weir design as outlined in the next chapter.
(*iv*) The discharge intensity being maximum in the undersluices the foundations, the floor and the Talus should be made extra strong.
(*v*) The floor downstream of the crest should not be kept level with the crest level because the formation of a proper hydraulic jump as destroyer of energy is very uncertain on a level floor. Moreover from the considerations of retrogression, it would be advisable to depress the downstream floor upto the likely retrogression bed level downstream which may be at least four feet.
(*vi*) A bridge downstream of the undersluices should always be provided to lift the gate by cranes if the Stoney's arrangement fails on any account.

Typical sections of the undersluices are shown Fig 6 and 7.

The length of impervious apron upstream of the sluice gate is got from the empirical formula.

$W = 7C\sqrt{\frac{H}{13}}$ where

C=creep coefficient

H=Height of the shuttered water level above floor level.

The length of the Talus below the undersluices is got from the formula on page 216 "Design of Irrigation works" by W. G. Bligh.

$$L = 15C\sqrt{\frac{H_b}{10}} \times \sqrt{\frac{q}{75}}$$ Where L = Length in feet

C = creep coefficient

H_b = height of the permanent weir crest.

and q = Discharge intensity per ft. run.

The thickness of the floor is calculated from the creep gradient theory like the weirs. The depth of the Talus is 4 to 6 feet, usually one and half times the depth of floor downstream of the weir.

10. Divide wall.

The divide wall is simply a long divide groyne built between the weir and the undersluices. It separates the turbulent river in maximum floods from the pocket in front of the canal head regulator. The wall extends upstream to a little distance beyond the beginning of the head regulator and downstream to the end of the Talus of the undersluices. This wall plays a very important part in controlling the entry of silt into the canal by enclosing a pocket of very nearly still water and by separating it from the turbulence and vageries of the alluvial rivers in floods.

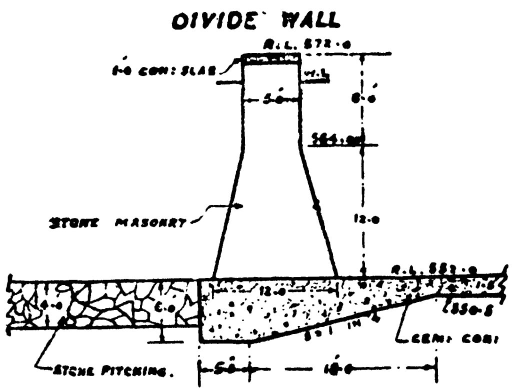


Fig. 8.

In the case of old headworks such as at Rasul the divide wall is simply an earthen bund protected with stone pitching on both sides and provided with stone apron as in the case of guide banks. The upstream end is provided with extra strong protection in the form of apron. An earthen divide groyne requires a lot of space for its section and shuts off a considerable effective water-way of the weir and it is, therefore, now usual to build the divide wall in solid masonry as shown in Fig. 8.

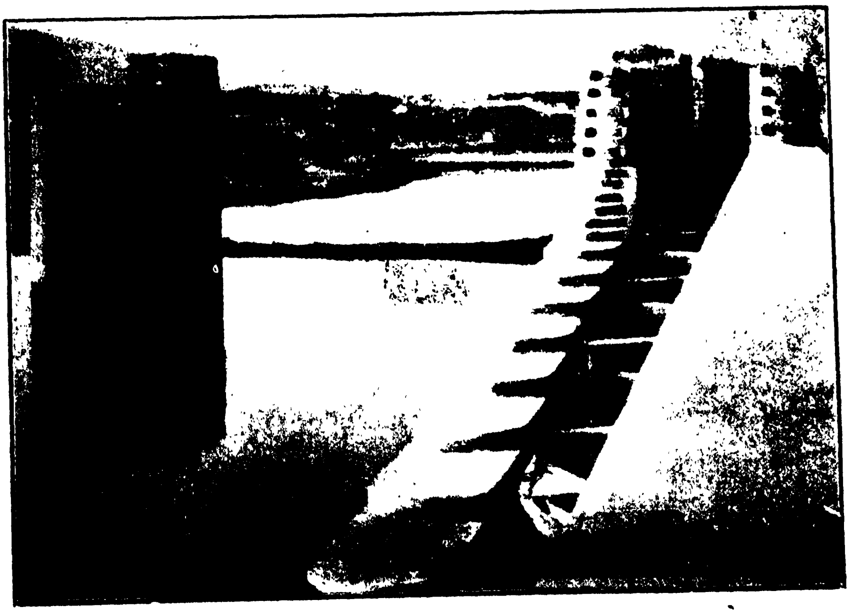
Normally there will be water on both sides but difference of pressure on both sides of the wall is likely, due to still water being in the pocket and a strong river current destroying its velocity head by striking against it. Similarly, if flood is being passed through the undersluices, there will be relatively less depth on the weir side. It is usual to design the wall for a pressure difference of 3·0 feet depth of water on either side. In the case of masonry divide wall, it is necessary to provide well foundations for at least 100 ft. length from the extreme upstream end taking them well below the deepest possible scour.

11. Fish lader.

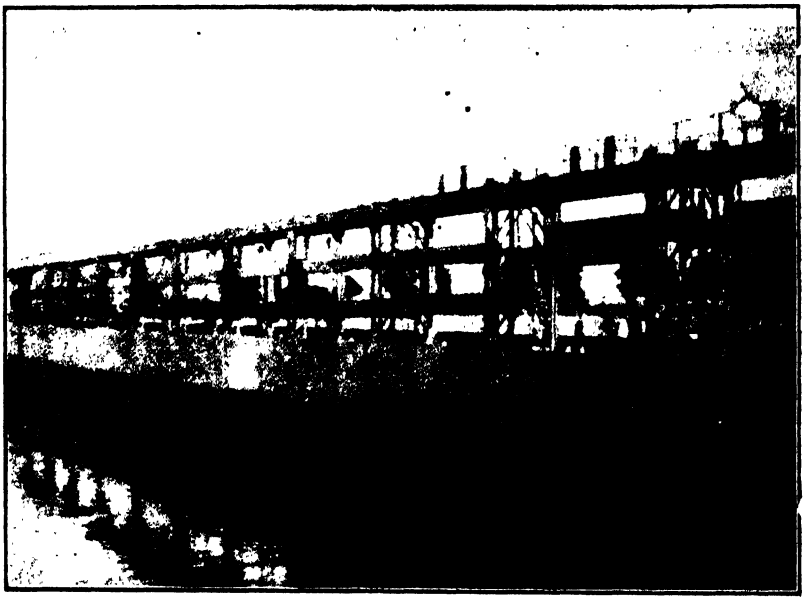
In the large rivers, the fish are always moving from one part to another. In the beginning of winter season they leave the cold water in the hills and move down to the relatively warm water in the plains. Before the monsoons in the months of May and June they move up again in search of clear water. In the months of July and August in Northern India, the female fish lays eggs in the water which meeting the juice created by the male fish in clear water fertilizes the eggs. The meat of the female fish is usually poisonous in the months of July and August on account of the presence of eggs in the body. The fish therefore travel up and down hundreds of miles along the large rivers and it is, therefore, essential that a space should be provided in the construction of the modern weir for the uninterrupted movements of fish.

Fish can easily travel against current of water with velocities up to 10 or 12 feet per second. The design of the fish ladder should be such that the velocity of the current against which the fish have to move shall not be higher than this limit. There is usually a head of about 16 to 20 feet from the upstream of a weir to the downstream water level on the river in the winter

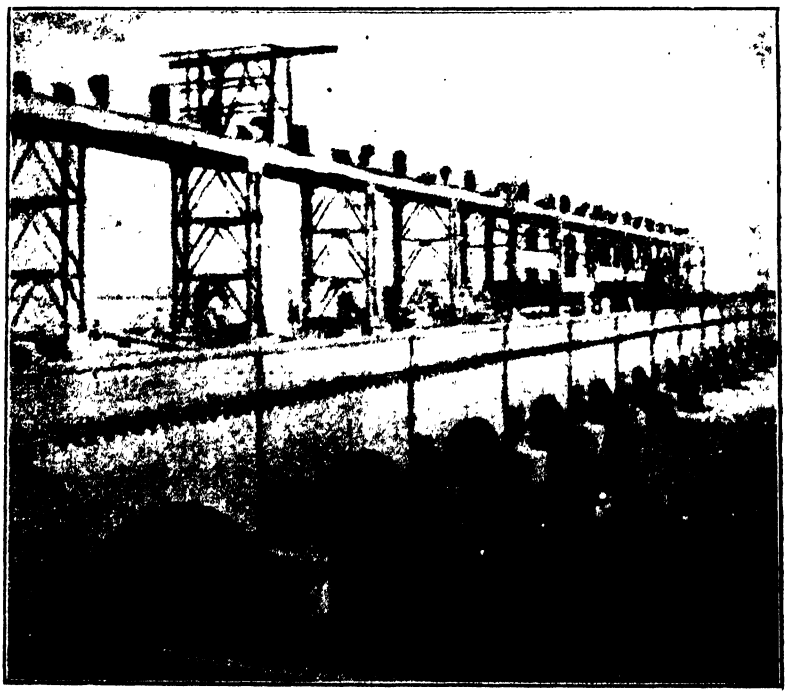
Fish ladder along Trimmu Weir Wall under construction. Fig. 10 (a)



A view of the fish ladder along the right divide wall Trimmu Weir. Fig. 10 (b).



Upper Chenab Canal Head Regulator as remodelled in 1937. Fig. 12 (a)



Canal Head Regulator Downstream view. Fig. 12 (b) months. In an open gap in the weir just like the one in the Ganges weir at Hardwar, the velocity of the current will be very high, and therefore even the strong fish will not be able to travel upstream. It was nothing short of cruelty to these poor animals to see swarms of large fish, 2 to 4 feet long, collecting downstream of fish gap in the weir at Hardwar and dying of sheer exhaustion in attempts to move upstream. A proper fish ladder has now been provided there.

In the modern weirs the fall of water is broken in small strips. Typical designs of fish ladders are shown in and Figs. 9 and 10.

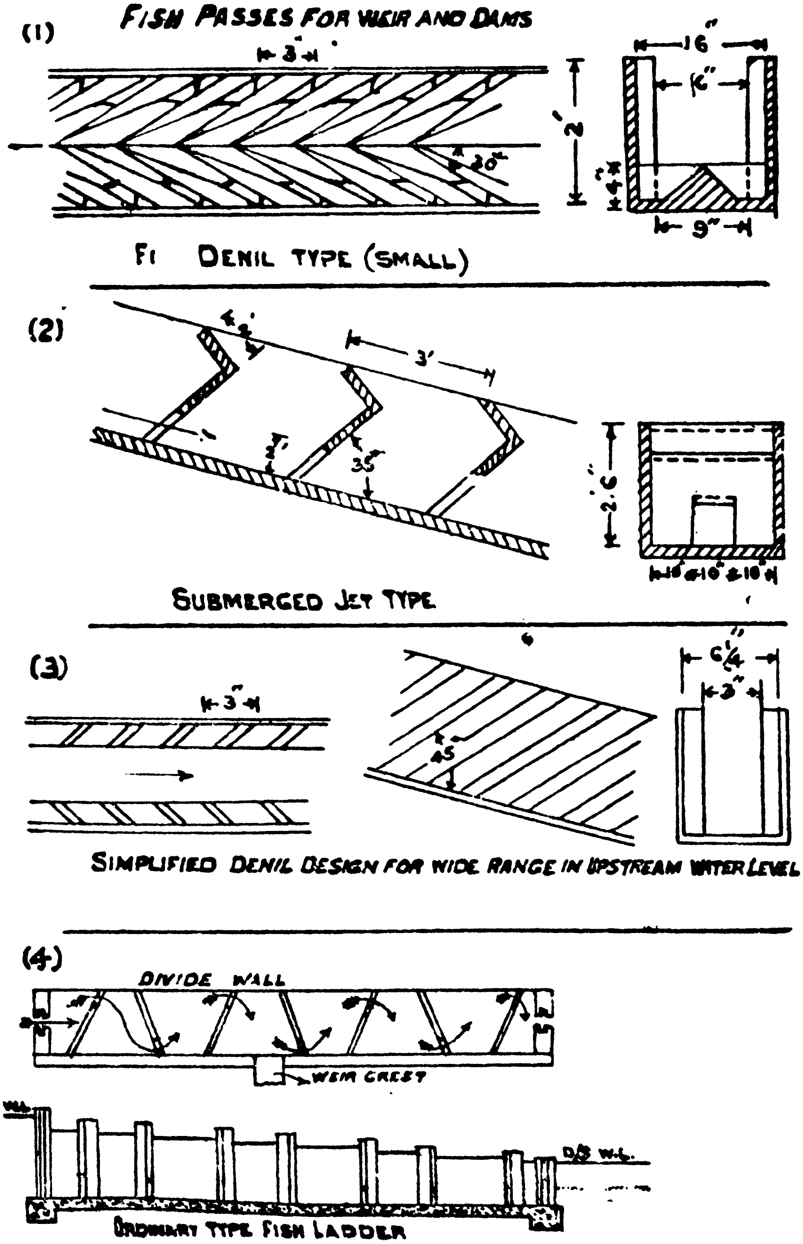


Fig. 9.

The suitable site for the fish ladder is near the divide wall because there is water throughout the year in river bed downstream of the undersluices only. It is usually located between the weir and the divide wall and in some cases it has been built within the divide wall. The walls are built oblique and holes are also staggered as shown on the plan so that the fish can take rest after passing one hole before they move on to the other. These walls also serve to break the fall in small steps as shown by water line in the section.

12. Head Regulator.

The object of a canal head regulator is three fold :—

(*i*) To regulate the supply entering the canal.
(*ii*) To control the amount of silt entry into the canal.
(*iii*) To shut out river floods entering the canal.

The water-way of the head regulator is fixed so that the full supply discharge of the canal could easily pass over the crest of the head regulator with the designed pond level, with ample factor of safety, to allow for any silting up of the canal. The regulation is arranged by providing gates sometimes in two sets, one rising and the other dropping, as shown in Fig. 11, as in the case of the Sirhind Canal regulator and Upper Chanab Canal head regulator in Fig. 12 (a) and 12 (*b*).

SIRHIND CANAL REGULATOR.

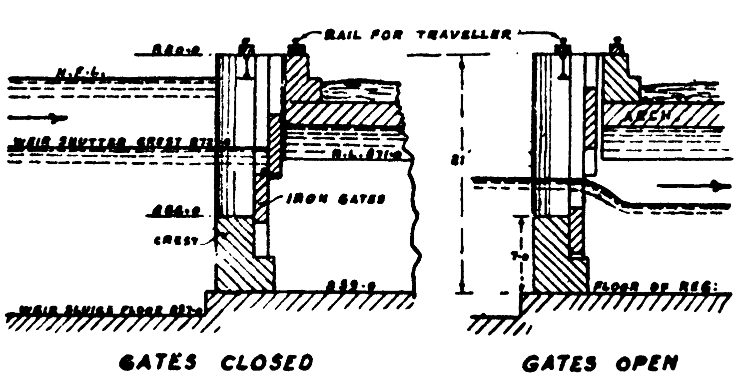


Fig. 11.

In the case of old Head regulators, the spans were small usually 5 to 7 feet, by providing dummy piers dividing the span of the downstream bridge because the gate lifting arragements were crude in the from of travelling winch mounted on rails. The original Sirhind canal head regulator had 39 spans of 5 feet each. In the modern canal head regulators, the modern arrangement of radial arm gates lifted by the rack and pinion method has reduced the number of the head regulator bays. The spans of 20 to 25 feet are now quite common. A section of the canal head regulator taking off at Haveli is given below in Fig. 13.

SECTION THROUGH HAVELI CANAL HEAD REG:

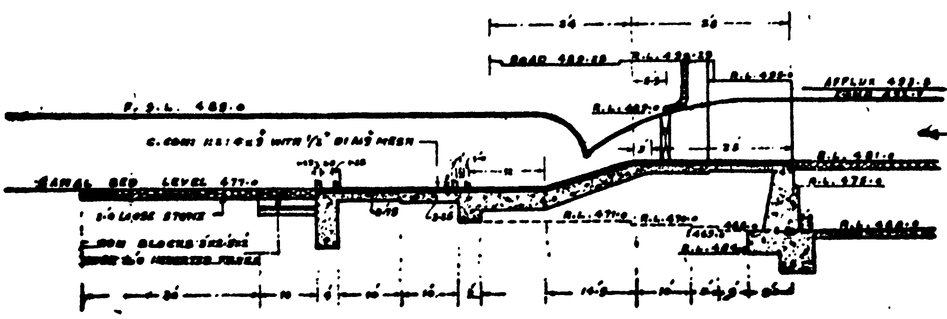


Fig. 13

The silt control in the head regulator design is provided by having a suitable raised crest. In the Sirhind Canal a crest wall of 7·0 feet height was put in due time to a serious trouble in the Canal. A permanent high crest shall need a high pond level to feed it in winter with high losses for m the pond and it is, therefore, usual to provide a rising cill gate as shown in Fig. 12.

The head regulators at Khanki and Panjnad have been provided with silt excluders in front of them in the pocket. A separate Chapter, No. XI of this part, has been devoted to the design of silt excluders and silt ejectors. As the river conditions in the pocket are usually very much disturbed, the silt excluders in front of the head regulators in the pocket have not proved so efficient as a silt ejector built in the canal with steady flow about a thousand feet downstream of the head regulator as in the case of the Haveli Canal.

In the case of the maximum floods, the head regulator of a canal is to be completely closed lest heavily silt-laden water of the river silts up the canal. There are now daily silt observations being carried nearly at all head works measuring the silt content of water passing into the canal and that of water escaped under the sluice-gate. When the percentage of silt charge in water entering the canal exceeds a certain figure fixed in each case, the canal is closed. This function of the canal head regulator has a very important effect on its design. In maximum floods, the worst hydraulic conditions are created. Hydraulic head causing flow under the regulator is of the order of 25 to 30 feet with the canal closed, which may be about double the head for which the weir floor is designed. Such a high head shall require a very long impervious floor with the modern introduction of sheet piles, and excessive floor thickness. The gravity sections will be impossible and even the inverted arch or reinforced concrete sections will be very expensive. The economical solution of this trouble in the case of large rivers would be to make the whole floor of the pocket impervious by providing a water-tight concrete of say about 1·5 to 2·0 feet depth ensuring that the joint of this floor with the crest of the Canal head regulator is absolutely water tight. Such a floor is required when it is desired to provide a silt excluder and shall also lessens the cost of the undersluice aprons.

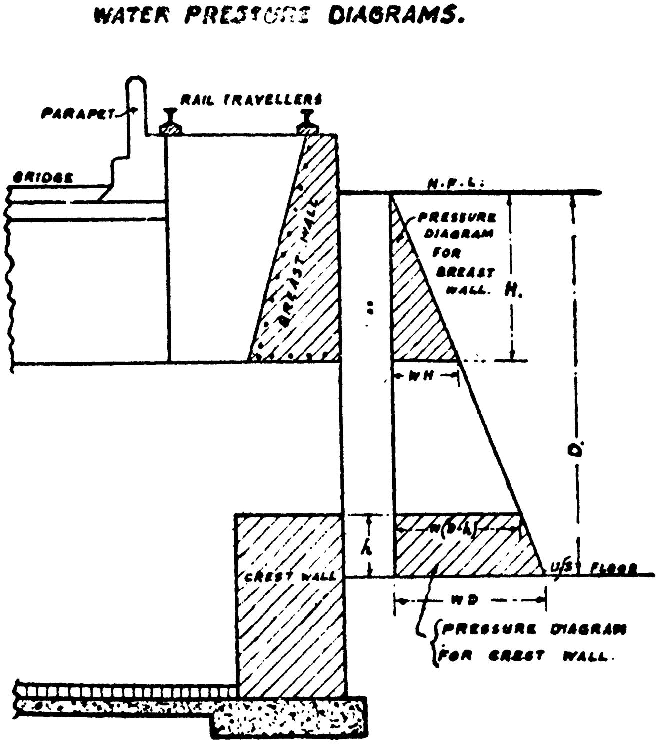


Fig. 14

The stabilty of the head regulator piers should be tested to withstand the overturning moments caused by the high pressure head in floods. The designs of the crest wall of head regulator, the gates and the breast wall will be worked out for the water pressure diagram shown in Fig. 14.

It is now usual to provide a solid masonry crest wall and a breast wall in reinforced concrete. The horizontal arched breast walls are now obsolete. The reinforced concrete breast wall can also be reinforced at its bottom to act as a beam to support its own weight and the weight of the gate lifting arrangements instead of providing additional girders as in the case of old head regulators. The layout of the head regulator is usually at right angles to the undersluices, parallel to the divide groyne wall. Sometimes oblique entry into the canal is considered desireable and the head regulator is splayed out 1 in 4 from the undersluices, taking off the canal at right angles to the splay. This has been tried at Sulemanki. This will provide additional still-pond water-way in the pocket and shall also insure relatively smooth entry into the canal. There is no empirical formula to determine the length of the Talus downstream of the head regulator. It would be ample to provide a Talus equal to 4 to 5 times the depth of canal and 2 to 3 feet thick in concrete blocks or stones of more than one foot diameter.

13. Silt control at Head Works.

There are three methods of regulation usually adopted at the canal headworks to control the silt entry into the canal.

(a) Still Pond System.

In this system the river water in excess of the canal requirements is not allowed to escape under the sluice gates. Only the supply required for the canal enters the pocket. The velocity of water in the pocket is very much reduced on account of its excessive water-way. The silt deposits in the pocket and clear water enters the canal. This system presupposes that the canal regulator is provided with a high crest wall as shown in Fig. 7. When the silt in the pocket has accumulated to say below 2 ft. of the cill level of the regulator, the canal is closed for about 24 hours and the sluice gates are opened to scour out the silt deposits in the pocket. This process is repeated when again the silt accumulates in the pocket.

(b) To Take Supply From The Curved River Approach Channel.

In this system, the undersluices remain closed and the river is allowed to develop a curved approach channel by escaping supply in excess of canal requirements over the weir by opening shuttered portion or by constructing another set of undersluices in the middle of the weir as at Khanki. The water is then taken from the outside of the curve when the channel section is deep as shown in Fig. 11, Part II Chapter III. There is crossflow of silt in a channel section on a curve which results in low silt charge on the outside of a curve as compared to that on the inside. After thus selecting relatively clear water from the river channel, this system can be combined with the still pond method for further silt selection in the pocket itself.

(c) To use Undersluice Gates to Escape Excess River Supply.

In this system, the river supply in excess of the canal requirements is escaped under the sluice gates. In this system reliance is placed on the fact that the coarse silt shall concentrate near the pocket floor and it can safely be escaped below the gates. This system though it looks sound in theory is very treacherous on account of the uncertain approach channel conditions in the river. When this system is used, there must be provided a fairly high crest of head regulator with suitable design. Even then the daily slit observations must be taken to find out the silt content of water entering the canal. When it exceeds a certain figure for the particular canal, it should be closed and the pocket cleared.

The still pond system has been very successfull at Rupar and Marala. This is by for the most efficient form of regulator for canal head works which are not provided with silt excluders with the only defect that the water has to be wasted in scouring the pocket for about a day in a month.

14. Guide Banks & Marginal Bunds.

The chief idea in the design of these works is to concentrate the river into a narrow channel by a pair of long guide banks. It is claimed for this arrangement that it not only induces a clear and direct flow to the weir but it also greatly reduces the number of spurs needed along the marginal bunds. Being forced into one restricted channel, the river has no room to swing about and there is no possibility of cross flow just upstream of the weir. If the river does swing about, it must be above the entrance to the guide banks which is so far from the weir that little or no harm can occur to the protective works behind the guide banks at the weir itself.

The following points should be kept in view in the design and layout of the guide banks :—

(*i*) The choice between parallelism, convergence and divergence must be dictated by the condition of the bed of the river, for construction purposes, during the working season. For it is better, if possible, not to be obliged to lay the apron in deep water.

(*ii*) But, if practicable, it is better that the guide banks should approach each other near their upper ends, before their upper curves begin. The amount of construction may be say, anything upto double the combined thicknesses of the bridge piers below low water.

(*iii*) The length of the upstream part of the guide banks may be made equal to, or say upto to a tenth longer than, the bridge. But attention should be paid to the possibility of the river bending round above one guide bank into the still water area at the back of it, and eroding

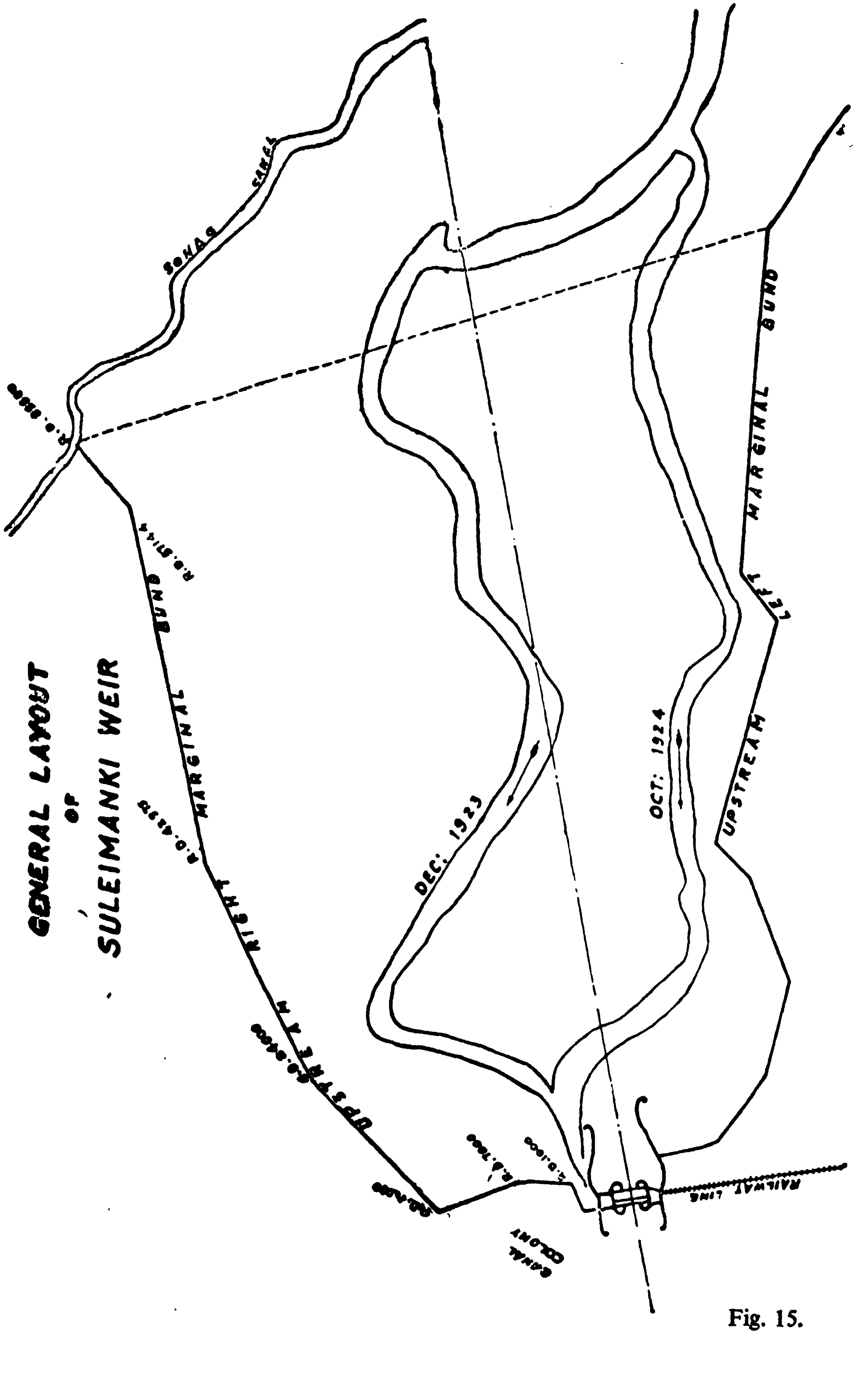

GENERAL LAYOUT OF SULEIMANKI WEIR
RIGHT UPSTREAM MARGINAL BUND
LEFT UPSTREAM MARGINAL BUND
DEC: 1923
OCT: 1924
RAILWAY LINE
CANAL COLONY

Fig. 15.

the main approach bank. In specially wide *khadirs* this may involve the use of very long guide banks.

(*iv*) The length of the downstream part of a guide bank may be a tenth to a fifth of the length of the bridge, according to the judgement that may be formed as to the activity of the swirl or disturbance likely to be caused by the splaying out of the water on leaving the bridge, for the swirl, if there is one, must be kept sufficiently far away so as not to endanger the approach banks.

(*v*) The radius of the curve of the downstream end of the guide bank may be such as the material trains can run on, say 200 to 600 feet ; because it is convenient to take the stone service line by this route.

(*vi*) The radius of the upstream curved part of the guide bank may be any thing from 500 to 1000 feet, according to the estimate of the probable velocity of the current past it. The curve should be carried well round to the back fully 120° to 140°.

Free board for guide bank should be 3·0 feet to allow full afflux head above the highest flood level. The thickness of the stone in slopes and aprons should be as given in the last chapter paragraph 17.

In the case of the Sulemanki head works the guide banks curve inwards forming bottle neck, the width between the noses being 1600 feet compared with 2223 feet between the weir abutments. The object was to induce central flow in floods and to prevent the formation of shoals. This object has not been attained, because the island formation on this headworks is as acute as on the others, but on the other hand, this has resulted in a strong flow in floods, round the guide wall towards the undersluices with the likely effect of damaging the bed protection upstream in the pocket. The marginal bunds with the protective spurs are required at all heads in addition to the guide banks. A typical layout is shown in Fig. 15 as used in the case of Sulemanki weir.

The underlying idea in the design of these works is that, firstly, a partially controlled and restricted flood by the marginal bunds or retired embankments should approach the river section between the guide banks, and secondly, they should protect the country upstream of the weir from river spills due to afflux caused by the construction of the weir. The design and layout of such protection works has already been dealt with in Chapter III. The free board is kept on similar considerations as for the guide banks.

15. Movable Weir Crests or Shutters.

(*a*) **Object :—**

The object of placing a movable crest on the permanent crest of a weir is to secure the maximum fair weather supply level required for the canal, and the minimum high-flood level, so as to lessen flooding, the height of flood embankments and the danger of outflanking. When these two levels correspond, the greatest efficiency of movable crest is obtained.

Another advantage of a movable crest is that it lessens the silting up of the backwater pool, which if it occurs excessively, may interfere with the supply to the canal. Movable crests are, however, very expensive, and may be difficult to work as their cost and manipulation increase greatly with their height. It is generally advisible to make them low, and to depend upon the solid weir to obtain the balance of the elevation necessary.

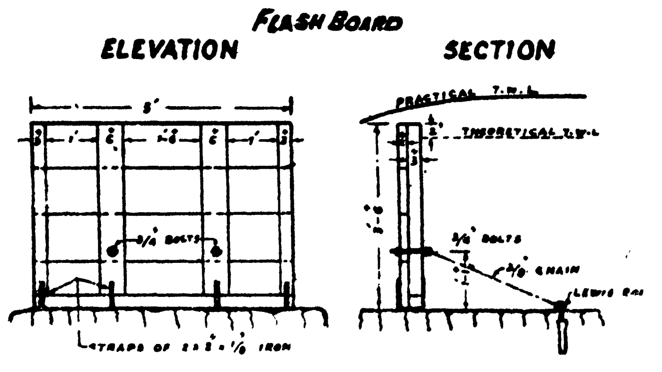


Fig. 16.

Movable crests may be divided into following categories :

(*b*) **Flash Boards.**

They are usually of wood as shown in Fig. 16.

The ordinary flash board is suitable

for all cases where the head held up does not exceed 5 feet, although, if the height is more than 4 feet shutters will probably be more efficient. This type is simple in construction and can be used in all cases. Even although boulders may damage the flash boards and their spports, these are cheap and can be easily replaced. Flash boards are, however, difficult to replace untill the water has fallen to the level of their base, and are therefore best adapted to high dams or escape weirs over which the water flows but rarely.

(*c*) **Hinged Shutters.**

These shutters are iron gates hinged at the base on the weir crest. The hinging or other guiding arrangement at the base of the shutters is a disadvantage, because many stones of fist size or greater, carried by most of the streams, are liable to lodge in the spaces in which the hinges and guide or spporting rods, work.

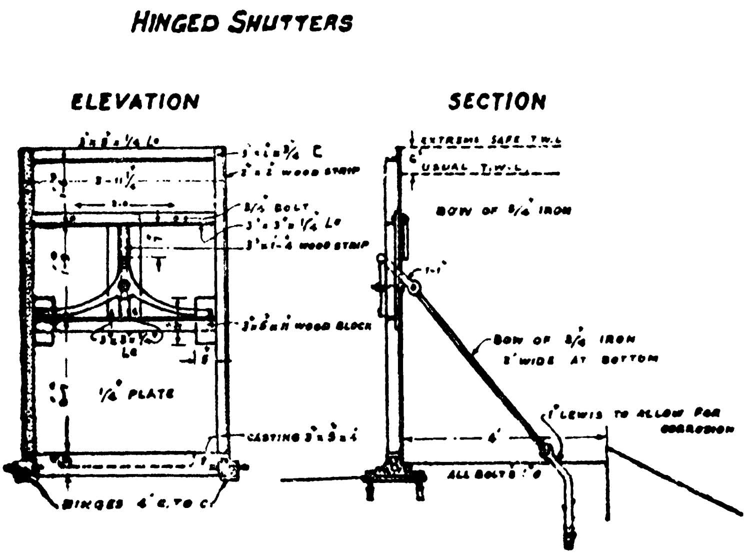


Fig. 17.

Fig. 17 shows a typical shutter used in India, which is dropped by releasing the curved horizontal lever. The raising of the shutter is effected by a hand crane which hooks on to the smaller (hanging) loop on the upstream face of the shutter. Trained men can raise and set these shutters with ease in 5, 6 or even 7 feet of water, provided that the backwater below the dams is not so high as to interfere with the working of the crane.

The dropping of these shutters is perfectly easy, provided that they are not over-topped. If once over-topped, it is almost impossible to get them down, although this has been effected with some risk when the river did not rise rapidly after over-topping the shutters. If the shutters are over-topped, and the rise of the water continues, the dam will probably be destroyed, not so much by the impact of the water falling over the shutters, as by cross-currents induced along the dam from the portion where the shutters are up, towards that where they are down.

(*d*) **Automatically Dropping Shutters.**

They are hinged Type Shutters up to 5 or 6 ft. height. As a rule the design consists of a projecting bevelled arm which is forced by the fall of one shutter and pulls out the key and sets the next one free. The arrangement is shown in Fig. 18.

They are raised by men with about 2 to 3 ft. depth of water, or raised by using traveling crane on the crest or from boats.

The shock produced by the simultaneous fall of a long line of shutters is a severe trial for a weir.

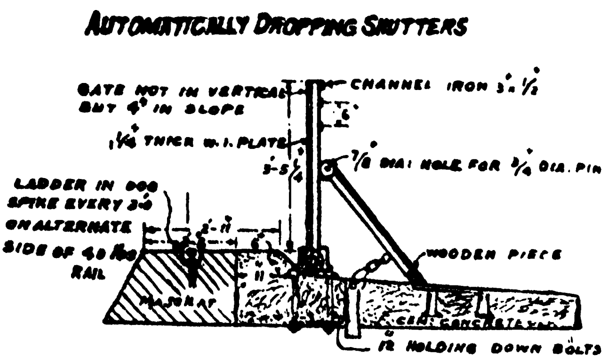


Fig. 18.

HINGED HYDRAULIC SHUTTERS

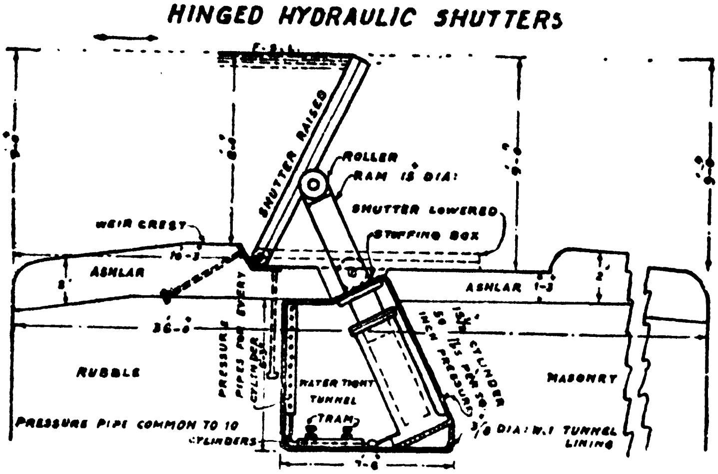


FIG. 19.

(*e*) **Hinged Hydraulic Shutter.** An expensive but reliable form of hinged shutter (Fig. 19) is worked by means of a hydraulic ram of which the plunger has a roller at its top bearing against the shutter, and is actuated by water under hydraulic pressure conveyed in a pipe in a water-tight tunnel formed below the weir crest. The shutter can thus be raised or lowered when desired, and is under complete control : it can be made so as to store up to 8 feet in depth.

(*f*) **Trestle Weir Crests.** This form is of French origin and has been made to store as much as 20 feet in depth but about one-half of this is a usual limit. The trestles (Fig. 20) are made of iron and are built up as lattices ; they are hinged at the base to the weir crest, to which they are normal, and are preferably spaceda part at a distance equal to the depth of water they store, so that they do not overlap each other when lying flat on the crest. When they have to be erected they are hauled up by tackle, and then needles are placed against them; the length and consequent weight of these determine the height of the trestle. Long needles may have to be lifted by machinery; their section and thus their weight, can be reduced by supporting them by removable wales hung on chains between the trestles. Some high trestles have wooden needles 2 to 3 feet wide stiffened by steel angle irons and are worked by machinery. Sometimes the trestles support horizontal shutters, and then, on account of the weight of these and the amount of water pressure on them, the trestles have to be spaced closer. This form has not been used inIndia. It is adapted to the case where there is a considerable depth of water in the river downstream.

TRESTILE WEIR CREST

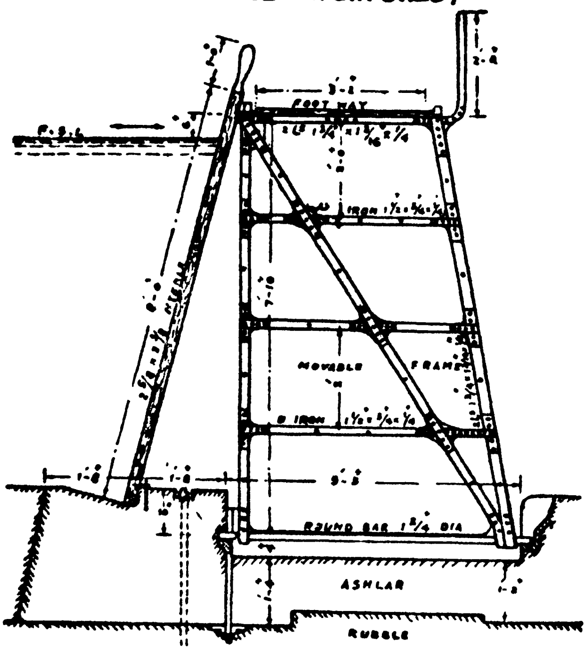


FIG. 20.

(*g*) **Machanical Weir Shutters.** In America a favourite type of mechanical weir is the "Bear Trap" of which there are several varieties. In its simplest form this consists (Fig. 21) of two levers pivoted at the base and working in a closed chamber ; these when raised for securing storage, make a triangle with the weir crest. To raise the leaves, water is admitted under the downstream one which is thus lifted and carries the upstream leaf with it ; to lower them a downstream valve in the chamber is opened and as the water level decreases, the leaves fall and lie flat in a recess on the bed. As the

BEAR TRAP WEIR.

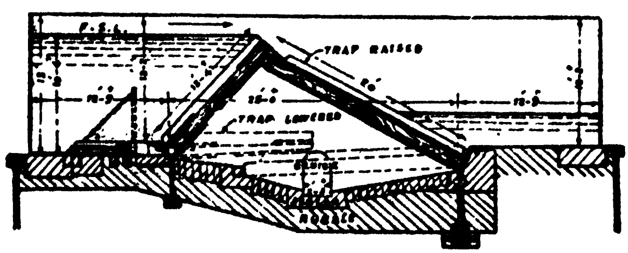


FIG. 21.

upstream leaf has no upward pressure on it, it is sometimes made in two parts with a loose "idler" covering the upper one to keep out floating debris ; in other forms the upstream leaf is joined. Formerly the leaves were made of wood, but recently they have been built up of iron plates and frames in lengths up to 70 to 80 feet, and are braced to secure rigidty. One objection to this design is its cost, as the total cross-sectional length of the shutters is about three times the storage depth ; another is that there may not be enough natural head available to raise the trap-artificial pressure is then supplied to overcome this defect. Bear traps have however been successful in use for many years and have not given trouble.

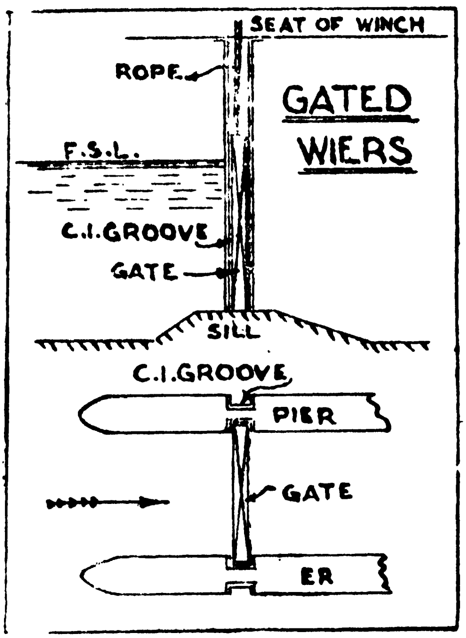


Fig. 22.

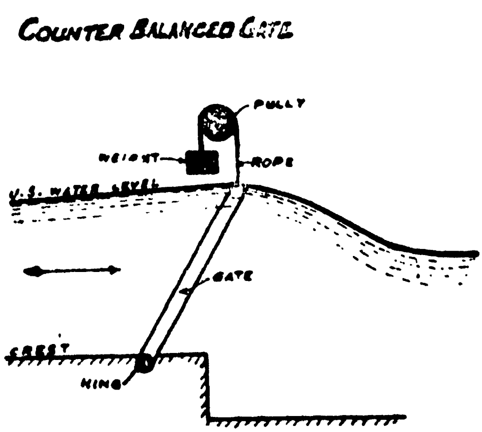


Fig. 23.

16. Gated Weirs.

(*a*) **Gates moving vertically in grooves.** These Weir gates are usually made of wood for small spans say 10—12 ft. or these are fabricated from steel for large spans, and raised or lowered by means of winch or a travelling crane. These gates usually operate in cast iron grooves embeded in the masonry of the piers. The general arrangement of this type of gate is shown in fig. 22.

(*b*) **Counter-balanced Gate.** This type of gate consists of a gate hinged at the bottom of the channel and counter-balaneed as shown in fig. 23. This type of arrangement has been used in the case of Hydro-electric schemes when excess had to be passed over. When the pressure on the upstream side of the gate increases due to increase in the upstream water-level, the gate is made to tilt with its fulcrum at the hinge, and it permits excess water to pass over when it is automatically back to its normal position by virtue of the counter-weight.

(*c*) **Walton Gate.** These are provided with a hinge at the bottom about which the gate revolves as its fulcrum. At the top of the gate a projecting lid about 2—3 ft. with a slope usually 1 to 5 is provided. This slope helps in smoothing out the path of the fluid in as much as it prevents presenting a sharp edge. This design has been found to give a high co-efficient of discharge which is about 3·6 as in the Weir formula. These gates are counter-balanced and are operated through a winch fig. 24.

(*d*) **Sluice Gate with Rollers.** Fig. 25. In the case of large gates or gates subjected to high fluid pressure, the frictional forces between the gate and the grooves in which it slides, are of sufficient magnitude to prevent the movement of the gate. In order to facilitate the working of the gates, the rollers are interposed between the groove and the gate in order to avoid sliding friction. The usual method is to mount the roller on the gate by means of a roller box and the roller rotates through a pin which passes through the roller box.

(*e*) **Stoney's Arrangement.** When, however, the pressure is very considerable due to a great head of water or the exceptional dimensions of the opening, another arrangement, *viz*,, that of free rollers, is made use of. These rollers run on axles, but there is no pressure induced on the latter, the axles are only used to keep the rollers at the proper distance apart, and are fixed in a frame. This roller cradle is suspended in the groove unattached to the gate. As the side of the gate bears against the rollers, when the former is moved, the rollers revolve and the whole frame of rollers rises and falls with the gate. The friction induced is pure roller friction which between smooth surfaces, is something very small indeed, so much so that its co-efficient can be entirely ignored, the lifting power being only the weight of the gate plus the friction of the lifting apparatus. Again under suitable conditions, the gate can be counterpoised by weights hanging from overhead pulleys, and in such cases gates of the largest size can be manipulated by one man. These free rollers are termed "Stoney's Patent Antifriction Rollers".

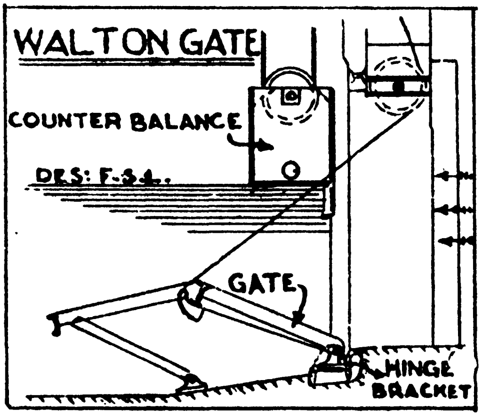


Fig. 24.

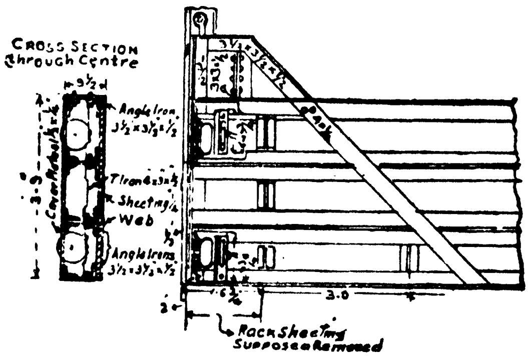


Fig. 25.

The diagram Fig. 26 represents a sluice gate fitted with free rollers in sectional plan and elevation. It will be seen that the frame of the gate on plan just clears the sluice wall, and projects beyond the groove. These spaces are closed by staunching rods. These are simple round rods of about 2 inches diameter, which are fastened to the gate, but have lateral play, so that when the gate is under pressure these are forced into the corner, effectually closing the

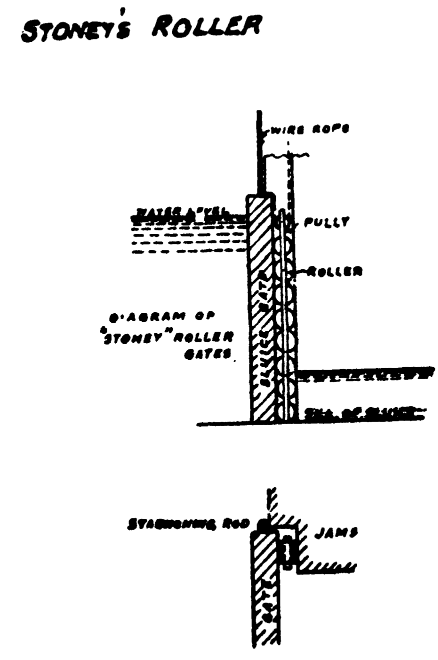


Fig. 26

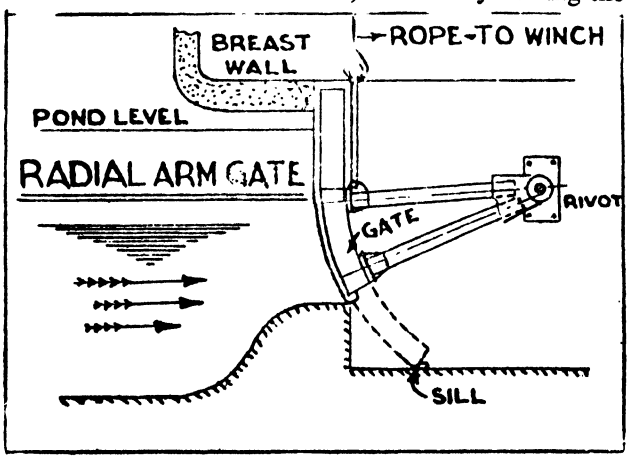


Fig. 27

aperture. On the gate being raised they are carried up with it, still bearing against the side of the gate and the sluice wall, the friction thus induced is, however, quite trifling. Owing to the suspending pulleys the roller frame moves half as fast as the gate.

(*f*) **Radial Arm Gates.** This type of gate is illustrated in fig. 27, and is hinged at a suitable point as shown in the drawing. It is raised or lowered by means of a winch, and this arrangement is used where the under-shot method of regulation is desired. This type of gate, therefore, finds general

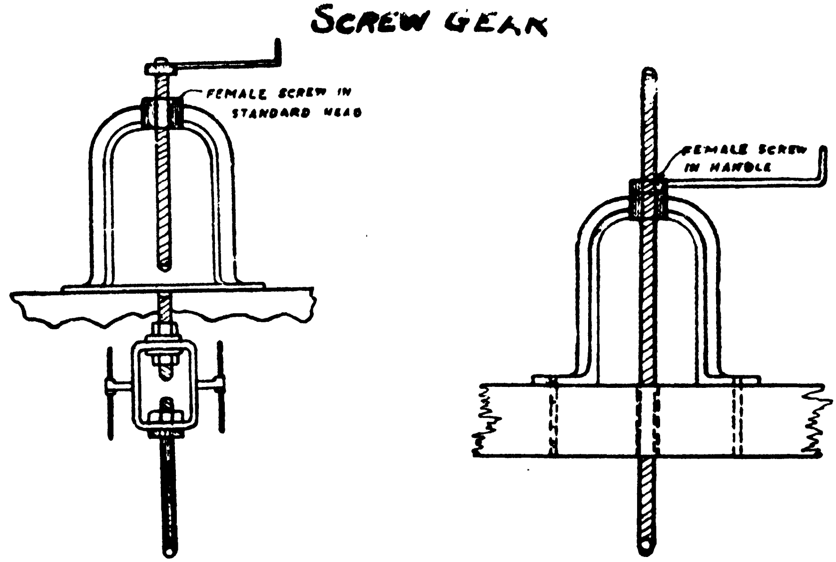


Fig. 28.

application in the regulation of Hydro-electric channels. The usual devices to reduce friction against the end of the gates usually fail due to rusting or debris catching in the rollers. The modern practice is to provide Radial Arm Gates. In the case of canal regulators, the gate is used behind the opening. It has no side grooves or roller bearing but is pivoted above downstream water level as shown in fig. 27.

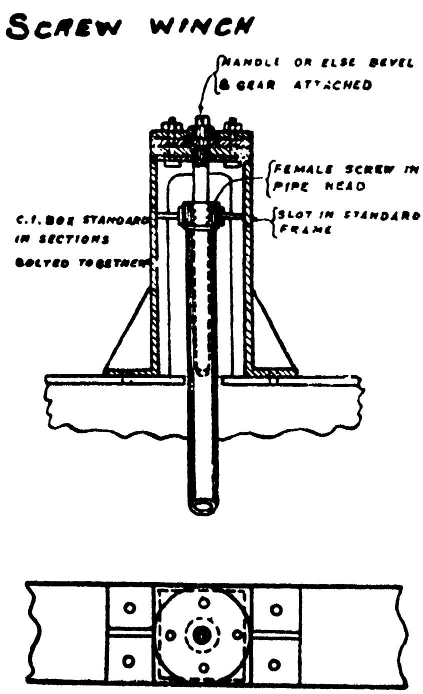


Fig. 29

17. Gate Lifting Arrangements.

(*a*) **Serew Gear.** A simple arrangement of screw gear is shown in fig. 28. This arrangement is used for lifting small gates with small lifts. The gate is suspended from a rod which is screwed at its upper end, and is raised or lowered by means of a nut to which is attached a handle or a hand-wheel. The screw is raised or lowered by rotation of the hand. wheel or the handle.

(*b*) **Screw Winch.** The usual winch used for lifting minor and distributary gates is sketched in Fig. 29. In this the power is applied to the male screw which can be quite short, a little over the lift in length. As shown in the illustration, the thrust collar and plates are situated at the rod head and consist of three plates, the upper, the distance plate, and the base plate. The screw is threaded through the base plate, while the upper plates are superimposed, and the whole bolted through as shown on plan (Fig. 29). The upper and lower plates should be of brass, the middle distance plate, which is subjected to no strain of friction, being of iron. The solid screw rod passes in to a pipe in the head of which is the female

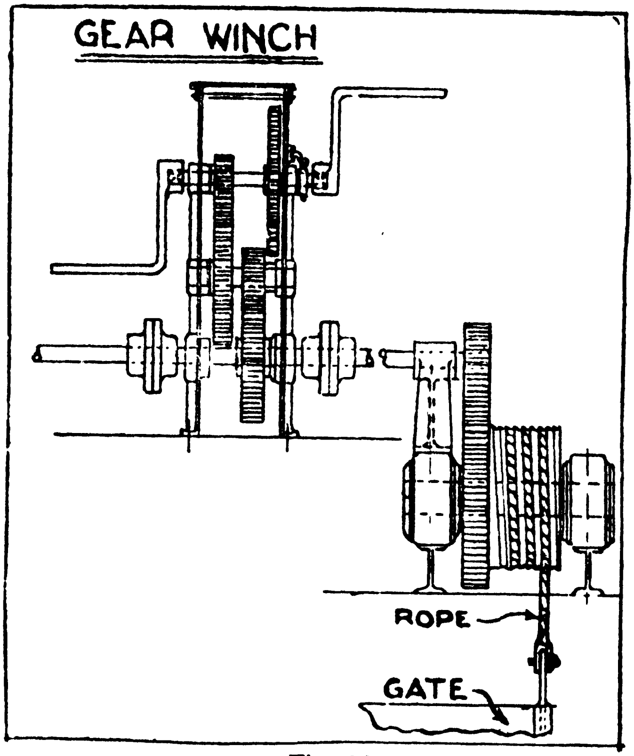


Fig. 30.

screw. This should be of brass and in some cases, the whole pipe is made of brass but this seems a needless extravagance. The pipe head is held rigid by two arms running in guide slots cut in the frame of the standard. Thus the torsional stress is at once absorbed and the position of the ends of the arms, one of which can be made to project beyond the frame, indicated by means of a pointer on a graduated scale, showing the exact height at which the gate stands above its cill. The pipe is rigidly connected with the gate and from its ring section is clearly much better suited to withstand compression than a solid rod of greater weight of metal. It can also be made of ordinary gas or water piping. The introduction of ball bearings above and below the thrust discs would be a further improvement.

(*c*) **Travellers.** In some canal the gates are not fitted with lifting arrangement individually, but the regulator is provided with a traveller winch, which is used for raising or lowering the gates when necessary. This travelling winch consists of a winch fitted over a trolley,

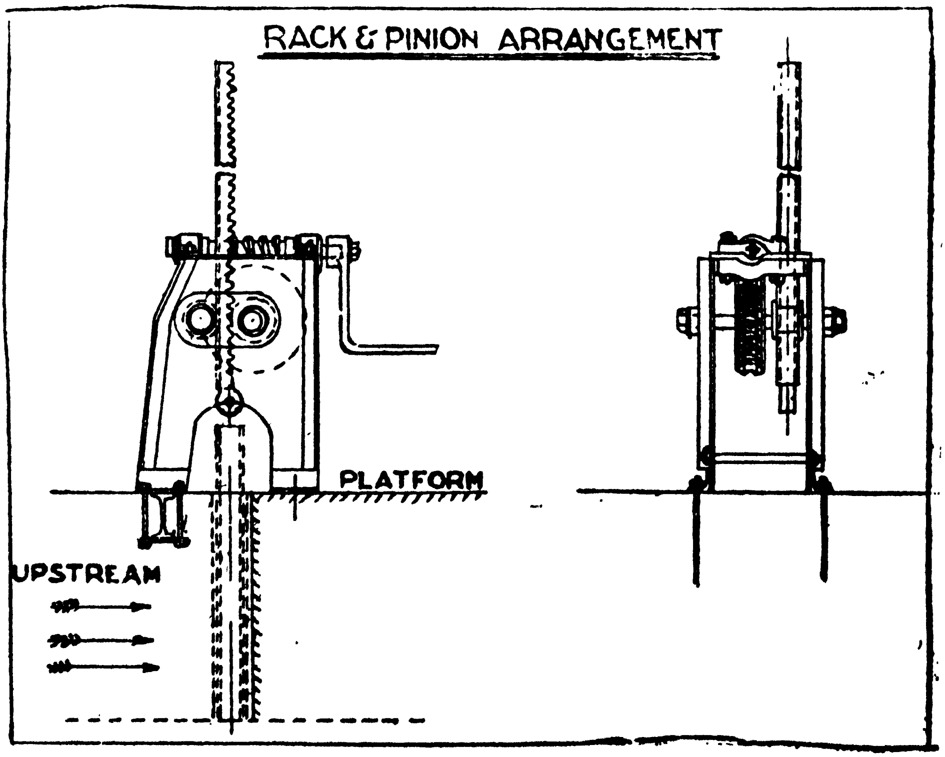


Fig. 31.

moving on rails over the regulation gates, and this trolley is brought over the gate to be operated upon. The winch consists of a simple reduction gear and a drum carrying a chain or wire rope for raising or lowering the gates.

(*d*) **Gear Winch.** In the case of large and heavy regulation gates it is desireable to provide each gate with its own lifting arrangement. In this case the gate is suspended from rope-drums, winch, in turn, are operated through a series of reduction gears as shown in Fig. 30.

(*e*) **Rack and Pinion Arrangement.** This arrangement is illustrated in Fig. 31. and is generally used for Distributary Head Regulator Gates. The lifting arrangement consists of a worm and worm wheel reduction gear, to which is attached a pinion which works against a rack attached to the gate. The rotation of the handle causes the pinion to rotate, and these raise or lower the gate depending on the direction of rotation of the handle.

PART II

CANAL IRRIGATION

CHAPTER V

Design Of Weirs On Permeable Foundations

1. Introductory.

Practically all the big canals of Northern India take off from rivers after they have left the hills. Rocky foundation are not available for the weirs constructed across them to pond up supplies for the canals. The weirs are usually of small height, 10 to 15 feet (as distinguished from the dams) and are constructed in the permeable river-bed. The irrigation engineering in India occupies a very conspicuous and poineer position in the experience of the design and construction of such works. The science of the designs of weirs on permeable foundations have developed by leaps and bounds in India as the student will find in the subsequent paragraphs in this chapter.

2. Development of the Theory of Design.

(*a*) The law of flow of water through permeable soils was enunciated for the first time in 1856 by H. Darcy who, as a result of experiments, found that the velocity of flow varied directly as the head and inversely as the length of the path of flow. This law is expressed by the equation.

$$V=k\frac{H}{L}$$

Where V = Velocity
H = Head
L = Length of the path of flow
k = A consant called the "transmission constant."

Darey's law is of the same form as the law for the movement of water in capillaries enunciated by Poiseuille in 1841-42.

The validity of this law in relation to weir design was tested by Col. Clibborn in 1896 in connection with the proposals for repairs to the damage to the Khanki weir on the Chenab river in 1895. This weir which feeds the Lower Chenab Canal, was completed in February 1892. In January 1895, 100 ft. of the weir crest in Bay No. 1 subsided by about 2 ft. This was the first major weir to be constructed on the alluvial bed of a Punjab river, and this damage occurring so soon after the construction, gave food for thought to the engineers responsible for its construction and maintenance. But this investigation, though affording the first rational basis for design, embracing as it did the conception of failure by undermining and by uplift due to the flow of water through the sub-soil of the weir, did not materially add to the knowledge for the purposes of practical design. It gave only qualitative indications which, however were generally in the right direction.

(*b*) **Hydraulic gradient theory.**

The Hydraulic Gradient Theory for Weir Design, apparently originated between Sir John Ottley and Thomas Higham and was developed as a result of experiments by Col. Clibborn (1895–97).

With the publication of the results of Col. Clibborn's experiments in 1902, the Hydraulic Gradient Theory came to be generally accepted in India. The following passage from Buckley's Book "Irrigation Works In India" (1905) Page 175 will bear testimony to this :—

"It has been maintained that, in those cases where the chief danger to a weir is from under scour and not from parallel currents, the true measure of security of a weir in a permeable bed is the distance through the soil which a current of water would have to travel before it could rise up below the weir, and that it is of little consequence whether masonry is laid horizontally on the weir bed or sunk vertically below it, so long as the currents passing through the soil below the structure are exposed to the friction of the same length of passage. This view appears to be sound, but it is essential to attach to the application of this principle, the condition that the weir must be protected from horizontal scour on the face of the toe, that it must have sufficient weight to resist the horizontal pressure of the head it supports, and also sufficient weight to oppose the upward pressure in the base of the foundations which that head may produce and, further, that the surface of it which is exposed to erosion should be of material sufficiently hard, and sufficiently heavy to resist that erosion."

(*c*) **Creap Theory of Bligh.**

In 1910, W. G. Bligh enunciated his creep theory in his book "Practical Design of Irrigation Works" and elucidated further in his book "Dams and Weirs" 1918. He did no experimental works, but analysed the failure of the Narora and the damage to the Khanki weirs. According to this theory, the creap was more important and destructive to undermine a weir than the percolation through the soil below it. The enginneers having the experience of closing breaches must have felt the truth of the statement when they find that it is most difficult or rather impossible to close a leakage by the side of a log of wood or roots in the soil. A bank of a distributing channel, otherwise strong, becomes unsafe if out-let walls be constructed right across it and it needs a special effort to make the joint between soil and masonry effective against creep. Bligh assumed the percolation water to "creep" along the contact of the base profile of the weir with the sub-soil, losing head enroute, proportional to the length of its travel.

The length of travel in a weir profile as given in Fig. 1, would be :—

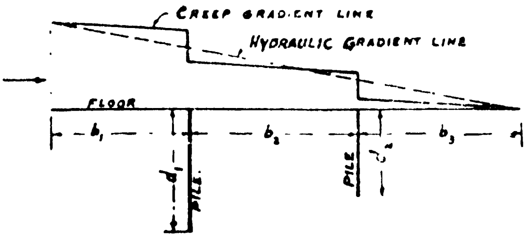


Fig. 1.

$$L = b_1 + d_1 + d_1 + b_2 + d_2 + d_2 + b_3$$
$$= b_1 + 2d_1 + b_2 + 2d_2 + b_3$$

If H = total head over the weir the loss of head per unit lenght of creep would be :—

$$C = \frac{H}{b_1 + 2d_1 + b_2 + 2d_2 + b_3}$$

He called this loss of head per unit length as Creep Co-efficient and assigned a safe value of C for different classes of soil. Thus, if in a given weir design, the value of C were less than the safe value assigned to it for the given class of soil, the design would be considered safe. The following were the values of C recommended by Bligh. (Page 155 "Dams & Weirs")

Class I : River beds of light silt and sand, of which 60% passes a 100 mesh sieve, as those of the Nile or the Mississppi, percolation factor ; C = 18.

Class II : Fine micaceous sand of which 80 per cent of the grains pass a 75 mesh sieve, as in the Himalayan rivers and the Colorado ; C = 15.

Class III : Coarse-grained sands, as in the Central and South India ; C = 12.

Class IV : Boulders or shingle, and gravel and sand mixed ; C varies from 9 to 5.

Because of its simplicity this theory found general acceptance. Some works designed on this theory failed while others stood, depending on the extent to which the then engineers ignored or took note of the importance of vertical cut-offs at the upstream and downstream ends.

(*d*) **Electricity Flow Analogy.**

Pavlovsky (1920) approached the problem of the flow of water through sub-soils of hydraulic structures from the analogy of flow of electricty through a conductor. According to Ohm's law :—

$$\text{Current } C = \frac{\text{Potential difference}}{\text{Resistance}} = \frac{E}{L} \cdot \frac{A}{\rho} = B \cdot \frac{E}{L}$$

Where E = Potential difference
A = Area of section of conductor
L = Length of conductor.
and ρ = Specific Resistance of the material of the conductor. This is identical, with Darcy's equation for flow of water through sand *viz.*,

$$V = k\frac{H}{L}$$

The work was published in Russian. Pavlovsky achieved success in solving a number of problems, but as the laboratory results could not be shown to agree with the field results, this method did not inspire confidence among the engineers and remained more or less of academic interest. The results obtained on the Panjnad model in conjunction with the researches of Dr. Vaidhianathan on the electric models have shown conclusively in Vol. No. 4, Irrigation Research Institute, Lahore, for the first time in the history of research on this subject that :—

(*i*) The distribution of pressures under works on sand foundations can be exactly reproduced on hydraulic or electric models.

(*ii*) All seasonal and other variations from the normal conditions can be reproduced on Hydraulic models by superimposition of silt, temperature, or both, and by simulating the stratification.

(*iii*) The problem is susceptible of mathematical treatment.

The conclusive proof about the reliability of model and mathematical results in application to field conditions marks a great advance and a definite land-mark in the development of this branch of engineering which has led to a final solution of the problem of design of weirs on permeable foundations.

(*e*) **Khosla's Investigation in the Punjab.**

In 1926-27 trouble at the syphons under the Upper chenab Canal (Punjab) became acute. Cracks appeared at the upstream and downstream ends due to the undermining of the sub-soil. Repairs were carried out on the accepted Bligh theory, but the trouble persisted. A set of pressure pipes with well points were inserted in the floors of two of these syphons and the observations disclosed that the pressures indicated by these pipes had absolutely no relationship with those calculated from the Bligh Theory. These researches were carried out by Khosla and are embodied in the Punjab Engineering Congress Papers Nos. 138 and 142 of 1930. The simple creep theory of Bligh was repudiated and some provisional and important conclusions were arrived at, notable among which were :—

(*i*) The outer faces of the end sheet piles were much more effective than the inner ones and the horizontal length of the floor.

(*ii*) The intermediate piles if smaller in length than the outer ones were ineffective except for local redistribution of pressures.

(*iii*) Undermining of the floors started from the tail end. If the hydraulic gradient at exit was more than the critical gradient for the particular sub-soil, the soil particles would move with the flow of water thus causing progressive degradation of the sub-soil, resulting in cavites and ultimate failure.

(*iv*) It was absolutely essential to have a reasonable deep vertical cut-off at the downstream end to prevent undermining.

(*v*) There was an urgent necessity for research work in the laboratory with regard to pressures under the existing and the new structures. The former could be done by inserting suitable located pressure pipes in these structures and by maintaining a continuous and comprehensive record of the observations of pressures form those pipes.

In 1929, it was decided to extend the Panjnad Weir (Punjab). This afforded the opportunity of putting in a comprehensive set of pressure pipes and of conducting full scale experiments as suggested in the Punjab Engineering Congress Papers 138 and 142 of 1930. This was the first full-size experiment in the world, and the results obtained from it (1932) paved the way to the final solution of the problem. The main conclusions of 1928-29 derived from researches at the Upper Chenab Syphons were confirmed and the following facts were established :—

(i) The flow of water through the sub-soil is in stream lines and therefore susceptible of mathematical treatment.

(ii) The ratio (ϕ) of uplift peresure (P), at any point along the base of a particular weir founded on premeable soil to the total head (H) is constant and independent of :—

(1) Head (H)

(2) Class of sub-soil so long as it is homogeneous.

(3) Upstream and downstream water levels.

(4) Temperature, provided it is uniform throughout the sub-soil ; but it varies with :—

(i) Silt deposit or scour upstream or downstream of the impervious floor.

(ii) Temperature which varies from point to point in the sub-soil and in different seasons of the year.

(iii) Law of loss under the floor was nearly a straight line and far the sheet piles something like logarithmic.

Sir R. P. Hadow suggested that the field results be tested on a scale model in the Lahore Research Laboratory. It was not till late in 1934 that a model of the Panjnad weir was set up. This investigation was carried out by Dr. Harbans Lal Uppal following Hele Shaw method and the findings were presented at the Punjab Engineering Congress in February, 1935 (Paper No. 185). The results clearly vindicated the correctness of the field observations made by Khosla at Panjnad.

The results of the field investigations made by R. B. A. N. Khosla, I. S. E., Punjab, supplemented by the work of Dr. Vaidianathan by electric methods and the work of tracing stream lines by Hele Shaw method by Dr. H. L. Uppal of the Irrigation Research Institute, were merely qualitative. Prof. Warren Weaver, Head of the Department of Mathematics at the University of Wisconsin and at the time working with the Rockfeller Foundation, developed his mathematical treatment of the flow of water through the permeable sub-soils under dams. Weaver's work, as will be seen later, provided the inspiration for the complete solution of the problem, which has been summarised in this chapter from Central Board of Irrigation, India Publication No. 12.

(*f*) **Weighted Creep Theory.**

In September, 1932, Lane analysed over 200 dams, all over the world, and evolved his Weighted Creep Theory, which in effect might be called the Bligh Creep Theory corrected for vertical cut-offs and slopping faces. Lane proposed a weight of three for vertical creep and one for horizontal creep. While this theory was an improvement on the original Bligh Theory, it was empirical and lacked the background for a rational basis for design, His memorandum on the subject appeared later, as Paper No. 1919, in 1935 Transactions of the American Society of Civil Engineers. This paper is valuable for the wealth of infomation about the numerous weirs it deals with.

(*g*) **Exit Gradient Theory.**

In 1929 a notable contribution came from Karl Terzaghi (U. S. A. and Vienna). He stated and proved by laboratory experiments that failures occured by undermining, if the hydraulic gradient at exit was in excess of what he called the "floatation gradient". This was the same as the critical gradient enunciated by Khosla, but it was more explicit in so far that it implied a state of floatation of the soil mass at the toe of the work if the exit gradient there exceeded the limit 1 : 1 at which limit the upward force due to the flow of water was almost exactly counter-balanced by the weight of the soil.

In 1935 Haigh (Paper No. 182 Punjab Engineering Congress, Lahore) and Harza (Paper No. 1920 American Society of Civil Engineers) independently produced two very useful papers on almost similar lines. Fundamentals were once again introduced and a definite attempt was made to break away from empiricism and to lay a rational basis for design. They took note of the exit gradiends as a controlling factor in the stability of weirs and discussed the distribution of pressures which could be considered as safe. Harza got agreement between theoretical values of uplift pressures and those obtained from the electric models for some of the simple cases dealt with by Weaver in his mathematical sections.

(*h*) **Failures by the uplift pressure due to the formation of the jump on the downstream apron.**

The worst condition for the sub-soil flow occurs when the downstream bed of the river is dry and water is headed up to the pond level. This received attention of almost all the investigators as shown above. The failure due to the uplift pressure also occurs in maximum floods in the region of the trough of the hydraulic jump formed on the downstream apron. The shaded area in Fig. 2. shows the amount of uplift pressure which is of the order of 10 to 15 feet in the case of large rivers of the Punjab. This causes damage in two ways, firstly by raking out the

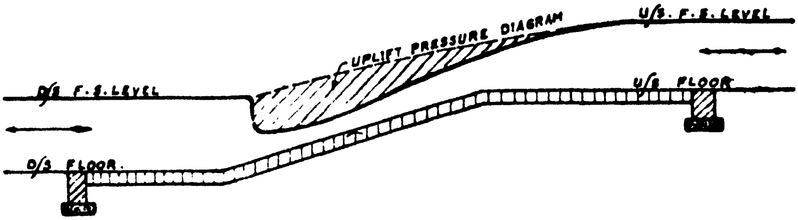


Fig. 2.

joints of the masonry floor and then lifting the *khwanja* and secondly by lifting the floor from below the bottom profile of the weir. The damage due to the first method is usual, but due to the second it has never been experienced, because before the full effect of the uplift pressure is felt, the flood subsides. This factor as influencing the design of modern weirs is dealt with in detail in the author's Publication No. 9. (Indian Engineering, October, 1936).

(*i*) **The pressure differences due to the river acting as a sink.**

Yet there is another very important factor determining the stability of a weir which has escaped the notice of the various investigators. The river and seepage drains act as strong sinks collecting the seepage flow. The pressure differences causing the flow produce uplift pressure if the bed be covered with impervious floor and also cause strong floatation gradients in the open bed. If a pipe provided with a filter point be sunk progressively in a river or seepage drain bed, the water level in the pipe rises, but after a sinking of 15 to 20 feet there is usually no rise in the water level on the further sinking. The pressure diagram becomes asymptotic to the highest water level recorded in the pipe. (This level has been definded to be the Basic Sub-Soil Pressure level in Chapter III Part V). The pressure difference thus recorded above the free water level in the river bed is the pressure determining the minimum thickness of the gravity floor in the river bed, and is a measure to determine the floatation gardients. The author observed 200 feet away from the downstream end of the weir at Khanki, a pressure difference of the order of 3 to 4 feet. These forces came into play irrespective of the hydraulic pressure by ponding up water against the weir crest causing flow under weirs and should, therefore, be separately determined and allowed for in the weir design in river bed, and the design of the floors and bridges and the meterflumes on the drains.

8. Practical Weir Designs.

(*i*) **Rock fill Weirs.**—In the original rock fill weirs of the type at Okhla in fig. 3. (a) there was no curtain wall projecting below the bed. The stability of the sand foundations is entirely dependent on its weight and effective base length. These weirs were not water-right and in floods the downstream aprons were very much damaged and had to be repaired every year. Clay puddle lining of the fore apron very soon followed as in Fig, 3. (*b*) and top skin of the downstream apron was also laid in mortar as in fig. 3 (*c*) to protect the loose stone from being ripped up in floods. The necessity of deep curtain walls under the crest and other points lower down was soon felt and walls were introduced as in Fig. 3. (*c*) and 3 (*d*). Inspite of these improvements the early type of weirs could not be called permanent structures, because the excessive leakage continued and enormous annual repairs were necessary after the floods.

(*ii*) **Permanent masonry weirs.**--The first attempt with vertical crest and *pacca* floors downstream was made in the case of the Narora weir, fig. 3 (*e*). The downstream floor and the previous apron were soon damaged which pointed out the necessity of an impervious rear apron and the insufficiency of the base length. Vertical drop of 13 feet at the crest was considered to be excessive causing river action in the downstream talus. Hydraulic gradient theory was then generally accepted.

Bligh, analysing the failure of Narora, brought out his creep theory stating that a creep gradient of 1 to 15 was necessary. The weirs which followed were designed according to Bligh's conception. The subsequent weirs had sloping downstream aprons with very small drop 7 feet and 6 feet at Rasul and Marala provided in the sloping aprons 1 in 15 in both cases instead of vertical drop. The weirs were provided with shutters, but no raised crest. These weirs shown in Fig. 3 (*f*) and 3 (*g*) were provided with deep upstream curtain walls but both of them suffered from the undermining of floors due to insufficient base length. The apron at Khanki was only 108 feet long, while it should have been 195 feet, allowing creep coefficient of 15. The Marala weir has 3 well lines which are pretty deep but it is doubtful if they are water tight in between the wells because this weir has been constantly giving trouble due to undermining. Khanki and Rasul Weirs were rebuilt in 1934 and 1931 respectively. The failure was nearly complete by undermining due to the well lines not being water tight and non provision of sound end curtain walls for controlling the exit gradient. They were hollowed out in course of time and the collapse came in flood probably due to the uplift pressure caused in the trough of the jump. Marala weirs actually did not give away, but in 1930 floods, the *Kharanja* of the downstream floor was ripped in the region of the hydraulic jump. The pile lines upstream and downstream have now been provided and the sections rebuilt.

A beautiful example of Bligh weir is provided by the Ganges weir at Hardwar shown in Fig. 3 (*i*), There are shallow curtain walls both upstream and downstream and the, gravity floors are provided according to Bligh's conception. This weir has stood very efficiently the test of time. The unprecedented floods in the Ganges in 1935 left the weir very nearly unscanned.

(*iii*) Haveli Project Weirs in the Punjab.

Two of the Sutlej Valley Project weir sections are given in Fig. 3 (*j*) and 3 (*k*). The era of sheet piles started. At Sulemanki, there was a clay layer below the weir and it was considered enough to put only one line in the beginning. Panjnad weir Fig. 3 (*k*) shows three very steep sheet pile lines. They were gravity weirs and in some cases top *Kharanja* was replaced by lightly reinforced concrete slabs to withstand effectively the hydraulic jump action on the floor. Sutlej Valley Project weirs, 4 in number, stood extremely well except the one at Islam which collapsed due to the retrogression of river levels downstream.

A comparison of the last Bligh's weir in Fig. 3. (*i*) and those which followed as in Fig. 3 (*j*), 3 (*k*) and 3 (*l*), shows that the introduction of deep sheet pile line introduced new factors which did not exist in Bligh's weir had shallow curtain walls, 6 and 10 feet deep, upstream and downstream and even the inside corners were filled with concrete. Bligh's creep theory could very well apply to shallow curtain walls. He could never imagine that his successors would put in sheet pile lines 40′ to 60′ deep. It is grave in justice to say that Bligh's theory had been repudiated, or that his conception had been proved absolutely wrong. Naturally with deep cut-offs creep would be very nearly be impossible, the loss of head shall have to follow some other law. It has already been stated that R. B. A. N. Khosla, I. S. E., Punjab did the necessary work in this direction. His theory to calculate pressures is explained later in this chapter. Even now the law of loss under straight floors is very nearly a straight line just like Bligh, and it is only in the case of deep cut-offs that the change has come in.

(*iv*) Khosla Weirs.

The Panjnad and the Emerson barrage Fig. 3 (*k*) and 3 (*l*) are beautiful examples of Khosla's weirs. Emerson Barrage is the cheapest weir which has so far been constructed in India. It is not a gravity weir. The floors are of reinforced concrete slabs to resist the, uplift pressures and are weighed down by the Berrage piers.

Khosla has brought out clearly the importance of exit gradients which had been the great source of trouble in the past weirs and has supplied the engineering world with a correct method of calculating the uplift pressures with deep cut-offs, and of designing weir floors economically.

OKHLA ROCK-FILL WEIR

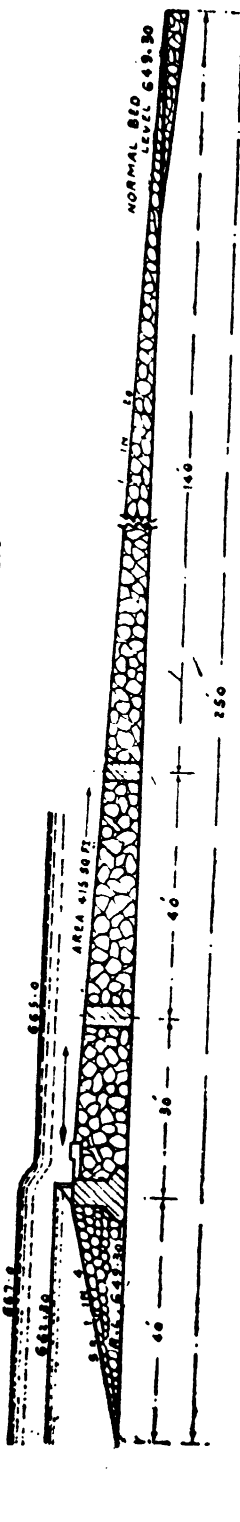


Fig. 3 (a)

MADAYA ROCK-FILL WEIR

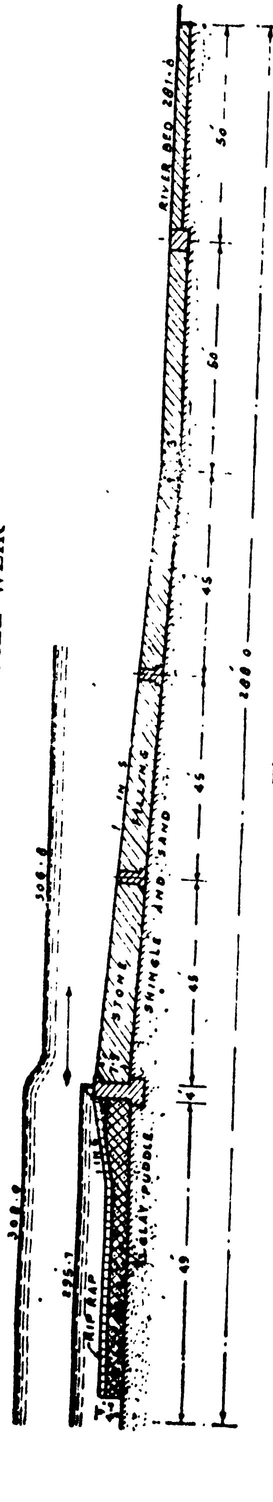


Fig. 3 (b)

Section of Dheri Weir Showing Steeper Gradient

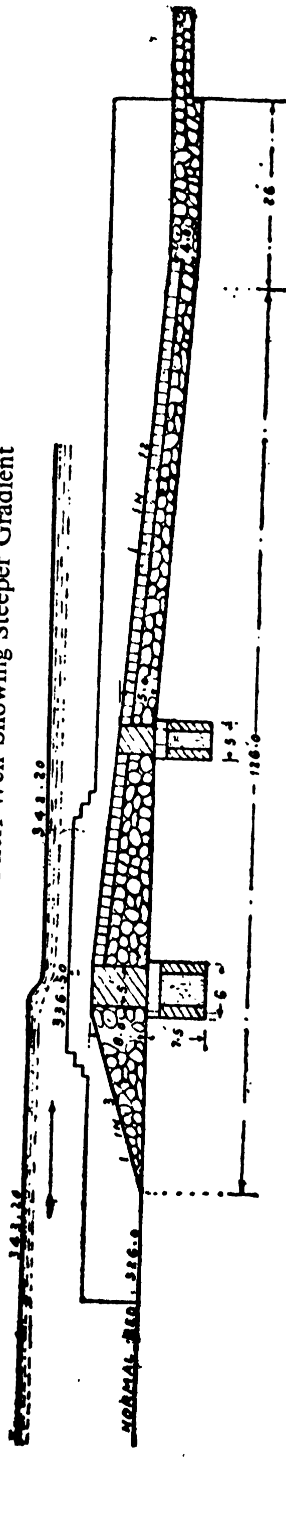


Fig. 3 (c)

Jobra Weir Mahanadi River

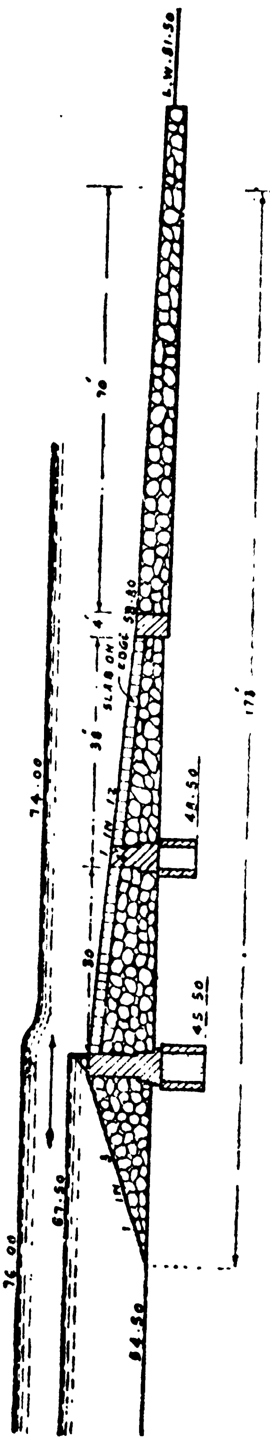


Fig. 3 (d)

Section through Narora Weir

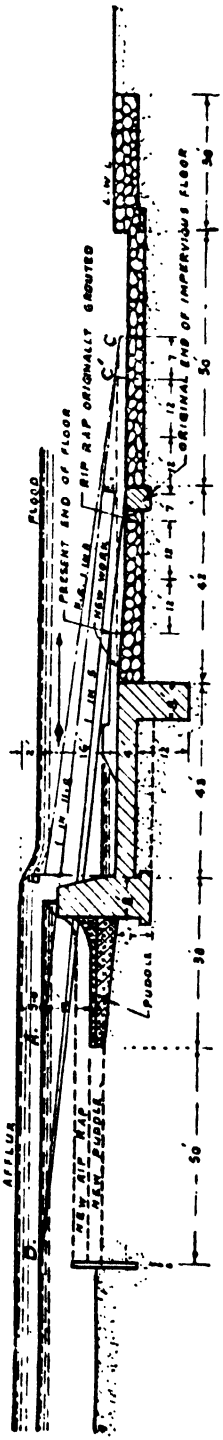


Fig. 3 (e)

Section through Chenab Weir at Khanki

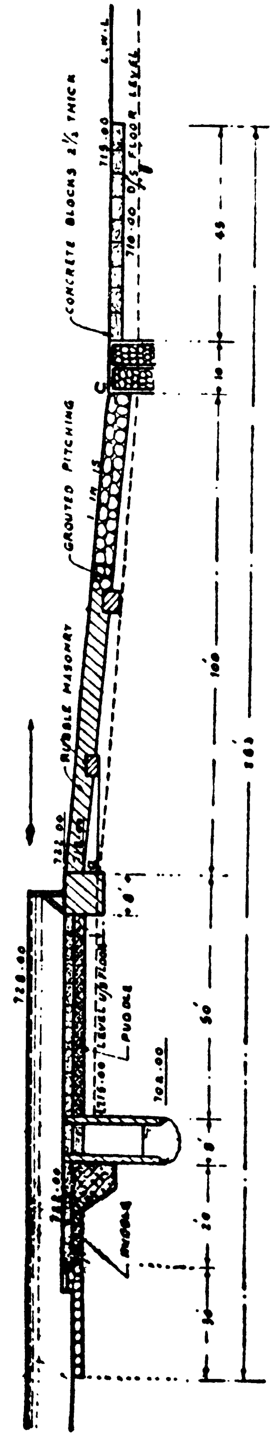


Fig. 3 (f)

Section through Jhelum Weir at Rasul

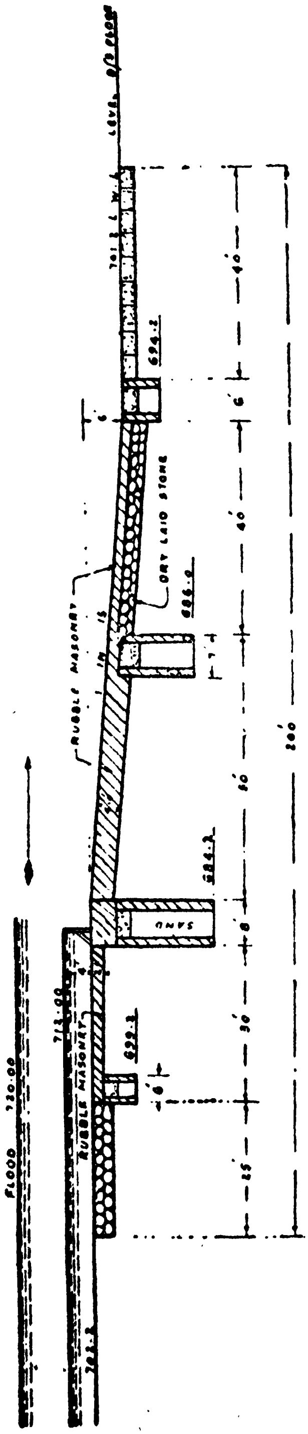


Fig. 3 (g)

Mrala Weir Section

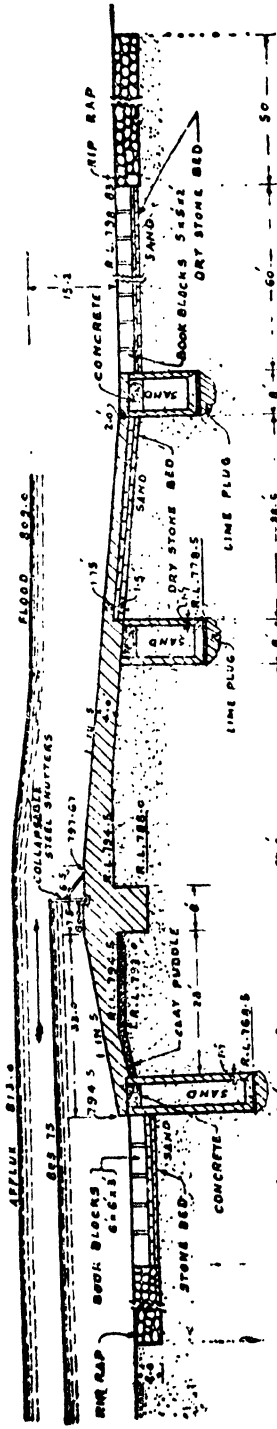


Fig. 3 (h)

Section of Bhimgoda Ganges Weir

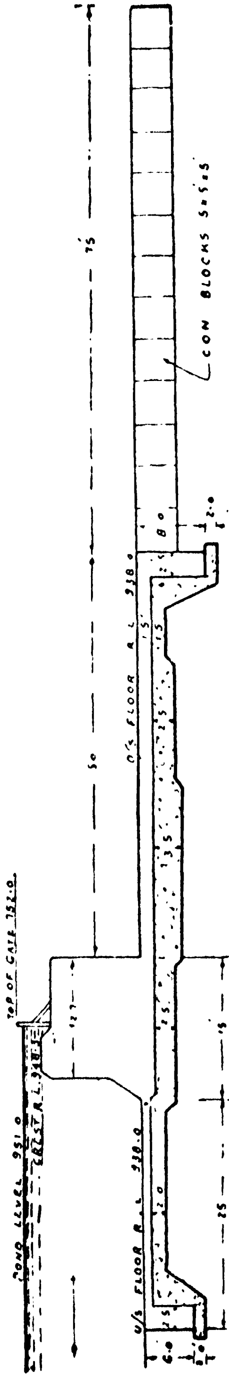


Fig. 3 (i)

Sulemanki Weir Section

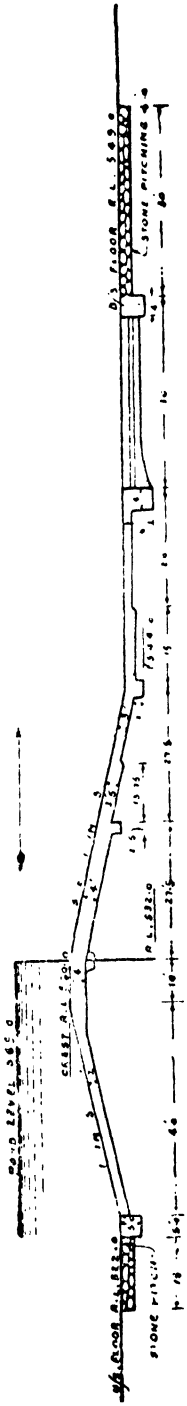


Fig. 3 (j)

Punjnad Weir Section

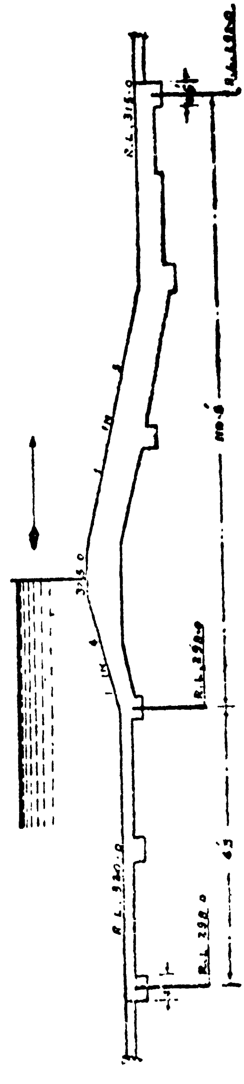


Fig. 3 (k)

Section of Emerson Barrage Weir (Haveli Project)

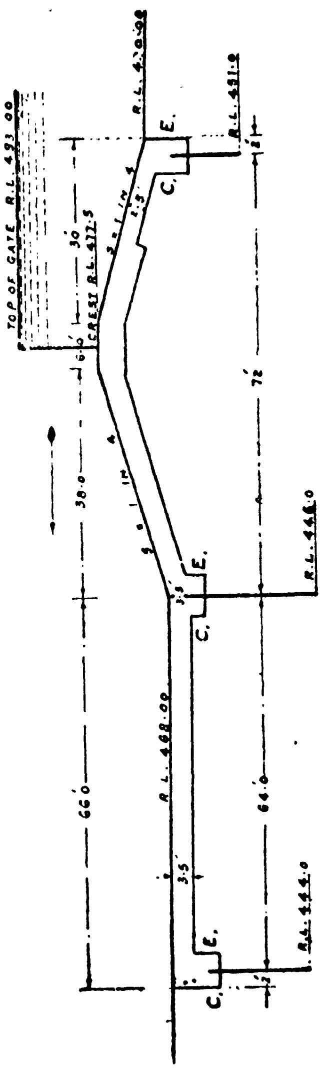


Fig. 3 (l)

4. Mathematics to determine uplift pressures.

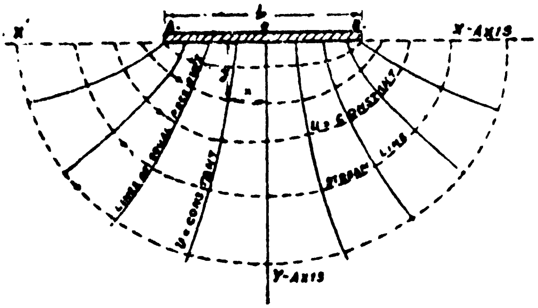


Fig. 4.

A very much abridged summary of the formula is taken from Chapter VII, Central Board of Irrigation, India, Publication No. 12 by R. B. A. N. Khosla, I. S. E., Dr. N. K. Bose, Ph. D. and Dr. E. Mchenzie Taylor O.B.E., D. Sc. The student should refer to the original book for the detailed knowledge of the subject and the proofs of the formulae stated herein.

It is generally known that the stream lines of flow under a floor AB as given in Fig. 4 are confocal ellipses with the centre at O, the middle of the floor AB $(=b)$ the major axis and with foci at A and B. One of the stream lines will be AB, which is the limiting form of this family of ellipses.

These ellipses are given by the equation

$$\frac{x^2}{\left(\frac{b}{2}\cos u\right)^2}+\frac{y^2}{\left(\frac{b}{2}\sin u\right)^2}=1 \text{...........(A)}$$

where u is stream line function.

From this equation any particular stream line can be determined by giving a suitable values to u, so that equation (A) will take the form :—

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

The stream line along the floor AB is given by putting

$$u=0$$

from which $\cos u=1$

$\sin u=0$

and equation (A) reduces to $y=0$, *i.e.*, the line AB.

The lines of equal pressure are at right angles to the lines of the flow and are in the above case given by

$$\frac{x^2}{\left(\frac{b}{2}\cos v\right)^2}-\frac{y^2}{\left(\frac{b}{2}\sin v\right)^2}=1 \qquad \text{...(B)}$$

where v is the pressure function.

From this equation any particular equi-pressure line can be obtained by giving suitable values to v, so that the equation (B) will take the form :—

$$\frac{x^2}{A_1^2}-\frac{y^2}{B_1^2}=1$$

At the upstream end A, $v=\pi$, the total head or pressure.

Substituting this value in (B) we get $y=0$, or that the full pressure line is AX′ which is horizontal and in plane of the floor at the upstream end.

The entire head or pressure $v=\pi$ is gradually lost along the direction of flow through the sand medium till it is finally reduced to zero at its exit along BX. The rate of loss is given by the spacing of the equi-pressure lines shown in Fig. 4.

If P is any point in the medium, and the pressure and stream line functions at this point are given by u & v, then the relationship (A) and (B) can be expressed as below :

$$Z=\frac{b}{2}\cos w \qquad \text{... (C)}$$

where $Z=x+\iota y$ and $w=u+\iota v$

The position of a point P in a plane (say Z-plane) is usually denoted by the two cartesian co-ordinates x and y. It is convenient to denote this number pair (x ,y) by the compound symbol $(x+\iota y)$, and this number pair is conveniently called (after Gauss) a complex number. In the fundamental operations of arithmetic the complex number pair $(x+\iota.0)$ may be replaced by the real number x defining ι to mean $(0-\iota.1)$ we have $\iota^2=(0,1)\times(0,1)=(-1,0)$ and so ι^2 may be replaced by -1.

$$\therefore \iota^2=-1$$

Substituting these values in (C) we get

$$x+\iota y=\frac{b}{2}\cos(u+\iota v)$$

$$=\frac{b}{2}(\cos u\cos v+\iota\sin u\sin v) \qquad \text{... (D)}$$

Equating the reals with the reals and the imaginaries with the imaginaries we get :—

$$x=\frac{b}{2}\cos u\cos v \qquad \text{... (E)}$$

$$\text{and } y=\frac{b}{2}\sin u\sin v \qquad \text{... (F)}$$

$$\text{or } \cos v=\frac{x}{\frac{b}{2}\cos u}$$

$$\sin v=\frac{y}{\frac{b}{2}\sin u}$$

Squaring and adding we get :—

$$\cos^2 v+\sin^2 v=1=\frac{x^2}{\left(\frac{b}{2}\cos u\right)^2}+\frac{y^2}{\left(\frac{b}{2}\sin u\right)^2}$$

which is the same as (A)

Similarly we can get equation (B) as :—

$$\cos^2 u-\sin^2 u=1=\frac{x^2}{\left(\frac{b}{2}\cos v\right)^2}-\frac{y^2}{\left(\frac{b}{2}\sin v\right)^2}$$

In the case of a simple floor the pressure at any point along it (where $y=0$) is given by equation (E) by substituting :—

$$u=0$$

or $x=\frac{b}{2}\cos v$

as $\cos 0=1$

or $v=\cos^{-1}\frac{2x}{b}$

or as $v=\pi\phi,\ \phi=\frac{1}{\pi}\cos^{-1}\frac{(2x)}{b} \qquad \text{... (G)}$

General Form. If under a floor we introduce a vertical obstruction like a line of sheet piles or wells, the configuration of stream and pressure lines of Fig. (4) will be distorted. But it is possible to bring back this distoration to the normal configuration of Fig. (4) by means of a transformation known as Schwarz Christoffel transformation.

The fundamental general epuations for pressure distribution under the foundation profile as sketched in Fig. 5 with the sheet pile at any point under the floor may be summarised as below :—

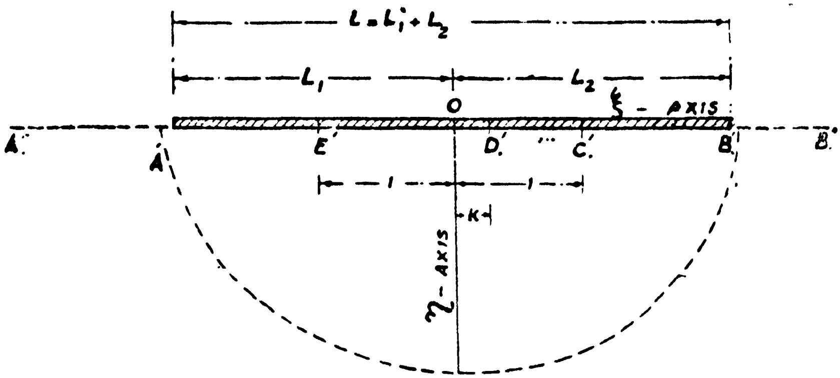


Fig. 5.

General Equations.

I $$x+\iota y=\frac{d_1-d_2}{\pi K}\sqrt{(\lambda\cos v-\lambda_1)^2-1}-\frac{d_1-d_2}{\pi}\log\left\{(\lambda\cos v-\lambda_1)+\sqrt{(\lambda\cos v-\lambda_1)^2-1}\right\}+\iota(d_1-d_2)$$

II. Pressure at E :—

$$P_e=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right)$$

III. Pressure at C :—

$$P_c=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$$

IV. Pressure at D :—

$$P_d=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1+K}{\lambda}\right)$$

Where $K=\cos\theta$ is given by
$\tan\theta-\theta=\pi\delta$

$$\lambda=\frac{L_1+L_2}{2}$$

and $$\lambda_1=\frac{L_1-L_2}{2}$$

where the values of L_1 and L_2 are given by epuations, $L_1=\cos\gamma_1$ and $L_2=\cos\gamma_2$

$$G_e \text{ at } C=\frac{dp}{dy} \text{ (exit)}$$

$$=\frac{H}{(d_1-d_2)\sqrt{\lambda}}\cdot\frac{K}{1-K}$$

V. $\quad \text{Lt.}\left(\dfrac{K}{d_1-d_2}\right)_{d_1=d_2=d}=\dfrac{1}{\pi d}$

We shall now deal with particular cases of this general form. As we are mainly concerned with the values of the pressures at the key points E, D and C and the values of the exit gradients, we shall obtain only the vaules of these quantities.

Case 1.

Floor with pile line not at end Fig. 6.

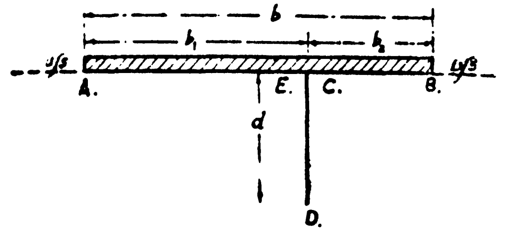


Fig. 6.

This is Weaver's general case.

where $\dfrac{b_1}{d}=\alpha_1$

and $\dfrac{b_2}{d}=\alpha_2$

Hence $L_1=\sqrt{1+\alpha_1^2}$

$L_2=\sqrt{1+\alpha_2^2}$

$$\lambda=\frac{L_1+L_2}{2}=\frac{\sqrt{1+\alpha_1^2}+\sqrt{1+\alpha_2^2}}{2}$$

$$\lambda_1=\frac{L_1-L_2}{2}=\frac{\sqrt{1+\alpha_1^2}-\sqrt{1+\alpha_2^2}}{2}$$

Pressures are as below

$$P_e=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right)$$

$$P_c=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$$

$$P_d=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

These values of P_e, and P_d and P_c are given in Plate II, Vol. III and Table I.

Case 2.

Floor with pile line at end as in Fig 7.

Here $\quad d_1=d_2=d$, $b_2=0$ and $b_1=b$

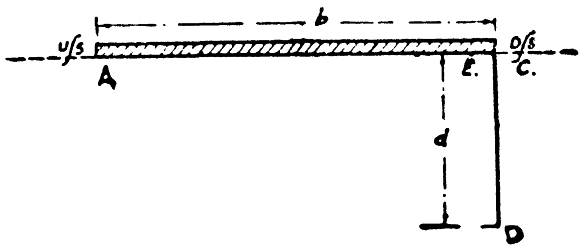


Fig. 7

$$P_e=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$P_d=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

and $P_c=\dfrac{H}{\pi}\cos^{-1}\left(\dfrac{\lambda}{\lambda}\right)=0$

where $\lambda=\dfrac{1+\sqrt{1+\alpha_1^2}}{2}$ because

$\alpha_2=\dfrac{b_2}{d_1}=0$. Values of P_e and P_d are given in Plate II A, Vol. III.

Exit gradient at C.

$$G_{exit}=\frac{H}{\pi d}\cdot\frac{1}{\sqrt{\lambda}}$$

values of G_e are given in Plate III. Vol. III. When the floor is absent

$$\lambda=\frac{\sqrt{1+\alpha_1^2}+\sqrt{1+\alpha_2^2}}{2}=1, \text{ as } \alpha_1=\alpha_2=0$$

$$G_{exit}=\frac{H}{\pi d}$$

Case 3.

Simple floor – No pile line, Fig. 8.
The pressures along the base are given by

$$P=\frac{H}{\pi}\cos^{-1}\left(\frac{2x}{b}\right)$$

$$G_{exit}=\frac{H}{\pi.0}\times\frac{1}{\sqrt{\lambda}}=\infty$$

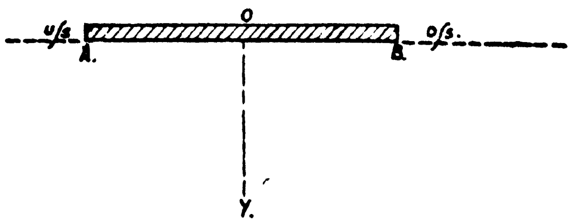


Fig. 8

In a simple floor the exit gradient at the immediate toe is infinity, which as will be explained in the next paragraph, contains a condition of definite instability. The particles at the toe must move.

TABLE I

Khosla's Table for Pile at end

$$P_e=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$P_d=\frac{H}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$G_{exit}=\frac{H}{\pi d}\cdot\frac{1}{\sqrt{\lambda}}$$

$$\phi_{d1}=\phi_d-\frac{2}{3}(\phi_e-\phi_d)+\frac{3}{\alpha^2}$$

$$\phi_{d1}=\frac{P}{H}\times 100$$

$\frac{1}{\alpha}=\frac{d}{b}$	ϕ_d	ϕ_e	ϕ_{d1} Theoretical	$\alpha=\frac{b}{d}$	λ	$\frac{1}{\pi\sqrt{\lambda}}$
·02	8·9	12·7	6·92	50	25 545	0 063
·04	12·6	17 9	9·56	45	23·006	0·066
·06	15·3	21·8	11·34	40	20·506	0·070
·08	17·6	25·1	12·90	35	18·007	0·075
·10	19·4	28·0	14·12	30	15·508	0·080
·12	21·2	30·6	15·16	25	13·010	0·088
·14	22·7	32·9	16·12	20	10 512	0·098
·16	24·1	35 0	16·97	18	9 514	0·103
·18	25·3	37·0	17·85	16	8·516	0·109
·20	26·6	38 0	18·61	14	7·518	0·116
·22	27·6	40 6	19·27	12	6·520	0·125
·24	28·7	42·2	19·80	11	6·020	0·130
·28	30·5	45·2	20·91	10	5·52	0·136
·32	32·1	47·9	21·84	9	5·03	0·142
·36	33·6	50·4	22·63	8	4·53	0 150
·40	34·8	52 7	23·41	7	4·04	0·158
·44	36·0	54·7	24·07	6	3·54	0·169
·48	37·1	56·7	24·71	5	3·05	0·182
·52	37·9	58·5	25·30	4	2·56	0·199
·56	38·8	60·1	25·81	3	2·08	0·225

Table 1 (Cont.)

$\frac{1}{\alpha}=\frac{d}{b}$	ϕ_d	ϕ_e	ϕ_{d_1} Theoretical	$\alpha=\frac{b}{d}$	λ	$\frac{1}{\pi\sqrt{\lambda}}$
·60	39·6	61·7	26·30	2	1·62	0·252
·64	40·3	63·2	26·72	1½	1·401	0·269
·68	40·9	64·5	27·18	1	1·207	0·289
·72	41·6	65·8	27·61	3/4	1·125	0·300
·76	42·1	66·9	28·00	1/2	1·057	0·309
·80	42·6	68·1	28·35	1/4	1·015	0·316
·90	43·7	70·7	29·21	0	1·000	0·318
1·00	44·6	72·8	30·00			
1·10	45·2	74·7	30·65			
1·20	45·8	76·4	31·21			
1·30	46·3	77·9	31·75			
1·40	46·7	79·3	32·23			
1·50	47·1	80·4	32·70			
1·60	47·3	81·4	33·10			
1·70	47·7	82·5	33·52			
1·80	47·9	83·6	33·90			
1·90	48·1	84·1	34·21			
2·00	43·2	84·9	34·60			

TABLE 2

Khosla's Table for pile not at end

$$\phi_e=\frac{P_e}{H}\times 100=100\times\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$\alpha_1=\frac{b_1}{d},\ \alpha_2=\frac{b_2}{d},\ \alpha=\frac{b}{d}$$

$\frac{b_1}{b}$ or base rates

b_1/b	0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
α				values of ϕ_e							
0·1	3 16	2·82	2·55	2 22	1·91	1·59	1·28	0·96	0·63	0 35	0
0·2	6·22	5 67	5 06	4 45	3 81	3·17	2·52	1·87	1·26	0 63	0
0·25	7·86	7·10	6·31	5·53	4·75	3 98	3·17	2·38	1·58	0·78	0
1/3	9·95	9·29	8·27	7·26	6·24	5·39	4·15	3·11	2·08	0·99	0
0·5	14·93	13·74	12·30	10·83	9·37	7·82	6·22	4·66	3·11	1·56	0
0·75	21·90	20 00	18·00	15·80	13·70	11·50	9·00	6·50	4 50	2·00	0
1·0	27·18	25·40	22·81	20 03	17·38	14·93	11·87	8·92	5·87	2·92	0
1·5	35·80	33·70	30·90	27·40	24·10	20·50	16·56	12·50	8·30	4·20	0
2 0	42·48	39·93	36·79	33·29	29·13	24·91	20·32	15·44	10·24	5·03	0
2·5	47·40	44·80	41·50	37·60	33·20	28·50	23·50	17·90	12·20	6·10	0
3·0	51·22	48 67	45 21	41 03	36 42	31·32	25·89	19·84	13·46	6·85	0
4·0	56·98	54·37	50·48	45·86	40·74	35 25	29·35	22·92	15·83	8·08	0
5·0	61·07	58·17	54·08	49 01	43·62	37·90	31·76	25·09	17·63	9·27	0
6·0	64·22	61·34	56 64	51·32	45·79	39·77	33 48	26·76	19 06	11·09	0
7·0	66·83	63 65	58·89	52 92	47 11	51·15	34·48	27·98	19·62	10·90	0
8 0	68·84	65·43	60·00	54·13	48·22	42·28	35·93	28·96	21·11	11·60	0
9·0	70·48	67·00	61·13	55·41	49·15	43·04	36·67	29·76	21·88	12·00	0
10·0	72·01	68·12	62·09	55·96	50·13	43·72	37·09	30·33	22·57	12·74	0
11·0	73·24	69·14	62·89	56·60	50·47	44·28	37·87	30·96	23·00	13·22	0
12·0	74·37	69·96	63·44	57·16	30·96	44·76	38·33	31·43	23·58	13·68	0
14·0	76·23	71·38	64·54	58·01	51·74	45·48	39·00	32·15	24·32	14·37	0
16·0	77·72	72·27	65·28	58·64	52·68	46·24	39·60	32·74	24·92	14·93	0
18·0	78·93	73·20	65·86	59.17	53·00	46·50	39·84	33·41	25·37	15·48	0
20·0	80·00	73·84	66·37	59·43	53·14	46·83	40·41	33·71	25·87	15·88	0
25·0	82·10	75·01	67·04	60·30	53·81	47·46	41·02	34·06	26·45	16·68	0
30·0	84·21	75·78	67·77	60·75	54·23	47·88	41·44	34·63	26·95	17·32	0

TABLE 3

Khosla's Table for pile not at end

Floor with pile not at end

$$\phi_d = \frac{P_d}{H} \times 100 = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$\frac{b_1}{b}$ / α	0	0·1	0·2	0 3	0·4	0 5	0 6	0 7	0·8	0·9	1·0
					values of ϕ_d						
0·1	50 08	50·07	50·04	50·03	50 02	50·00	49·98	49·97	49·96	49·93	49·92
0·2	50·32	50·25	50·19	50·13	50·07	50·00	49·92	49·87	49·81	49·75	49·68
0·25	50·48	50·40	50·30	50·20	50·10	50·00	49·90	49·80	49·70	49·60	49·52
1/3	50·83	50·65	50·50	50.34	50·17	50·00	49·83	49·66	49·50	49·35	49·17
0·5	51·72	51·44	51·13	50·72	50·38	50·00	49·62	49·28	48·87	48·56	48·28
0·75	53·70	52·94	52·34	51·62	50·80	50·00	49·20	48·38	47·66	47·06	46·30
1·0	55·46	54·50	53·61	52·43	51·22	50·00	48·78	47·57	46·39	45·50	44·54
1·5	59·20	58·00	56·37	54·44	52·29	50·00	47·71	45·56	43·63	42·00	40·80
2·0	62·50	60·99	58·77	56·11	53·17	50·00	46·83	43·89	41·23	39·01	37·50
2·5	65·18	63·50	60 70	57·50	53·82	50·00	46·18	42·50	39·30	36·50	34·82
3·0	67·29	65·46	62·44	58·61	54·37	50·00	45·63	41·39	37·56	34·54	32·71
4·0	71·06	68·61	64·75	60·02	55·07	50·00	44·93	39·98	35·25	31·39	28·94
5·0	73·56	70·87	66·26	60·94	55·50	50·00	44·50	39·06	33·74	29·13	26·44
6·0	75·47	72·53	67·07	61·55	55·83	50·00	44·17	38·45	32·93	27·47	24·53
7·0	77·08	73·81	68·38	61·93	55·89	50·00	44·11	38·07	31·62	26·19	22·92
8·0	78·43	74·68	68·50	62·17	56·02	50·00	43·98	37·83	31·50	25·32	21·57
9·0	79·57	75·44	68·86	62·33	56·07	50·00	43·93	37·67	31·14	24·56	20·43
10·0	80·53	76·01	69·11	62·39	56·15	50 00	43·85	37·61	30·89	23·99	19·47
11·0	81·33	76·52	69·34	62·57	56·21	50·00	43·79	37·43	30·66	23·48	18·67
12·0	82·16	76·92	69·47	62.65	56 22	50·00	43·78	37·35	30·53	23·08	17·84
14 0	83·39	77·25	69·77	62·79	56·28	50·00	43·72	37·21	30·23	22·75	16·61
16·0	84·44	77·93	69·93	62 85	56·28	50·00	43·72	37·15	30 07	22·07	15·56
18·0	85·14	78·22	70·04	62·89	56·29	50·00	43 71	37 11	29·96	21·78	14·86
20·0	86·01	78·40	70 12	62·93	53·34	50·00	43·66	37·07	29·88	21·60	13·99
25·0	87·43	78·78	70·26	63 02	56·37	50·00	43 63	36 98	29·74	21·22	12·57
30·0	88·74	79·02	70·32	63 03	56·40	50·00	43·60	36·97	29·68	20 98	11·26

5. Theory of Exit Gradient.

A brief summary is given below from Chapter VIII of Central Board of Irrigation Publication No. 12 as mentioned before.

"Weir failure from seepage flow can occur by :—

(*a*) Undermining of subsoil.

(*b*) Uplift due to pressure under the floor being in excess of the weight on the floor.

The failure due to undermining is most common, so that a knowledge of its causes and of the measures to prevent it, is of utmost importance both for design of new works and for ascertaining the safety of existing ones. Even in the second case if the floor be burst due to excessive pressure, the final failure is due to the reduction of the effective length with the consequent inerease in the exit gradient.

The undermining of the sub-soil starts from the tail-end of the work. It begins at the surface due to the residual force of seepage water at this end being in excess of the restraining forces of the sub-soil which tend to hold the latter in position. Once the surface is disturbed the dislocation of sub-soil particles works further down and, if progressive, leads to the formation of cavities below the floor into which the latter may collapse. According to the commonly accepted ideas, this underminig is supposed to result from what is known as "piping" that is, the erosion of sub-soil by the high velocities of water through it, when such velocities exceed a certain limit. But as will be shortly explained, this conception of undermining is incomplete. Water has a certain residual force at each point along its flow through the sub-soil which acts in the direction of flow, and is proportional to the pressure gradient at that point. At the tail end this force is obviously upwards, and will tend to lift up the soil particles, if it is more than the submerged weight of the latter. The frictional resistance, cohesion etc., of the adjacent soil will have to be considered in certain cases. Once the surface particles are disturbed, the resistance against upward pressure of water will be further reduced, tending to progressive disruption of the sub-soil. The flow gathers into a series of pipes in the latter case and dislocation of particles is accelerated. The sub-soil is thus progressively undermined. Soil erosion can also occur through natural pipes, or faults in the sub-soil.

As water flows through the sandy sub-soil, it is well known that the velocity of flow at any point in the medium is given by Darcy's Law :— $Vk=\frac{H}{L}$

This water exerts a force (F) on the sandy medium along its line of flow. Besides this force, the particles of the medium are subject to two other forces :—

(1) the force of gravity, or weight (W) which acts downwards,

and (2) the force of buoyancy (B) which acts opposite to gravity. These two latter forces can be combined into one and if :—

w = Weight of unit volume of water

ρ = Specifice gravity of sand particles

ε = Pore space in unit volume, then the weight of soil particles per unite volume

$= W(1-\varepsilon)\rho w$

The weight of displaced water $= W(1-\varepsilon) = B$

Hence the resultant of the forces, that is, the weight of the soil particles less buoyancy
$W_s = w(1-\varepsilon)(\rho-1)$

This may be called the submerged weight W_s which will obviously always act downwards as ρ is greater than unity.

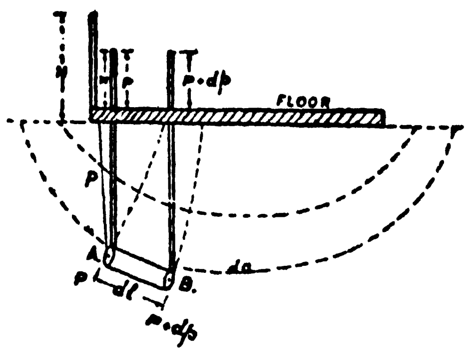


Fig. 9.

The next step is to determine the value of F. Let us assume a cylinder of soil along one of the stream lines as shown Fig. 9.

Let da = sectional area of cylinder and dl = length of cylinder.

The pressure at face A = p. da and acts along the stream line from left to right.

The pressure at face B = (p+dp) da and acts along the stream from right to left.

Neglecting the curvature of the stream line we get the force acting on the cylinder along the stream line as.

$$pda-(p+dp)da=-dp.da$$

The force per unite volume of the cylinder $=-\frac{dp.\,da}{dl\,.\,da}=-\frac{dp}{dl}$

Hence $F=-\frac{dp}{dl}$ = Pressure gradient at that point. So that, the action of water on any point in the sandy medium through which it percolates can be given by a force F, which, per unit volume, is equal to the gradient of pressure at that point and which acts in the direction of flow.

Thus the force acting on any particle in the sub-soil consists of as shown in Fig. 10.

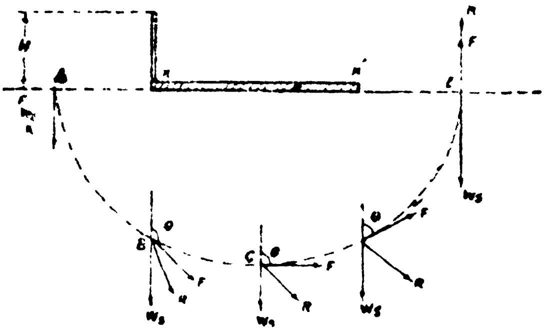


Fig. 10.

(*a*) A force $F=-\frac{dp}{dl}$ acting in the direction of flow, and (*b*) the submerged weight $W_s=w(1-\varepsilon)(\rho-1)$ acting downwards.

For stability of the particles, the resuluant R of F and W_s must have no upward component.

Now the vertical component of R is given by

$$R_v=F\cos\theta-W_s$$

From Fig. 10 R_v in the region A to C must necessarily be downward as F has no upward vertical component. From C onwards, F has an upward vertical component, which goes on increasing rapidly as the stream line approaches E the exit end. At E, F is vertically upwards ; at X′ it is infinite, beyond which point it rapidly falls off as shown in Fig. 10. Thus for some length beyond X′ at the surface, where F acts vertically upwards and W_s vertically downwards, the resultant $R_v=F-W_s$ will be vertically upwards, so that, the soil particles in this region will be lifted up and be in a state of unstable equilibrium. This region of unstable equilibrium will extend to the limit where R=0 or $F=W_s$. At this point the soil particles will be just stable, and F acting upwards will have its critical value which will be just resisted by the submerged weight of the particles acting downwards. The slightest increase in this value of F will lead to instability. The soil particles will start to be lifted up or "float" so to speak. The gradient of pressure at which this occurs has been aptly called by Terzaghi the "floatation gradient". Haigh calls it the bursting gradient. Another author has called it the critical gradient. This is equal to the critical value of the Force F_c and is given by.

Floatation gradiant $=F_{critical}=W_s,\ G_f=w(1-\varepsilon)(\rho-1)$

If the pore space=40 per cent and $\rho=2{\cdot}65$ and w=1,

$$G_f=(1-{\cdot}4)(2{\cdot}65-1)={\cdot}6\times1{\cdot}65={\cdot}99, \text{ say unity.}$$

Thus for the class of soil mentioned above the floatation gradiant G_f=unity or 1/1. This floatation gradiant will vary with the pore space of the sub-soil and the density of the soil particles. The density of the soil particles though generally in the neighbourhood of 2·6 to 2·71 for the Punjab sands may vary in extreme cases from 1·8 to 2·8, similarly, the pore space which for the Punjab sands ranges between 37 and 42% (may be as low as 20 per cent and as high as 45 per cent). Working on these limits the extreme values of G_f may range from 0·44 to 1·44.

Table for G_f

Pore space	Densities					
	2·8	2·6	2·4	2·2	2·0	1·8
0·20	1·44	1·28	1·12	0·96	0·80	0·46
0·25	1·35	1·20	1·05	0·90	0·75	0·60
0·30	1·26	1·12	0·98	0·84	0·70	0·56
0·35	1·17	1·04	0·91	0·78	0·65	0·52
0·40	1·08	0·96	0·84	0·72	0·60	0·48
0·45	0·99	0·88	0·77	0·66	0·55	0·44

The theoretical value of the floatation gradients is given by the formula at this page and the table on page 128, but in actual weir design there are other factors which cannot be allowed in theory such as the formation of scour holes and retrogression etc. All the under mentioned factors increase the value of critical gradient :—

(*i*) Scour holes can be caused by the formation of swirls by simply a tree sticking in the bed bar the downstream end of the weir.
(*ii*) The exit gradient at the lower end can be increased by the formation of the local surges or waves which resemble the action of the intermittent pump. The rising and the ebbing of waves produces suction on the soil.
(*iii*) The sudden application and reduction of head also tends to increase the value of the the exit gradient.
(*iv*) The pressure difference between the free water level in a drain or a sink and the Basic Sub-Soil Pressure level produces flow into the river bed causing floating gradient and critical flow disturbing the soil particles.
(*v*) Retrogression results in the lowering of the downstream bed level, with the consequent increase in the value of the exit gradients.

To allow for such uncertain river conditions, a factor of safety is allowed as given below in fixing the value of the exite gradients :

Nature of soil	Factor of safety	G_f
Shingle	4 to 5	·25 to ·2
Coarse sand	5 to 6	·2 to ·17
Fine sand	6 to 7	·17 to ·14

6. Sheet Pile

(*a*) **Sheet Pile Section**

In the modern weirs the sheet piles replace the curtain walls of masonry or well lines of Bligh's weirs, because they can be driven to any depth without the tedious process of well sinking or the very expensive unwatering required in case of curtain walls. A typical section of sheet pile is shown in fig. 11. The piles are interlocked as shown therein.

To all intents and purposes they are supposed to be water-tight.

Fig. 11.

(*b*) **Depth of sheet piles.**

In paragraph No. 7 Chapter V formulae have been given to determine the depth of normal scour. The scour depths usually calcutated from Lacey's formula are somewhat higher than those from Kennedy's. It is on the safe side to accept Lacey's figures. According to Spring, scours are classified as :—

(*i*) Normal scour, 30 to 40 ft., caused by the unobstructed currents.
(*ii*) Abnormal 60 to 80 ft., caused by the deflected but nevertheless onwards moving currents.
(*iii*) Extraordinary 80 to 100 ft., caused by swirls set up in forward moving water.

Lacey has given a clearer definition of classes of scour.

Let R be the depth of scour in a channel flowing straight with level bed.

Class A	Straight reach	1·25 R
Class B	Moderate Bend	1·5 R
Class C	Severe Bend	1·75 R
Class D	Right-angled Bend	2·0 R

Class A is likely to occur just below or upstream of loose aprons, class B along the aprons of guide banks in a straight reach and C & D at the noses of guide banks. The top level of the aprons should be fixed allowing scour depth according to Kennedy's and the bottom of the sheet piles should go below R as calculated from Lacey's formula Plate III, Vol. III.

(*c*) **Function of upstream and downstream pile line.**

Scour holes can occur both upstream and downstream so that a pile line is required at the upstream end of the floor just as at the downstream end in order to prevent failure by slipping of the sub-soil into the scour holes by simple earth pressure. In the case of a scour hole at the upstream end, the flow of seepage water will tend to keep the soil in position, as its force will oppose slipping due to earth pressure. At the downstream end this force will assist and accelerate earth slipping due to earth pressure as the two forces will act together.

The depth of the upstream pile will be governed by the depth of scour only and the downstream one by the depth of scour and the exit gradient.

Under normal conditions the bottom of the scour upstream will be higher than the bottom of the scour downstream by the amount of afflux or difference between upstream and downstream levels in maximum flood. The sheet piles at the upstream end can therefore, be higher than those at the downstream end by that amount.

The necessity for piles at the upstream and downstream ends is obvious. They are to protect the work against undermining of the foundation soil. The pressures under the downstream floor increase as the depth of the downstream pile line increases. The upstream pile line is relatively more effective to destroy pressure head and reduces uplift pressures under the weir while the pile line at the downstream end is most effective to reduce the exit gradient.

(*d*) **Function of intermediate piles.**

The intermediate sheet pile lines are required neither to prevent undermining of the floor at the upstream nor at the downstream end nor do they materially alter the pressure distribution to give less uplift pressures under the downstream floor. But they act as important seccndary lines of defence so that even if the *pacca* (impervious) floor is damaged at either the upstream or the downstream end by failure of the end piles under abnormal scour. The rest of the floor and the superstructure will be saved from collapse by the intermediate piles. The uplift pressures will increase but the undermining will be arrested as the exit gradients will still be below the critical value. The structure will thus be saved from complete failure which on a major work would be disastrous. Opinions differ as to whether there should be one or more intermediate piles The best value will be obtained with one deep pile line dircetly under the crest.

7. Khosla's Method of Design of Modern Weirs.

(*a*) The two essentials to be considered in weir design, therefore, are :—

(*i*) Residual or uplift pressure under weir, and, (*ii*) Exit gradients.

These two essentials are inter-connected. For any given foundation profile of a weir in a given class of soil, there will be a definite distribution of pressure and exit gradient. To safeguard against undermining, the exit gradient must not be allowed to exceed a certain safe limit, generally 1/5 to 1/7. The upstream pressures must be kept as low as possible, consistent with safety at the exit so as to keep the floor thickness at the minimum.

(*b*) In paragraph (4) the mathemtical solutions have been given for simple cases to determine the uplift pressures and the exit gradients. A complete mathematical solution of a weir section with sloping or level aprons with 3 to 4 sheet pile lines is not yet available. Khosla evolved a method of design known as "The Method of Independent Variable's" the justificatian of which has been methematically given by Dr. J. K. Malhotra of the Irrigation Research Institute, Lahore.

In this method, a complex weir section is split up into its elementary standard forms—The entire length of the floor with any one of the pile lines, etc., making up one such form. Each elementary form is then treated as independent of the others. The pressures at the key points are then read off the curves of Plate II, Vol. III. These key points are junction points of the floor and the pile line of that particular elementary form, the bottom point of that pile line and the bottom corners in the case of a depressed floor.

The exit gradients can be obtained from Plate III, Vol. III.

The readings at the junction points are then corrected for :—

(*a*) the mutual interference of piles,
(*b*) the floor thicknesses,
and (*c*) the slope of the floor.

The method of spliting up a complex weir section is shown in the calculations

(*c*) **Mutual interference of the Piles.**

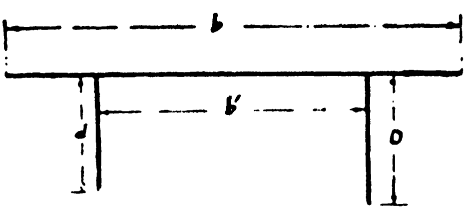


Fig. 12.

The mutual interference of piles is worked out from Khosla's formula, Fig. 12.

$$C=19\sqrt{\frac{D}{b'}}\cdot\frac{d+D}{b}$$

where C=the correction to be applied as percentage of head,
b′=distant between the two piles,
D=the depth of piles the influence of which has to be determined on the neighbouring pile of depth (d),
d=depth of pile on which the effect of pile (D) is sought to be determined,
and b=total floor length.

This correction (C) is additive for points in the rear or back-water and subtrative for points forward in the direction of flow. This equation gives results within about 2½ per cent of those obtained by experiments and almost exactly those obtained from theory by Dr. Malhotra.

But this equation does not apply to the effect of an outer pile on any intermediate plie, if latter is equal to or smaller than the former and is at a distance less than twice the length of the outer pile. Subject to these limitations this equation can be applied to find the influence of outer piles on intermediate ones and *vice versa* irrespective of their depth or spacing.

The mutual influence of the piles is local. It mainly extends up to a distance equal to the depth of the pile beyond which it gradually falls off till the residual effect at twice that distance is negligible in most cases. Where the spacing is close and the depth of the piles great, the residual effect may be considerable. The above equation gives this influence for all cases. It takes note of the depth of either pile of the disance between the two and the floor length.

(*d*) **Correction for the floor thickness.**

The pressure percentages so obtained refer to the join of sheet pile line with the surface at E′, C′, Fig. 13. To obtain the corresponding pressure at E and C it is necessary to determine the pressure drop along each face of the sheet pile line. The pressure drop along the face of a sheet

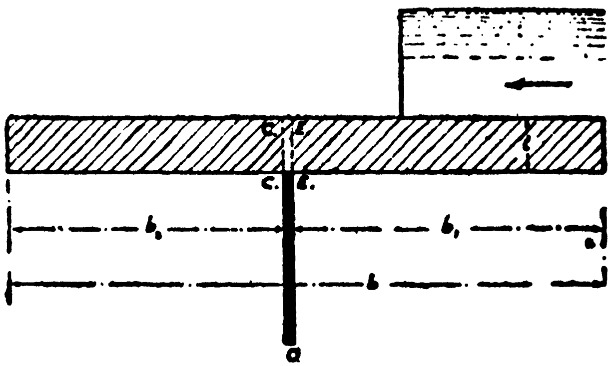


Fig. 13.

pile line, facing direction of seepage flow, at any point distant b_1 from the upstream edge of the floor$=\left(70-40\frac{b_1}{b}\right)$ per cent of the total drop along both the faces of the sheet pile, the pressure drop being assumed to be uniform along each face.

(*e*) **Correction for slope.**

The pressure percentages obtained from Plate II, Vol. III are to be corrected in case of sloping floors. The pressure percentages under a floor sloping down or sloping up in the direction of flow are respectivly greater or less than those under a horizontal floor for the same base ratios. As such the percentage corrections would be positive for the down and minus for the up slopes following the direction of flow. The correction curve is given in Fig. 14. The correction applied is $\frac{1}{b_1} \times c$ where 1 is the horizontal length of the sloping floor. c is a factor as read from Fig. 14. and b_1 is the horizonal distance apart of the piles.

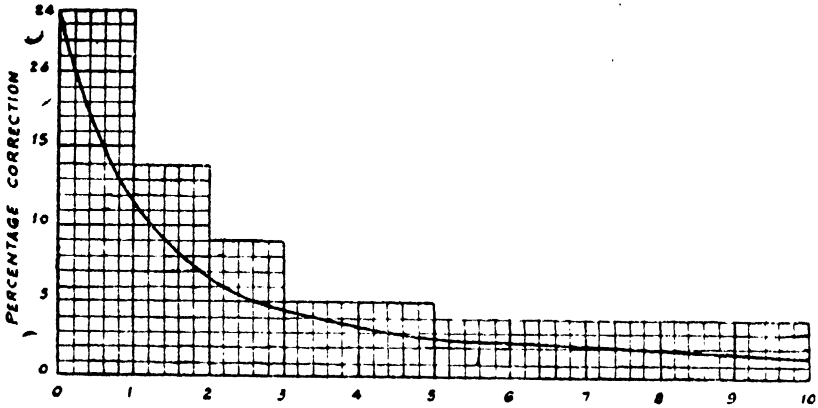


Fig. 14.
Slope (horizontal divided by vertical)

8. Design of wing walls.

(*a*) In the case of Flank walls and return walls design is complicated by the fact that the soil behind, gets saturated by the rise of spring level when the river rises in floods. When the river subsides both water and earth pressures act behind the walls. It is usual to put in filter points or inverted filters connected to weep holes to drain away water and to relieve water pressure behind them. The provision of weep holes should be considered as additional factor of safety and the wall section should be designed both for the earth and the water pressures behind them as shown hereafter.

(*b*) The foundations of abutments and flank walls between any pair of pile lines should go down to the level of the bottom of those pile lines. If these latter are at different levels the flanks foundation can step down from one end to the other to suit these levels. This, in fact, means of complete boxing in one of the foundations of the entire weir by means of pile lines of suitable depth upstream, downstream and on the two flanks. With such an arrangement of levels for the flank foundations, the uplifts under the weir due to flank flow will not be more than those due to direct flow under the weir. The foundations of the upstream and downstream return walls of the flanks should similarly go down to the levels of the bottom of the upstream and downstream pile lines respectively.

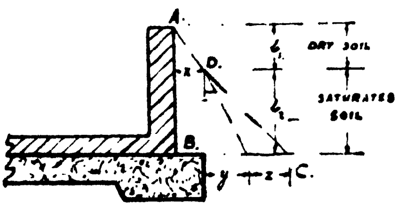


Fig. 15.

(*c*) A wall for dry and saturated soil. Fig. 15

(*i*) Dry earth pressure

$$P_1 = w_1 h_1 \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

where w_1 = weight of dry soil,
h_1 = height of dry soil,
ϕ = angle of repose of that soil,

(*ii*) Pressure due to saturated soil

$$P_2 = w_2 h_2 \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

where w_2 = submarged weight of the soil

$=(\rho-1)(1-\varepsilon)\times w$ lbs, (if $\rho=2{\cdot}65$ for sand, $\varepsilon=40\%$)

$\omega=62{\cdot}5$ lbs.

ϕ=angle of respose of saturated soil (5° to 10° less than ϕ usually)

and h_2=Height of saturated soil

(*iii*) Water pressure height h_2

$P_3=wh_3$, where w is weight of one cft. of water.

The total pressure at the base $=P_1+P_2+P_3$

The resultant horizontal pressure is the area of the figure ABCD acting at the C. G. of this figure horizontally. Resultant out of the weight of the wall and the horizontal resultant pressure must pass within the middle third of the base of the wall.

(*d*) **A wall with saturated soil upto its top.**

The same formula shall apply, the pressure at the base shall be P_2 plus P_3 as calculated above.

9. Calculations.

(*i*) **Fixation of maximum flood level.**

Let the maximum flood level be fixed from local flood marks of the highest flood. Let it be taken as 669.

(*ii*) **Maximum flood discharge.**

(*a*) By observations of cross sections as governed by the local highest flood marks at the site of the weir, one mile upstream and downstream. Take average section and slope as given by flood marks and calculate discharge using Kutter's formula.

(*b*) By calculations from catchment area and the average annual rainfall.

(*c*) By consulting flood records of the same river at other sites, as available in the discharge Division, Lahore.

Let the discharge be taken as 3,00,000 cusecs.

(*iii*) **Pond level and Crest Level.**

Pond level depends on the Full Supply Level of the off-taking canals. Let there be a canal with full supply level at head 667. Allowing 2·0 ft. working head, the pond level required will be R. L. 669.

This level must be made up by the raised crest and the shutters. As the shutters cannot be of more then 6 ft. height, let the gates be used of 10 ft. height giving thus a crest level of the weir R. L. 660·0 with one foot free board above pond level. Let there be 24 bays of the weir with 60 feet clear opening, the available gate size shall fix the bay width.

Similarly, the available size of gates shall fix the undersluice bay width and the crest level. If the size of gates available is 30′ × 18′, the crest level will be, pond level minus 17 feet *i.e.*, 652. Let there be 16 bays of 30 ft. each.

(*iv*) **Afflux.**

The Afflux is usually 3·0 to 5·0 ft. and let it be 3·0 ft. in this case to given H. F. L. 672 upstream.

(*v*) **Calculated discharge capacity.**

Let 1·0 be taken as velocity of approach head. The Discharge of the weir per foot $=3{\cdot}1(12+1)^{3/2}=145$ cusecs. Total discharge over the weir $=24\times60\times145=208800$ cusecs.

Drowning ratio for the undersluices $=\frac{17}{17+3}=\frac{17}{20}={\cdot}85$. Using Fance's curve, the Punjab Engineering Congress, Paper No. 111 of 1927, the value of C is 3·1 for this drowning ratio, plate IV, Vol. III.

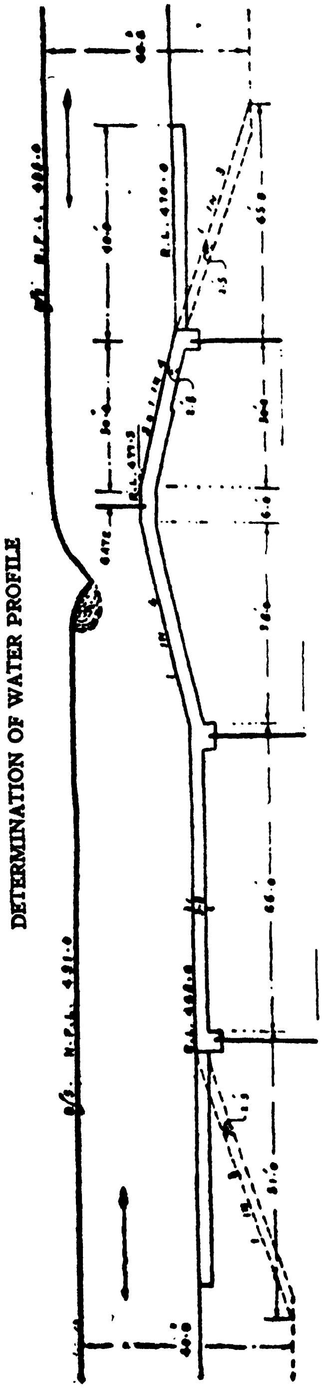


Fig. 16.

The probable velocity of approach head $=1{\cdot}5$ & take $C=2{\cdot}8$ on the safe side as per latest practice.

Undersluice discharge per foot run

$$=2{\cdot}8\,\{(H+h_a)^{3/2}-h_a^{3/2}\}=2{\cdot}8\,\{(20+1{\cdot}5)^{3/2}-1{\cdot}5^{3/2}\}=275 \text{ cs.}$$

Total discharge through undersluices

$$=16\times30\times275=132{,}000 \text{ cusecs}$$

Total discharge $=208800+132000=340800$ cusecs which is more than the requied *i.e.*, 3,00,000 cusecs.

(*vi*) **Scour depth.**

Assuming 7·0 ft. as width of the pier, the discharge intensity upstream of the weir $=\frac{60}{67}\times145=130$ cusecs. Scour depth using Kennedy's formula $D=1{\cdot}11q^{\cdot61}=1{\cdot}11\times130^{\cdot61}=21{\cdot}5$ ft. say 22·0 ft.

Using Lacey's formula :—

$$R=0{\cdot}9\left(\frac{q^2}{f}\right)^{\frac{1}{3}}=0{\cdot}9\left(\frac{130^2}{1{\cdot}0}\right)^{\frac{1}{3}}=30 \text{ ft.}$$

Lacey's formula gives excessive results assuming eleptic section.

The upstream floor level should be fixed according to the Kennedy's scour depth, while the upstream curtain wall or pile line should go deeper than the depth worked out according to Lacey's conception.

Floor level upstream $=672-22=650$

Floor level downstream $=669-22=647$

(*vii*) **Determination of water profile.**

Above is the section of the Emerson Barrage Fig. 16. The discharge intensity $=271$ cusecs per foot run.

The scour according to Lacey's formula $=39{\cdot}2$ say 40 ft.

The floor level upstream is arbitrarily kept at R.L. 470 and it is supposed that the upstream apron will be launched as shown dotted in Fig. 16. with minimum thickness of 2·5′. The crest level has been fixed as shown in the previous example from considerations of pond level and the height of gates available. Afflux assumed $=4{\cdot}0$ feet.

Assuming a glacis slope of 1 in 4, the length of the glacis is to be found so that the jump remains always on the glacis.

Apply weir formula :—

$$q=271=3{\cdot}1\,E^{\frac{3}{2}} \quad \therefore E=19{\cdot}7 \text{ feet.}$$

H.F.L. fixed $=491$ which shall be level downstream.

Total energy level upstream $=$ crest level $+E=477{\cdot}5+19{\cdot}7=497{\cdot}2$

First the calculations will be made according to Crump's method Fig. 17 (neglecting friction) Plate No. VI and VII, Vol. III and then the complete water profile will be worked our according to author's method as per Plates VIII to X, Vol. III, taking friction into account.

$K=19{\cdot}87$

$L=4{\cdot}0$

$F=$ Drop in feet to the point where jump takes place

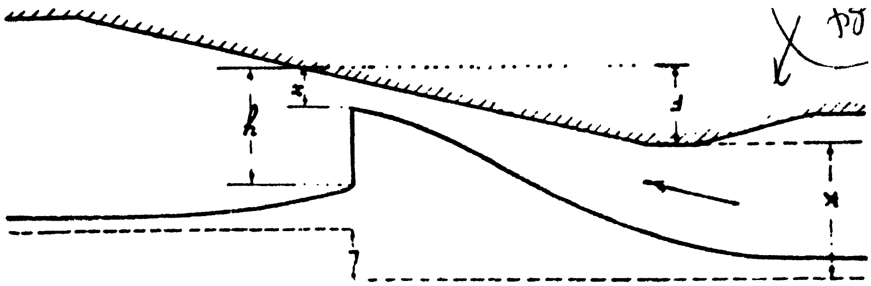


Fig. 17.

x = Upstream depth and y = Downstream depth

$$C = \text{Critical depth} = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{271^2}{32{\cdot}2}} = 13{\cdot}15$$

$$\frac{L}{C} = \frac{4}{13{\cdot}15} = {\cdot}305$$

for $\frac{L}{C} = {\cdot}305$, from Plate VI, $\frac{K+F}{C} = 2{\cdot}1$.

$$\frac{x}{C} = {\cdot}35 \text{ and } \frac{y}{C} = 1{\cdot}35$$

$\therefore K+F = 2{\cdot}1 \times 13{\cdot}15 = 27{\cdot}6$ ft.

$x = {\cdot}562 \times 13{\cdot}15 = 7{\cdot}4$ ft. and $y = 1{\cdot}605 \times 13{\cdot}15 = 21{\cdot}2$ ft.

The same values of depth upstream and downstream of the jump are obtained from Plate VI Fig. 1, for level floor.

Drop from crest = 27·6—19·7 = 7·9

Distance from the begining of the glacis = 4 × 7·9 = 31·6 say 32 feet.

Actual length is 38 feet which is O.K. for H.F. conditions.

Downstream floor level = 468

(*ii*) Test for half discharge

$$K = \left(\frac{271}{2} \times \frac{1}{3{\cdot}1}\right)^{\frac{2}{3}} = 12{\cdot}6 \text{ ft.}$$

Depth on downstream floor in H.F. conditions = 491 − 468 = 23 feet

$q_1 = k \times 23^{5/3} = 271$

$q_2 = k \times D^{5/3} = 135{\cdot}5$

$\therefore D = {\cdot}64 \times 23 = 14{\cdot}72$ feet

$$\text{velocity} = \frac{135{\cdot}5}{14{\cdot}72} = 9{\cdot}20 \text{ ft. per second.}$$

$$h_a = \frac{9{\cdot}20^2}{2 \times 32{\cdot}2} = 1{\cdot}3 \text{ feet.}$$

The total Energy level downstream = 468 + 14·72 + 1·3 = 484·02

L = (477·5 + 12·6)—484·02 = 490·1 − 484·02 = 6·08 feet.

C = critical depth = $\frac{2}{3}$ × 12·6 = 8·4 feet

$\frac{L}{C} = \frac{6{\cdot}08}{8{\cdot}4} = {\cdot}744$ from plate VI, Vol. III.

$$\frac{K+F}{C}=2{\cdot}73 \text{ in } K+F=2{\cdot}73\times 8{\cdot}4=23{\cdot}0 \text{ feet}$$

Drop from crest level $=23-12{\cdot}6=10{\cdot}4$ ft.
which means 41·6 ft. from beginning of crest.

$x=467\times 8{\cdot}4=3{\cdot}92$ feet

$y=1{\cdot}83\times 8{\cdot}4=15{\cdot}4$

The downstream depth is only 14·72 feet and the length of a glacis only 38 and therefore there will be no hydraulic jump on the glacis but it will take place somewhere on the level floor when depth upstream shall drop to the corresponding available depth of 14·72, this cannot be calculated neglecting friction.

(*viii*) **Author's method**: - taking friction into account.

(*a*) Upstream flow

It is assumed that the loose apron shall launch down in floods to a slope of 1 in 3 and the maximum scour depth below floor level is 15 feet.

T.E. level upstream $=497{\cdot}2$

Velocity with 40 feet depth $=\frac{271}{40}=6{\cdot}8$ feet per second,

$$h_a-\frac{6{\cdot}8^2}{64{\cdot}4}={\cdot}617 \text{ feet say } {\cdot}6 \text{ feet.}$$

Actual water level $=497{\cdot}2-{\cdot}6=496{\cdot}6$

Flow is above critical conditions against a negative slope of 1 in 3.

$$\text{Average velocity}=\left(\frac{271}{40}+\frac{271}{25}\right)^{\frac{1}{2}}=9{\cdot}7 \text{ feet per second.}$$

From Plate X, Vol. III, the corresponding value of $f={\cdot}011$

$$C \text{ in Chezy's formula}=\sqrt{\frac{2g}{f}}=76{\cdot}4$$

$$Y_n=\text{neutral depth}=\sqrt[3]{\frac{q^2}{C^2s}}=\sqrt[3]{\frac{271^2}{76{\cdot}4^2\times\frac{1}{3}}}=3{\cdot}35 \text{ feet.}$$

$$Y_c=\text{Critical depth}=\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{271^2}{32{\cdot}2}}=13{\cdot}15 \text{ ft.}$$

The equation of the water profile

$$L=\frac{Y_n}{g}Z+Y_n=\left(\frac{1}{s}+\frac{C^2}{g}\right).\ F(Z)$$

(*i*) let $Y_1=40'$ in the beginning, $\frac{y}{y_n}=Z_1=\frac{40}{3{\cdot}35}=12{\cdot}0$

For $Z_1=12$, the value of $F(Z_1)$ from Plate X $=0{\cdot}91$

$$L_1=\frac{3{\cdot}35}{\frac{1}{3}}\times 12+3{\cdot}35\left(3+\frac{(76{\cdot}4)^2}{g}\right)\times 0{\cdot}91$$

$$=40\times 3+3{\cdot}35(3+180)0{\cdot}91=120+612\times 0{\cdot}91=120+556=677 \text{ ft.}$$

(*ii*) Let $y_2=33{\cdot}5$ feet, $Z_2=\frac{y}{y_n}=\frac{33{\cdot}5}{3{\cdot}35}=10$ & $F(Z_2)={\cdot}911$

$\therefore L_2=3\times 33{\cdot}5+612\times 0{\cdot}911=100{\cdot}5+557=657{\cdot}5$ ft.

$L_2-L_1=677-657{\cdot}5=19{\cdot}5$ feet.

(*iii*) Let $Z_3=8$; $y_3=8\times 3{\cdot}35=26{\cdot}80$ and $F(Z_3)$ from plate X $=0{\cdot}9137$

$\therefore L_3=3\times 26{\cdot}8+612\times 0{\cdot}9137=80{\cdot}4+559=639{\cdot}4$ ft.

$L_2-L_3=657{\cdot}5-639{\cdot}4=18{\cdot}1$ ft.

Plotting these three points in Fig 18 we get the last point at B from the plate with $y=25{\cdot}0$ ft.

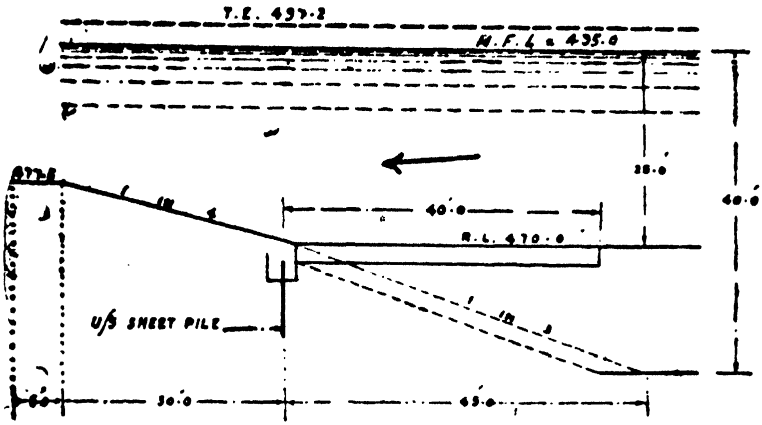


Fig 18

(*b*) **Flow over upstream approach.**

Negative slope $=\frac{1}{4}$ and cement plastered

Average velocity $=\frac{1}{2}\left(\frac{271}{25}+\frac{271}{13{\cdot}15}\right)=15{\cdot}6$ ft. per second From Plate X, $f=0{\cdot}0095$

C in Chezy's formula $=\sqrt{\frac{2g}{f}}=\frac{64{\cdot}4}{0{\cdot}0095}=83$

Natural depth $=y_n=\sqrt[3]{\frac{q^2}{C^2\times S}}=\left(\frac{271^2}{83^2\times\frac{1}{4}}\right)^{\frac{1}{3}}=3{\cdot}5$ feet.

The equation for negative slope and flow above critical

$$L=\frac{y_n}{s}.\,Z+y_n\left(\frac{1}{s}+\frac{C^2}{g}\right)F(Z)$$

(*i*) Let $y_1=25$ ft. in the beginning $\therefore Z_1=\frac{y}{y_n}=\frac{25}{3{\cdot}5}=7{\cdot}14$

& $F(Z_1)={\cdot}9148$ from plate X, Vol. III

$$L_1=y_1\times4+13{\cdot}5\left(4+\frac{83^2}{g}\right)(0{\cdot}9148)$$
$$=25\times4+3{\cdot}5\,(4+214)\times{\cdot}9148=100+763\times{\cdot}9148=798{\cdot}0$$

(*ii*) Let $Z_2=5{\cdot}0$, $Y_2=3{\cdot}5\times5=17{\cdot}5$ and $F(Z_2)={\cdot}927$

$L_2=17{\cdot}5\times4+763\times{\cdot}927=70+707{\cdot}3=777{\cdot}3$

$L_1-L_2=798-777{\cdot}3=20{\cdot}7$ feet.

(*iii*) Let $Z_3=4$ and $Y_3=14{\cdot}0$ and $F(Z_3)={\cdot}939$

$L_3=14\times4+763\times{\cdot}939=716{\cdot}3+56=771{\cdot}2$

$L_2-L_3=777{\cdot}3-771{\cdot}2=6{\cdot}1$ feet.

Plotting these points, extend the curve to get the depth at the beginning of the crest, it comes to be 12·6 feet while the critical depth is 13·15 feet. The flow against a negative slope with depth above critical cannot drop below critical dedth – this indicates that the heading up upstream will be higher than 496·6 by about a foot and the total energy line cannot be 97·2 but about a foot higher.

(*c*) **Flow on level crest 6 ft. wide.**

This length is too small for $E=19{\cdot}7$. Double this length shall ensure control section and parallelism at the middle of crest (Author's Publication No. 10) and crest length equal to E shall ensure it to be at the end of the crest.

The insufficient length of crest and the energy losses in friction upstream of crest are likly to drown the weir long before it could be expected because 4 ft. drop on a properly designed weir should be sufficient for depth crest of the order of 30 feet.

The actual position of the control section, cannot be calculated in this case according to the equation for level floor.

$$L=\frac{C^2}{g}\left(y-\frac{y^4}{4y_c^3}\right)$$

(*d*) **Flow on the down stream glacis 1 in 4.**

Assuming that the levels upstream shall adjust themselves as stated above and that the critical depth shall be attained at the end of the crest.

Let depth in the beginning be 13-12 feet.
Value of f=·01 from Plate X, Vol. III.

$$C=\sqrt{\frac{2g}{f}}=\sqrt{\frac{64\cdot4}{\cdot01}}=80\cdot0 \text{ nearly}$$

$$y_n=\sqrt[3]{\frac{q^2}{C^2\times s}}=\left(\frac{271^2}{80^2\times\frac{1}{4}}\right)^{\frac{1}{3}}=3\cdot6$$

The equation for water profile with positive slope with depth below critical

$$L=\frac{y_n}{s}Z-y_n\left(\frac{1}{s}-\frac{C^2}{g}\right)F(Z)$$

(*i*) $y=13\cdot12; Z_1=\frac{y}{y_n}=\frac{13\cdot12}{3\cdot6}=3\cdot65$ and $F(Z_1)$ from Plate X ,Vol III is ·948

$$L_1=\frac{y_1}{s}-y_n\left(\frac{1}{s}-\frac{C^2}{g}\right)F(Z_1)=13\cdot1\times4-3\cdot6\ (4-200)0\cdot948$$

$$=52\cdot48+3\cdot6\times196\times\cdot948=52\cdot48+706\cdot0\times0\cdot948=721\cdot7$$

(*ii*) Let $y_2=3\times3\cdot6=10\cdot8$; $Z_2=3\cdot0$ and $F(Z_2)=\cdot963$
$L_2=10\cdot8\times4+706\times0\cdot963=722\cdot2$

(*iii*) Let $y_3=2\cdot5\times3\cdot6=9\cdot0$; $Z_3=2\cdot5$, $F(Z_3)=\cdot989$
$L_3=9\times4+706\times\cdot989=734\cdot3$
$L_2-L_1=722\cdot2-721\cdot7=0\cdot5$
$L_3-L_2=734\cdot3-722\cdot2=12\cdot1$

(*iv*) Let $y_4=2\cdot0\times3\cdot6=7\cdot2$; $Z_4=2\cdot0$ & $F(Z_4)=1\cdot039$ from Plate No. X.
$L_4=7\cdot2\times4+706\times1\cdot039=28\cdot8+733\cdot9=762\cdot7$
$L_4-L_3=762\cdot7-734\cdot3=28\cdot4$

(*e*) **Water profile on the glacis downstream of the jump.**

Water on the level floor is fixed as 491
supposing glacis floor=468
Depth=23·0 feet

Depth above critical, positive slope of $\frac{1}{4}$

$y_n=3\cdot6$ feet as before for the slope

(*i*) $y_1=23\cdot0$ ft.; $Z_1=\frac{y_1}{y_n}=\frac{23}{3\cdot6}=6\cdot4$

$\therefore F(Z_1)=\cdot918$

$$L=\frac{y_n}{s}Z-Y_n\left(\frac{1}{s}-\frac{C^2}{g}\right)F(Z)$$

$$L_1=23\times4+706\times\cdot918=92+648\cdot1=740\cdot1$$

(*ii*) $Z_2=5$; $Y_2=5\times3\cdot6=18\cdot0$ & $F(Z_2)=\cdot927$

$L_2=18\times4+706\times\cdot927=72+654\cdot4=726\cdot4$

$L_1-L_2=740\cdot1-726\cdot4=13\cdot7$ feet

(*iii*) Let $Z_3=4$; $y_3=4\times3\cdot6=14\cdot4$ & $F(Z_3)=\cdot939$

$L_3=14\cdot4\times4+706\times\cdot939=720\cdot5$

$L_2-L_3=726\cdot4-720\cdot5=5\cdot9$ ft.

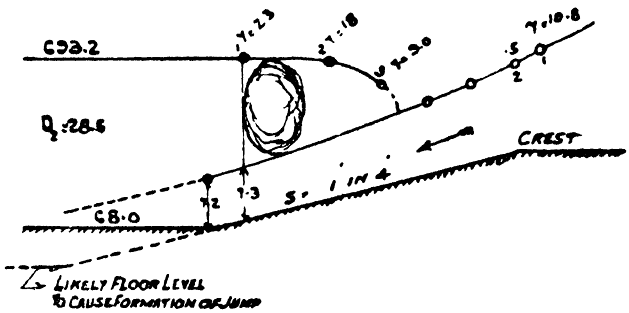


Fig. 19.

Now plot both these profiles as shown in Fig. 19 and find out the position of the jump using curves of plate VIII, Vol. III with slope 1 in 4 and D_1 on the first curve gives D_2 on the other. Fig. 19 shows that no hydraulic jump is formed in the highest flood condition with floor level 468 but it could be formed at the end of the glacis if the downstream floor were lowered to R.L. 62·5. The weir shall work drowned with discharge intensity 271 with upstream levels shooting up from 3·0 to 4·0 feet. The uplift pressure due to jump would be of the order of 20·0 feet but *Kharanja* damage is not likely as the floor is R.C. slab doubly reinforced.

(*ix*) **Determination of uplift pressures.** (Khosla's method)

The worst condition will be when the water upstream is ponded up to the pond level *i.e*, R.L. 493 and downstream water level is at floor level 468.

Head across weir $=493-468=25$ feet.

(*a*) **Exit Gradient.**

To determine exit gradient, the weir is considerd to be of an elementary form.

Depth of downstream vertical cut-off, $d=468-444=24$ feet. Total length of impervious floor, $b=140$ feet.

$$\therefore\ \alpha=\frac{140}{24}=5\cdot83;\quad \frac{H}{d}=\frac{25}{24}=1\cdot04$$

From Plate III A, Vol. III corresponding to $\frac{H}{d}=1\cdot04$ and $\alpha=5\cdot83$

Value of $G_e=0\cdot184$

The critical value of exit gradient for Trimmu sandy soil being 1·0, the factor of safety$=5$ which is ample.

(*b*) **Uplit Pressure 1st Pipe Line.** Elementary form Fig. 20.

Percentage Pressures.	Percentage corrections.
Upstream sheet pile line :—	Thickness. :—
$\alpha=\frac{140}{19}=7\cdot37.$	Total drop along the sheet pile line $=100-67\cdot5=32\cdot5.$
$\frac{b_1}{b}=0$	Drop towards $C=32\cdot5\times\frac{30}{100}=9\cdot8$
$\therefore\ \phi_c=67\cdot5$	Proportionate drop over floor thickness of $1\cdot5$ feet$=\frac{1\cdot5}{19}\times9\cdot8=0\cdot8$

Interference of the intermediate sheet pile line :—

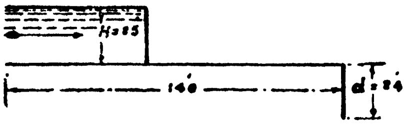


Fig. 20.

ϕ_c (corrected) $=67{\cdot}5+0{\cdot}8+3{\cdot}0-1{\cdot}2=70{\cdot}1$

$$\frac{D}{b'}=\frac{22{\cdot}5}{72}=0{\cdot}312$$

$$\frac{D+d}{b'}=\frac{22{\cdot}5+17{\cdot}5}{140}=0{\cdot}286$$

$\therefore\ \phi_c=3{\cdot}0$ (from Plate VI)

1:4 slope :— $\phi_e=3{\cdot}4\times\frac{26}{72}=1{\cdot}2$

Percentage corrections due to thickness of floor as well as interference of intermediate sheet pile line are additive : the effect of the intermediate sheet pile line is to pond up the seepage flow so that the pressure at C would rise.

The percentage correction due to the upslope in the direction of flow is negative.

(*c*) Intermediate sheet pile line :—

Section reduces to elementary form Fig. 21.

$$\alpha=\frac{b}{d}=\frac{140}{468-446}=6{\cdot}36$$

$$\frac{b_1}{b}=\frac{74}{140}=0{\cdot}53$$

$$\frac{b_2}{b}=0{\cdot}47$$

$\therefore\ \phi_c=38{\cdot}0$

$\phi_e=100-42=58$

Corrected values :—

$\phi_c=38{\cdot}0+1{\cdot}6+3=42{\cdot}6$

$\phi_e=58-1{\cdot}6-1{\cdot}9+1{\cdot}7=56{\cdot}2$

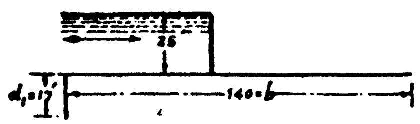


Fig. 21.

Thickness

Total drop along the pile line $=20\%$

$$\frac{\text{Drop along DC}}{\text{Drop along ED}}=\frac{1\times(0{\cdot}7-0{\cdot}53\times0{\cdot}4)}{0{\cdot}70-0{\cdot}53\times0{\cdot}40}=\frac{0{\cdot}512}{0{\cdot}188}$$

$\therefore$ Drop along $DC'=20\times0{\cdot}512=10{\cdot}24$

along $E'D=20\times0{\cdot}488=9{\cdot}76$

$$\phi_c=\frac{3{\cdot}5}{22}\times10{\cdot}24=1{\cdot}6$$

$$\phi_e=\frac{3{\cdot}5}{22}\times9{\cdot}76=1{\cdot}6$$

Interference of sheet piles :—

$$\phi_e=19\sqrt{\frac{13{\cdot}5}{72}}\times\frac{32{\cdot}0}{140}=1{\cdot}9$$

$$\phi_c=19\sqrt{\frac{20{\cdot}5}{64}}\times\frac{39}{140}=3{\cdot}0$$

1 : 4 slope :—

$$\phi_e=3{\cdot}4\times\frac{36}{72}=1{\cdot}7$$

The effect of down slope in the direction of flow would be to increase the pressure and as such the correction is additive.

(*d*) For the last pile elementary form of the weir is sketched in Fig. 22 and the same is used to work out the exit gradient.

Downstream sheet pile line :—

$$\alpha=\frac{140}{24}=5{\cdot}83\ ;\ \frac{b_2}{b}=0$$

$\therefore\ \phi_e=100-64=36$

Corrected value :—

$\phi_e=36-1{\cdot}6-2{\cdot}8=31{\cdot}6\%$

Thickness correction

Total drop $=36\%$

$\therefore$ Drop along ED $=36\times0{\cdot}3=10{\cdot}8$

$\therefore\ \phi_e=\frac{3{\cdot}5}{24}\times10{\cdot}8=1{\cdot}6$

Interference of intermediate sheet pile line:-

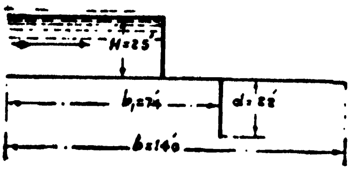


Fig. 22.

$$\phi_e = 19\sqrt{\frac{464\cdot5 \times 446}{64} \times \frac{18\cdot5+(24-35)}{140}}$$

$$= 19\sqrt{\frac{18\cdot5}{64} \times \frac{39\cdot5}{140}} = 2\cdot8$$

Total Head across weir=25·0 feet.

Location	Percentage Head.		Head in ft. of water above R.L. 468·0	
	E	C	E	C
Upstream pile line	100	70·1	25	17·5
Intermediate pile line	56·2	42·6	14·1	10·7
Downstream pile line	31·6	0	7·9	0

9. Floor thickne ss.

(*a*) Floor up to crest.

The downward water pressure is 25 ft. to 15·5 ft. The maximum uplift pressure is 17·5 ft. The downward water load is more, the thickness may be 2 to 3 feet from practical considerations.

(*b*) Glacis downstream of crest.

Uplift pressure=14·1 ft.

$$t = \frac{4}{3} \times \frac{H-h}{\rho}$$

(as the pressure calculated below the floor, the dividend is ρ not $(\rho-1)$

t=thickness for gravity section.

H−h=uplift pressure

ρ=Specific gravity of material=2·25

$$t = \frac{4}{3} \times \frac{14\cdot1}{2\cdot25} = 8\cdot3 \text{ ft.}$$

(*c*) Thickness of the downstream level floor.

$$t = \frac{4}{3} \times \frac{H-h}{\rho} = \frac{4}{3} \times \frac{7\cdot9}{2\cdot25} = 4\cdot7 \text{ ft.}$$

(*d*) The gravity sections are very thick. It is a typical case where reinforced concrete sections will be not only cheaper but also are desirable, because masonry *kharunja* has not proved safe for the back pressure in the jump. Reinforced concrete section can be designed as aconti-nuous slab under the piers which are 7 ft. thick and 60 ft. apart. It will serve as the founation slab for the piers and will also take up the uplift pressure. In the middle, theoretically, steel at top only will be required but at least half of the steel should be provided at the bottom so that it is capable of taking up downwards loads due to the formation of probable cavities.

10. Design of Talus or Pervious Floor.

(*a*) The length and thickness are now no longer fixed according to Bligh's formula as stated in chapter V.

The upstream stone apron should contain enough stone so that when it settles down to floods it should take a slope of 1 in 3 with a minimum thickness of 2·5 ft. The scour below the floor level=15 ft. in this case 40 ft. length of stone with 4 ft. thickness will do.

(*b*) **Downstream Talus.**

This has to serve a triple function.

(*i*) To serve as an inverted filter.

(*ii*) To settle down to maximum scour with a slope of 1 in 3 with minimum thickness of 3·0 feet.

(*iii*) To withstand very high velocities downstream of the hydraulic jump. The size of the blocks should be heavy.

In this case the scour below floor level is only 15 ft. and therefore a length 30 ft. to 50 ft with 5 ft. depth will do. In addition a length 30 ft. to 50 ft. at least should be provided just downstream of last pile to serve as inverted filter. First a layer of shingle and spawls about a foot thick over the river bed sand, then boulders about 2·0 ft. thick of 1 ft. size and then concrete blocks $5' \times 5' \times 5'$ size. In the second portion of the Talus which is meant to sink, the blocks may be of small size *i.e.*, $2' \times 2' \times 2'$ because the velocities will be relatively low and also for uniform and even sinking, a small size is desirable

10. Examination questions.

1. Sketch a design for a weir with shutters across a Tarai stream to divert water into a small canal. Width of stream 80 ft., side slope 1/2 : 1, bed level 650·01 H F L. 650 0. Flood discharge 6800 cusecs H. Gradient 1 in 12. Top of banks 655 0. Bed level of canal 655 0, F.S.L. 658·0. (*T.C.E. 1933*)

2. (*a*) Sketch a design for the head of the canal in question No. (1) given above. Bed width = 10·0 ft. (*T.C.E. 1933*)

(*b*) Design and give a dimensioned sketch of the cross section of a weir to be built on fine micacous sand (C = 15) R.L. of F. Supply in canal 630·0 R L. of bed river = 620 0. Height of drop shutters = 3 ft. maximum, discharge per foot of weir = 75 cs. Winter discharge of river is just sufficient for canal requirements and the weir is just submerged when 4 feet of water is passing over the crest. (*T.C.E. 1935*)

3. What are the functions of upstream and downstream pile lines in a weir on sand foundation and what is the objection to a downstream pile line ? (*P.I.B.* 1936)

4. Describe briefly Bligh's creep theory. (*P.I.B.* 1936)

5. (*a*) You are required to construct a canal Headworks. What points will you consider in selecting the most suitable site ?

(*b*) What observations are necessary to make in connection with such a site ?

(*c*) Why are trough head-works preferable to head-works in boulder reach ?

6. State why you cannot do away with underslucies in the design of headworks ? (*P. U. 1942*)

7. Would you prefer a hill boulder tract for the headworks of a canal or the trough site ? Discuss in detail. What points would you consider while selecting site at each of the three places ? (*T C.E. 1934*)

8 (*i*) Sketch a typical lay-out for a canal headworks in the sandy trough stage of a Himalayan river showing the position of (*a*) weir and undersluices (*b*) Canal Head Regulator. (*c*) afflux bunds (*d*) Divide wall (*e*) River training works and (*f*) And other works that are usually required

(*ii*) What points would you consider in selecting a suitable site for headworks ? (*T.C.E.* 1935)

9 Explain the significance of exit gradient in the design of a weir ? What do you understand by the term unity gradient ? What factors of safety are allowed in fixing exit gradient and why ? (*P. U. 1943*)

10. Describe the various methods of control for silt entering the canal usually adopted at canal Headworks ? (*P. U. 1943*)

11. Design a flank wall upstream of a weir in masonry retaining dry sand up to 10 feet from top and saturated sand for the next 12 feet to the bottom of wall

12. Why is a fish ladder required in a weir ? Sketch a suitable design for 12 feet difference of level in the pond and that downstream of the weir. (*P. U. 1944*)

13. Describe the effect of constructing a weir across an alluvial river on its regime with special reference to retrogression of levels.

14. Explain the working of Stoney's gates as used for the undersluices. Sketch the roller bearing, and staunching bar arrangements and explain their working ?

15. Explain the following terms as used in canal headworks, (*a*) Divide wall, (*b*) Barrage, (*c*) Intake, or diversion weirs (*d*) Talus (*e*) Pond level (*f*) Breast wall (*g*) still pond method.

16. Define uplift pressure and Floatation gradient and explain Khosla's method of "Independent Variables" for the determination of uplift pressure on weirs. Work out from first principles the value of floatation gradient or exit gradient at the tail of the structure if para space E = 40% and P = 2·65 and W = 1. What factors of safety should be used for fine sand as bed material. (*P. U. 1949*)

17. Describe breifly Bligh's creep theory and state its short comings. State why you cannot do away with undersluices in the design of headworks. What points would you consider in selecting a suitable site for Headworks. (*P. U. 1950*)

18 In what way to modern Barrages on a river differ from the earlier type of headworks. Give the chief reasons responsible for the changes in design. What works on a headwork serve as the armour to receive the spearheads of the attack caused by a record flood. What vigilance can the Officer-in-charge exercise at such a crisis. (*P. U. 1952*)

19. Explain the significance of exit gradient in the design of a weir. Draw a sketch. What factors of safety are allowed in fixing exit gradients and why ? (*P. U. 1952*)

20. What are the various types of gates and gearing in use in India for control of flow. What is the function of rollers in the stoney type gates. (*P. U. 1952*)

21. What are the various devices used for regulation of irrigation supplies on (*i*) Head regulators.

(*ii*) Undersluices. (*iii*) Canal falls. (*iv*) Minor Heads (*v*) Outlets. (*P.U. 1952*)

22. In ths case of a barrage founded on sand, indicate by sketches the significance of (*a*) Uplift (*b*) sub-soil flow and (*c*) exit gradient. How are these kept within safe limits in a structure. (*P. U. 1953*)

23. State Bligh's and Khosla's theories for the design of weirs on sand foundations. Sketch the section of a barrage to withstand 20 feet of head. Show the approximate dimensions of sheet pile cut-offs to counteract uplift pressure and to give safe exit gradient (*P. U. 1952*)

24. Sketch the layout of a headworks in the Punjab indicating how its main features are adopted to the needs of the river and offtakes. (*P. U. 1953*)

25. Sketch a typical section of a weir on permeable foundations and explain the design and functions of various parts. (*P. U. 1954*)

26. What are different types of gates used in canal. Headworks and what are suitable depths and spans for each type. (*P. U. 1955*)

27. Explain the significance of exit gradient in the design of weirs on permeable foundations. Prove that in average soil the value of critical gradient equals unity. What factors of safety are used in fixing exit gradient and why ? (*P. U. 1955*)

28. Sketch a typical section of a weir on permeable foundations clearly indicating the various parts and explain the significance of (*a*) Uplift (*b*) exit gradient. How are these kept within safe limits in a structure ? (*P. U. 1956*)

29 Explain the terms same exit gradient, Inverted Filter, Pond Level, Retrogression, Silt excluder, Silt ejector, Drowning ratio, Afflux.

30. What factors govern the following :—

(*a*) Whether the floor of the weir be gravity or non gravity type.

(*b*) Whether to provide sheet pile cut-offs or deep curtain walls at U/S and D/S ends of a weir. Briefly state the methods of silt control followed are headworks. (*P. U. 1956*)

31. Explain the significance of exit gradient in the design of weirs. (*P. U. 1957*)

32. Sketch the layout of a typical barrage type headworks and explain the function of the following :-

(*a*) Undersluices (*b*) Fore-apron (*c*) Flexible apron.

33. Describe Khosla's theory of exit gradient. Explain why the theoritical exit gradient is kept less than unity in actual design of weirs ? What factor of safety is generally adopted for weirs founded on fine sands ? (*P. U. 1958*)

PART II

CANAL IRRIGATION

CHARTER VI

Design of Irrigation Channels

Introduction.

In hydraulics the students have already learnt to work out the sections of open channels using Chezy, Bazin or Manning slope formula. There was no reference to the soluble or the suspended material carried in the flowing water. In irrigation earthen channels or in natural streams in erodable bed, water carries a certain cargo of suspended or soluble stuff known as silt.

Silt is defined to be the solid material carried by flowing water whether in suspension or solution.

Sand is defined to be the coarse stuff generally rolled by flowing water and deposited in the bed of a channel.

The problem of silt transport in irrigation channels baffled the early irrigation engineers and is still a subject which has not yet reached the final stage of research. There are numerous controversial theories put forth by various authors. This problem is very important to a canal engineers who designs irrigation channels which often silt up and somtimes scour. In the case of channels which are silting, the water levels change and the distribution of water is disturbed. There is discontent and agitation among the cultivators. There is shortage of supply at the tails. The efficiency of the canal system is so much impaired that the frequent silt clearances are found necessary which entail a lot of expenditure and labour and the consequent botheration. Similarly the scouring of the channels upsets distribution and endangers the safety of works by undermining. This troubled has led to various investigations in order to arrive at designs of irrigation channels which should neither silt up nor scour.

2. Hydraulic formulae.

Before the development of various silt theories is described, it is considered advisable to summarise the formulae for ready reference which apply to the the design of sections in hydraulic flow.

(*a*) Chezy 1854.

Chezy, a French Engineer, produced his empirical formula of flow in open channels as below by fitting his observations of some channels

$V=C\sqrt{RS}$ where V = meam velocity flow

R = Hydraulic mean depth

S = Slope

C = a co-efficient

Subsequent hydraulicians have been trying to work out the mathematical basis to derive this formula as below. In an open channel, the pressure is atmospheric and may, therefore be neglected, head due to the slope in the channel is assumed to be lost in friction. Hence the hydraulic gradiant is equal to the slope of the channel if the latter is uniform.

Let S = slope of channel

A = Area of the cross section of the channel

P_w = Wetted perimeter

V = mean velocity of flow

W = weight of one cft. of water in lbs.

F = Fritional resistance in lbs. per second

f=absolute co-efficient of friction between water and sides of the channel per square foot per foot velocity.

f'=Co-efficient of friction in lbs. (per square ft. at one foot velocity)

Consider a section of water of length l moving along the channel as in Fig. 1.

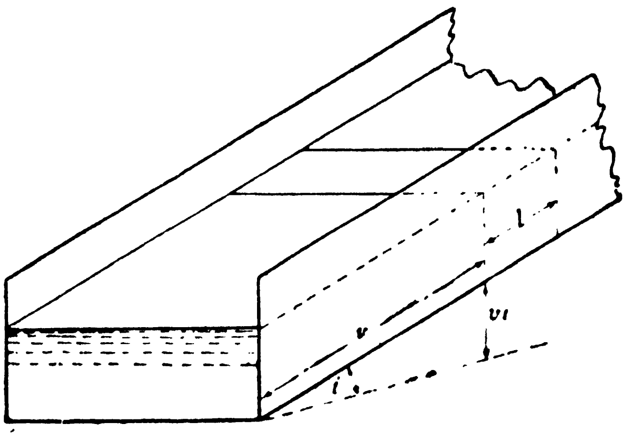


Fig. 1.

Assume slope of the channel uniform and equal to S.

Frictional resistance of the section for length l,

F=wetted area × frictional resistance per unit area in lbs.

$$F=P_w\left(f.\ W.\frac{V^n}{2g}\right) \text{ in lbs.}$$

The value of n is taken as 2 as the average. Actually it varies with the nature of the surface and the velocity from 1·7 to 2·3

Work done in overcoming friction

E_w=friction resistance × distance moved

=F × V foot lbs.

$$=Pl.\ f.\ W.\frac{V^3}{2g} \text{ ft. lbs.} \qquad \text{(A)}$$

loss of potential energy per second

=weight × change of altitude in foot

=$WAl \times VS$ ft. lbs. **(B)**

Equating equations (A) and (B)

$$Pl\,fW\,\frac{V^3}{2g}=WA\,l\,V\,S$$

$$V^2=\frac{2g}{f}\cdot\frac{A}{P_w}\cdot S=\frac{2g}{f}R.S.$$

$$\therefore V=C\sqrt{RS} \text{ where } C=\sqrt{\frac{2g}{f}}$$

Note :—In friction experiments to determine the value of frictional resistance in lbs. per square foot, f' is used to express the unit of frictional resistance. Its value in terms of f is given by the relation.

$$f'=\frac{fw}{2g}$$

NOMOGRAM FOR KUTTERS FORMULA

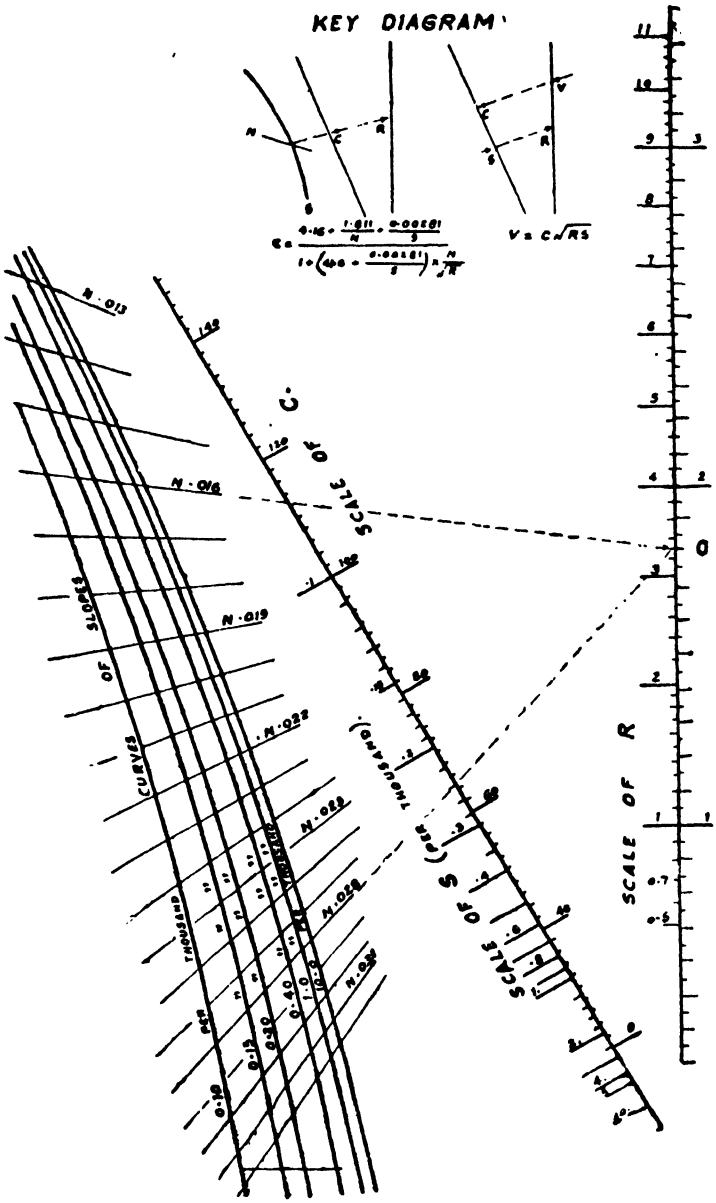


Fig. 2.

In terms of f′ the value of $C=\sqrt{\frac{w}{f'}}$

The Chezy's equation of the fundamental energy equation of flow of water and is the time-honoured practice. Solution of Chezy's equation can be got from Nomogram Fig. 2.

(*b*) Bazin, 1897.

From the results of his experiments on flow on water through channels, Bazin deduced the following formula.

$$C = \frac{157 \cdot 5}{1 + \frac{k}{\sqrt{R}}}$$ where R=H.M.D. & k=a constant

The value of k depended on the nature of surface of the channel and has the following values

Clear smooth sides of wood, bricks, stones etc. k= ·2
Dirty sides of wood, bricks, stones etc. k= ·5
Sides of natural earth k=2·35

(*c*) Manning 1889, Transactions of the Institute of Civil Engineers, Ireland.

Manning analysed some of the Channels in his charge and produced his empirical formula in C.G.S. units which when expressed in British units takes the form.

$$V = \frac{1 \cdot 4858}{N} R^{2/3} S^{1/2}$$

where N is the co-efficient of rugosity,

This formula is simplified furthar in practice and is used as an exporational formula of discharge of a channel. $Q = kd^{5/3}$ the solution of which is given in monogram in Fig. 3.

(*d*) Kutter, 1870

Kutter, an American Engineer, from his analysis of the Mississipi River sections, produced an empirical formula for C in Chezy's equation wherein he made C to depend on (a) nature of bed (*b*) Hydraulic mean depth and (*c*) slope.

$$C = \frac{41 \cdot 66 + \frac{1 \cdot 811}{N} + \frac{\cdot 0028}{S}}{1 + (41 \cdot 66 + \frac{\cdot 0028}{S}) \frac{N}{\sqrt{R}}}$$ in ft. lb. units.

In metric units

$$C = \frac{23 + \frac{1}{N} + \frac{\cdot 00155}{S}}{1 + \left(23 + \frac{\cdot 00155}{S}\right) \frac{N}{\sqrt{R}}}$$

In actual practice the Manning's coefficient of rugosity and the Kutter's coefficient of roughness are taken to have the same values as given above. In fact they have the same values only for one meter depth.

(*e*) Barnes 1916.

A.A. Barnes produced empirical formula in his book. "Hydraulic flow reviewed." London, for earthen channels in average conditions $V = 58 \cdot 4\ R^{\cdot 694}\ S^{\cdot 496}$

This was the first attempt to find out a formula where the variation of the roughness of the channel section was ignored and a formula covering 130 observations was found without N.

3. (*a*) **Rectangular Channel for Maximum Discharge.**

Using Chezy's formula.

$$\text{Discharge} = AV = AC\sqrt{RS} = A.C.\sqrt{\frac{A}{P_w}} . S$$

Let B be the bed width and D as depth.

Discharge is Maximum for the same slope when P_w is minimum.

The condition is when $D = \frac{B}{2}$ (vide Page 163 of Lewitt Hydrulics)

DISTRIBUTARY DISCHARGE DIAGRAM

For $Q = kd^{5/3}$

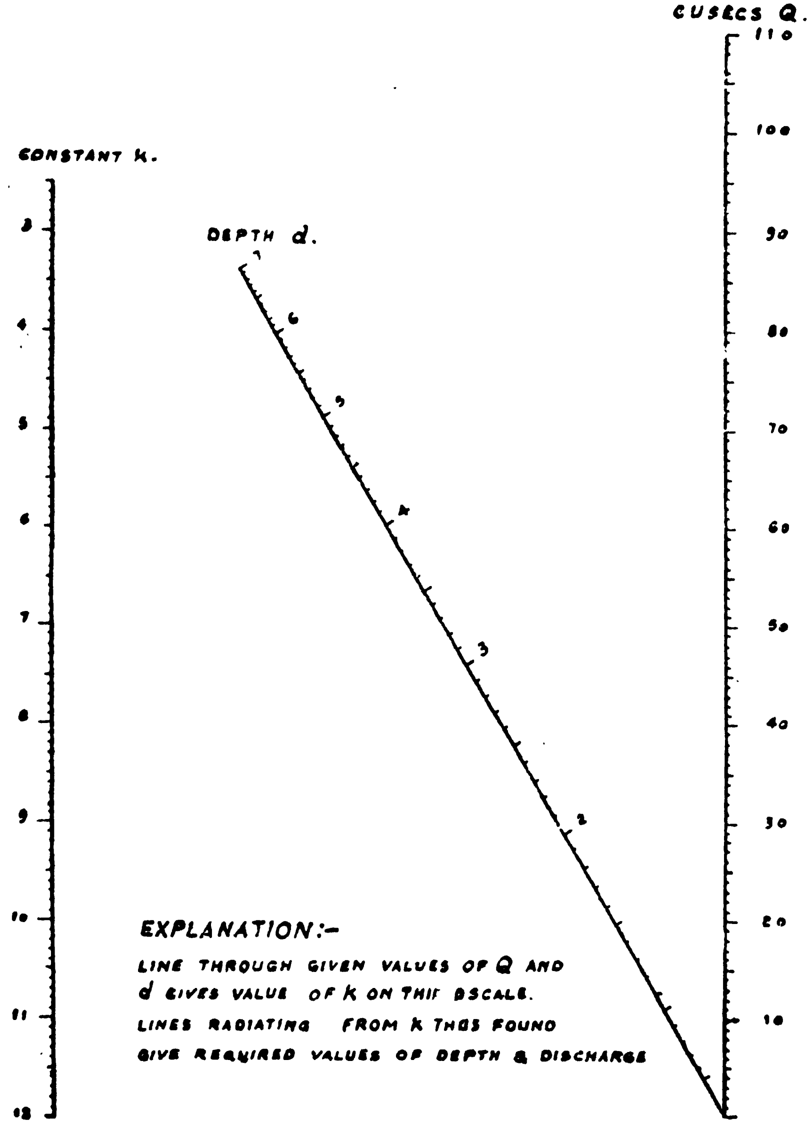


Fig. 3.

The generally accepted values of the co-efficient of roughness 'N' are :—

0·009 Well planed timber in perfect order and alignment.
0·010 Plaster in pure cement and other glazed surfaces.
0·011 Plaster in sand and cement. Iron and other smooth pipes in good order.
0·012 Unplaned timber.
0·013 Ashlar and well laid brickwork.
0·015 Rough Brickwork. Good stone-work in fair order.
0 017 The same in inferior condition.
0·020 Rubble masonry. Coarse brickwork and masonry.
0·0225 Canals in earth above the average in order and regimen.
0·025 Canals and rivers in earth in tolerably good order and regimen.
0·030 Canals and rivers in bad order and regimen.
o·035 Canals and rivers obstructed by detritus and in bad order and regimen.
0·050 Torrents encumbered with detritus.

(*b*) **Trapezoidal channel for economical section.**

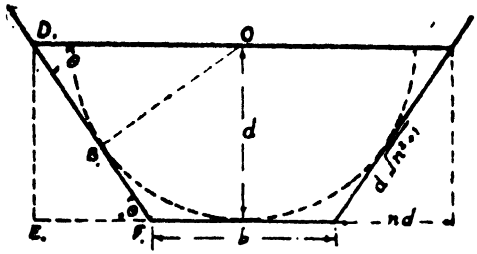


Fig. 4.

The most economical section for a trapezoidal channel will be when discharge is maximum for a given excavation. This will be the case when P_w is minimum

Necessary condition (Lewitts Hydraulics Page 165.)

(*i*) If a semicircle is drawn with centre at O (mid point of surface width) and depth 'd' as radius three sides of the section shall be tangential to it, Fig. 4.

(*ii*) The hydraulic mean depth R shall be equal to $\frac{D}{2}$ where D is the depth of the channel

(*c*) **Circular section for maximum velocity.**

Depth for maximum velocity is given by the maximum value of $\frac{A}{P_w}$ as velocity is proportional to $\sqrt{R}$ when S is constant

$D = 1{\cdot}62\ r$; where r is the radius of circular section.

(vide Page 167 of Lewitt Hydralics)

(*d*) **Circular section for maximum discharge.**

Depth for maximum discharge is got when $\frac{A^3}{P_w}$ is maximum, because discharge

$= A \times C\sqrt{RS} = C\sqrt{\frac{A^3}{P_w}}\,.\,S$; the value of S remaining constant.

$D = 1{\cdot}90\ r$, where D = depth

r = radius of the section (vide Page 168 of Lewitt Hydraulics)

4. Kennedy's Silt Theory.

The above mentioned formula apply to design of open channel sections without any reference to the silt which the water carries in the earthen irrigation channel sections. It was pointed out in the introduction to this chapter that research work was very essential to arrive at the correct section of canals transporting silt so that they, for the successful working of the irrigation system, do neither silt nor scour. The pioneer research work on these lines was done by Mr. R. G. Kennedy, Executive Engineer, Punjab Irrigation, Upper Bari Doab Canal. This is one of the oldest canals in the Punjab constructed in about 1850. Mr. Kennedy selected about 20 sites on the channel of this canal system in about the middle reach. He selected channels which had not been silt-cleared for the past 30 years and had become stable. These channels were neither silting nor scouring. His observations were published in Paper No. : 2826, 1894-95 in the Institution of Civil Engineers, London, England.

(*b*) Sediment in the flowing canal is kept in suspension solely by the vertical component of the constant eddies, which can always be observed over the full width in any stream boiling up gently to the surface. These eddies rise on account of the roughness of the bed and work up against the depth of the channel. From the sides also, some such eddies may occur to a much smaller degree but any such eddies must be for the greater part horizontal and of no silt supporting power. The silt supporting power in a stream is, therefore proportional to the width of the stream and not the total wetted perimeter. According to Kennedy, a Regime Channel is that which neither silts up nor scours.

(*c*) **Mean or the critical velocity.**

The critical velocity is defined to be the mean velocity which will just keep the channel free from either silting or scouring. This is denoted by the letter V_o.

The expression to find out critical velocity was found by Mr. Kennedy by ploting observations with velocity as axis of X and depth as axis of Y. From this the following equation was deduced :—

$$V_o = CD^m = \cdot 84\ D^{\cdot 64}$$

This expression gave the non-silting velocities for channels which had silt of the same character as in the observed channels of Kennedy on the Upper Bari Doab Canal, Table 2. But if the channels had silt of different size and grade, they run non-silting with a velocity different from V_o (mean velocity).

C. V. R. (critical velocity ratio) -Critical velocity ratio is the ratio of the actual velocity in a channel to the critical velocity calculated according to the Kennedy's V_o formula :—

$$V_o = x.C.D^m$$

C. V. R. represents a factor which does not reflect departure from regime, but a factor which is a measure of the variation in the silt conditions from the standard silt conditions on Upper Bari Doap Canal in the Kennedy's observations. It is also a factor which takes into account the scale effect of the dimensions of the channels. Kennedy considered the question of bank erosion quite distinct from the silt question, though of undoubted importance in fixing the limiting depth and the safe limiting velocity of a channel. He considered the safe velocity in the Punjab soil to guard against side erosion as 3·5 feet per second. This would mean a depth not more than 10·0 feet.

(*d*) **Silt transportive power of a channel.**

Kennedy found expression for silt transportive power in a channel as below :—

Let it vary as V_o^n — A = a factor

Total silt carried = $A.B.V_o^n$ — B = width

V_o = non silting velocity

Let p = a percentage of silt by weight in water

Q = Discharge

$\therefore A.B.V_o^n = pQ$ and $Q = B.D.\ V_o$

$A.B.\ V_o^n = p.B.D.\ V_o$

$$V_o = \left(\frac{P}{A}\right)^{1/n-1} \times D^{1/n-1}$$

This is f the same form as Kennedy's

$$V_o = C.D^m$$

equating the values of index of D ; $\cdot 64 = \dfrac{1}{n-1}$

Therefore n = 2·56 or Say 2·5

$\therefore$ the silt transportive power varies as $V_o^{5/2}$

5. Kennedy's Diagrams.

Kennedy carried out no investigation to find out the correct slope formula which applied correctly to the design of irrigation channels. He simply took the Kutter's formula and gave the value of N (coefficient of roughness) equal to ·0225 as the average value for all irrigation channels. He designed the hydraulic diagrams known as kennedy Diagrams, printed at the Thompson College Press at Roorkee, India. He gave diagrams for different slopes showing discharge, depth, velocity and C.V.R. Kennedy's diagram applicable to varying slopes is given in Plate No. XI Vol.III.

(*b*) It was soon found out that the value of N in Kutter's formula was not constant. Kennedy himself suggested the value of ·02 for large canals. Similarly in the case of small irrigation channels, the value of N has been found to be ·025. Evidently arbitrary selection of the Kutter's formula and the value of N = ·0225 was not satisfactory.

(*c*) The author had the occasion to work out the value of N in the Manning's formula for the irrigation channels and found N to vary from ·018 to ·028 as published in author's paper on "Design of Irrigation Channels" in the proceedings of the Institution of Engineers, India, 1936. The variation of N in Manning's formula for irrigation channels in regime was much more than that in the case of Kutter's. Evidently exhaustive and sound research work necessary to find out the correct slope formula applicable to the irrigation channels in regime. This deficiency was made good by Gerald Lacey as explained later.

6. Woods' normal data of design of Kennedy's channels.

From the Kennedy's diagrams for the same value of C.V.R. lot of designs can be got out for the given discharge as shown in an example below. A channel of 60 cusecs may be designed as below with unity C.V.R.

Bed	Depth	Slope	Bed with ratio to depth
21·5	2·0	1 in 5000	10·75
11	2·9	1 in 4000	3·8
4·0	4·3	1 in 2000	·93

Evidently all these designs are not suitable. Kennedy in his note dated 25th August, 1904 admitted the necessity of fixing proportion of bed to depth. Mr. F.W. Woods, Chief Engineer, analysed a lot of data of channels of L.C.C. and issued a table of bed ratio of width to depth in a note of 1917. Table No. 3 shows the normal data of design for Kennedy's channels as given by Mr. Woods.

The Woods permissible bed width depth ratio is plotted in Plate No. XI, in chain dotted line. The suitable channel design will be near about this line. If the silt conditions (silt grade and silt charge) are such that a channel would run non-silting with a C.V.R. lower than unity say ·9, relatively narrower sections will be required and *vice versa* for channels requiring C. V. R. more than unity.

Mr. E. S. Lindley Executive Engineer, Punjab Irrigation, took some observations in Jhang Divison and some in Lyallpur Divison and wrote Paper No. 49 in 1919 Punjab Engineering Congress, Lahore, on "Regime Channels". He worked out therein the following empirical relations.

$$B=3{\cdot}8D^{1{\cdot}61} \quad \text{(A)}$$
$$V_d={\cdot}95D^{{\cdot}57} \quad \text{(B)}$$
$$V_b={\cdot}59D^{{\cdot}55} \quad \text{(C)}$$

where B = bed width in feet
D = depth in feet
V_d = velocity in terms of D in feet per second.
V_b = velocity in terms of B feet per second.

The relation No. (A) confirmed the belief that ratio of bed width and depth did play an important part in determining regime sections. The defect in his obsevations was that they did not relate to channels known to be in regime. The author was in charge of these channels soon after and found that none of them was stable. However, it was a valuable mass of data and has been used by Lacey in his analysis.

7. Lacey's Silt Theory.

(*a*) Gerald Lacey B. Sc., M.I.C.E., I.S.E., P.W.D. Irrigation, United Provinces, India, considerably advanced the subject of silt transportion in channels and their design worked out in his publications as given below :—

(*i*) Proceedings Inst : C.E. London Volume 223 1928-29
(*ii*) do Volume 229 1930-31
(*iii*) do Volume 237 1934
(*iv*) Technical Paper No. 1. P.W.D. United Provinces India 1932
(*v*) Central Board Irrigation Publication No. 20 Simla, India 1939

He has been developing the subject in all these publications by stages. The student is advised to refer to his latest publication No. (*v*) mentioned above as the authoritative verison of his theory even though it gives views and formulae very much in variance with his original conception of the problem as published earlier. A brief summary is given here to introduce the student to the subject.

(*b*) Lacey's conception of Regime.

According to Lacey, the regime conditions shall be established when the following fundamental requirements are fulfilled.

(*i*) Discharge should be constant.

(*ii*) Channel flowing uniformly in unlimited incoherent alluvium of the same character as that transported. Incoherent alluvium is supposed to be the loose granular material which can be scoured out as readily as it can be deposited.

(*iii*) Silt grade (and silt charge) constant.

TABLE NO. 1

Exponential formula of flow of water in open channels in use

'D' Whole Number.	·0	·1	·2	·3	·4	·5	·6	·7	·8	·9
0	0·000	0·023	0·069	0·135	0·217	0·315	0·427	0·552	0·690	0·839
1	1·000	1·172	1·355	1·548	1·756	1·966	2·189	2·421	2·663	2·915
2	3·175	3·444	3·721	4·008	4·302	4·605	4·916	5·235	5·562	5·897
3	6·240	6·591	6·949	7·315	7·688	8·068	8·456	8·851	9·254	9·663
4	10·079	10·503	10·933	11·371	11·815	12·26[illegible]	12·273	13·188	13·659	14·136
5	14·620	15·111	15·608	16·111	16·621	17·137	17·660	18·188	18·723	19·264
6	19·812	20·365	20·924	21·490	22·061	22·639	23·222	23·812	24·407	25·008
7	25·615	26·228	26·846	27·471	28·101	28·737	29·378	30·025	30·678	31·336
8	32·000	32·669	33·344	34·025	34·711	35·402	36·099	36·801	37·509	38·222
9	38·941	39·664	40·394	41·128	41·868	42·613	43·363	44·118	44·879	45·645
10	46·416	47·192	47·973	48·760	49·552	50·349	51·151	51·957	52·768	53·585
11	54·407	55·234	56·064	56·905	57·745	58·591	50·443	60·300	61·161	62·027
12	62·898									

$Q = kd^{5/3}$; Table of power :—$D^{5/3}$, *i.e.* $(\text{depth})^{5/3}$

TABLE No. 2

Table giving values of $V_o = 0{\cdot}84\ D^{\cdot 64}$ (Kennedy's V_o) for all velues of D (the depth) from 1 to 11·9.

D	·0	·1	·2	·3	·4	·5	·6	·7	·8	·9
0	0·000	0·192	0·300	0·389	0·467	0·539	0·605	0·668	0·728	0·735
1	0·840	0·893	0·954	0·994	1·042	1·088	1·132	1·179	1·223	1·266
2	1·310	1·352	1·392	1·433	1·472	1·511	1·549	1·587	1·624	1·661
3	1·696	1·732	1·767	1·803	1·837	1·873	1·907	1·941	1·975	2·007
4	2·040	2·073	2·105	2·137	2·168	2·200	2·231	2·261	2·292	2·323
5	2·352	2·382	2·412	2·441	2·470	2·500	2·529	2·558	2·586	2·615
6	2·645	2·673	2·701	2·729	2·756	2·785	2·813	2·840	2·866	2·893
7	2·920	2·947	2·973	3·000	3·026	3·050	3·076	3·101	3·127	3·153
8	3·178	3·203	3·228	3·254	3·279	3·305	3·330	3·355	3·380	3·404
9	3·428	3·453	3·478	3·502	3·526	3·550	3·574	3·598	3·622	3·645
10	3·667	3·690	3·713	3·736	3·759	3·783	3·806	3·829	3·852	3·875
11	3·897	3·920	3·943	3·966	3·988	4·010	4·033	4·055	4·077	4·099

TABLE NO. 3.

NORMAL DATA OF DESIGN FOR "KENNEDY" CHANNELS

1	2	3	4	5	6	7	8
Discharge	Ratio bed/depth = $\frac{B}{D}$	Bed width (B)	Depth (D)	Gradient 1 in—	Kutter's N	Critical Velocity Ratio	Mean Velocity
2	2·0	2·0	1·0	2500	·0225	1·00	0·80
4	2·3	2·7	1·2	2500	·0225	1·03	1·00
6	2·6	3·5	1·4	2857	·0225	1·00	1·02
8	2·75	4·0	1·5	2857	·0225	1·03	1·07
10	2·9	4·75	1·6	3333	·0225	0·99	1·13
12	3·0	5·25	1·75	3333	·0225	1·00	1·18
14	3·1	5·5	1·80	3333	·0225	1·01	1·22
16	3·15	6·0	1·93	3636	·0225	1·00	1·25
18	3·20	6·25	1·95	3636	·0255	1·00	1·28
20	3·3	6·6	2·0	3636	·0225	1·00	1·32
25	3·4	7·25	2·15	3636	·0225	1·01	1·39
30	3·5	8·0	2·25	3636	·0225	1·02	1·46
35	3·55	8·75	2·45	4000	·0225	0·98	1·48
40	3·6	9·25	2·55	4000	·0225	0·99	1·50
45	3·65	9·75	2·65	4000	·0225	1·00	1·53
50	3·7	10·25	2·75	4000	·0225	1·00	1·56
60	3·8	11·0	2·9	4000	·0225	1·00	1·67
70	3·9	12·0	3·0	4000	·0225	1·01	1·73
80	4·0	13·0	3·20	4000	·0225	0·97	1·76
90	4·1	13·5	3·35	4000	·0225	0·98	1·79
100	4·2	14·5	3·40	4444	·0225	1·00	1·82
125	4·3	16·0	3·65	4444	·0225	1·00	1·91
150	4·4	17·0	3·95	4444	·0225	1·00	2·00
175	4·5	18·5	4·05	4444	·0225	1·01	2·07
200	4·6	19·5	4·30	4444	·0225	1·01	2·15
225	4·7	22·0	4·60	4444	·0225	1·01	2·23
250	4·8	22·0	4·70	4444	·0225	1·01	2·30
300	5·0	24·0	4·8	4444	·0225	1·03	2·38
350	5·15	26·5	5·15	5000	·0225	0·97	2·33
400	5·3	28·5	5·3	5000	·0225	1·00	2·44
450	5·5	30·5	5·5	5000	·0225	1·00	2·49
500	5·7	32·0	5·65	5000	·0225	1·00	2·54
600	6·0	35·5	6·0	5000	·0225	1·00	2·60
700	6·4	39·0	6·1	5000	·0225	1·01	2·70
800	6·8	42·0	6·3	5000	·0225	1·01	2·81
900	7·2	46·0	6·4	5000	·0225	1·02	2·85
1000	7·6	50·0	6·5	5000	·0225	1·03	2·89
1100	8·0	54·0	6·8	5714	·0225	0·98	2·82
1200	8·33	58·0	6·9	5714	·0225	0·99	2·85
1300	8·67	61·0	7·0	5714	·0225	1·00	2·88
1400	9·0	64·0	7·1	5714	·0225	1·00	2·89
1500	9·4	67·5	7·15	5714	·0225	0·98	2·90
1600	9·8	71·0	7·2	5714	·0225	1·00	2·98
1700	10·2	74·0	7·3	5714	·0225	1·00	3·00
1800	10·5	77·0	7·35	5714	·0225	1·01	3·04
2000	11·3	83·5	7·4	5714	·0225	1·03	3·10
2200	12·0	90·0	7·5	5714	·0225	1·04	3·13
2400	12·8	97·0	7·55	5714	·0225	1·03	3·15
2600	13·5	103·0	7·65	5714	·0225	1·02	3·18
2800	14·2	112·5	7·9	6666	·0225	0·96	3·16
3000	15·0	120·0	8·0	6666	·0225	0·98	3·02
3300	16·1	130·0	8·0	6666	·0225	0·97	3·08
3600	17·2	139·0	8·1	6666	·0225	0·97	3·11
3900	18·3	148·0	8·1	6666	·0225	0·99	3·17
4300	19·8	158·0	8·15	6666	·0225	0·98	3·18
4500	20·5	168·0	8·2	6666	·0225	0·98	3·19
5000	22·5	185·0	8·2	6666	·0225	0·98	3·20
6000	26·0	215·0	8·4	6666	·0225	0·99	3·25
7000	29·8	250·0	8·4	6666	·0225	1·00	3·27
8000	33·5	281·0	8·4	6666	·0225	1·01	3·35
9000	37·2	315·0	8·4	6666	·0225	1·01	3·35
10000	41·0	345·0	8·5	6666	·0225	1·02	3·38
10000	41·0	360·0	8·8	8000	·0225	0·93	3.13
15000	60·0	530·0	8·85	8000	·0225	0·94	3·19
20000	78·0	696·0	8·9	8000	·0225	0·94	3·20

Evidently all of these requirements are difficult or rather impossible to be fulfilled in nature. Lacey, therefore, qualified the regime conditions as below :—

True Regime—The channel should flow in an unlimited alluvium plain of the same grade as the material transported. There should in theory be complete freedom for lateral movement. Sandy rivers in alluvium plains achieve to some extent this freedom and by meandering adjust there length and slope.

A constant discharge transporting silt of a given grade and flowing in a self transported alluvium plain of the same grade, tends eventually to assume a gradient solely determined by the discharge and silt grade, the mean velocity, hydraulic mean depth and wetted perimeter will also tend to uinque determination. Such a constant discharge will tend also to transport a fixed regime silt grade.

Artificial channels such as irrigation cnannels can never be in their true regime unless they are neglected like rivers in plains. They may acquire initial regime generally or final regime in some cases.

Initial Regime :—Channels excavated in the first instance with defective slopes and to somewhat narrow dimensions are free by immediately throwing down the incoherent silt on the bed to increase their slopes and by generation of the increased velocity to achieve a non-silting 'Initial Regime'. Such channels if berms are grassed which is usually the case of irrigation channel, will be subject to a considerable lateral restraint. The channels of this type in the Initial Regime attain a working stability and do neither silt nor scour, but the are not in final regime.

Final Regime :—It represents the conditions set up eventually in theory when all the variables are equally free to vary. In the final regime, the wetted perimeter, far from being a variable, is constant.

(*c*) **Lacey's mean velocity relation.**

Lacey like Kennedy recognizes that the silt is suspended by the vertical component of eddies, and advocates that they are generated in a channel section at all points by forces normal to the wetted perimeter. For this reason he adopts the hydraulic mean depth as a variable rather than the vertical depth. He discarded his original reasoning in his latest publication that R being the bed rock of hydraulicians must replace D (the depth).

(*i*) In wide channels, there is hardly any difference between R & D. In eliptic or semi-circular channel section there is no side in the true sense and therefore R is the correct basis, but Kennedy considered that the irrigation channel sections were trapezoidal in shape in which case the eddies produced from the sides do not appreciably help the silt transportation. Kennedy rightly neglected the side in trapezoidal channel sections and therefore, he evolved the critical velocity formula in terms of D and took the total silt transported proportional to bed-width.

(*ii*) There is a great divergence of opinion about the shape of the irrigation channels. The author considers that the irrigation channels are generally trapezoidal in shape when they transport silt or sand, but when they carry fine silt or clay, they tend to be eliptical or sem-circular in shape near the tail reaches of the distributaries and all in the case *Kharif* channels. Rivers in alluvium in plains are not channels like the irrigation ones and the shape of their section in floods may be eliptical even though forming many tributaries and encroached by unyielding islands and vegetation. The term channel can hardly be applied to large rivers of the Punjab subject to avulsions and meanderings as explained in Chapter II of this part. Author does not consider that one set of formulae could be evolved which could be applicable to large rivers of the Punjab and the small irrigation channels known as minors. Lacey claims that his formulae are applicable to channels of all sizes.

(*iii*) Lacey evolved the following formula for the mean velocity (non silting velocity to regime conditions) for the Punjab data of Kennedy and Lindley :—

$$V = K\sqrt{F_l R} = 1{\cdot}1547\ \sqrt{F_l R} \qquad (A)$$

where V = mean regime velocity
K = a constant
f_l = silt factor
R = Hydraulic mean depth

It will be noticed that Lacey has taken his silt factor "f_l" under the root sign. In his own words it is preferable, to denote silt grade by a linear ratio rather than by a velocity ratio and for this purpose the silt factor "f_l" has been introduced by the writer.

In the Punjab conditions, taking the value of f_l equal to unity, he evolved the formula by logarithmic plottings in the form

$$V = 1{\cdot}138\ R^{\cdot 4995} = 1{\cdot}138\ R^{\cdot 5} \text{ nearly} \qquad \text{(B)}$$

Critical velocity ratio as used by Kennedy is equal to the square root of the silt factor f_l.
$V = {\cdot}790\ R^{\cdot 508}$

(*iv*) Lacey's equation for silt factor

$$f_l = X^2 = {\cdot}75\ \frac{V^2}{R} \qquad \text{(C)}$$

where f_l = Lacey's silt factor ; X = C. V. R. (Kennedy)
V = mean velocity ; R = Hydraulic mean depth.

The silt factor is nearly three quarters of $\frac{V^2}{R}$ which is the turbulent acceleration and the silt factor when unity is also consistant with the rugosity co-efficient $N_a = 0{\cdot}0225\ f_l^{1/4}$

This equation can be utilized to calculate the depth of maximum scours for the bridge foundation in large rivers.

$$R = 0{\cdot}75 \frac{V^2}{f_l} = {\cdot}4725 \left(\frac{Q}{f_l}\right)^{\frac{1}{3}} \qquad \text{(D)}$$

(*d*) **Lacey's Wetted Perimeter Discharge Relationship.**

From the logarithmic plottings of large amount of data, Lacey derived the empirical relation in this form.

$$P_w = 2{\cdot}67 \times Q^{\frac{1}{2}} = \frac{8}{3} \times Q^{\frac{1}{2}} \qquad \text{(E)}$$

$$\text{and } \frac{P_w}{R} = 7{\cdot}111 \qquad \text{(F)}$$

$$V_o = 0{\cdot}141 \frac{P_w}{R} \qquad \text{(G)}$$

where P_w = wetted perimeter
Q = discharge in cusecs.
R = H.M.D.
V_o = mean velocity.

The constant 2·67 is not a true constant as it varies with different localities. In the Punjab, the Research Institute, Lahore, found its value to vary from 2·2 to 3·12 from 24 regime sites kept under observations for a long time. Lacey remarked in the Indian Engineering, Calcutta, August, 1937 as below :—

"It must be remembered that the expression $P_w = 2{\cdot}67\ Q^{\frac{1}{2}}$ gives the value of the minimum stable perimeter in channels in incoherent alluvium with considerable accuracy. All the data so far avilable with considerable accuracy tends to confirm the expression. The value of the constant in individual channels varies within limits roughly from 2·20 to 3·20.

If the banks are tenacious, the width may be less, if the bed is tenacious, the width may be greater, therefore, there is no absolute rigidity in the constant 2·67 when applied to small channels with even a small admixture of clay in the bed or banks.

In large channels it is to our interest to make them as deep and narrow as is consistent with stability. In small channels approaching the limiting velocity there is no objection to making them wider if the clay present provides a small additional measure of stability."

The upper limit of the wetted perimeter constant is about 3·16 and the lower one 2·2 for the Punjab Canals.

(*e*) **Lacey's general ragime equation.**

(*i*) It has been said that Kennedy's acceptance of Kutter's formula with value of N = ·0225 was not very satisfactory. Lacey's attempt to evolve regime flow equation is a great advance to our knowledge. Ignoring the attempts of Mr. Lacey to give mathematical proof of his general regime equation using the dimensionless numbers he analysed a large amount of data and his regime equation fits the data very well. Lacey's general regime equation entirely fitted with the author's observations published in paper read in the Institute of Engineers, India, 1936, giving variation of N in Manning's formula. Lacey rightly claims on page 12 Central Board of Irrigation Publication No. 20 "it is very improbable that any improvement can be made in this relation in the near future."

Lacey's formula for regime flow

$$V = 16\, R^{2/3}\, S^{1/3} \qquad \text{(H)}$$

This equation applies to perfect regime and has the decided advantage that it applies to all earthen channels in regime irrespective of any rugosity consideration.

(*ii*) In the case of channels not in perfect regime but postulating between the initial and the final regime. Lacey derives his flow equation in the form of Manning type in which Chezy's co-efficient is represented by $\frac{R^{1/4}}{N}$ in metric units and by $\frac{1{\cdot}3458R^{1/4}}{N}$ in foot units.

Lacey denotes his rugosity co-efficient, as N_a and terms it an absolute rugosity co-efficient that is a co-efficient determined solely by the average size and density of the incohorent bed material of the channel. The standard grade of silt is that which cannotes a rugosity co-efficient of ·0225 when the hydraulic mean depth is one meter. It is, therefore, permissible to write in terms of the Lacey's definition.

$$N_a = {\cdot}0225 m_d^{1/2} = {\cdot}0225 f_l^{1/4} \text{ where } m_d = \text{mean diameter} \qquad \text{(I)}$$

(*iii*) **Vertical Exaggeration.** Lacey pictures the hydraulic channels as the wetted surface spread out on the horizontal plane, with the hydraulic mean depth erected thereon. Prototype and model may, therefore, be depicted as in Fig. 5.

perspective view of wetted surface flattened
Out by a Dimensioned Hydraulic Mean
Radius erected thereon

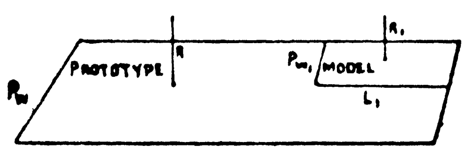


Fig. 5.

Let the horizontal scale of the model be $\frac{1}{X}$ and the vertical scale be $\frac{1}{Y}$ then the ratio $\frac{X}{Y}$ is termed the "exaggeration" E (*i.e.*, the ratio between the vertical and horizontal scales). Then, in the horizontal.

$$\frac{L}{L_1} = \frac{P_w}{P_{w1}} = X$$

in the vertical, $\frac{R}{R_1} = Y$

$$\therefore E = \frac{X}{Y} = \frac{P_w}{P_{w1}} \cdot \frac{R_1}{R}$$

But from equation (F)

$$\frac{P_w}{R}=7{\cdot}111\ V$$

$$\therefore \frac{P_w}{R}\cdot\frac{R_1}{P_{w1}}=\frac{V}{V_1}=E$$

Again H_l and H_{l1}, be the differences in levels between corresponding points on prototype and model, at the end of lengths L and L_1 respectively then $\frac{H_l}{L}=S$ and $\frac{H_{l1}}{L_1}=S_1$

$$\therefore \frac{S}{S_1}=\frac{H_l}{L}\cdot\frac{L_1}{H_{l1}}=Y.\ \frac{1}{X}=\frac{1}{E}$$

$$\therefore \frac{S}{S_1}=\frac{1}{E}=\frac{V_1}{V} \tag{J}$$

This relationship is of vital importance in model experiments.
Returning now to the Chezy's formula

$$\frac{V}{V_1}=\frac{C}{C_1}\cdot\frac{R^{1/2}S^{1/2}}{R_1^{1/2}S_1^{1/2}}$$

$$\text{or } \frac{C}{C_1}=\frac{V}{V_1}\cdot\frac{R_1^{1/2}}{R^{1/2}}\cdot\frac{S_1^{1/2}}{S^{1/2}}$$

Substitute for S from equation (J)

$$\frac{C}{C_1}=\frac{V^{3/2}}{V_1^{3/2}}\cdot\frac{R_1^{1/2}}{R^{1/2}}$$

But Lacey pastulates that streams flowing in envelopes of the same silt (*i.e.*, whose f_l value is the same) must have the same co-efficient of absolute rugosity N_a, hence for such channels, from equation (A).

$$\frac{V}{V_1}=\frac{R^{1/2}}{R_1^{1/2}}$$

Substitute value of $\frac{V}{V_1}$

$$\frac{C}{C_1}=\frac{R^{1/4}}{R_1^{1/4}} \tag{K}$$

Therefore C in the Chezy's formula when applied to regime channels varies as $R^{1/4}$
Hence the Chezy's farmula may be written

$$V=KR^{3/4}\ S^{1/2}$$

$$\text{or } V=\frac{K_1}{N_a}.\ R^{3/4}\ S^{1/2} \tag{L}$$

where N_a is a true co-efficient (a measure of the absolute rugosity of the silt envelope).

Determination of the value of K_1

Now the values of Cutter's N and Manning's N coincide at a depth of one metre. Lacey decided to retain this coincidence, familiar to all hydraulicians.

$$V=\frac{1{\cdot}4858}{N}R^{2/3}\ S^{1/2}\ \text{(Manning)}$$

$$=\frac{1{\cdot}4858}{N}(3{\cdot}208)^{3/4}\ S^{1/2}$$

$$=\text{(Lacey)}\ \frac{K_1}{N_a}(3{\cdot}208)^{3/4}\ S^{1/2}$$

whence $K_1 = 1{\cdot}3458$

$$\text{and } V = \frac{1{\cdot}3458}{N_a} R^{3/4} S^{1/2} \qquad \text{(M)}$$

the above formula is strictly applicable to regime channels, but may be used freely as a substitute for either Kutter's or Manning's in non-regime alluvial channels or channels with rigid boundaries etc., with improved accuracy and greater facility, by adjusting the value of N_a.

(*iv*) **Regime Slope.**

Lacey evolved his regime slope formula in 1932 in Technical Paper No. 1. United Provinces, as $S = \frac{1}{1788} f_l^{5/3}/Q^{1/6}$ but in 1932 in Central Board Irrigation Publication No. 20, it has been modified by him to

$$S = \frac{1}{1844{\cdot}3} f_l^{5/3}/Q^{1/6} = {\cdot}000542\, f_l^{5/3}/Q^{1/6}$$

(*v*) **Lacey's Shock theory.**

The use of all such equations presents great difficulty in assigning values to the rugosity co-efficient. There is no limit to Kutter's "over-all" co-efficient N in obstructed non-regime channels, and for such channels no rational equation can be framed. One source of error is in measuring the slopes. This if measured locally over the discharge run, may be too small to be accurately assessed. If measured as gross slope over a large slope base, the hydraulic mean depth at the discharge site, owing to the natural tendency to select narrow reaches for measurements, will often be in excess of the average hydraulic mean depth throughout the slope base. The slope as measured will, thus be in excess of the slope applicable to the hydraulic mean depth as measured. The slope may also include "shock" due to bends or irregularities in the channel, and also "shock due to channel condition" as opposed to channel material. In such circumstances it is preferable to assign to N_a a value which the bed material warrants, and to account separately for the energy destroyed in shock by making an appropriate deduction from the gross slope as measured. The equation can then be written in the form.

$$V = \frac{1{\cdot}3458}{N_a} R^{3/4} (S - s)^{1/2} \qquad \text{(N)}$$

The following table of values of the rugosity co-efficient N recommended for different channel conditions demonstrates the futility of seeking other than an approximate empirical solution and the advantage of employing a simple exponential equation of the Manning type. Value of "s" the fraction of the slope "S" lost in shock due to channel conditions.

Channel condition	N	"s"	Description of channel
Perfect	·0250	·000 S	Natural stream channels,
Good	·0275	·174 S	Straight bank, full stage,
Fair	·0300	·306 S	no rifts or deep pools,
Bad	·0330	·426 S	"Engineering News" Vol. 75,1916 Page 373.
Very good	·0225	·000 S	Earthern channels and canals
Good ,	·0250	·190 S	under ordinary conditions,
Indifferent	·0275	·331 S	Buckley's Irrigation
Bad	·0300	·437 S	Pocket Book.

The table shows that more than forty per cent of the energy destroyed in the channel may be dissipated by channel irregularities. It is important to note that if a main canal is excavated in the first instance to an excessive slope it will adjust itself by the creation of channel irregularities. Such a canal would eventually achieve some kind of balance and remain stable, the slope however, would be in excess of that which the garde of silt transported would demand in a regime channel.

8. Lacey's Diagrams.

Lacey published his diagrams in Technical Paper No. 1, P.W.D., United Provinces, India. These diagrams were based on his original equations.

$V_o = 1{\cdot}17\, f_l^{1/2} R^{1/2}$ (A) $\qquad V_o = \left(\frac{Qf_l^2}{3{\cdot}8}\right)^{1/6}$ (B)

$P_w = 2{\cdot}67\, Q^{1/2}$ (C) $\qquad R = \frac{{\cdot}7305\, V_o^2}{f_l}$ (D)

$$S = \frac{f_l^{5/3}}{1788\, Q^{1/6}} \qquad \text{(E)}$$

$$D = \frac{A}{2RK} - \left\{\left(\frac{A}{2RK}\right)^2 - \frac{A}{K}\right\}^{1/2} \qquad \text{(F)}$$

$$B = \frac{A}{D} - sD \qquad \text{(G)}$$

$$K = 2(rs^2+1)^{1/2} - s \qquad \text{(H)}$$

The channel section was assumed to be trapezoidal with sides ½ to 1 though he believed that it would become cup-shaped after a few years running.

Side slopes	S	K
Vertical	0	2·0
One half to one	0·5	1·736
60° slope	0·5774	1·7321
One to one	1·0	1·8284
1½ to 1	1·5	2·105

The diagrams are given in Plate XV, Vol. III but they need to be revised for the equations finally arrived at by him as summarised in paragraph (8) above. The original diagrams can be used till new ones are produced.

9. Examples.

(*a*) Design a regime channel section disharge 500 cusecs and $\frac{V}{V_o} = 1{\cdot}1$

$$f_l = \left(\frac{V}{V_o}\right)^2 = 1{\cdot}1^2 = 1{\cdot}21$$

$$V_o = \left(\frac{Qf_l^2}{3{\cdot}8}\right)^{1/6} = \left(\frac{500 \times 1{\cdot}21^2}{3{\cdot}8}\right)^{1/6} = 2{\cdot}4 \text{ feet per second}$$

$$A = \frac{500}{2{\cdot}4} = 208 \text{ sft.}$$

$$R = \frac{{\cdot}7305 V_o^2}{f_l} = \frac{{\cdot}7305 \times 2{\cdot}4^2}{1{\cdot}21} = 3{\cdot}47 \text{ feet}$$

For side slope ½ to 1, K=1·736

$$D = \frac{A}{2RK} - \left\{\left(\frac{A}{2RK}\right)^2 - \frac{A}{K}\right\}^{\frac{1}{2}} = 17{\cdot}26 - (298 - 120)^{1/2} = 3{\cdot}93 \text{ ft.}$$

$$B = \frac{A}{D} - sD = \frac{208}{3{\cdot}93} - \frac{1}{2} \times 3{\cdot}93 = 51{\cdot}04 \text{ ft.}$$

The channel section should have bed width=51·0 feet and depth=3·93 feet

$$S = \text{Regime slope} = \frac{f_l^{5/3}}{1788 Q^{1/6}} = \frac{1{\cdot}21^{5/3}}{1788 \times 500^{1/6}} = {\cdot}00027$$

say=·27 feet per thousand.

It is to save these calculation that the diagrams are used.

(*b*) Design a channel section for 200 cusecs and $\frac{V}{V_o} = {\cdot}9$

Lacey's silt factor $= f_l = \left(\frac{V}{V_o}\right)^2 = \cdot9^2 = \cdot81$

From diagram XV (b) for $f_l = \cdot81$ and $Q = 200$ cusecs.

$B = 30$ ft. and $D = 3\cdot45$ feet.

From diagram XV (*c*) for $f_l = \cdot81$ and $Q = 200$ cusecs.

Regime slope $S = \cdot16$ feet per thousand feet.

(*c*) Section for 80 cusecs with $\frac{V}{V_o} = \cdot8$

Lacey's silt factor $f_l = \left(\frac{V}{V_o}\right)^2 = \cdot8^2 = \cdot64$

From diagram XII (A) for $f_l = \cdot64$ and $Q = 80$ cusecs.
Depth $D = 3\cdot05$ feet and bed width $B = 16\cdot5$ feet.
From Diagram XII (*c*) for $f_l = \cdot64$ and $Q = 80$ cusecs.
Regime slope $S = \cdot1$ feet per thousand.

(*d*) Design Kennedy's section for discharge 200 cusecs and $\frac{V}{V_o} = 1\cdot0$ and given slope of $\cdot2$ feet per thousand.

From Plate XII for slope $\cdot2$ feet per thousand and $Q = 200$
$B = 26$ feet and $D = 3\cdot8$ feet.
From the same diagram, according to Wood's normal data of design for $Q = 200$ cusecs $B = 21$ feet and $D = 4\cdot3$ feet.
The slope required $= \cdot216$ feet per thousand.

10. A summary of Lacey's Conclusions.

"In all regime channels in incoherent alluvium the primary fundamental variables are the mean velocity, the hydraulic mean depth, the water surface slope and the bed silt grade.

Of the four variables V, R, S, and f_l, two only are necessary in order to obtain the third and fourth. The first epuation is $V = (4/3)^{1/2} \sqrt{f_l R}$

In this equation when the value of the silt factor is unity the grade of sand has a rugosity co-efficient N_a of $\cdot0225$ and standard sand is thus defined. When the hydraulic mean depth is equal to one metre ($3\cdot2808'$) the value of N_a is identical numerically with the value of N assigned by the Kutter, Manning or Forchheimer equations for flow.

The second equation is $V = 16\cdot05\ R^{2/3}\ S^{1/3}$

In this general regime equation the rugosity is implicit, and the equation is applicable to every type of regime alluvial channel from very fine silt to boulders. From these two fundamental equations a variety of other equations can be derived. If a flow equations of the Manning or Forchheimer type is sought it must take the from $V = \frac{1\cdot3458}{N_a} R^{3/4}\ S^{1/2}$

The value of N_a is given by the equation $N_a = \cdot0225\ f_l^{1/4}$
If it is desired to apply the flow equation to non-regime channels in which shock and other channel conditions play a part, the flow equation takes the from $V = \frac{1\cdot3458}{N_a} R^{3/4}\ (S - s)^{1/2}$ in which s represents the fraction of the gross slope destroyed by shock.

In regime channels, free from shock, and of which the silt is of standard incoherence, the silt factor and mean bed silt grade are correlated by the equation $f_l = 1\cdot76 m^{1/2}$ the diameter 'm_r' being measured in millimetres.

If shock is present in a regime channel as a variable, the silt factor becomes an inverse function of the silt grade and $m_r^{1/2}$. $f_{lr} = m^{1/2}$. $f_l = m'^{1/2}$. f'_l etc. In the modified Bose relation

where $q = R\ V = \cdot375\ Q^{1/2}$: $S \propto m^{5/6}/q^{1/3}$
the slope is a function of bed silt grade and the discharge intensity 'q' the shock being implicit. In the equivalent expression of Lacey $S \propto f_l^{5/3}(f_r/f_l)^{5/3}/q^{1/3}$; the shock is explicit. When there is no shock the expressions are identical. The modified Bose equation applies to silt of a standard coherence. The equation of Lacey $S \propto f_l^{5/3}/q^{1/3}$

$$\text{or } S = 1/1844{\cdot}3\ F_l^{5/3}/Q^{1/6} = {\cdot}000542F^{5/3}/Q^{1/6}$$

when shock it is absent it is applicable irrespective of the precise coherence being implicit.

The simplest dimensioned expression for the general regime equation is the Lacey Malhotra equation $V/V_o \propto (RS/m)^{1/2}$; in which the kinematic viscosity is implicit.

Under ideal conditions when all the variables are equally free to vary, the wetted perimeter and discharge are correlated by the Lacey's expression $P \propto Q^{1/2}$; from this equation coupled with the two first fundamental equations in terms of V. R. S. and f_l, all other regime relations follow vide page 38 of C. B. I. Publication No. 20.

11. Comparison of Kennedy and Lacey Theories.

(*a*) The theoretical conception of silt transportation is the same in both cases. The silt is carried by vertical eddies caused by the friction against perimeter. Kennedy considers irrigation channel section generally trapezoidal and, therefore, neglects the sides and derives his critical velocity formula in terms of d(depth). Lacey believes that the regime sections in alluvium are cup-shaped and therefore, derives his regime velocity formula in terms of R (Hydraulic mean depth). Lacey considers that the irrigation channels cannot be in perfect regime on account of the imposed sections in the beginning and the subsequent berm growth. They cannot therefore, be regime channels flowing in self-silted alluvium and hence they can only be called in initial regime. Kennedy deals with irrigation channels only and calls them in regime when they do neither silt nor scour.

The author considers the change of D to R as a change of form only. In wide channels they have very nearly the same value. Both claim to satisfy the same data. The empirical relations derived by both are essentially correct. The superiority of Lacey's work over that of Kennedy is due to other reasons mentioned hereafter.

(*b*) In the regime velocity equation $V = 1{\cdot}13\sqrt{F_l \cdot R}$ Lacey used f_l, as a silt factor instead of Kennedy's C.V.R., Lacey's $f_l = \left(\frac{V}{V_o}\right)^2$ of Kennedy. Kennedy simply stated C. V. R. $\left(\frac{V}{V_o}\right)$ varried according to the silt conditions (silt charge and silt grade). Lacey did not leave f_l as a guess work but correlated it with mean diameter of the bed silt in his formula $f_l = 1{\cdot}76m^{1/2}$ for regime channels and in non-regime channels $N_a = {\cdot}0225\ f_l^{1/4}$. This is a very distinct improvement made by Lacey. This omission in Kennedy's work has, however, been made good in author's paper read in the Institution of Engineers, India, 1936, the summary of which is given in paragraph 13 of this Chapter.

(*c*) The selection of Kutter's formula and giving N an arbitary value of ·0225 by Kennedy in his work was not correct. Lacey has produced a general regime flow equation in formula $V = 16\ R^{2/3}\ S^{1/3}$ after analysing a very large mass of data of regime channels. Mr. R. K. Khanna, Assistant Engineer Punjab Irrigation, also worked on similar lines and evolved a regime flow equation in the form $V = 13{\cdot}07\ R^{\cdot58}\ S^{\cdot3}$ which is of the same form as Lacey's. Lacey's work in working out Slope equation for initial regime and in investigations of shock in irrigation channels stands out conspicuously in the domain of research.

(*d*) Kennedy's work suffered from the defect that he did not notice the importance of bed-width-depth ratio. This deficiency was made good by the work of Woods and Lindley as mentioned before. Lacey produced his formula to this effect in terms of wetted perimeter *i.e.*, $P \propto Q^{1/2}$ and the value of K in the formula $P = KQ^{1/2}$, is got out by Lacey as $K = 2{\cdot}67$ for average regime conditions. He admits that there is no absolute rigidity of the constant value and accepts its variation from 2·2 to 3·2 for regime irrigation channels. Similarly in the case of Woods' bed width ratios there is no absolute rigidity because they are for unity C. V. R. and the ratio must change for channels carrying silt requiring C. V. R. other than unity. It is, therefore, apparent that the regime channel sections can have wetted perimeters 20% more of less than the Lacey's formula $P_w = 2{\cdot}67\ Q^{1/2}$ and similarly following Woods' normal data of design of Kennedy's channel a variation of 20% is quite likely. We do need a guide to limit our selection of bed width and depth ratios whether it is in the form of Lacey's wetted perimeter formula or in the form of Woods' bed width and depth ratios.

(*e*) Kennedy did not fix regime slopes for his channels. However, his diagrams clearly show that steeper slopes are required for small channels and the flatter ones for large channels. In his original work, Lacey produced a regime slope formula in the form $S = \frac{f_l^{5/3}}{1788\ Q^{1/6}}$ and in his final work, Central Board Publication No. 20, he put this in the form $S \propto \frac{f_l^{5/3}}{q^{1/3}}$ where f_l is the silt factor and q discharge intensity per foor width of the channel. The final formula shows that there is no rigidity about the constants. The original formula of Lacey allows very excessive slopes. There is not a single channel on the Lower Jhelum Canal, Lower Chanab Canal and Lower Bari Doab Canal where Lacey's regime slopes are available. These canals would have never been constructed if the designers had cared for the Lacey's regime slopes. The author worked out in Paper No. 154, Punjab Engineering Congress 1932, in reply to Lacey's criticisms that if Lacey's regime slopes were allowed in the Mithalak distributary, the head supply shall have to be raised by 4·5 ft. and if the same process were extended to head of Lower Jhelum Canal, it would have to be raised about 15 feet even after consuming all existing falls. The author considers that a further limitation in the form of regime slopes is not required. It is enough to design the regime channel sections with the available slope based on the general regime equation and the regime or critical velocity formula limited by the wetted perimeter or bed width depth ratio considerations.

If a regime slope formula is to be used as a guide, then the Punjab Irrigation Research Institute (1937) formula $S = {\cdot}00209\ m_d^{\cdot 86}/Q^{\cdot 21}$ is better applicable to the Punjab conditions (1937, Annual Report) where m_d=mean diameter in millimeters. In designing irrigation channels according to Kennedy's theory, the author's regime slope formula (paragraph 13 below); $S = \frac{X^3}{7500}\left(1 + \frac{1{\cdot}74}{P + {\cdot}5}\right)^2$ is the best guide as it is dimensionally more correct than Lacey's regime slope formulae mentioned above, for all the quantities in this are dimensionless.

12. Defects in Kennedy's and Lacey's Theories.

(*a*) Both of them aimed to find out the average regime conditions. None of them considered the effect of the varied silt conductive powers of the outlets and the off-takes on the regime of a channel. Mr. Lacey stated that the silt factor was constant for a canal system which is not a fact, while Kennedy said that C. V. R. decreased towards the tail of a channel.

(*b*) They also took no account of silt left in the channel by water that is lost in absorption which is as much as 12 to 15% of the total discharge of a channel.

(*c*) The effect of silt attrition was also ignored in both of these theories. The silt size does actually go on decreasing by the process of attrition among the rolling silt particles dragged along the bed.

(*d*) They also took no notice of the scale effect. The Ravi river at Sidhnai, Indus at Sukkur and a small irrigation minor may have Lacey's silt factor unity or Kennedy's C.V.R. unity but actually they carry silt charge and silt grade very many times different.

(*e*) Neither Lacey nor Kennedy has been able to define the silt grade and silt charge and the size of the channel for unity silt factor or unity C.V.R.

13. Author's Design of Irrigation Channels.

(*a*) The author published his paper on "Design of Irrigation Channels" in the proceedings of the Institution of Engineers, India, 1936. The first 4 factors as enumerated in paragraph

12 above were dealt therein as influencing the change of C.V.R. in Kennedy's theory. The selection of C.V.R. is no longer a guess work which was the major defect in Kennedy's theory. A brief summary is given here and the student should refer to the original publication for a detailed study and for the proofs of the formulae.

(*b*) **Notations.**

a_1 = Silt charge in water at the beginning of a reach and represents the ratio of the amount of the silt carried to the volume of water containing it ; a_2 = Silt charge in water at the end of a reach ; f_1 = Grade of silt charge in the beginning of a reach and represents the average diameter of all the silt particles above 0·04 milimeter ; f_2 = Grade of silt charge at the end of a reach ; p_1, p_2 etc. = The silt conducting powers of outlets in the reach expressed as a fraction with respect to unity in the reach by weight ; q_1, q_2 etc. = The corresponding discharge in cusecs of outlets having silt conducting power as p_1, p_2 etc. ; A = Absorption in cusecs in a reach ; Q_1 = Discharge in cusecs at the beginning of a reach ; Q_2 = Discharge at the end of a reach.

$r_1 = \frac{a_2}{a_1}$ = Ratio of silt charge at the end of a reach to that at its beginning.

$r_2 = \frac{f_2}{f_1}$ = Ratio of silt grade at the end of a reach to that at its beginning.

$r_3 = \frac{Q_1}{Q_2}$ = Ratio of discharge at the beginning of a reach to that at its end.

$\lambda = \frac{D_1}{D_2}$ = Ratio of depth at the beginning of a reach to that at its end.

$R_c = \frac{x_2}{x_1}$ = Ratio of C.V.R. at the end of a reach to that at its beginning.

B = Bed width in feet in trapezoidal section with side slope 1/2 to 1.

D = Depth in feet trapezoidal section with side 1/2 to 1.

$= d\ \frac{B}{D}$ = Ratio of bed to depth.

X = C.V.R. (Kennedy) in formula. $V = xV_o = xc\ D_m = x{\cdot}82D^{\cdot 66}$

(*c*) Formulae.

Change in silt charge by varied silt conduction by off-takes and absorption losses.

$$r_1 = \left\{ \frac{Q_1 - (p_1q_1 + p_2q_2 + p_3q_3 + \ldots\ldots)}{Q_1 - (A + q_1 + q_2 + q_3 + \ldots\ldots)} \right\} = \frac{Q_1 - (p_1q_1 + p_2q_2 + p_3q_3 + \ldots)}{Q_2} \tag{A}$$

$$r_2 = \left\{ \frac{Q_1 - (p_1{}^3q_1 + p_2{}^3q_2 + p_3{}^3q_3 + \ldots\ldots}{Q_2} \right\}^{\frac{1}{3}} \tag{B}$$

Change in C. V. R. for r_1 and r_2 ; $R_c = \frac{x_2}{x_1} = r_1{}^{\cdot 33}.r_2{}^{\cdot 3}.r_3{}^{\cdot 1} = r_1{}^{\cdot 33}.r_2{}^{\cdot 3}.\lambda^{1/6}$ (C)

$$S = \frac{x^3}{7500}\left(1 + \frac{1{\cdot}74}{p + {\cdot}5}\right)^2 \tag{D}$$

$$D = \left\{ \frac{Q}{{\cdot}82 \times (p + {\cdot}5)} \right\}^{3/4} \tag{E}$$

The ratio of C. V. R. at the end of a reach to that at its beginning can be calculated from equation (C) or taken from plate XIII. (A) Vol. III.

Actual non-silting velocity of a channel section is a multiple of V_o by some factor which depends upon the silt-charge and its grade. Kennedy called it Critical Vetocity Ratio (C. V. R.) and in Lacey's theory it appears as silt factor. From the formula given above for the change in C.V.R. it is clear that it not only depends on the change of silt grade but also on the change of silt-charge and the channel section inversely as $D^{1/6}$. It can only be a factor, as used by Kennedy and may conveniently be called Critical Velocity Ratio (C.V.R.) and not a silt factor. A unity C.V.R. or unity silt factor (Lacey) have not the same significance for a river, canal, distributary and a minor. If a canal was running non-silting with C. V. R. $=\cdot85$ (for example Lower Chanab Canal at head) it does not follow that its distributaries, drawing the same silt charge and grade, would run non-silting with C.V.R. $=\cdot85$. They would require C.V.R. as influenced by the factor λ in equation (C) above. It is therefore, that the off-taking channels from the Lower Chanab Canal are actually running with C. V. R. more than unity, even after considerable side exclusion at their head regulators. It is nothing but misnomer to call C. V. R. or any power of it as silt factor.

(*d*) The design diagrams XIII (B) and (C) for equations (D) and (E) are based on Kennedy's critical velocity formula and Lacey's general regime equation $V=16\ R^{2/3}\ S^{1/3}$. With the worked out value of C. V. R. from XIII (A) as explained above, and the available slope, ρ (the ratio of bed width to depth) is got from plate XIII (B).

It is just watched that the value of ρ is not off by more than 20% from Wood's bed width-depth ratio or Lacey's relation $\frac{P_w}{R}=7\cdot2V$. If ρ is out by more than permissible, the tentative longitudinal section of the channel should be changed. If it is a case of excess slope giving relatively low value of ρ, it should be flattened by designing suitable falls. If the slope is flat giving value of ρ more than the permissible and also if it could not be steepened, then maximum permissible value of ρ should be selected and design completed from equations (D) and (E). In the latter case, the non-silting design is not possible and the section designed should be declared to need annual silt clearance just before the period of keen demand for equitable distribution of supplies to the outlets. With this known value of ρ, depth is got from Plate XIII (C) and bed width is then equal to ρD.

(*e*) It will be interesting to mention here that the regime slope formula as worked out in the said publication is in the form of equation (D).

Slope is a function of x and ρ only. Both are ratios and have no dimensions. The slope, which is dimensionless is uniquely determined by these two dimensionless numbers. The part played by x is much more predominant than that by ρ, as explained in the discussion of the paper.

Slope is essentially independent of discharge. The channels could certainly be run non-silting even with flat slopes, if permitted by dimensionless numbers x and ρ. The bed width-depth ratio is determined independent of discharge for a given slope and silt conditions.

14. Examples Illustrating the Use of Author's Nomograms.

(1) Head reach from Head to R. D. 16000 Mithalak Disty. Parent channel section U. S. of the head, discharge=1710 cusecs, Bed=74 ft., Depth=7·6 and side slopes ½ to 1.

$$\text{Area}=7\cdot6(74+\frac{7\cdot6}{2})=591\cdot3 \text{ sq. ft.}$$

$$\text{Velocity}=\frac{1710}{591\cdot3}=2\cdot892 \text{ ft. per second and C.V.R.}=\frac{2\cdot892}{3\cdot08}=\cdot94$$

Silt conductive power of the head regulator by weight as determined by actual experiments is equal to 90%.

$$\therefore r_1=0\cdot9; \lambda=\frac{\text{Depth in parent channel}}{\text{Depth in off-take}}=\frac{7\cdot6}{3\cdot7}=2\cdot05 \text{ and let } r_2=\cdot9$$

Using formula, $R_c=r_1^{1/3}.\ r_2^{\cdot3}.\ \lambda^{1/6}$, from Nomogram Plate XIII (A); $R_c=1\cdot05$
$\therefore$ C.V.R. required in the head reach of distributary $=\cdot94\times1\cdot05=\cdot985$.
Discharge of Mithalak Distributary at head $=143\cdot5$ cusecs

Slope=1 in 4444, C.V.R. $=0\cdot985$, N $=\cdot023$, from Plate XIII (C) using the nomogram. $\rho=5\cdot0$. Similarly from Nomogram No. XIII (B) when

$Q = 143{\cdot}5$ cusecs, $\rho = 5{\cdot}0$ and C.V.R. $= {\cdot}985$
$\therefore$ depth $= 3{\cdot}68$ feet
Now Bed width $= \rho \times D = 3{\cdot}68 \times 5 = 18{\cdot}4$ feet
Use $D = 3{\cdot}7$ and Bed width $= 18{\cdot}25$ feet.

(2) Reach from 16000 to 24500
Discharge $= 133$ cusecs, $N = 0{\cdot}023$
Slope $= 1$ in 4200
Discharge of outlets in upper reach $= 7{\cdot}5$ cusecs
Absorption in outlets in upper reach $= 2{\cdot}8$ cusecs
Silt conductive power of outlets in upper reach $= 122\% = 1{\cdot}22$

$$r_1 = \frac{Q_1 - (p_1 q_1 + p_2 q_2 + \ldots\ldots)}{Q_1 - (A_1 + q_1 + q_2 + \ldots\ldots)} = \frac{143{\cdot}5 - (7{\cdot}5 \times 1{\cdot}22)}{143{\cdot}5 - (2{\cdot}8 + 7{\cdot}5)} = 1{\cdot}010$$

$$r_2 = \left\{ \frac{Q_1 - (p_1^3 q_1 + p^3 q_2 + \ldots\ldots\ldots)}{Q_1 - (A_1 + q_1 + q_2 + \ldots\ldots\ldots)} \right\}^{1/2} = \left\{ \frac{143{\cdot}5 - (7{\cdot}5 + 1{\cdot}22^3)}{133{\cdot}7} \right\}^{1/2} = {\cdot}99$$

$$r_3 = \frac{Q_1}{Q_2} = \frac{143{\cdot}5}{133{\cdot}7} = 1{\cdot}08$$

$\therefore$ R_c = Ratio of C.V.R. at 16000 to that at its head $= r_1^{1/3}.\ r_2^{\cdot 3}.r_3^{1/10}$
From Nomogram No. XIII (A) $R_c = 1{\cdot}011$
$\therefore X = {\cdot}985 \times 1{\cdot}011 = 1{\cdot}0$
Let ρ be the ratio of bed width to depth,
From Nomogram XIII (B), $\rho = 4{\cdot}66$
From Nomogram XIII (C), $D = 3{\cdot}57$
$\therefore B = \rho \times D = 3{\cdot}57 \times 4{\cdot}6 = 16{\cdot}6$; Keep $D = 3{\cdot}6$ and $B = 16{\cdot}5$ feet

(3) Section Down stream R.D. 39275
Discharge $= 107$ cusecs, Slope $= 1$ in 4000
Absorption up to $39275 = 2{\cdot}8 + 1{\cdot}4 + 1{\cdot}6 + 0{\cdot}8 = 6{\cdot}6$ cusecs.
Discharge of A.P.M. outlet set at bed level with silt conductive power $122\% = 6{\cdot}0$ cs.
Discharge of A.P.M. outlets set at 8/10th with silt conductive power 110%
$= 6{\cdot}36 + 9{\cdot}36 + 6{\cdot}31 = 22{\cdot}03$ cusecs.

$$r_1 = \frac{143{\cdot}5 - (1{\cdot}22 \times 7{\cdot}5 + 1{\cdot}1 \times 22)}{107} = 1{\cdot}020$$

$$r_2 = \left\{ \frac{143{\cdot}5 - (1{\cdot}22^3 \times 7{\cdot}5 + 1{\cdot}1^3 \times 22)}{107} \right\}^{1/2} = {\cdot}97 \; ; \; r_3 = \frac{143{\cdot}5}{107} = 1{\cdot}34$$

From Nomogram No. XIII (A) for the values of r_1, r_2 and r_3, $R_c = 1{\cdot}035$
$X = {\cdot}985 \times 1{\cdot}035 = 1{\cdot}02$
From Nomogram XIII (B), $\rho = 4{\cdot}5$
From Nomogram XIII (C), $D = 3{\cdot}37$ feet
$B = \rho \times D = 3{\cdot}37 \times 4{\cdot}5 = 15{\cdot}17$, Keep $D = 3{\cdot}35$; $B = 15{\cdot}25$ feet

(4) Section at R.D. 58,000
Discharge $= 88$ cusecs, Slope $= 1$in 4000
Absorption $= 6{\cdot}6 + 1{\cdot}3 + 1{\cdot}0 + 1{\cdot}1 = 10{\cdot}0$ cusecs
Discharge of A.P.M. outlets at bed level with silt conductive power $122\% = 7{\cdot}5$ cusecs.
Discharge of A.P.M. outlets at 8/10th with silt conductive power $110\% = 22$ cusecs.
Discharge of A.P.M. outlets at 9/10th with silt conductive power $115\% = 16$ cusecs,

$$r_1 = \left\{ \frac{143{\cdot}5 - (122 \times 7{\cdot}5 + 1{\cdot}10 \times 22 \times 1{\cdot}15 \times 16{\cdot}0 + \ldots\ldots)}{88} \right\} = 1{\cdot}030$$

$$r_2 = \left\{ \frac{143{\cdot}5 - (1{\cdot}22^3 \times 7{\cdot}5 + 1{\cdot}1^3 \times 22 + 1{\cdot}15^3 \times 16{\cdot}0 + \ldots\ldots)}{88} \right\}^{\frac{1}{2}} = \left(\frac{75{\cdot}0}{88}\right)^{\frac{1}{2}} = {\cdot}92$$

$$r_3 = \frac{143{\cdot}5}{88} = 1{\cdot}63$$

For these values of r_1, r_2 and r_3 from Nomogram XIII (A) $R_c = 1{\cdot}025$

$\therefore X = 1{\cdot}025 \times {\cdot}985 = 1{\cdot}01$

C.V.R. goes on decreasing up to the tail of the channel, the maximum being at R.D. 39275.

From Nomogram XIII (B) ρ is found.

When $S = \frac{1}{4000}$, $N = {\cdot}0235$, $X = 1{\cdot}01$

$\therefore \rho = 4{\cdot}6$

From Nomogram XIII (C), $D = 3{\cdot}15$

$\therefore B = 3{\cdot}15 \times 4{\cdot}6 = 14{\cdot}3$, Say $14{\cdot}5$

$$V = \frac{88}{3{\cdot}15\left(14{\cdot}5 + \frac{3{\cdot}15}{2}\right)} = 1{\cdot}75 \text{ ft. per second.}$$

15. Velocity and Silt Distribution in a Channel Section.

(*a*) Variation of velocity over cross section of a channel.

The velocity of flow varies at different points of the cross section of a channel. The frictional resistance of the sides causes the water to slow down towards the sides of the channel, and the frictional resistance between the water surface and the atmosphere causes a slight reduction of velocity at the free surface. The maximum velocity will be on the vertical centre line of the channel at a point a little below the free surface.

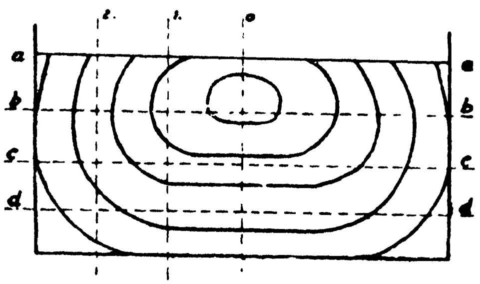


Fig. 6

The variation of velocity over the cross section of a rectangular channel is shown in Fig. 6.

The curves shown are lines of equal velocity ; they have the greatest value at the centre just below the water surface, and decrease towards the sides and base. In Fig. 7, are shown the variations of velocity on horizontal section lines taken at different depths. The velocities at different points of the section lines, a, b, c, & d are plotted on a base representing the width of the channel.

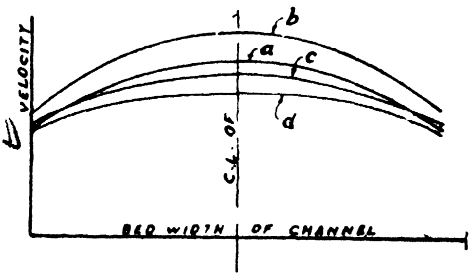


Fig. 7

Fig. 8 shows the variation of velocity on the vertical section lines, 0, 1 and 2 The horizontal ordinate represents the velocity and the vertical ordinate the depth.

The mean velocity on any vertical section occurs at approximately 0·6 of the depth ; it varies with the type of channel and with the nature of the sides. The discharge of the whole channel may be obtained by dividing the section into vertical rectangles and finding the mean velocity of each rectangle. Using this mean velocity, the discharge through each rectangle may, be obtained. The sum of all these discharges will be the total discharge of the channel.

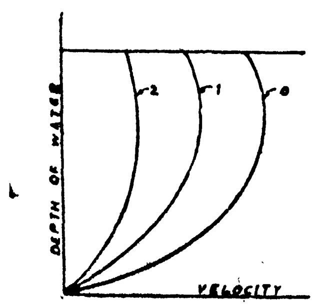


Fig. 8

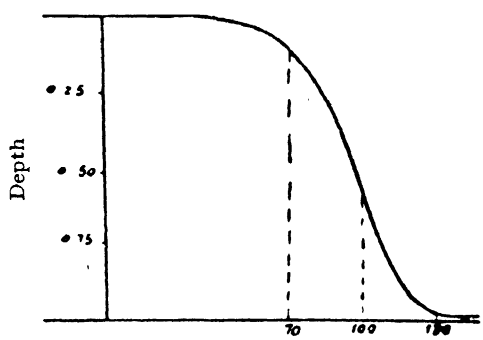
The mean velocity of each rectangular strip may be taken approximatley, as the velocity at a depth of 0·6 of the total depth. In a branch canal of over 50 feet bed width the mean velocity in the central segment has been found to be just double the mean velocity of the slope segment. The ratio reduces to 1·5 on small channels.

(*b*) **Silt distribution in vertical plane.**

Author carried out observation to find out silt distribution in vertical plane. The observations were taken in the middle of the

Bhek distributary, Lower Jhelum canal, Punjab from a platform by means of silt sampling bottles and compared with the total silt charge by weight passing over the crest of a fall. The results were published in Fig. II Page 4, Author's paper on Design of Irrigation Channels in the Institution of Engineers India 1936. They are reproduced in Fig. 9.

If the silt charge intensity over the section be 100, 60 to 70 per cent intensity of silt by weight will be near the surface and about 130 per cent near the bed. The cent per cent silt charge intensity will be at about 0·6 D from the surface. The portion DE represented the silt dragged along the bed. It was observed that coarse silt was also available near the surface as near the bed though in a very small amount. This showed that the vertical eddies do work from bed right up to the surface in all depths. If some disturbance in the form of obstruction be placed causing vertical eddies, the silt charge intensity at the surface may be cent per cent and even more.



Percentage of silt charge by weight

Fig. 9

(*c*) **Silt lifting eddies in vertical plane.**

Kennedy stated that the silt was carried by vertical eddies produced by the roughness of the bed. The eddies are definitely produced on this account as proved in Reynold experiments, and they convert the stream line motion into turbulent motion. Lacey also states that silt is suspended by the vertical component of eddies, but urges that they are generated at all points by forces normal to the wetted perimeter. Can such eddies be strong enough to work up against depth of 10 to 12 feet in large canals ? The silt carrying capacity would thus be more in channels with shallow depths and less in case of those with large depths.

The author considers that very much stronger silt lifting eddies are produced by other factors than by friction against the perimeter. One factor is velocity distribution as explained in para 15 (*a*) above. The velocity distribution in the vertical plane gives a rolling motion. Top water moving with relatively higher velocity topples over and becomes the bed water, and the bed water lagging behind rises up to take the place vacated by top water. This explains the strong boiling up to the surface which is usually seen in the case of large canals. In addition to the forward motion, there is also the rolling motion. Such silt lifting eddies are many times stronger than those caused by the roughness of the perimeter. It is therefore that the large canals are capable of carrying relatively higher silt charge be weight and grade.

K.B. Khushlani in his article in Indian Engineering, Calcutta, September, 1937 explains the rolling theory of water considering that the forces acting are in the form of a couple produced by the frictional resistance and the forward motion of the mass of water and works out the lever arm of the couple $Z = \frac{D}{2}$ and his regime velocity formula in terms of Z. This would be the case if the moving mass was a solid, but the author considers that the explanation of the rolling water due to velocity variation is relatively more sound. The similar lateral effect has already been found in his observations by A.S. Gibb as explained below.

(*d*) **Lateral silt distribution in a channel section.**

A.S. Gibb, Executive Engineer Punjab Irrigation, carried out observations on Gugera Branch, Lower Chenab Canal, published in his paper No. 28, 1916, Punjab Engineering Congress Lahore. He mand observations by floats, "The path of surface floats gave surface lines directly, and rod floats were assumed to follow lines giving a mean between surface and bottom stream. The lines of bottom flow were deduced from the observed surface and mean lines. Fig. 10 merely illustrates what is well known, namely that in an ordinary straight canal the bottom water flows from the middle towards the margin, the surface water flows back towards the middle, and there is an upward current in the marginal strip.

Gravitation is the only other known force acting on the silt particles. Consequently from these cross circulating currents, allowing for gravity, it is possible to get an idea of the activities of the silt particles in a channel. Sand suspended near the bottom of the canal, as it is carried downstream, will be gradually pushed along on or near the bottom (traveling by saltation as an American experimenter terms it) into the strip of water near the margin, there it will come under the influence of the rising current, and as gravity is acting against the current, the ascent will be considerably retarded with the result that water there will be more densely charged with silt than elsewhere. At last on arriving near the surface the sand is carried away towards the middle of the stream, falling through the water as it goes, its fall being assisted by the ultimate downward tendency of the current. Eventually the sand grains again come under the influence of the bottom current and start off on their rounds again. Heavy particles of silt, it may be assumed always remain near the bottom rather than light ones, and they are thus more under the influence of the bottom current towards the bank. The density of the silt charge in the marginal strips of water will be still further increased, owing to the fact that the forward velocity is less there than it is in the middle, and a given quantity of silt, while going the round above described, must be contained in a smaller volume of water when it is near the margin than when it is near the middle of the stream. It appears, then that the marginal strip of the parent canal, which is to be drawn off into a distributary, normally, contains the most highly silt charged water in the whole cross section, and tends continually to be fed with a specially selected supply of the heaviest sand available. The cross currents in question result from the presence of the boundaries of the channel or in other words due to the relatively reduced velocity near the sides and higher mean velocity in the midstream. If the velocity near the side could be increased by providing pitching the difference of velocity relative to the mid mean velocity shall be reduced and the amplitude of the cross waves shall increase."

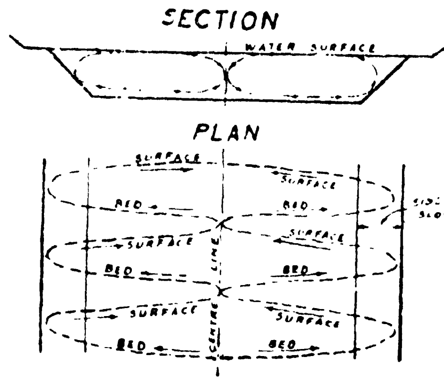


Fig. 10.

(*e*) **Effect of Convergence.**

The effect of convergence has been studied at length by C.C. Inglis, Director Research Institute, Poona, India. If a bell mouth approach is constructed as is usually done upstream of contracted meter flumes, the silt tends to concentrate in the mid-stream. If water be abstracted from the bell mouth approach, the silt charge on the off-take is relatively less.

(*f*) **Silt Distribution at Bends.**

The cross section of a channel at bends is as shown Fig. 11.

There is shallow depth on the inside of the curve and greater depth towards the outside of a curve at a bend in a channel. The inside tends to grow berm, while the outside is eroded. There is flow of silt along the bed as shown by arrows in the Fig. 11. This results in the concentration of silt charge on the inside of a curve and there is reduced silt charge towards the outside of a curve, where the channel is deeper. An off-take taking off water from the outside of a curve contains relatively low silt charge by weight and grade.

Fig. 11

16. Diurnal and Seasonal Variations in the Silt Charge in a Channel.

(*a*) There is considerable variation in the silt charge and silt grade of the silt carried daily in an irrigation channel. A few observations from the author's paper on Design of irrigation channels are given below showing variation of the silt charge carried at R. D. 13500 of the

Bhek distributary, Lower Jhelum Canal. These observations were taken by the author while engaged in experiments for determining the silt conducting power of different outlets.

Date	Silt charge in Disty : expressed as percentage by weight of water
16-5-1931	·202
25-5-1931	·171
20-6-1931	·132
9-7-1931	·275
13-8-1931	·331
21-8-1931	·350
27-8-1931	·165
3-9-1931	·14
13-9-1931	·182
18-9-1931	·150
1-10-1931	·203
2-10-1931	·120
5-10-1931	·180
7-10-1931	·150
8-10-1931	·160

There is a free fall at R.D. 13500. The longitudinal section of the reach upstream of this fall was observed occasionally to see if it was silting or scouring during the course of experiments. The observations showed that the bed of the channel did neither silt up nor scour during the period in question, in spite of the fact that there was a considerable variation in the silt charge carried in water from time to time. The Bhek distributary is a regime channel. It has not changed its levels during 40 years of its running. The variation in the silt charge by weight was from ·12 to ·35, *i.e.*, about 3 times. Two points are explicitly clear from these observations.

(*i*) In the formula for the change in C.V.R., ($R_c = r_1^{1/3}\, r_2^{\cdot 3}\, \lambda^{1/6}$) if the depth does not change, and if r_1 the silt charge changes, there must be corresponding reduction in r_2 silt grade in order that R_c does not change causing neither silting nor scouring.

(*ii*) Silting or scouring does not take place immediately as the silt charge or silt grade is changed. There is some natural resistance offered by bed before it is eroded.

(*b*) The seasonal variation of silt charge is well known. In my observations on the Lower Jhelum Canal, I found it to vary from ·5 percent by weight in summer to a little below 1 percent in winter. The silt charge by weight is maximum in the rainy season when the parent river is in floods. The summer silt is muddy and winter silt is fine and coarse sand. The irrigation channels pass the high silt charge in the months of June, July and August without silting up. The channels berm up most in the months of July and August, when the fertilising fine silt charge is very high. Deterioration of channels in the monsoons is mostly due to berming up. The irrigation distributing channels silt up most in the months of October and November, mostly due to the falling parent river picking up coarse silt in digging the winter channel and partly due to the water picking up of the coarse silt in the canal itself due to the large reduction in the silt charge by weight.

17. Silt Attrition.

(*a*) The subject of silt attrition in canals has received little recognition so far and it is generally argued that on account of the low velocities in the canal, there is no possibility of silt attrition. More over, it is sometimes argued that the progressive reduction in the silt grade is not due to the silt being worn down by attrition, but due to the silt selective behaviour of flow in dropping coarse stuff and picking fine stuff. This would be the case if the velocity was reducing but in long lengths of canals with no change in velocity, as the lined Bikaner canal, the reduction in silt grade must mean nothing but silt attrition, In rivers the part played by attrition is evident. The pebbles (grit) are ground down first to coarse sand and then to fine sand by the process of rolling and impact against the bed and among themselves, as they are transported from the hills to the plains. The silt attrition is therefore an established phenomenon, but the limits of its operation remain to be determined. The velocity at which turbulence starts is known to be about ·80 foot per second. There must be a second critical velocity beyond which the silt would drag along the bed and the conditions necessary for silt attrition would come into play. Dr. Owen determined such a velocity to be equal to about 2·5 feet per second. Observations and experiments could alone determine such a velocity in actual canal practice. The author saw from below plate glass bed in the course of some other experiments that the silt was rolling and dragging along the bed of the Mithalak Distributary. Lower Jhelum Canal, when the velocity was only 2·0 feet per second. It is probable that the second critical velocity necessary for silt attrition might be variable with the quantity of the silt to be dragged along the bed. There can be no denying the fact that the dragging of the silt along the bed always takes place at least in the case of main and branch canals. The effect of the silt attrition must at least come in the design of these channels. The author in his paper on ' Design of Irrigation channels', worked out the effect on the value of C. V. R. if the silt grade changed from f_1 to f_2 by attrition. There was then no authoritative record of experiments to determine the actual change in silt grade.

(*b*) Experiments have been carried out by C.C. Inglis C.I.E. Director, Central Irrigation and Hydrodynamic Research Station, Poona.

"The object of this experiment with silt abrader is to see whether abrasion can be an explanation of the feature of large rivers that the bed sand becomes progressively finer from the hills to the sea. As described in the annual report for 1937-38, the silt abrader consists of a circular tank of 2 ft. radius, 4 feet high, with a central shaft to which three tiers of blades are fixed, each tier being set at 40 degrees to the other two. The shaft was revolved 30 times per minute, so the velocity of the blades at the outer edge was 6·9 feet per second.

Sukkur silt was placed in the tank to a depth of 6″ and water maintained at a constant level, 3′-6″ over the bottom of the tank.

The abrader was worked for 1637 hours (68 days) since last year's report, the total working hours up-to-date being 8787 hours (366 days). At the end of this time, the silt on the bottom was thoroughly mixed and analysed by the Puri siltometer.

It will be seen, that between 7150 and 8787 hours abrasion of silt particles of 0·25 to 0·35 mm. grade took place, resulting in the increase of percent silt between 0·11 to 20·5 mm. diameter. There was very little percentage increase of 0·08 mm. silt, which shows the limit of abrasion, because below that limit, the silt goes in suspension. "Table II shows the progressive reduction in the grade of silt at various periods during the experiment."

Table II.

Total hours of working	d_{mm}	$f_d = 1{\cdot}76\sqrt{d_{mm}}$ based on particles 0·075 mm. diameter
Original sample	0·229	0·84
3775 hours (157 days)	0·216	0·82
5382 hours (224 days)	0·196	0·78
6220 hours (259 days)	0·193	0·77
7150 hours (298 days)	0·181	0·75
8787 hours (366 days)	0·169	0·72

(*c*) The Poona experiments clearly show the effect of silt attrition or abration, but actual observations in canals are needed where the discharges and the velocities do not change in long lengths as main line Upper Bari Doab Canal, Bikaner lined canal or Haveli lined canal to determine the actual reduction in silt grade.

18. Silt Movements.

The phenomenon of silt movements is peculiar to the large distributing channels in the tail divisions of a large canal. The channels of the Jhang Division Lower Chenab Canal, in the Punjab (Jhang Branch lower in its tail reach). Bhango Branch, Dhaular Distributary and Sultan Pakha distributary are especially notorious in this respect. The water levels suddenly rise from ·5 ft. to 1·0 ft. depending on the size of the channel in its middle reaches even though no excess supply has been admitted at the head. This results in serious leakages and breaches. The author had a personal experience of this trouble in the channels mentioned above.

It was at first argued that there was actual increase in the discharge of the channel at head due to relatively muddy water coming in the canal. This is quite likely when the regulation is done with reference to a guage fixed downstream of a head in an earthen channel. There is about 10 to 12% increase in discharge, if muddy water follows a clear water flow in earthen channels without any change in the section due to reduction in the co-efficient of friction. J. P. Gunn Executive Engineer, Punjab Irrigation carried out detailed observations on meter flumes and found that an increase in discharge due to muddy water was 3 to 4 percent. The remedy would therefore be to construct meter flumes near the head of such channels for precise regulation. The Dhauler distributary was provided with one at R. D. 500 in 1925. This has not, however, cured the silt movement trouble which is now (1943) the worst ever experienced.

There is one peculiar point about these silt movements that they are the worst in the beginning of November and about the end of February every year, synchronising with the change of seasons. The author considers that the silt movements are primarily due to temperature changes and the consequent viscosity changes in the channel having fine bed silt. The bed silt is lifted up in water and the piling up of water is due to the unerodable bars in bed or the drowned bridges and the increased friction on account of relatively coarse silt in suspension.

The silt momements are no doubt intensified in the lower reaches of a canal, if there are a lot of raised crested falls in the upper reaches subject to long periods of low supply and also if lot of silt exclusion has been resorted to in the case of distributary head regulators in the upper reaches of the canal by constructing skimming platforms.

19. Berming and Silting-up of Channels.

The distributary channels silt up most in the head reaches and berm up in the tail reaches. The middle reaches are generally free from silt trouble.

The reasons for silting up in the head reaches :—

(*i*) **Non-regime section.**

If the section given is not suitable, the channel water would drop its cargo of silt which it cannot carry in the head reach and the lower reaches will thus deal with silt in water, out of which objectionable silt had already been dropped in the head reaches.

(*ii*) **Defective head regulator.**

If the head regulator design is such that excessive silt charge enters the distributary, the coarse silt would naturally drop in the head reach.

(*iii*) **Insufficient slope.**

If the channel has been given insufficient slope, it would naturally tend to increase the slope by silting up in the beginning of every reach in the channel dewnstream of the control points.

(*iv*) **Defective outlets.**

If the outlets do not draw their due share of silt, the channel would silt up in the head reach mostly and in other reaches to a less extent.

(*v*) **Fluctuations in the supply.**

If the channel runs long periods of low supply it would silt up most in head reach to adjust its cargo of silt from the reduced depth and velocity.

After the adjustments of the silt cargo in the head reach of a channel, the middle reaches pass off the the supply safely without silt trouble. The lower or the tail reaches of a channel are again the source of great trouble due to berming up. The reasons are enunciated below :—

(*a*) **Low velocity.**

On account of small depth and small velocity water is capable of carrying a reduced cargo and low grade of silt. The side velocity being very low, fine silt deposits there to begin with.

(*b*) **Growth of grass on the berms.**

The grass growth of berms flourishes most in the tail reaches due to low velocity and the fertilising fine silt deposited on the sides. The deterioration of the channel section starts from the sides. Water is headed up on account of the reduced section, and spreads over the berms with the consequent rise of the bed. The berm rises up by grass growth and silting in turn in bed encourages further rise in levels. It is, therefore, that berm cutting in the tail reaches is very necessary to feed the tails in the months of July and August.

20. Silt Sampling and Analysis.

(A) Silt is transported by a stream either in suspension or by rolling on its bed. Silt samples are analysed as below :—

(*i*) Coarse sand, *i.e.*, particles above ·2 mm. diameter.
(*ii*) Medium sand, *i.e.*, particles between ·2 and ·08 mm. diameter.
(*iii*) Fine silt, *i.e*,, particles between ·08 and ·02 mm. diameter.
(*iv*) Dissolved material and clay below ·02 mm. diameter.

Item (*iv*) is ignored and the summation curves are got for the first three items to work out the mean diameter. Silt intensity can be calculated from the mean value of coarse sand, medium sand and fine silt. observations taken at a particular site by the formula : —

Silt intensity in cft. per 100 cft. = grams per litre $\times$ 5/8
and silt intensity in cft. per cusec-day = grams per litre $\times$ 54.

(B) Sampling of Suspended Silt.

(*a*) **Bottle sampler.**

It consists of a brass frame holding a one-litre bottle fitted with a rubber stopper. The stopper is operated by a lever at the top of the suspension pipe. When the lever is pressed down, the bottle is opened. The length of the suspension pipe of the sampler is varied to suit the depth of water in the channel at the sampling site. The two essentials regarding the working of the sampler are :—

(*i*) The mouth of the bottle should be opened only when it reaches the required depth.

(*ii*) The mouth of the bottle should be kept open for the minimum time required to fill the bottle.

If the bottle is kept open for a longer time than that required for actually filling the bottle, coarse silt particles keep on falling into the bottle even after it is full. The time required to fill the bottle should be ascertained experimentally by the observer. The diagram and detailed description is given in the Punjab Irrigation Research Institute Publication Volume II No. 15 of 1936.

(*b*) **Tait Binckley sampler.**

The Tait Binckley Sampler consists of a central pipe with rubber extensions on both sides. The extensions are twisted by an arrangement fitted at the top of the suspension pipe to enclose a sample of water.

(*c*) **Uppal sampler.**

The Uppal Sampler, as developed by Dr. H. L. Uppal of the Irrigation Research Institute, Lahore, consists of a brass barrel with guides fitted at each end. Brass diaphragms move in the guides, vertically, like the shutters of a photographic camera. Both the diaphragms and guides are given a slight taper to secure a leak tight contact. The top end of the diaphragms are connected together by an iron rod which is worked by a lever at the end of the suspension pipe. Detailed description with photographs is given in the report for the year ending April, 1940, of the Punjab Irrigation Research Institute, Lahore.

(C) Bed Silt Sampling.

The old method of filling cigarette tins from the bed of a canal in a closure is now discontinued as silt thus sampled out is not considered to be a representative sample of bed silt carried in water, because when the canal supply is dropping before a closure, some fine silt in suspension also drops down which is again picked up in full supply conditions.

The samples of bed silt are taken by means of an apparatus which is described in detail, in the Punjab Irrigation Research Institute Publication Volume II No. 15. This consists essentially of an eccentrically mounted scoope which digs into the bed when revolved and a cowl which protects the sample from being washed away when the apparatus is brought to the surface.

To take a sample, the apparatus is lowered to the bed and the handle at the top rotated which in turn rotates the scoope by means of wire. It should be noted that the pipe forming the handle is kept clear of the cowl so as to avoid the water in the pipe interfering with the sample. The sample is air dried and approximately one lb. kept for examination. The following hydraulic datas are obtained at the time of sampling :—

Q (Discharge), A (Area of cross section), S (slope of water surface $\times 10^{2}$), R (Hydraulic mean radius), D(Mean depth), P_w(Wetted perimeter), B(Bed width), N(Kutter's co-efficient for roughness) and G(mean gauge of the channel during discharge observations). The temperature of water is also recorded at the same time.

(D) **Total Silt Charge Sampling in a Canal.**

The methods described in paragraph (B) above omit the silt rolling along the bed and the method described in (C) above omits the silt in suspension. The author used a scoope in a trough intercepting the jet at the downstream end of a free fall long crested weir to collect a total silt charge sample. This method is described in detail in the author's paper No. 168, Punjab Irrigation Congress, Lahore, 1933.

(E) **Silt Analysis.**

(*a*) **Kennedy's siltometer.**

Detailed description is available in Paper No. 9 Technical Publication class A, P.W.D., Irrigation Branch, Punjab. It is not obsolete as this method gave qualitative results only. The dropping silt in a tube was collected in a graduated bottle and the times of filling the various divisions were noted.

(*b*) **Puri's siltometer.**

This is an improvement over the former. A detailed description is available in the Punjab Irrigation Research Publication Volume II Nos. 7 and 9. The bottom of the siltometer tube opens into a circular trough provided along its circumference with detachable cups which are just large enough to be covered by the bottom of the tube, *e.g.*, after 22.26,30 seconds, so that each cup comes in turn under the tube and receives the particles which reach the bottom in the interval between two movements of the trough. When all the silt has passed down, the cups are taken out and the silt in each is washed into a separate glass tube with a narrow long bottom graduated in cubic centemeters. The volume of the silt which settles down is read assuming that the density of particles of all sizes is the same, we get a proportion to the weight of the particles received in each cup.

Optical Lever Siltometer.

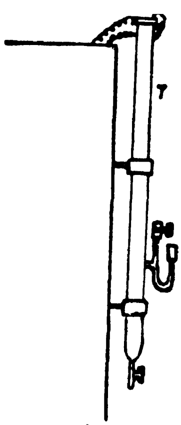


Fig. 12

The sizes are read from a table prepared and the volumes for the cups are successeive totalled up to give the total volume below each size. This is then plotted against the size to give a summation curve similar to the pressure size curve obtained from the optical siltometer described below.

(*c*) **Vaidianathan's optical lever siltometer.**

This is the most correct method of silt analysis. It is described by Dr. Vaidianathan in paper No. 167, Punjab Engineering Congress, 1933. A brief description is given below :—

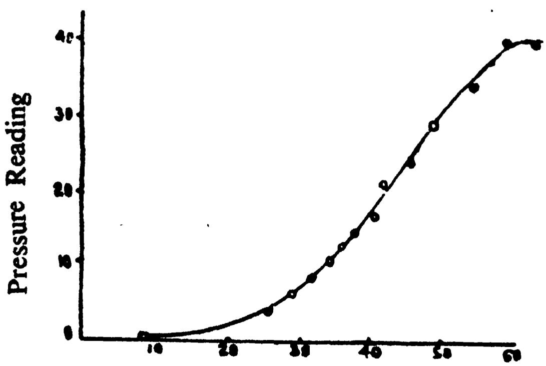
(*i*) When a given sample of silt is released from the top of the siltormeter tube the arrival of various particles at the bottom of the water column is indicated by a change in the pressure as shown by the fall of the mercury meniscus (M) in the monometer Fig. 12. This is magnified by the optical lever arrangement and the movements recorded on photographic paper. The light is intercepted at known intervals (usually one second) so that a spiral curve broken at the corresponding intervals is obtained. A base line from which measurements have to be taken, is obtained when the silt has completely passed down.

The ordinates of the spiral curve are measured from the base line at the various breaks in the curve. They give numbers proportional to the pressures of various instants.

The theory connecting the time of fall of the particles at a given temperature of the water column with their average size is described at some length in Paper No. 167 Punjab Engineering Congress, 1933. Here it may only be stated that from a knowledge of the time and the temperature it is possible to read the average size of the corresponding particles from a previously prepared table. This table can, however, serve only for a given length of water column and has to be separately prepared for each siltometer of a different length.

The rate of change of pressure at any given instant on the other hand has been shown to be directly proportional to the rate of change of the weight of particles deposited at the bottom of the siltometer tube and consequently the rate of change of weights is directly known from the pressure readings.

Pressure-Summation-Curve



Particle size in mm.
Fig. 13

Thus we get the corresponding values at any instant of a number proportional to the total weight of particles deposited up to that instant and of the average diameter of the particles arrived at the bottom since the previous measurement of time. These values are entered on a sheet and are plotted on a graph. A free hand curve passing very closely through the plotted point is now drawn and the curve and the values are passed on to the statistical section. Form of curve is shown in Fig. 13.

(*ii*) **The percentages.**

The plotting and the curve are checked first and then the values of the pressures read from the curve for integral values of the diameter of the particles. If the value of the pressure for the datum line is not zero it is then subtracted from each value. We consequently get a series of numbers proportional to the weights of all the particles whose mean diameter is less than the corresponding integral value entered in the previous column. The last number in the column (say L) is proportional to the total weight of the silt sample.

Each number is then divided by the number L and multiplied by 100 to give the percentages by weight of the particles below each size. These are what are known as the "summation" percentages, *i.e.*, they represent the sums of the separate percentages by weight of all the particles whose sizes are less then the given size. If the summation percentages are plotted against the values of the mean diameter we get the so called summation curve.

(*iii*) **The size distribution curve.**

Each value in the summation percentage column is next subtracted from the value just succeeding it. The differences from a new column giving the "distribution percentages." These give the proportions by weight of the fractions of particles whose diameters lie between the value of the diameters entered in the previous column in the same row and its next lower value. It is thought sufficient, for all practical purposes to take the mean of these two values as the mean diameter of the fractions.

The distribution percentages are next plotted as ordinates against the diameters of the fractions as abscissa. A free hand curve passing closely through the individual points is next drawn. It shows how the particles are distributed according to the mean sizes and is known as the size distribution curve. A typical siltogram or the photographic record of the analysis of the silt sample is shown in Fig. 14 and a typical summation curve in Fig. 15.

(*d*) **Uppal's air siltometer.**

Siltometer for quick and accurate analysis of silt samples has been developed by Dr. H.L. Uppal. A jet of air has been utilized to separate the particles belonging to different grades.

SILTOGRAM OF BED SILT

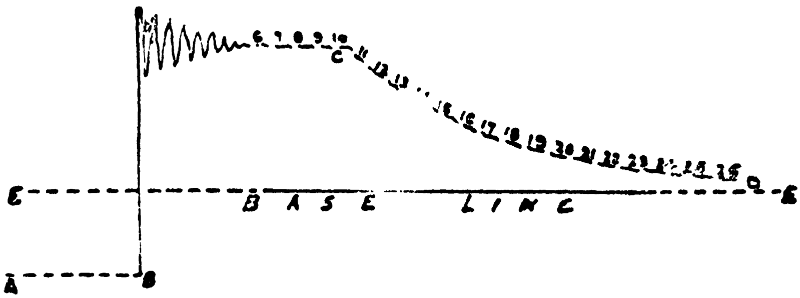


Fig. 14

SIZE DISTRIBUTION CURVE

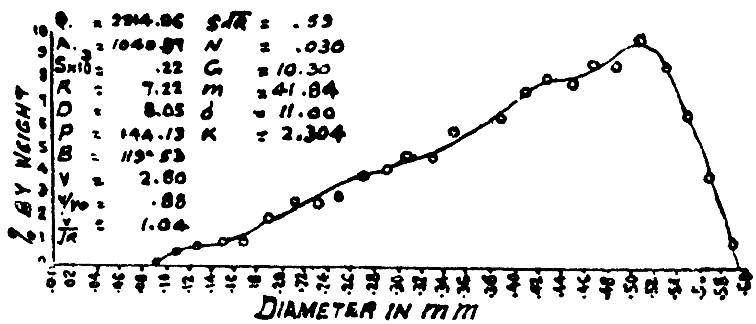


Fig. 15

The apparatus consists of the following parts :—

(*i*) A blower.
(*ii*) A stilling chamber.
(*iii*) An air tank.
(*iv*) A collection chamber.

A standard blower supplied by Messrs A. Gallenkemp has been used. The speed of the motor is adjusted by means of a tachometer. In the stilling chamber, which consists of a wooden box 2′ × 2′, the air is stilled before it begins to flow in the air tank. The air tank is made of glass plates 4′ × 3′ and 5/2″ apart. The top two feet portion of the tank is used for the passage of a stream of air, while the bottom one foot is occupied by the collection chamber. A sample of the silt is introduced at the top left hand corner of the air tank. The air blast separates it into different grades which collect in the collection chamber below the air tank.

The collection chamber is divided into one hundred compartments by means of celluloid strips. At the top-ends of the strips fine needles are fitted so as to avoid any turbulence which might be caused by the jumping of the particles. Each compartment of the collection Chamber has been calibrated for a known speed of this blower. A direct distribution curve is obtained by the use of this siltometer. The bottom of the collection chamber is removable so that after analysis the sample can be readily taken out.

A large number of samples have been examined in this apparatus and it has been shown that duplicates agree well and that the method is rapid since it saves a considerable amount of laborious calulations.

21. Correlation Between Silt and Hydraulic Data.

Size Distribution Curves though characteristic of various silts are not convenient for defining them for purposes of correlation with the other hydraulic data of the channel. For this purpose it is necessary to define the silt by means of "single values" capable of being derived from the summation of distribution curve. The single values now being used are :—

(*i*) **Weighted mean size "m".**

The summation curve of a silt sample gives the percentage of paricles below any given size. By taking the readings for two given sizes and subtracting we can ascertain the percentage of particles whose diameters lie between the two sizes and which may very nearly be assumed as having a diameter lying midway between the those two sizes. If this last value is multiplied by the corresponding percentage, and the sum of all such products is divided by the sum of percentages (usually 100) we get a value for the mean diameter of all the particles contained in the sample. This is the weighted mean size or "m" and furnishes a useful measure of the degree of coarseness of the sample.

(*ii*) **Standard deviation "σ"**

Every sample does not consist of particles distributed in exactly the same way, and it is quite possible that two samples with the same mean size may differ, one having a preponderance of particles with diameters near the mean size, and the other with diameter varying much more widely. Hence it is necessary to know how the various sizes are distributed about the mean size, and consequently the "standard Deviation" is calculated as a measure of their "dispersion." To obtain this the deviation of each size from the weighted mean size is squared and multiplied by the corresponding percentage and than the sum of such products is divided by the sum of the percentage. The square root of the quotient gives "σ" and the smaller it is the more uniform can the sample assumed be.

(*iii*) **Schoklitsch number or "k".**

The maximum diameter of a particle that can be determined by a siltometer is 0·6 mm. so that in practice the mass curve is bounded by the 0 and 0·6 mm. ordinates and the line (Distribution Curve) may be taken to represent a normal mass curve. If then the area A lying above and to the left of the mass curve, be divided by the area B lying below and to the right of the mass curve the fraction A/B remains constant as long as the limits 0 and 0·6 remain unaltered.

This was pointed by Professor Schoklitsch, who found that this fraction worked quite well for specifying shingles, (though open to many theoretical objections). This number is referred to as the Schoklitsch number or "K" and when not otherwise specified is taken to refer to diameters lying between 0 and 0·6 mm. Should it be desired to specify these limits more particularly it may be written as 0^k-0·6 in the same way $0{\cdot}5^k$ -2·0 would refer to mixtures whose mass curves lay between 0·5-2·0 mm.

It has been often asserted that the presence of fine suspended matter in the water will alter the viscosity, so a series of rough tests were made on water containing from ½ oz. to 8 oz. of fine suspended matter per cubic foot. A plot of these values is shown in Fig. 4 of the Punjab Irrigation Research Publication No. 15 Volume II and with the exception of one point they lie on a fairly good line, examination of the wide point by statistical methods showed that Chauvonets criterion justified its reduction (Brunt D "The Combination of Observation" 1923 page 130). The viscosity increases from 852 to 914 from pure water to oz. of silt per cubic foot, which may be seen on a reference to the approximate tables to be that produced by about 2°C difference of the temperature. It may be concluded, therefore, that the normal amount of suspended matter which rarely exceeds 2 oz. per cubic foot in canals makes no practical difference to the viscosity. The temperature, however, is recorded at the time of observation so that suitable correction may be applied at any time, if found necessary.

It might be argued that though the fine suspended matter has no effect on viscosity, it would exert some direct influence on the movement of bigger particles. Silt was, therefore, analysed in the Puri Siltometer using water with increasing quntities of suspended matter. The results below show that silt constants are uneffected by the presence of fine particles.

TABLE

Effect of Suspended Matter in the Water on Silt Constants.

(Sample from Lower Gugera Branch R. D. 15,000).

Suspended matter Ounces per cft.	m.	σ	K
0	·3011	·892	1·008
·12	·2894	·890	0·932
·16	·2952	·904	0·969
·39	·2998	·916	0·999
·62	·3002	·946	1·001
1·11	·3022	·988	1·015
4·09	·2958	·890	0·972
5·11	·2950	·862	0·967

It has already been shown that Lacey's silt factor $F_l = 1{\cdot}76m$ where m is weighted mean diameter of bed silt excluding matter below ·07mm diameter, and Kennedy's critical velocity ratio X varies as below between two sections Nos. 1 and 2 :—

$$\frac{X_1}{X_2} = \left(\frac{W_1}{W_2}\right)^{\cdot 33} \cdot \left(\frac{m_1}{m_2}\right)^{\cdot 3} \cdot \left(\frac{Q_2}{Q_1}\right)^{\cdot 2}$$

Where X = C.V.R.

W = Weight of total silt charge carried and rolled.

m = Weighted mean diameter of silt carried and rolled excluding soluble matter and clay below ·07 mm. diameter.

Q = Discharge in cusecs.

22. Weed Growth.

The weed trouble in the Punjab canals does not exist because they are fed directly from the rivers. The weed trouble in the seepage drains in the Punjab is very acute. The Deccan canals fed from the storage reservoirs usually suffer from serious weed trouble. A very authoritative account of this trouble has been described by C.C. Inglis C.I.E. Director Poona Research Institute and V.K. Gokhale in Bombay P.W.D. Technical Paper 1937-38 on "The eradication of water weeds from the Deccan Canals". A brief summary is given here and the student should refer to the original publication for a detailed account.

(*a*) During the monsoon, the Deccan Canals obtain most of their supply from river flow; but for the rest of the year they are dependent on supplies from large artificial lakes formed by the construction of masonary dams across river valleys in the foot-hills on the eastern side of the Western Ghats. These dams are about a mile long and vary in height from 100 to 270 feet. The canals, except the Mutha canals, take off from above pick-up-weirs constructed across the rivers some 20 to 53 miles downstream of the storages. These weirs vary from 10 feet to 40 feet in height; and from 890 feet in length in the case of the Paravara pick-up-weir at Ojhar, to 3618 feet in the case of the Godavari pick-up-weir at Nandur Madhameshwar. In the case of the Pravara left, the canal when first opened in 1919 was capable of giving a discharge of 500 cusecs with only 6 feet of water, whereas by 1922 only 272 cusecs could be obtained with that depth and only 73 cusecs or 15 percent of the 1919 figure after the weeds had grown within 2½ months of a full weed clearance. Weed growth is not uniform throughout the year. It generrlly starts in October, and by December it is heavy and under the old method of running canals, another clearance was required early in March and another about the end of April.

(*b*) The following water weeds are found in the Deccan Canals :—

1. Vallisneria spiralis,
2. Potamogeton pectinatus,
3. Potamogeton perfoliatus.
4. Potamogeton indicus,

5. Ceratophyllum demersum,
6. Hydrilla verticillata,

but for simplicity, water weeds may be divided into only 2 groups :—

1. Vellisneria spiralis—a grass like weed, up to 14" in length, which grows in tufts, and
2. Potamogeton varieties and associates—the worst of which is Potamogeton perfoliatus which grows to a length of 12 feet or even more.

No. 1 is the first weed to appear, and is comparatively, unimportant except in small channels.

No. 2 indicates a much more serious condition of weed.

(*c*) Factors which may be expected to affect the growth of water weeds.

(*i*) Infection,
(*ii*) Temperature of water,
(*iii*) Chemical composition of water,
(*iv*) Velocity-effect on weeds due to silting velocity and mechanical, or drageffect.
(*v*) Depth of water,
(*vi*) Deposit of silt,
(*vii*) Reduction of light due to turbidity and depth.

Several inherent difficulties make the proof of relative effect very difficult.

(*i*) **Infection.** Under Deccan conditions, water weeds grow profusely in all *nalas* and ponds—*i.e.*, infection is universal and this enquiry showed that there is no relation between weed growth in canals and intensity of infection.

(*ii*) **Temperature of water.** In the Deccan water the temperature varise between 20° and 30° C, and hence it is highly favourable for weed growth.

(*iii*) **Chamical composition of water.** There is no traceable relation between intensity of growth in various canals and composition of water, but alkalinity by causing despersion of clay has a secondry effect ; because it increases turbidity and reduces the amount of light at the bed.

(*iv*) **Velocity** has little effect except where it exceeds 2 feet per second ; but it has an indirect effect, in that, when the velocity is in excess of regime velocity, deposition of seeds and cuttings does not occur.

(*v*) **Depth of water.** Water weeds can grow in clear water at depths down to 18 feet.

(*vi*) **Deposition of silt.** It is undoubtedly favourable to weed growth, but deposition only occurs where the velocity is considerably below regime velocity and such low velocities do not occur in properly designed channels. When canals are newly opened, they are generally run with low discharges and water has to be headed up at regulators. This reduces the velocity and causes deposition of silt especially along the banks, rendering the water clear ; so that all conditions are then favourable for weed growth. Therefter the weeds tend to persist.

(*vii*) **Reduction of light due to turbidity.** There is reason to anticipate that exclusion of light will reduce weed growth, in that, light is required for carbon assimilation and tissue building (photo-synthesis) and there are no cases in which weeds grow where the water is turbid throughout the year whereas weeds grow profusely when water is clear. Weeds generally grow luxuriantly where silt is depositing, several factors being favourable then ; but weeds do not grow where silt is depositing, unless the water is also relatively clear.

It will be seen that turbidity was greatest at the head of the Nira Left Bank Canal (except in a fair season of 1928-29) when there was a marked drop (due to discharge let out from Bhatgar being more constant than in other years). This canal has always been more free from weeds than any other. The Godawari Right Bank was the only other canal in which weeds were, at this time, decreasing ; and the disc readings were also small. In 1930 there was a marked increase of disc reading in the Pravara Left Bank Canal and thereafter weed growth became serious. The worst canal for all weed-growth was the Mutha Right Bank Canal in which the disc reading generally exceeded 4 feet.

(*d*) Conclusion : -

(*i*) In every case examined, weed growth and turbidity bore a close inverse relationship ;

(*ii*) No other single factor inhibited weed-growth under normal canal conditions.

(*iii*) Where $D > 3d$, no weeds grew, where d = Disc reading and D = depth.

(*iv*) Where $D > d^2/2$, only Vallisneria grew.

(*v*) Turbidity, which leads to exclusion of light was the dominant factor controlling weed growth in Deccan Canals.

The growth of water weeds in the Deccan Canals has hitherto interfered to a serious extent with Canal administration, due to the fact that closures for weed clearance were necessary in the hot weather at intervals of 40 to 50 days. These closures, which took place when the discharge had fallen to minimum supply, necessitated an increase in the period between waterings and considerable dislocation of normal supplies, with the danger of mistakes and the certainty of water being taken out of turn by some irrigators. The consequent dislocation of normal conditions did not end when the canal was reopened, but persisted for about 2 rotations after each closure. These closures inevitably led to serious loss to irrigators, a severe strain on staff and a considerable amount of unavoidable friction.

Temporary labour had to be collected, often a difficult matter, and work had to be carried out against time, because no mechanical method of weed clearance had been found which is as satisfactory as doing the work by hand.

The rush rotation system has been so successful (on most of the Deccan canals) that there is danger of its being looked upon as a panacea. Where, however, the slope of a canal is very small, it may be very difficult to drain off the stagnant water completely, and, where this occurs, the weeds will not die off and may even increase. There is also a danger that some of the weeds will accommodate themselves to changed closure conditions, and persist, because they have a marked capacity for adaptation, and are descended from land plants.

In certain cases it may not be desirable or even possible, to adopt the rush rotation system. Weeds will, however, be inhibited where there is adequate turbidity.

Under Deccan condition no weeds grow where the depth exceeds three times the depth at which a 3″ white disc just disappears from view ; and where the depth exceeds $d^2/2$ (d = Disc reading) the Potamogetion group does not persist.

Where natural turbidity in the parent river is inadequate, it can be created by providing a pick-up-weir designed to hold about three days supply, for range of 2 to 5 feet in water level.

Water can then be let out from storage, intermittently, flow being run for say 1 day in 3. Wavewash will then erode the exposed face of silt, deposited during the flood season and will keep this silt in suspension until it reaches the head of the canal. Where, however, the pick-up weir basin is too large (Lake Fife) or silted berms with fairly steep faces have not had time to form (Nandur Madhmeshwar) or where the pick-up weir basin is too small (Ojhar weir at head of Pravara canals) turbidity will be in defect. In such cases, weed growth can be reduced by running velocities considerably in excess of regime velocities throughout the year.

23. Lined Channel Sections.

K. B. S. I. Mahbub Executive Engineer, Published a very comprehensive paper No.260 on "Lining of Channels" in the Punjab Engineering Congress Proceedings, 1943. A brief summary on design of sections is given here

(A) Design of sections.

The best form of lining section would seem to be an arc having sloping sides, more or less at the same slope as the angle of repose of the soil. The arc in the bed should be tangential to the side slopes. It can be easily shown that we get the most economical section, *i.e.*, the maximum area for the minimum wetted perimeter if the centre of the arc is at the F. S. Line with radius equal to depth.

This section is also useful as it has a higher silt carrying capacity than a wide shallow one. During low supplies, heading up has to be done at off-take sites which would cause some silting. This in turn reduces the velocity and also the value of 'f_l'. In the section proposed, however, with no level bed, the silt can deposit on and effect the rugosity co-efficient of a relatively small portion of the perimeter and hence cannot greatly affect the velocities. The effect on 'f_l' also would thus be correspondingly less.

The side slopes may be kept as 1 : 1 for radii less than 12′ and 1¼ : 1 for radii over 12′ provided that the angle of repose of the soil is not flatter than 1½ : 1 as shown in Fig. 16.

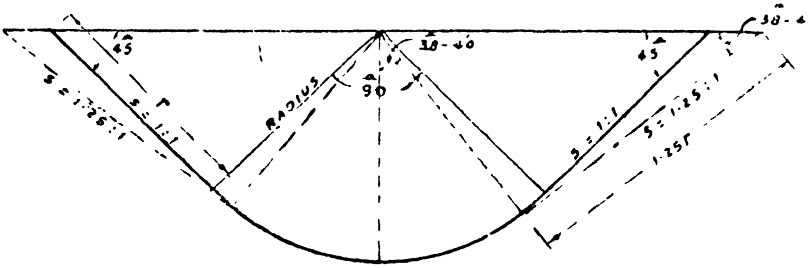


Fig. 16.

The sectional data would be given by the following formulae :—

	Distributaries	Canals
Side slopes, 's'	1 : 1	1¼ : 1
Section area	$1{\cdot}78\ s^2$	$1{\cdot}925\ s^2$
Wetted perimeter	$3{\cdot}56\ s$	$3{\cdot}85\ s$
Hydraulic mean radius (R)	$0{\cdot}5\ s$	$0{\cdot}5\ s$

The velocity according to Manning's formula

$$=\frac{1{\cdot}486}{N}R^{2/3}S^{1/2}=\frac{1{\cdot}486}{N}R^{2/3}\frac{S_1^{1/2}}{10{,}000}$$

where S_1 is the fall in 10,000′

with $N=0{\cdot}18$ we have

$V=0{\cdot}826\ R^{2/3}\ S_1^{1/2}$ and its solution is given in nomogram Fig. 17.

$\therefore$ Discharge$=AV=0{\cdot}92s^{2\frac{2}{3}}S_1^{\frac{1}{2}}$ for side slopes 1 : 1 and $0{\cdot}99s^{2\frac{2}{3}}S_1^{\frac{1}{2}}$ for side slopes 1¼ : 1 and Lacey's $f_l=0{\cdot}75\frac{V^2}{R}=0{\cdot}40s^{\frac{1}{3}}S_1$

Graphs connecting Q, S, s, f_l and V have been plotted for side slopes 1 : 1 with $N=0{\cdot}018$ (Plate XIII—A) and these can be conveniently used for design.

The free board may vary from 1′ for $s=4$ and 2′ for $s=16$ which would be equivalent to $0{\cdot}5\ s^{0\cdot5}$

(B) The limiting depth.

The limiting depth of this section for all practical purposes may be taken as 15′ and so for discharges exceeding 2,000 cusecs, an alternative type as shown in Fig. 18 may be adopted. This was adopted on the Haveli and is also proposed to be used on the Thal Main line.

In this the sectional data is given by the following formulae.

When $\rho=\frac{B}{D}$; $C=\frac{R}{D}$ and N (Manning)$=0{\cdot}018$

Side slopes 's'	1 : 1	1¼ : 1
Section area	$D^2(\rho+1{\cdot}7854)$	$D^2(\rho+1{\cdot}9248)$
Wetted perimeter	$D(\rho+3{\cdot}5807)$	$D(\rho+3{\cdot}8496)$
Velocity	$0{\cdot}826\ R^{2/3}\ S_1^{1/2}$	$\cdot826\ R^{2/3}\ S_1^{1/2}$

LINED CHANNELS

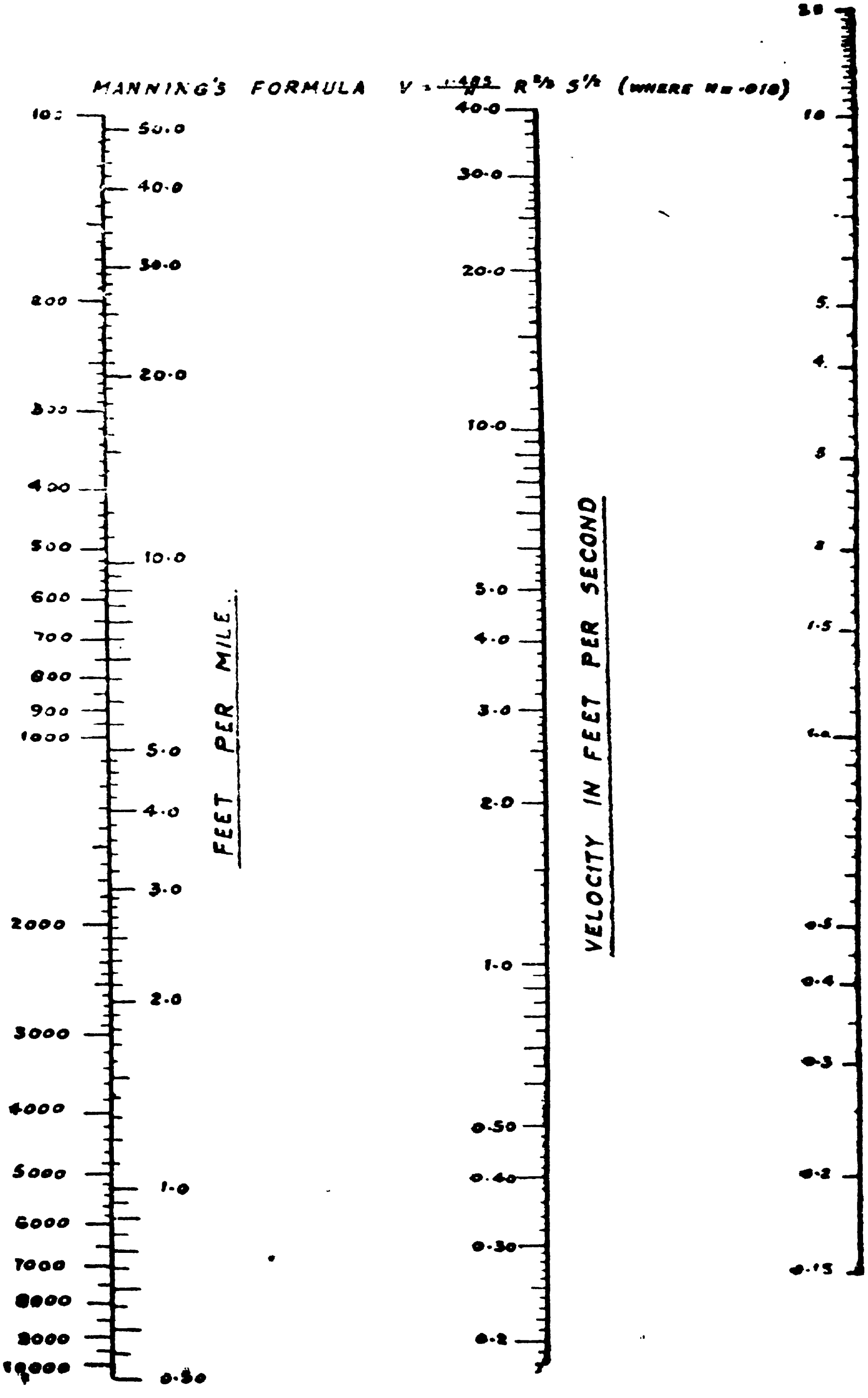


Fig. 17.

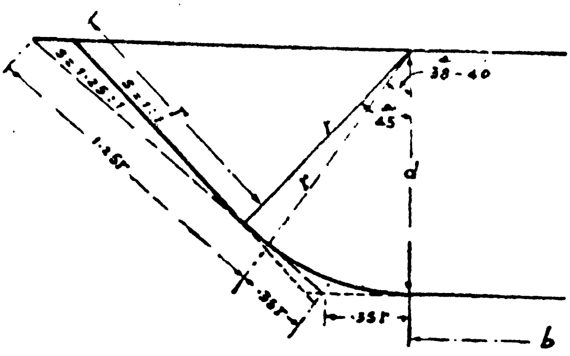


Fig. 18.

With $S_1 = 1$

Discharge $= (\rho + 1{\cdot}7854)\ 0{\cdot}8225\ C^{2/3}\ D^{2/3}$	$(\rho + 1{\cdot}9248)\ 0{\cdot}8255\ C^{2/3}\ D^{2\frac{2}{3}}$
Lacey's $f_l = \left(\frac{0{\cdot}75\ V^2}{R}\right) = 0{\cdot}511\ C^{1/3}\ D^{1/3}$	$0{\cdot}511\ C^{1/3}\ D^{1/3}$
$C = \frac{\rho + 3{\cdot}5708}{\rho + 1{\cdot}7854}$	$\frac{\rho + 3{\cdot}8496}{\rho + 1{\cdot}9248}$

Graphs with N=0·18 for side slopes 1 : 1 have been plotted corelating the various data (Plates XIII (b) Vol. III), which would be found useful in designing any section.

This section is better than a trapezoidal section, as it more stable and economical and also does away with the need of having a toe wall to support the sides.

The slope may be kept as the steepest practicable subject to a maximum velocity of 6 ft. per second and a minimum value of $f_l = 1{\cdot}2$.

Iron or concrete rings, flush with masonry may be provided in the sides every 1,000 ft. spaced 2′ apart, fixed vertically to enable any person who falls in the channel to come out.

(C) **Kennedy C. V. R. or Lacey's silt factor in lined section.**

Kennedy's C. V. R. (X= 1·0) and Lacey's silt factor ($f_l = 1{\cdot}0$) connote regime channel conditions in the earthen irrigation channels normally maintained with self-silted berms and silted bed. The roughness of the perimeter of the earthen channel section is greater than that of the lined channels. The silt supporting eddies in earthen channel sections are relatively stronger and more efficacious to support and to roll the silt charge in channel as compared with the lined channel. In actual practice 10 percent higher value of Kennedy's C. V. R. or Lacey's silt factor is allowed in the brick lined sections and 15 percent more in the cement concrete or cement plastered sections.

24. Super-Elevation in Water Roads.

(A) It is a well known fact that when a body moves in a circular path, it gets deflected form its tangent to the circle by a force acting towards the centre of the circle. In the case of the canal sections on circular curves, the centrifugal effect of the mass of water moving round the curve causes damage and slips to the banks on the concave side (outside of a curve). The curve, thus soon developes into a horse shoe bend impeding regular regime flow and inducing silt deposits on the inside of a curve. To maintain equilibrium and to counteract this tendency of the water moving round a curve to attack its concave bank under the action of centrifugal force, super-elevation or raising of the outer side is most essential.

The centrifugal force developed on a curve in the case of canals with velocities ulways less than 3·0′ per sec. (which is the limiting velocity for side erosion for the soil conditions in the Punjab) is too small to have any effect on the concave side of the channel section. The centrifugal force results in the acceleration of the velocity of water near the concave side of the curve and retardation of velocity near the convex side of the curve. The discharge intensity per foot width of the channel section does not remain constant as in a normal section in a straight reach. Both, the increase in the discharge intensity on the concave side and the acceleration of the velocity may result in producing velocity near the concave side higher than the safe scouring velocity of 3·0 ft. per sec. The maximum safe-scouring velocity has been taken to be 3·5 ft. per sec. in extreme cases in the case of tenacious clayey soil crust. The silt distribution at bends is described in paragraph 15 (*f*) of this Chapter. To design the non-silting bends, the bed of the channel should be super-elevated as shown in Fig. 19 so that the discharge intensity is the same per foot width of the channel. The depth of the channel on the concave side should be reduced according to the increase in the velocity. If the developed maximum velocity exceeds the safe-scouring velocity of the soil of the bed and the sides, the channel section should be protected by pitching or by paving.

The minimum radii of curves in irrigation channels are fixed in practice based on the experience of earthern channel sections and their values are given below :—

Capacity of channel.	Minimum radii to be given.
above 3000 Cusecs	5000′
3000 to 1000 ,,	3000′
1000 to 500 ,,	2000′
500 to 100 ,,	1000′
100 to 10 ,,	500′
Below 10 ,,	300′

As far as possible larger radii should be given.

(B) **Calculations.**

If water be moving on curve, the pressure varies from one stream tube to another. The centrifugal force on a stream tube is balanced by a difference of pressures on the two ends. Let 'r' be the radius of a small stream tube of width dr and let 'V' be the velocity in feet per second. It can easily be proved that the change of pressure caused by the centrifugal force shall be :—

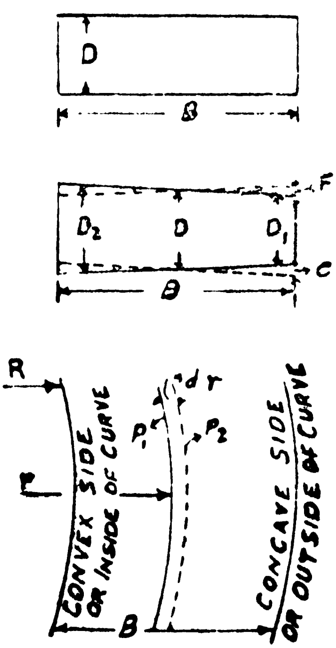


Fig. 19

$$\frac{dp}{dr} = \frac{wV^2}{gr}$$ where w = weight of unit cft. of water.

V = Velocity ft. per sec.

$= \omega r$ (where ω is the angular velocity in circular units).

r = radius of the stream tube in ft.

F = Centrifugal pressure difference in ft.

$$\therefore \frac{1}{w} \cdot \frac{dp}{dr} = \frac{\omega^2 r}{g} \qquad \text{(A)}$$

In a channel of width B, the limits of r are, R and (R+B).

Integrating equation (A) within limits R and (R+B).

$$\frac{P_1 - P_2}{w} = \frac{\omega^2}{2g}\left\{(R+B)^2 - R^2\right\}$$

$$F = \frac{\omega^2}{2g}(B^2 + 2RB), \quad \text{But } \omega^2 = \frac{V^2}{R^2}$$

$$\therefore \quad F = \frac{V^2}{2g}\left(\frac{2RB + B^2}{R^2}\right) \qquad \text{(B)}$$

In the straight reach, let Q = total discharge in cusecs

$q = \frac{Q}{B}$ discharge per foot.

$V = \frac{q}{D}$ Velocity in feet per sec.

D = Depth in feet.

Let V_1 and D_1 be velocity and depth at the middle of the curve on the concave side (outside of the curve) and V_2 and D_2 be velocity and depth at the middle of the curve on the convex side (inside of the curve). F represents the total pressure differences on the outside of a curve relative to the inside and let us assume for the sake of simplicity that the water surface is depressed to the extent of F/2 on the inside and elevated on the outside by F/2. The depth D reduces to D_1 Fig. 19, near the outside of the curve in the outfall beyond the middle of the curve.

$$\therefore \frac{(V_1-V)^2}{2g}=\frac{F}{2}$$

$$V_1=V+\sqrt{gF} \qquad (C)$$

similarly depth D expands to D_2 on the inside of the curve

$$V_2=V-\sqrt{gF} \qquad (D)$$

On the inside of a curve, the average velocity in a channel section, is retarded by the critical velocity due to net pressure difference caused by the centrifugal force on a curve and on the outside of a curve, *vice versa* the velocities accelerated by the same amount.

$$\therefore D_1=\frac{q}{V_1} \qquad (E)$$

$$\text{and } D_2=\frac{q}{V_2} \qquad (F)$$

$$\text{super--elevation}=e=(D_2-D_1) \qquad (G)$$

In channels with sloping sides, the super-elevation can be calculated neglecting the slopes. In a large canal, the error thus introduced is extremely small and insignificant.

(C) **Limitations.**

It is evident from the above treatment that the design of non-silting and non-scouring bends with non-changing discharge intensity per foot width to avoid swirls downstream of the curve is governed by the following conditions :—

(*i*) Due to retardation, the velocity on the inside of the curve should not drop below the non-silting velocity for the depth there.

(*ii*) The maximum velocity developed on the outside should not exceed the safe-scouring velocity of the material of which the section is made of.

(*iii*) The full supply levels are not changing.

The first condition cannot at all be fulfilled in earthen channel section because, if the depth be increased more than the depth in the straight reach, velocity must drop below the non-silting velocity. It is only in lined channel sections that this condition can be fulfilled. Because these can be flumed with the available slope to about two-third of bed-width on account of the reduced co-efficient of rugosity of the lining material (brick work). In cement plaster or cement concrete, even greater reduction of the section is possible.

It is also impossible to fulfil the second condition in the sandy soil or silted bed conditions because the velocity developed on the outside of the curve is certainly going to be more than the scouring velocity of the earthen channel section in the straight reach of the canal. If soil of the bed is clay which is not liable to be eroded up to certain maximum velocity, say 3·0 ft./sec. and roughly assuming 20 percent increase in velocity on the outside of a curve, the maximum velocity in the normal straight reach of a channel could at the maximum be $3{\cdot}0\times\frac{1}{1{\cdot}2}=2{\cdot}5$ ft. per sec. which is Kennedy's V_o for 5·5 ft. depth. It is evident that it is only possible to design non-silting super elevated bends in earthen channels with depth less than 5·5 ft.

The third condition is fullfilled only in the regime channels. In non-regime channels the design of non-silting bends on the inside and non-scouring bends on the outside is impossible in earthen channel sections.

Let us assume that the bed is unerodable clay and will not be depressed on the inside of

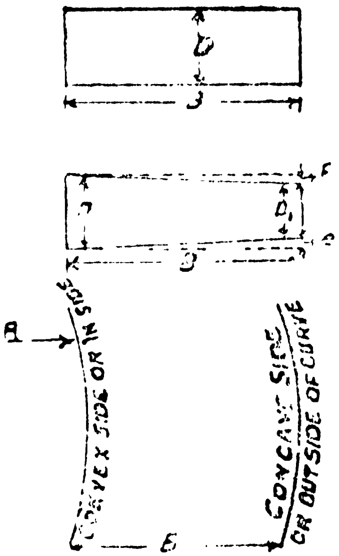


Fig. 20

the curve to satisfy the conditions (*i*) and (*ii*) above, then water surface on a curve takes the shape as sketched in Fig. 20. Depth D_1 reduces to depth D on the outside and the depth D does not change in the inside.

$$\frac{(V_1-V)^2}{2g}=F$$

$$\text{or } V_1=V+\sqrt{2gF} \qquad \text{(H)}$$

$$D_1=\frac{q}{V_1};\ D=\frac{q}{V}$$

$$\text{Super-elevation}=e=(D-D_1) \qquad \text{(I)}$$

(D) **Example 1.**

Design a super-elevated curve in a lined section for the following data of a channel :—

Discharge	$=1000$ cusecs.
Bed width	$=60{\cdot}0$ ft.
Depth	$=6{\cdot}0$ ft.
Slope	$={\cdot}175'$ per thousand
Side Slope	$=\frac{1}{2}$ to 1.
V	$=2{\cdot}64$ ft. per sec.
N	$=0{\cdot}0225$.
C.V.R.	$=\frac{2{\cdot}64}{V_o}=1{\cdot}0$

Brick pitched sections or rough masonry$=N=0{\cdot}015$

Neglecting side slopes for lined section, using Manning's Formula :—

$$\text{Effective bed width}=\frac{60\times 0{\cdot}15}{0{\cdot}0225}=40{\cdot}0 \text{ ft}.$$

Usual side slope for economy is kept 1 to 1 in lined channels for side soil compaction.

$$\text{Average velocity}=\frac{1000}{(B+D)D}=\frac{1000}{46\times 6}=\frac{1000}{276}=3{\cdot}65 \text{ ft. per sec.}$$

Let the radius of the curve$=2000$ ft.

$$F=\frac{V^2}{2g}\times\frac{2RB+B^2}{R^2}=\frac{3{\cdot}65^2}{64{\cdot}4}\times\frac{2\times 2000\times 46+46^2}{(2000)^2}=0{\cdot}00955 \text{ ft. say } 0{\cdot}01 \text{ ft.}$$

$$V_1=V+\sqrt{gF}=3{\cdot}65+\sqrt{32{\cdot}2\times{\cdot}01}=3{\cdot}65+0{\cdot}56=4{\cdot}21 \text{ ft. per sec.}$$

$$V_2=V-\sqrt{gF}=3{\cdot}65-0{\cdot}56=3{\cdot}09 \text{ ft. per sec.}$$

$$\text{Discharge per foot run}=\frac{1000}{46}=21{\cdot}8 \text{ cusecs.}$$

$$D_1=\frac{21{\cdot}8}{4{\cdot}21}=5{\cdot}18 \text{ ft. };\ D_2=\frac{21{\cdot}8}{3{\cdot}09}=7{\cdot}03 \text{ ft.}$$

$$\text{Super-elevation}=e=D_2-D_1=7{\cdot}03-5{\cdot}18=1{\cdot}85 \text{ ft.}$$

$$\text{Slope of super-elevated bed}=\frac{1{\cdot}85}{40}=1 \text{ in } 21{\cdot}6$$

Since $V_2=3{\cdot}09$ feet per sec. is greater than the Kennedy's V_o for $7{\cdot}03$ feet depth, that is $2{\cdot}92$ feet per sec. the designed curve section is nonsilting ; and the discharge intensity shall remain constant throughout the width. The above calculations give the maximum super-elevation in the middle of the curve. It shall be zero at the beginning and at the end of the curve.

The contraction in the upstream of the curve and expansion on the downstream respectively should be 1 in 5 generally and 1 in 3 in the extreme case. The expansion should preferably be designed as per paragraph 5(F) Chapter XII part II.

Example II.

Design a super-elevated non-silting curve in an earthen channel section with the following data.

Discharge	= 265 Cusecs.
Bed width	= 20 ft.
Depth	= 5·00 ft.
Sides	= $\frac{1}{2}$ to 1.
Slope	= 1 in 4444.
Velocity	= 2·35 ft. per sec. and Kennedy's $V_o = 2{\cdot}35$
Kennedy's C.V.R.	= 1·0

Effective bed width $= B + \frac{D}{2} = 20 + \frac{5}{2} = 22{\cdot}5$ ft.

Discharge intesity per foot width $= \frac{265}{22{\cdot}5} = 11{\cdot}6$ Cusecs.

Let radius of the curve = 1000 ft.

$$F = \frac{V^2}{2g} \times \frac{2R\hat{B} + B^2}{R^2} = \frac{2{\cdot}35^2}{64{\cdot}4} \times \frac{2 \times 1000 \times 22{\cdot}5 + 22{\cdot}5^2}{1000^2} = 0{\cdot}004 \text{ ft.}$$

From equation (H) $\quad V_1 = V + \sqrt{2gF} = 2{\cdot}35 + \sqrt{2g \times (0{\cdot}004)}$
$= 2{\cdot}35 + 0{\cdot}505 = 2{\cdot}855$ ft. per second

$D_1 = \frac{11{\cdot}8}{2{\cdot}855} = 4{\cdot}1$ ft. say 4·0 ft. But $D = 5{\cdot}0$ ft.

Super-elevation $= 5 - 4{\cdot}1 = 0{\cdot}9$ ft., say 1·0 ft.

The section designed is non-silting but the discharge intensity may not be exactly the same throughout the width. It is not so injurious as the swirls (vertical rollers) which shall die out downstream of the bend of the curve in the straight reach.

Super-elevation in rivers.

The super-elevation in the water surface as calculated in the foregoing examples of irrigation channel sections is rather trifling but in rivers very high velocities of the order of 20 to 25′ per sec. are possible especially in the boulder reach with steep slope of the order of $\frac{1}{500}$. The Ganges weir at Bhimgoda, Hardwar, in the United Provinces, India, is situated on a curve. In the floods of 1923 the difference of water level on the left side Flank was recorded to be 3·0 feet higher than that on the right side Flank. In order to counteract the effect of the cross flow and the varying discharge intensity, the crest level of the weir bay on the left side had to be raised by 3·0 ft., the next bay by 2·0 feet and the third one by 1·0 ft. There are 8 bays of about 500 feet each. This showed that the velocities, which developed on the outside of the curve (left Flank), were of the order of 35 to 40 feet per sec. in this boulder-bedded river at Hardwar.

24. Examination Questions.

1. Describe Kennedy's silt theory. A channel is silting badly in the head reach. How would you proceed to determine its cause and what remedies would you suggest ? *(T.C.E. 1934)*

2. Quote Bazin's revised, Kutter and Manning formula for flow of water in open earthen channels. Which of these is employed in India and Egypt ? Derive the Chezy's formula from the first principle. *(T.C.E. 1928)*

3. Design an earthen channel in ordinary soil to carry 500 cusecs.

Kennedy's $\frac{V}{V_0} = 0{\cdot}9$. Use any method or formula you like that gives reasonable results ? *(T.C.E. 1928)*

4. What do you understand by (*a*) regime channels, (b) initial and permanent regime of channels ? Determine the hydraulic mean depth and water surface slope of a regime channel to carry 80 cusecs.

$$\frac{V}{V_0}=0\cdot8$$

(*T.C.E. 1935*)

5. Complete the design of a channel on the enclosed L. Section form which gives N. S. level etc. Draw type cross sections of this channel at R. D. 5000 and R. D. 12000. Inspection road will be at the top of the bank on the left up to the fall and on the natural surface below that. Work out the land-width required on the left from the centre line from head up to the fall. (*P.I.B. 1939*)

6. A minor of 30 cusecs discharge and a bed slope of 1 in 5000 gives constant silt trouble. Give possible reasons for the silt trouble and state what measures you would take to stop it. (*P.I.B. 1935*)

7. (*a*) What is critical velocity ratio ?
(*b*) What are the velocities for various type of soils ordinarily met with in the Punjab ?
(*c*) What points will you bear in mind in aligning and designing an earthen irrigation channel ? (*P. U. 1942*)

8. Describe a brief essay on the design and grading of irrigation channels. (*P.U. 1942*)

9. (*a*) Why do the irrigation channels silt most in the head reaches ?
(*b*) What do you understand by silt movements in irrigation channels ? Explain their causes.

10. Compare Kennedy's and Lacey's silt theories. Why is Laceys conception superior to that of Kennedy ?

11. How do the diurnal and seasonal variations of silt charge in the irrigation channels affect their working ?

12. Sketch the best form of a lined channel section. Design a lined channel section for 200 cusecs discharge, slope 0·3 per thousand, N=0·018 in Manning's formula.

13. A channel is to be designed to carry a full supply of 600 cusecs with a depth of 4·5 feet. A velocity of 3·0 ft. per second is considered suitable. Side slope 1 to 1, sketch a dimensioned channel and calculate the slope required for this channel. The co-efficient of velocity can be selected from the following table.

H.M.D.	2·5	3	3·5	4	4·5
C	67	70	73	76	78

(*F.S.C. 1937*)

14. How would you design a stable channel in coherent material ? Describe briefly the factors involved.

An earthen channel of trapezoidal section with sides ½ to 1 has a bed width of 20 feet and depth 3·5 ft. The slope is 0·00025. What will be the discharge ? Will this channel be stable one ? (*F.S.C. 1941*)

15. Quote Manning's formula, explaining the various terms and say to what it is applicable. A drainage culvert under a canal has a level barrel 200 ft. long faced with brick masonry, 4 ft. wide and 6 feet high. The floor is extended horizontally beyond the end and the wings are splayed at an angle of 30° with the axis. There is a vertical breast-wall at each end. Using Manning's or any other formula, derive an expression connecting the discharge with the upstream and downstream water levels. Also obtain an expression for the pressure under the roof of the culvert at a point 50 ft. upstream, of the downstream and, assuming the upstream water surface to be 10 ft. above the floor level, when the discharge is 140 cusecs. (*F.S.C. 1940*)

16. A canal lined with concrete has a section consisting of 45 degree side slopes joined by a quadrant of a circle of 10 feet radius centred in the water-course. Calculate its discharge when running full with a slope of 1 in 5000.

17. Derive a formula for the depth of water in a circular pipe which will give the maximum discharge for a constant slope.

18. State Bernouli's theory. How would you calculate the velocity head in a channel knowing the velocity distribution ?

19. Draw a typical cross section for a lined channel with a discharge of 12500 cuescs, and give a gist of the formulae used by you to derive the section. What are the advantages of lining canals. (*P.U. 1952*)

20. Design and sketch the section of a lined canal for a discharge of 10,500 cusecs. The canal is liable to be subjected to heavy draw-down conditions. Devise suitable drainage arrangements for safety of the lining. (*P. U. 1954*)

21. Design the most economical section of the lined canal for a discharge of 12,000 cusecs with water slope of 1 in 10,000. Sketch typical cross section of such a canal with N. S. below bed. (*P. U. 1956*)

22. What are the various ways of controlling excessive silt entry in (*i*) canals. (*ii*) Branches. (*iii*) distributaries (*iv*) minors. (*v*) outlets.

What would be an ideal irrigation channel in respect of silt transportation and to what particulars it should conform. (*P. U. 1952*)

23. Describe the relation of a depth, width and surface slope in a natural and artificial irrigation channel and the extent to which each of these factors governs the transportation of silt. What are the remedies for a distributary of 200 cusecs discharge that silts badly in the head reach.

24. What principles govern the design of an earthen channel of 5000 cusecs discharge to keep it free from excessive silting or scouring troubles ? Work out its principal dimensions where value of Manning's N=0·025. Show also the freeboard, width of berms and banks. (*P. U. 1953*)

25. Write a brief essay on the design and grading of irrigation channels. Derive Chezy's formulae from first principles. (*P. U. 1951*)

26. Explain the significance of Lacy's 'f' in design of earthen channels. (*P. U 1957*)

27. Describe Kennedy's silt theory. Design a trapezodial regime channel section for Punjab conditions on Kennedy's theory for a discharge of 1100 cusecs. (*P. U. 1958*)

PART II

CANAL IRRIGATION

CHAPTER VII

Design and Maintenance of Banks

1. The banks of irrigation channels should be strong enough to withstand the water pressure due to the depth of water in the channel. The earthen banks are porous and, therefore, water can percolate through them. The normal pore space in the Punjab soil is about 40% by volume. The banks as originally constructed are rather loose, but when consolidated in course of time in a couple of years, are compacted better than the normal soil crust. The width of the bank is determined from the considerations of the hydraulic gradient which is usually kept 1 in 5 as shown in Fig. 1.

The actual gradient of flow is 1 in 2·5 to 1 in 3 as shown dotted, and the permissible hydraulic gradient presupposes a factor of safety equal to about 2. The height of the bank above full supply level in the channel is called free board. It is usually allowed 1·25 feet for the minors, 1·5 ft. for the distributaries and 2·0 ft. for the branch and main canals.

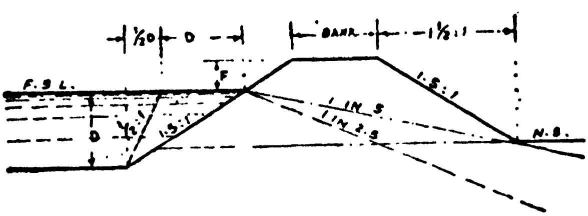


Fig. 1.

2. Typical Distributary Cross Sections.

Typical distributary and minor cross sections are sketched in Fig. 2 to 4.

(*a*) Fig. 2 gives the cross section for a distributary when bed level is below natural ground surface *i.e.*, N. S.

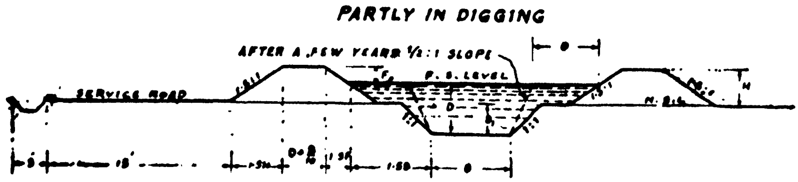


Fig. 2

The bank width $= Z + D + \frac{B}{10}$

where Z = height of bed above N. S. ; D = depth in feet ; B = bed width in feet

Free board $= F = 2 + \frac{D}{10}$ in feet and the berm width = D

The digging is 1 : 1 up to N. S. and the side slopes of the banks inside and outside are 1½ to 1. It is assumed that the berms shall silt up to the dotted lines to a slope of 1/2 to 1, thus leaving a clear berm of length equal to D at full supply level in the distributary. The inner edge of the berm is kept at water level by cutting the lip. In course of time the other edge is sloped up 1 in 10 by the earth made available by cutting the lip. The earthwork dug from the bed is used on the banks and the additional earthwork required for them shall be obtained from borrowpits usually placed in the fields at least 10 feet away from the canal land limits.

The compensation for the earth got from the cultivators' land is paid according to the rules in force framed by the Local Government. The aim of the ideal design should be that the earth obtained from digging in the bed is equal to the earth required in the banks. The channel is then said to be designed with the balancing depth. If the digging is more than the earth required on the banks, then either the bank dimensions are increased or it is dressed half a foot lower than the bank level as a spoil behind the bank on the side other than the boundary or service road..

(*b*) Fig. 3 shows the cross section of a distributary when it is in filling with bed level above ground level. The height of the free board is kept the same. The bank width is kept to suit the permissible hydraulic gradient line of 1 in 5.

In filling with bed above N. S.

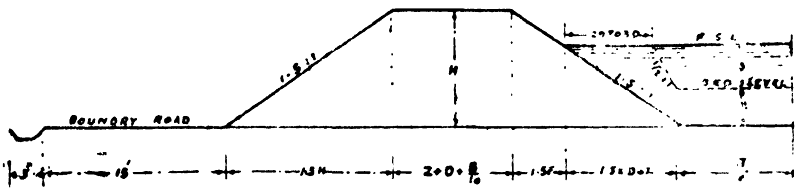


Fig. 3.

The berm width is kept 2D to 3D according to the requirements depending on the embankment height.

The side slopes are 1½ to 1 both inside and outside. All earthwork for banks is to be obtained from the borrowpits sufficiently away from the banks.

In the case of large channels the borrowpits can also be put in the bed leaving five feet berm from the inner toe of the banks on either side and 10 feet wide barriers accross the bed after every 90 feet.

(*c*) Fig. 4 shows a cross section when the channel is in digging with full supply level below the ground level *i.e.*, N. S.

In Digging F. S. below N. S.

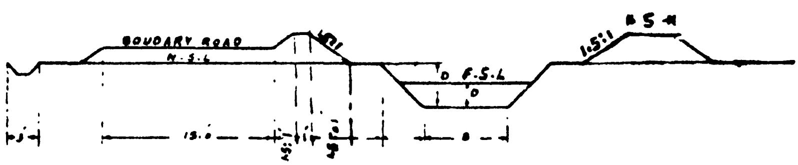


Fig. 4.

In this case the earth obtained from digging is more than required. The digging is 1 to 1 on sides and usually a berm 2 to 3 feet is left between the inner toe of the spoil banks and the digging limits. The excess earthwork can be used to raise the boundary service road so that it is not flooded in rains as shown in Fig. 4 and if the earthwork still in excess of these requirements has to be dressed in the form of a spoil bank on the right side as shown therein.

3. Main Branch and Canal Cross Section.

A typical cross section is sketched in Fig. 5. The free board is 2·0 feet. The bank widths are 17′ to 22′ on the patrol road side and 12 feet on other side exclusive of the dowel which is 1 foot high and 4 feet wide at the base with 1 foot top width, sides 1·5 to 1. Usually there are two plantation lines on either side with boundary road for cart traffic in been them as shown therein. Side slopes of banks are 1½ to 1 and digging 1 to 1.

The inside berm dimensions shall change according to the considerations outlined for the distributary sections in paragraph (2) above.

4. (A) Earthwork Specifications.

(*a*) **Setting out for earthwork.** On canals and large channels, previous to the commencement of work, the centre line is marked by pegs at every chain, curves are properly laid out, all half-breadths are carefully set out, the top and bottom edges of the excavation and the toe of all the embankments and spoil bank are clearly lockspitted.

(*b*) **Profils for earthwork.** Before construction, a complete profile should be set up on every 5000 feet of distance and at every change of section. This profile should be a 10 feet length of the actual completed channel or embankment, the excavation being dug to the proper level, banks thrown up to the correct height and width and all slopes dressed to true form. Care should be taken that the ends of the profile banks are stepped so that they be picked over at the time of construction of the banks adjoining them.

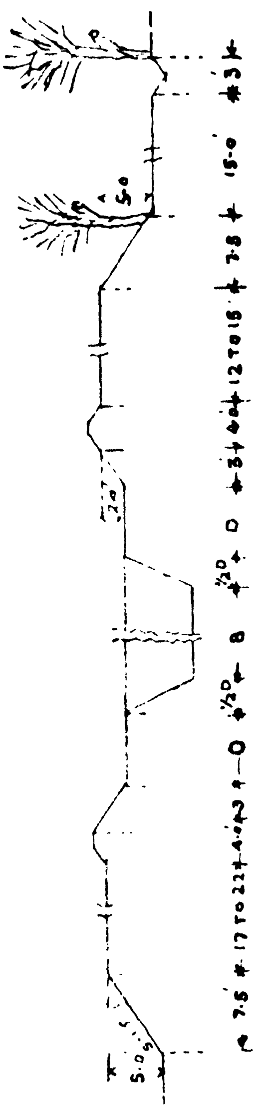


Fig. 5.

(*c*) **Stripping soil prior to earthwork construction.** Before beginning work, the surface area of ground to be occupied by all banks and spoil shall have all its jungle and roots grubbed and be ploughed over so as to completely eradicate roots and grass, and other jungle from it. Where the earth below the top layer is not suited for making substantial covering to roadways and embankments, the top layer of good soil should be set aside for this purpose.

(B) **Excavation of channel.**

(*a*) The excavation to be dug in lifts of from 2 feet to 5 feet, as may be ordered by the officer in charge, and in each chain, each lift to be completed, as far as possible, before the one below is commenced. Care should be taken that the finally completed width of the channel is in no place exceeded. All gangways, roads and stepping should be left within the channel and not cut into the slope. The final dressing of the slope will then consist of digging only and no filling or making up will be necessary.

(*b*) **Deadman.** Deadman or such other marks as the engineer-in-charge may direct shall be left at points indicated by him. These should remein in tact till measurements are completed, but final payments should be deferred till all the marks are removed. Where natural surface is regular, deadman or benchehes shall be left at equai-distant intervals.

Note—This paragraph applies to borrowpits as well as to channel excavation.

(C) **Slopes for earthwork.**

The standard slopes best suited for the ordinary average soil met with in the Punjab plains, are 1 : 1 in excavation and $1\frac{1}{2}$: 1 in embankment, but where inferior soil is met with and in the case of flood embankments, special section shall be proposed by the local officers in the estimates for the work or reported for orders of sactioning authority when the work is put in hand if the estimate was made out and sanctioned for ordinary conditions.

(D) **Borrowpits (outside borrowpits)**

(*a*) Borrowpits should be dug only where unavoidable spoil for the formation or the banks being led along the channel, if possible, in preference to taking it form borrowpits. No borrow-pits should be within ten feet of the toe of the bank and if its depth exceeds 2 feet, the distance to top edge of pit should not be less than 15 feet. Where borrowpits extend for a considerable distance, a bar, separating them, 10 feet wide at the top, shall be left in every chain so as to prevent drainage from running along the back of the canal bank. Borrowpits shall be as shallow as possible, and they will therefore extend over the whole area available for their formation. No borrowpits should be dug in the canal bed below bed level except under special sections. All borrowpits should be properly laid out by the subordinate-in-charge before diging is begun.

No borrowpits should at all be made in the berms of channels. Approximately 2½ feet width of the lip of the berm near the water edge may however be dug away not in the form of borrowpit but it should be continuous without "*taties*" for at least a chain. The lip can be dug to about half the full supply depth but not lower.

(*b*) **Table of quantities of earthwork.**

In distributary estimates the table of quantities shows, for each 500 or 1000 feet length, the quantity of earth that will be obtained from channel excavation, and the quantity that must come from borrowpits.

(*c*) **Widening of distributary bed to obtain earth.**

In the new construction extra earth required for banks shall preferably be obtained by widening the bed of the canal itself. The bed may be widened to three times the normal width without causing defects in future working and maintenance, such widening to be of the same amount through each length of low ground, and not to vary frequently.

(E) **Embankments Laying.**

(*a*) All embankments should be thrown up in layers which should never exceed half a foot in height, and to the full completed width. Each layer should be commenced from the edge farthest from the excavation so that all earth is throne into the slope and not tipped over it. Care should be taken that the top of each layer is level or slightly hollow in the centre ; any rounding should be dressed down level before the next layer is begun.

(*b*) **Breaking of clods.**

All large clods should be broken up in the borrowpits and care should be taken that no clods larger than a man's fist, nor any roots, grass, jungle or other rubbish are brought in baskets to be burried in the banks.

(*c*) **Allowance for settlement of banks.**

The height of the bank should be in all cases one-tenth greater than that shown on the drawings, to allow for settlement.

(*d*) **Dressing earthwork.**

When thrown up to full height, the bank should be dressed to the slopes and dimensions ordered.

(*e*) **Watering earthwork.**

Whenever water can be let down in a channel, the embankments of which are under, formation, the area reserved for borrowpits to be moistened to obviate all risk of colds being introduced into the banks.

(F) **Spoil Banks.**

(*a*) Spoil should be tipped as directed by the engineer-in-charge, and spread evenly over the whole area available in layers of not more than 1 foot thickness. It should be dressed to the slopes and form shown in the type section of the work. Bad soil and sand where possible should be faced and topped with good earth.

(*b*) **Spoil banks for plantations.**

Spoil banks intended for plantations to be provided with long and cross dowels, forming compartments 50′ × 50′ so that no rain water can flow off.

(G) **Ramps (Earthwork).**

The plans of canal masonary works shall include such a site plan to a suitable scale to show the ramps. Such site plans should show the ramps to any subsidiary works in the neighbourhood, such as bridges on ditch distributaries, and mill channels ete. The plans of distributary masonry works should also show how the ramps are to be arranged. Before construction is started, the ramps shall be marked on the ground by lockspits

(*a*) **Foundation pit (Excavation)** The foundation pit should be dug truly to the level shown on the drawings. Where the soil is strong and the pit is above spring level the whole area, may, at the discretion of the engineer-in-charge be opened out prior to the commencement of constuction. In such soil the bottom of the pit should be dug truly to the plan of the bottom of the foundations, and the side slopes should be as steep as in compatible with safety for slips, under the shock of ramming.

(*b*) **Inspection of pits prior to laying of foundations.** No construction to be commenced without the order of the engineer-in-charge and such order in case of all works in the channel of the canal or branch must not be given until the pit has been inspected by him and passed as true in depth, form and length.

(*c*) **Foundation plans.** For large works a foundation plan should be prepared by the Engineer-in-charge showing reduced levels of each part of the pit.

(*d*) **Watering of pit.** Prior to the cemmencement of work, the bottom of the pit should be thoroughly watered but all ramming of dry ballast or laying of bricks or stone over its area should be absolutely prohibited except in the case of slushy foundation.

(*e*) **i. Soft foundations.** In laying concrete in slushy clay foundation it well found that the lowest layer of 3 inches depth will on ramming sink into the soil and mixup with earth and be rendered useless as concrete. In such cases, to ensure the full thickness of concrete being good, excavation should be carried out to such extra depth as may be ordered by the officer-in-charge and dry ballast should be rammed down to from a satisfactory bed for the concrete to rest on. In the case of foundations in bad soil or below spring level, only as much of the pit should be dug down to final level as can be rendered safe during the day by the laying of the bottom courses of concrete or masonry. Extreme accuracy of form or level cannot be looked for, but care shall be taken that the level is nowhere higher than that on the drawings, and that the edges of the lower course of concrete or masonry as laid are nowhere within outer footings as designed.

(*e*) **ii. Foundation pit (filling).** The backing of walls and fillng of excavation should be carried up in level layers of not more than ½ foot in thickness. The top the filling should be, as a rule, about 2 feet below the top of the masonry. Before commencing any layer of the filling, the surface of the layer below should be cleared of all spawls, bats, and other debris, and each layer should be flooded to a depth of 2 inches over night so as to be consolidated and to prevent absorption of moisture from the new masonry or concrete. Care should be taken that no clods, root, grass or other rubbish, is buried in the earth filling which must be laid gently to prevent dust getting on to the top of the completed course of masonry. Where the filing consists of shingle, care should be taken that enough sand or finely broken up loam is mixed with it to ensure complete filling of all interstices. No silt or soil having angle of repose when wet, less then 2 to 1, should be used for filling.

(H) **Turfing earthwork.**

Turfing should be carried out over the areas in the manner directed by the Engineer-in-charge. It should be done in all cases immediately after the setting in of the monsoons and should be kept well watered until the seeds or roots have sprouted. It should be protected from cattle.

Watering by spray being preferable to watering by flow, should be adopted, where possible.

5. Earthwork Specifications for Repairs.

(*a*) **Repairs to banks (Holes and ravines).** All holes (*gharars*) and ravines should be, wherever possible, first fully opend out to the bottom, all lumps of fallen earth dug away, and the sides dug down in steps of not more than 1½ feet depth. All jungle, grass, roots, or other rubbish, should be thorougly cleared and the work when ready for filling should be inspected and passed by the engineer or subordinate, deputed for the purpose, before filling is begun. Filling should be done in accordance with the specifications given for foundation pits except that constant wetting during the work may take the place of flooding each layer. Flooding the top layer of the day's work should, however be done every night, ramming with wooden rammers while the work is in progress, should be done when directed by the engineer-in-charge.

(*b*) **Repairing of banks by earth from berms.** Where a silt berm exists, earth for filling and for repairs generally should be obtained as far as possible, by cutting away such berm, care being taken that a layer of at least six inches thick of silt next the bank is left untouched, except under special orders of the engineer-in-charge and that cross dowels are left at close intervals in the silt berm, to permit such borrows silting up quickly. Any bank which is to be widened, should be ploughed or cut into steps.

(*c*) **Raising of driving banks.** Raising of driving bank shall not be done with sandy earth from silt berms.

(*d*) **Repairing of banks by earth from spoil banks.** It there be no berm, or if the soil obtainable from the berm be insufficient, earth shall be obtained from the spoil bank, if such exists, or from outside excavation. In getting earth from the spoil bank borrowpits on top shall be rigidly prohibited, as in wet weather they form tanks and lead to damage by breaching. Earth, therefore, is to be obtained from the back of the spoil, or by widening the drainage gaps in the spoil banks, where such exist.

(*e*) **Repairing of banks by earth from borrowpits.** Where there is no spoil, earth should be obtained by levelling down any high lumps, if there be such, and last of all from borrowpits. Where borrowpits are unavoidable, they must be dug as far from the toe of the bank as possible (the minimum distance to be 10 feet), must not exceed one foot in depth and must be neatly set out parallel to the banks. The long slope which forms at the toe of all banks by washing down of soil from the top and slope shall in no case be dug away.

(*f*) **Silt clearance.** The spoil from silt clearances of channels should be spread out evenly in the neighbouring borrowpits, if such exist. If they do not, the spoil should be spread evenly along the back of the bank, thus widening and strengthening it. Care should be taken that the spoil is not heaped up on the top of the bank, or thrown in lumps on the outside so that it may not be blown in by the winds.

6. Specifications for Puddle.

The clay should be dug and exposed to the air for two or three days. The clay containing sodium carbonate (called sodium clay) is the best for puddle.

If necessary, sand should be added until the mixture is suitable, and the whole should be wetted and well worked up in a pug mill or by men's feet into a smooth plastic mass. The working with a pug mill or kneading with feet tends to convert sodium carbonate into a colloidal solution and the collods tend to choke the fine pore spaces to make the puddle impervious.

7. Construction of High Embankment.

Special precautions have to be taken in the construction of high embankments, such as approaches to aqueducts in the case of large canals for examples Solani aqueduct, of the Ganges canal in the United Provinces.

(*a*) Specifications of earthwork in general as described in paragraph 5 above, apply to the earthwork in high embankments concerning the layout, removal of jungle, the size of the clods, location of borrowpits etc.

(*b*) The usual section adopted in the good old days was as shown in Fig. 6 with a puddle core. The earth was laid in 6 inches layers and consolidated by wooden rammers and the puddle core about 5 feet thickness rose as the bank progressed. The main reliance was placed on the efficiency of the puddle core as a water tight substance. This efficiency of the puddle depends on the retention of the moisture in it because the densification of the puddle is due to the chocking of the pore space by the sodium clay being pugged into colloidal state to fill the pores. The dried puddle is, therefore, even worse than ordinary soil banks because it cracks. The dry soil around the puddle core extracts the moisture in course of time, reducing thereby its efficiency. It is therefore, that high embankments in the past have always been a sourse of great trouble.

(*c*) The modern practice is to stabilize the whole mass of the soil in the bank. Various methods of soil stabilisation have been described in Chapter I Part VI such as :—

(*i*) To manipulate the component parts in such a way that the mixture will produce compact soil.

(*ii*) Use of various admixtures such as electrolytes, chemicals binders, and adhesives.

(*iii*) By the eletrochemical process such as the application of heat.

(*iv*) Compaction and densification at the opitimum moisture content.

Only the last method is of practical utility when earthwork is to be carried out on a large scale. In this method the soil is made impervious not by chocking the pores but by compaction.

Compaction results in reduction of the film of water around the soil particles. When the soil is compacted at the optimum moisture content the dry density of the soil is maximum. There are various methods of compaction at optimum moisture content. The one described below can be used in the field on a large scale.

(*i*) Measure the sand content in the soil by drying a soil specimen, then pulverising it and passing it through sieve No. 270, calculate the percentage by weight of sand left on the sieve.

(*ii*) Calculate the optimum moisture content in each case according to K.B.S.I. Mahbub formula, Punjab Engineering Congress, Lahare, Paper No. 257.

$$W = 25 - \cdot 14S$$

where W = percentage optimum moisture content
S = percentage of sand in the soil.

(*iii*) Determine the hygroscopic moisture content of the soil.

The difference of the optimum moisture content and the hygroscopic moisture will give the amount of water to be added while compacting.

(*iv*) Lay the soil in 6 inches layers and then add water as worked in (*iii*) above. With a little experience, the variations due to the Prevailing temperatures can be easily allowed.

(*v*) The soil should then be rolled by means of 1·3 ton dentated roller. The roller is made of concrete having staggered teeth projecting three inches and driven by bullocks. The soil is supposed to be consolidated when the impression by the projecting teeth is not more than ½" deep or when the surface has been rolled by 16 to 20 times.

(*vi*) After consolidation of each layer, the density of the soil is tested. It should be about 1·48.

(*vii*) Each layer should be covered with a couple of inches of sand after consolidation which should be removed when the next one is laid. This is just to stop soil evaporation during the interval.

8. Sand and Puddle Core.

In the case of banks which are otherwise strong having hydraulic gradient flatter than 1 in 5, but subjected to leakage due to rat or insect holes, an economical method to remove the trouble is to provide a sand core as shown in Fig. 6.

SAND AND PUDDLE CORE

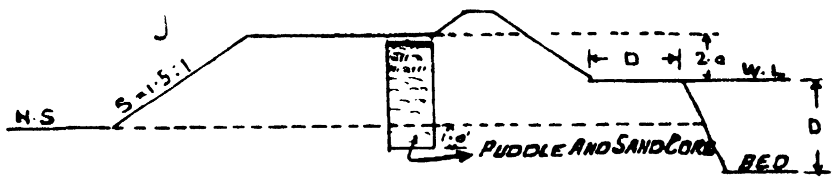


Fig. 6.

The sand collapses and fills the rat or ant holes and the leakages stop.

Sometimes due to very porous soil, even the hydraulic gradient 1 in 5 might allow seepage water coming out from the outside slopes. The banks do not generally fail on account of such leakage unless the exit gradient or the floatation velocity is more than that required for the particular soil. The economical remedy to stop the trouble is to provide a puddle core instead of sand core as shown in Fig. 6.

In the case of high embankments when the bank width has been fixed according to practical considerations and when hydraulic gradient line falls outside the toe of the normal outer slope of the bank, the width at the base can be increased providing a *pushta* as shown in Fig. 14. Chapter III of this Part. A large economy in earthwork can thus be effected, because earthwork from borrowpits is not generally available in low places, requiring high embankment.

9. Berm Formation.

The author described the formation of berms in detail in his paper No. 154, Punjab Engineering Congress 1932 on "Remodeling of Mithalak Distributary". The Mithalak disty.

was 34 to 35 feet wide in the head reach against the required bed width of 16·5 feet according to Kennedy's theory. The contraction was attained by encouraging berm formation by the use of bushing. A brief summary of the proccess is given here.

(*a*) **Action of bushing in formation of berms.**

According to Kennedy's silt theory, the silt carrying capacity of water depends upon the vertical eddies which are formed on account of roughness of the bed. The rougher the bed the greater is the number and magnitude of these eddies. The aim should be to nullify these eddies on the sides so that their silt carrying capacity is reduced to the minimum.

Moreover it is well known that when a fixed obstruction is placed in a channel, there is scour upstream and silting up on downstream side of the obstruction. The obstruction serves to roughen the bed, causing vertical eddies upstream of it, which lift up the silt resulting in erosion upstream of the obstruction. The water as it leaves the obstruction with reduced velocity is unable to carry its burden of silt which it originally carried and which was further supplemented by what it picked up upstream of the obstruction and so unloads itself just downstream of the obstruction. It is for this reason that we need flexible obstruction to reduce the velocity on the sides.

(*b*) **Three types of bushing are usually used.**

(*i*) **Longitudinal bushing.** It consists of a line of stakes, 3 feet apart on one or both sides parallel to the length of the channel with branches inter-twined behind the stakes and providing cross spurs 50 feet apart. This form of bushing is very expensive. It can be laid properly in a closure. If long reaches are rushed through by putting tiny branches between and behind the stakes, it is the least effective. The tiny bushing behind the stakes is useless as it floats when the water comes. The stakes aggravate the evil. They are fixed obstructions. As explained before, they produce eddies and the consequent erosion around them and when the toe is not formed, no berming up is possible. These remarks apply only to the longitudinal bushing improperly done.

(*ii*) **Cross fixed spurs.** They consist of lines of stakes with jungle around projecting form banks across the channel to the designed bed line. Sometimes there are double lines of stakes with jungle in between them. The cross fixed spurs are no doubt effective. Their function is, however, one sided. They serve to reduce the horizontal velocity of water on the sides which results in silt deposits bedind them, but they do not stop the vertical eddies and are accompanied by erosion just upstream of them especially near the nose. The berm formation thus obtained is very zigzag.

(*iii*) **Hanging Spurs.** The hanging spurs are found to be the best solution. Branches are hung from banks as shown in Fig. 7

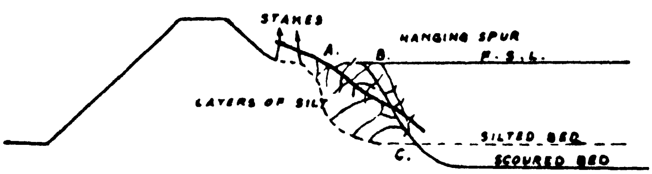


Fig. 7

Their action is two-fold. Their leaves and branches suppress the vertical eddies on the sides and they serve as flexible obstructions to slow down the velocity of water on the sides without causing any erosion. The hanging spurs are at once the cheapest and the best. Being flexible obstructions, they cause the minimum heading-up. The formation of the berm is uniform and the work can be done in running water.

The progress of the rise of the berms is rapid to beginwith, but it diminishes as the berms rise to the full supply level. The berms attain the final shape A.B.C., as shown in the sketch. The process is additive, where the length of the spurs is reduced and the bushing is repeated more than once. Where full berms are formed, the protruding branches are allowed to rot, as cutting them would disturb the berm formation. The inner slopes of the berms formed in the first state are generelly 1 to 1. The further growth of the berms to conform to ½ to 1 slope is gradual and in some cases it needs further artificial aid in the from of small hanging spurs. As the berm growth progresses the bed scours as shown in the Fig. above. The hanging spurs prove the best method when berming up accompanies the scouring of bed. Complete success of the berm formation depends upon the efficiency of the bushing and its renewals. In a fortnight the branches dry up with the result that the spurs become ineffective. New branches are put in by the side of the old ones from time to time. Similarly twisted branches are renewed. It is the persistent and patient repairs of the spurs, which lead to success.

(*c*) The longitudinal bushing has an advantage of its own in certain conditions. If a channel is silt-cleared to the designed bed and the contraction is not complete, the flexible obstruction will yield morê waterway and lead to silting up of the channel. Here the longitudinal bushing offers the best solution. This sort of bushing is, therefore, used with adventage, where the channel was locally wide, with the important precaution that the rolls of the branches round the stakes are well pressed, so that erosive action of the stakes is reduccd by the leaves around them. It is found that the leaves and the branches of the rolls round the stakes extend considerably in the channel and that the berm formation is about 1 to 1 inside with the highest point at the stakes. This results in over-contraction, when the distance between the stakes on either side is kept equal to bed-width plus half depth. It has been tried and found in the case of large scale contraction on the Mithalak distributary that the distance between stake lines should be bed width plus depth.

10. Strengthening of Banks by Silting Tanks.

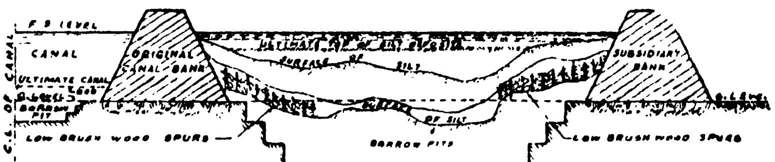
Strengthening canal banks by silt formed the subject of a note by A.G. Reid in 1894 and of a paper read before the Simla Irrigation Conference in 1904. The object is to strengthen canal banks when they stand in low ground. The system of doing this by silting was orginally started in the N.W. Provinces and subsequently adopth and largely extended in the Punjab, where extensive reaches of the canals have had their banks strengthened by this process. There are three systems described in paper :—

(*a*) **The 'In and out' system.**

Under this system additional banks are constructed. outside the original canal banks and parallel to them, with cross banks at intervals of 500 feet to 1,000 feet apart. The series of compartments, which are thus formed, have inlets from the canal at the upstream ends and outlets to the canal at the downstream ends of the compartments. A portion of the canal discharge is passed through each compartment by these inlets and outlets, and it is thus gradually filled up with silt.

(*b*) **The 'Long Reach' system.**

Under this system (Fig. 8) external paralled banks are constructed, as in the first case, with similar cross banks, but these are at 4000 to 5000 feet intervals, Head inlets and tail outlets



"LONG REACH" AND "IN AND OUT" SYSTEM

Fig. 8.

are constructed, and not only a portion of the canal supply but the whole of it is diverted into one silting reach at a time. The canal channel contiguous to the silting reach in operation, is closed at its head and tail to prevent the deposit of silt in the canal. Where this system is employed, spurs also are constructed to encourage deposits.

(*c*) **The system of 'Internal Silting'.**

Under this system (Fig. 9) the canal banks are set back at a little distance from the normal section of the canal channel. Inducements are laid down to encourage the deposit of silt internally on the berms. This system is the simplest, the best, and the most economical of the three, but it can only be applied to a new canal which is so constructed.

The papers give full details of the operations, with illustrations. It was found in the operations of the Rakh Branch of the Lower Chenab Canal in the Punjab that in 22 months

16,000,000 cubic feet of silt was deposited in a length of about seven miles. It was found best to commence with the "long reach" system and complete the operation with the 'In and out' system. The proportion of silt deposited to water passing was about 1/2200. The sitled portions were eventually ploughed and sown with *shisham* very successfully.

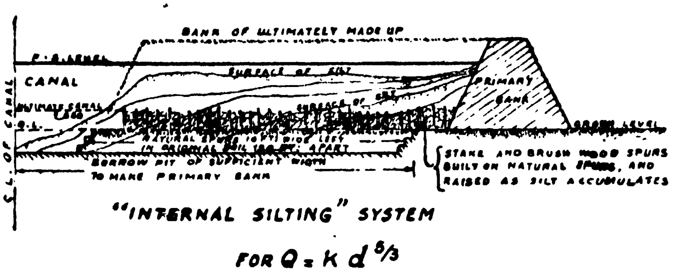


Fig. 9

To obtain the most speedy and satisfactory results with a long series of reaches, each reach should be closed and made reasonably secure by obtaining a sufficiently heavy deposit. Other reaches should then be taken in hand, and the completion of the warping in the reaches first opened, which being a slow process should not be undertaken until all the reaches have received their first heavy silting. The inner bank should, therfore, be made sufficiently strong to withstand the head of pressure when the canal is full and there is no water in the reach. One other point to be noted is that silting reaches should not be made too narrow. A.G. Reid says that the width of the tank will of course be the whole of the width between the canal bank and the boundary road, but it may often be desirable to increase the usual width between the outer boundaries when passing through low ground in which silting reaches are contemplated.

A. M. R. Muntagu, Superintending Engineer Western Jamna Canal, evolved a system of silting tanks on the Western Jumna Canal with a view to remove the silt trouble of the canal. Clear water after dropping silt in the silting tanks, picked up silt from the reaches of the canal and distributaries lower down and thus caused scouring of channels. This method gives a great relief to the silt trouble in a canal where proper silt Extractors and Ejectors cannot be constructed on account of the supplies being not available for escapage.

11. Accidents to Canal Banks.

Breaches in the canal banks are fairly common. They may be due to (*a*) Intentional cuts by the cultivators to irrigate the adjoining lands or (*b*) the usual causes for normal breaches.

The accidents of the former case, can be avoided by patrolling banks at night, by levy of additional charges for the water thus wasted, and by strict judicial action against the persons at fault.

The breaches can occur due to the following causes :—

(*a*) **Weak Bank.** Insufficient bank width should be made good by periodical strengthening of banks which is normally required after every fifth year.

(*b*) **Overflows.** The overflow can also be avoided by periodical raising of the banks to the designed sections.

(*c*) **Leakages.** The leakages occur through insect or rat holes. They take a considerable time usually more than 24 hours before they develop into a breach. If patrolling by *beldars* is efficient, no breach should occur on this account.

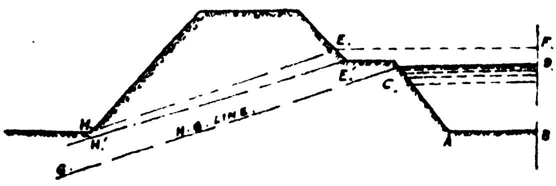


Fig. 10

(*d*) **Piping due to excess supply.** Even though the bank be strong the excess supply over-topping the berms results in failure of the banks by piping as explained in Fig. 10.

Let CD be the surface of canal water, where it normally runs full supply. CG represents the saturation line in the bank. The bank as sketched above is extra safe. Let there be an excess in the canal so that water level rises to EF. What actually happens is that the line of saturation is changed to EH. A breach should not occur even if the point H be higher than the toe of the bank. At the most water should seep out from the canal. The velocity of the seepage cannot be high enough to dislocate the soil particles of the bank. A failure never occurs by percolation through the banks.

Under normal conditions the soil of the bank above CG is dry and has probably been never wetted. The rise of the saturation line from CG to EH wets for the first time the soil of the trapezium CGHE of the bank section. Dry soil on first wetting contracts and the bank stands at places by arching action. Thus an open pipe is formed between EH and E′H′ as shown in the sketch above. Water flows out of it as a leakage as if an open connection exists from water in the canal to that outside. The water comming out starts washing out the soil and eventually develops into a breach. If in the beginning the mouth E of the pipe EH could be located and closed, the breach would never occur.

12. Closing of Breaches.

(*i*) Closing a breach in a small distributary or minor.

Water of the breach spreads on the adjoining lands and usually there is no place to take earth for closing the breach. The earth has to be obtained by cutting the outer slope of the existing bank. Enough earth should be collected on both sides of the breach on the existing bank. The earth baskets should never be thrown in the water. All jungle should be removed from the breach site by men in running water. The process starts from both ends by slipping the earth from the heap and protecting the channels side by grassy clods usually available from the berms. No grassy clod should be allowed to be washed down into the breach site. With a rush of eartbwork at the end, the breach can be closed straightaway progressing from the bank.

(*ii*) Closing a breach in a major distributary or small branch canal.

In this case it is necessary to reduce the flow through the breach, otherwise a lot of earth will be washed away before the breach is closed. This is usually done by driving a double line of stakes as shown in Fig. 11 and then putting planks or mattresses against them if available and if not, then filling jungle in between the stakes pressing it down with bags filled with sand and by men walking over them. No earthwork should progress before the flow through the breach has been arrested to some extent in this way.

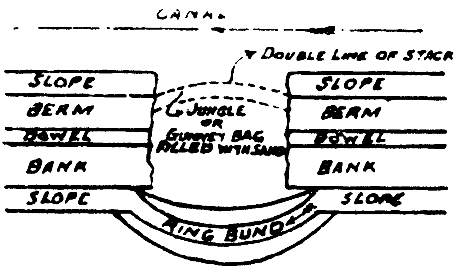


Fig. 11

Meanwhile earth is piled up on both sides. The closing is started from both sides by slipping earth from the heap in form of a ring *bund* as shown in Fig. 11. All jungle from the ring *bund* site should be removed before earth work progresses. No earth basket should be thrown in water, it must always be slipped from a heap. The last gap of about 10 feet should be closed with a rush when enough earth has been collected on both sides. Straight closure in large channels is not possible.

(*iii*) Closing a breach in a canal.

The closing of a breach in a canal follows the same method of the ring *bund* but the jungle or planks do not serve the purpose of arresting the flow through the breach. The double line of stakes should be driven as before if depth permits and a double line of gunny bags filled with sand is put in. The inter-spaces are plugged with berm earth. A temporary bank of gunny bags is raised in the position of stakes and bushing as shown in Fig. 11. The closing of the breach is then done by constructing a ring *bund* behind.

13. Examination Questions.

1. How would you consolidate a new canal bank in filling if it has clods and is full of *kallar* ? (P. I. B. 1941)
2. How would you increase the amount of silt entry into a silting tank inlet ? (P. I. B. 1941)
3. Draw to scale the cross section you would adopt for building a two miles long embankment reach from a new distributary of the following dimensions : the soil is light and contains a small amount of *kallar*.

Bed width 25 feet	F. S. depth 3 feet
Stope 1 in 4000.	F. S. L. 5 feet above N. S. L.

State what precautions you would take on opening the channel and running it to ensure that the embankment reach would become safe and water-tight as quickly as possible. (P. I. B. 1940)

4. Describe what method you would adopt for each of the following operatiens :—

(*i*) To form berms on a distributary.
(*ii*) To check side erosion on a large channel. (P. I. B. 1939)

5. Give specifications for :—
(*i*) Repairing rat holes and ravines in a canal bank.
(*ii*) Deadmen for borrowpits. (P. I. B. 1936)

6. Draw cross sections of a channel of dimentions given below for the fallowing cases :—
Bed width 20 feet. F. S. Depth 4 feet. F. S. Discharge 160 cs.
(*a*) When natural surface is 3 feet lower than the bed level.
(*b*) When natural surface is 2 feet higher than the bed level but soil is very bad. (P. I. B. 1935)

7. How much labour should be arranged in each day for the construction of a fall on a distributary in a ten days closure, having the following quantities of work to be done ?
Earthwork in foundations. 200 cft. ; Cement concrete 400 cft.
Brick masonry 1500 cft. ; Dry brick pitching 300 cft. (P. I. B 1935)

8. Give specifications for *Gharabandi* and describe how you would carry out and check this class of work to prevent fraud. (P. I. B. 1936)

9. You are required to make embankments of a channel leading to an aqueduct. Lay down specifications for earthwork. (P. U. 1942)

10. What is "balancing Dept" of cutting as applied to canals ? Why are sides of canals taken as having slopes of $1/_2$: 1 for purposes of computing discharges although they are cut to slopes of 1 : 1 or flatter. Design and sketch the cross section of a distributary of 200 cs capacity showing berms of standerd dimensions when the distributary is (*i*) wholly in banks (*ii*) wholly in cutting (*iii*) partly in bank and partly in cutting. (*P. U. 1949*)

11. How would you close a breach in a branch canal having 500 cs. discharge. (*P. U.1955*)

12 What is the nature of damage expected when a canal bank has numerous rat holes what preventive measures are indicated. (*P. U. 1953*)

PART II

CANAL IRRIGATION

CHARTER VIII

Lining of Channels

1. The lining of channels has progressed only recently in India. It was brought up for the first time in 1917 by T. A. Curry in paper No. 32 Punjab Engineering Congress Messrs. R.S. Duncan and Som Nath Kapur contributed their papers No. 221 and 253 in the Punjab Engineering Congress on the "Haveli Lining". F. F. Haigh Superintending Engineer, incharge of Haveli lining wrote various notes on the subject. A comprehensive paper No. 260, Punjab Engineering Congress, Lahore 1943 by K.B.S.I. Mahbub on "Lining of channels." was published. A brief summary is given here and the student should refer to the original publications for a detailed study.

2. Advantages of Lining.

The main advantages of lining are :—

(*a*) To save water for extension of irrigation or increasing the water supply in areas already served.

(*b*) To prevent water from reaching the water-table and raising it, thus avoiding water-logging.

(*c*) To improve command owing to flatter possible slopes.

It may thus be worthwhile in some cases to line only the head reach of a distributary, so as to command the adjoining high areas and leave the channel unlined in the lower reaches where the command is ample.

(*d*) The stability of section, which in the case of distributaries should reduce remodelling and alteration of outlets.

(*e*) Reduction in maintenance costs.

3. Suitability of Lining.

The principal factors which have to be considered in deciding the suitability of any design are :—

(*a*) Permeability, *i.e.*, reduction in absorption losses. Generally speaking, bitumen lining or a suitable form of concrete or brick lining would reduce these losses appreciably. The presence of any cracks, however, would also have a material effect on these losses.

(*b*) Co-efficient of Rugosity, which will determine the carrying capacity of the channel.

It is preferable to use Manning's formula in designing the section as it is simple and fits closely to Kutter's, for which values of N have been determined by various experiments and deduced from several observations.

The values of N generally accepted are :—

Surface	*Perfect*	*Good*	*Fair*	*Bad*
Cement mortar surfaces	·011	·012	·013	·015
Brickwork in cement mortor	·012	·013	·015	·017
Concrete lined channel	·012	·014	·016	·018

Refer to article 23 chapter VI of this part for finding out lined channel section and Plate XIV of Vol. III.

(*c*) **Durability.**

This may be described as the resistance to the following disintegrating forces :—

(*i*) **Weathring.** This is caused by the disruptive action of alternative freezing and thawing and by expansion and contraction resulting from temperature variations and alternate wetting and drying. The resistance of any surface to weathering is thus a function of its water-tightness and volume-change characteristics. It may be remarked in this connection that the co-efficient of expansion and contraction of brickwork is about ½ that of cement concrete or cement mortar.

(*ii*) **Chemical attack.** The most important destructive agencies under this head are sulphate of sodium and magnesium, commonly encountered in so called alkali soils and water. Corrosion of concrete by alkali water can be materially reduced by the use of sulphate resisting cement. Surface coating treatments are also generally effective in prolonging the life of concrete or bricks, though this may not be a permanent remedy.

Alkali solutions increase in strength in dry seasons when dilution is at the minimum. The United States Bureau of Standards specifies that concentrations greater than 0·1 of 1 percent endanger concrete.

(*iii*) **Wearing.** In the case of concrete and brick-lining this wearing action is negligible for all velocities under ordinary conditions.

(*d*) **Intial cost and subsequent maintenance.**

This would vary with locality and the availability of various materials. Generally brick masonry lining would be cheaper than stone concrete lining.

(*e*) **Structural stability.**

The following factors may be considered under this head : –

(*i*) **Reinforcement.** In America, it is a common practice to reinforce the lining concrete. Reinforcement is designed to reduce the size of contraction cracks and to assist in preventing failure of the lining due to settling of the sub-grade or to back pressure from a saturated subgrade.

On the other hand this may delay relief being obtained by local failure in small patches, in the case of heading up of water pressure, thus causing extensive damage. This was found to be the case on the Haveli Main Line.

(*ii*) **Thickness of lining.** The sides of a channel to be lined should preferably be left nearly of the same slope as the angle of repose of the natural surface, which is of the following order :—

Material	Angle of repose	Slope
Firm clay well drained	45°	1 : 1
Clay loam, average sandy loam	36°—33°	1⅓ – 1½ : 1
Sandy or gravel soil	33°—26°	1½ – 2 : 1

Where the slopes are left steeper, the sides have to be designed as sloping retaining walls. Etchevery (Irrigation Practice and Engineering Vol. II by B.A. Etchevery) has worked out the following thickness of concrete lining corresponding to various depths of canals, based on Coulomb's formula :—

Maximum depth of canal in feet.

Side slope of canal.	Angle of repose of earth.	No surcharge and thickness of lining of.				Maximum surcharge and thickness of lining of			
		1″	2″	3″	6″	1″	2″	3″	6″
1/2 : 1	1 : 1	5·3	10·6	16·0	32·0	1·6	3·3	5·0	10·0
1/2 : 1	1/2 : 1	1·6	3·2	4·8	9·6	0·6	1·2	1·8	3·6
1/2 : 1	2 : 1	1 0	2·0	3 3	6·0	0·4	0·8	1·2	2·4
1/2 : 1	3 : 1	0·5	1·1	1·6	3·2	0·3	0·6	0·9	1·8
1 : 1	1·5 : 1	15 8	31·6	47·5	94·8	4·8	9·7	14·5	29·0
1 : 1	2 : 1	3·8	7·7	11·5	23·0	1·9	3·8	5·7	11·4
1 : 1	3 : 1	1·9	3·8	5·7	11·4	0·8	1·7	2·5	5·0
1·5 : 1	2 : 1	37·0	74·0	111·0	222·0	11·3	22 6	34·0	68·0
1·5 : 1	3 : 1	6·2	12·4	8·6	17·2	2·5	5·1	7·6	15·2

These results may be considered as useful for practical application only within reasonable limits.

The thickness required would be much more, if provision has to be made for any hydrostatic pressure across the lining

In the worst conditons, the lininig is subject to pressure due to saturated soil and the differential waterhead across it. Assuming angle of repose of earth to be 30° and angle of slope of lining as 45°, the differential head that a lining of 6″ and 9″ thickness would stand as obtained from calculations, is indicated in the graph in Fig. 1.

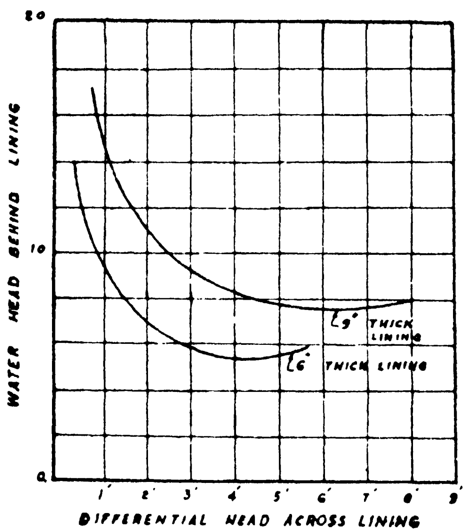


Fig. 1

It would be seen that with greater depths the lining would only stand a slight difference of hydrostatic pressure on the two sides. This clearly indicates the importance of proper drainage so as to keep the difference of hydrostatic pressure across the lining as low as possible.

(*iii*) **Strength of lining.**

Any reasonable strength should be good enough if the materials used are such as do not deteriorate in course of time. A piece of Bikaner lining shows on test a compressive strength of only 690 lbs/sq. inch.

The compression strength of 1:4 cement, sand mortar after 3 months is considered to be over 3,000 lbs/sq. inch. Both cement concrete and cement brickmasonry would thus give more strength than the Bikaner lining.

(*iv*) **Earth backing.**

In the U.S.A. Bureau of Reclamtion specifications, it is laid down that lining should only be placed on an undisturbed material or thoroughly compacted back fill. It is also necessary to provide adequate facilities for drainage of rain water to keep the backing in proper form. The American practice is to lay open joint pipes in gravel-filled trenches to serve as longitudinal drains. Feeder lines are brought from the side slopes where necessary and outlets are provided as required to discharge the accummulated water.

4. Various types of Lining.

The types generally used for lining so far are :

(*a*) Cement or lime concrete – 2″ to 6″ thick.
(*b*) Cement mortar—½″ to 1½″ thick.
(*c*) Stone Masonry set in cement or lime mortar—6″ to 12″ thick.
(*d*) Road oils.
(*e*) Sodium carbonate plaster.
(*f*) Clay puddle – 3″ to 6″ thick.
(*g*) Brick lining combined with cement mortar—3″ to 6″ thick.
(*h*) Precast cement concrete blocks.
(*i*) Tar or bitumen impregnated cloth covered by masonry.

(*a*) **Concrete lining.**

This is a very useful type and is generally adopted. It is durable if laid properly, and reduces the absorption losses by nearly 95%. The co-efficient of rugosity is vary low, and in view of the high velocities possible, the section is considerably reduced. The All American Canal is a concrete lined channel.

On the Gang canal (Concrete lining of Gang Canal by C.E. Jefferies, Punjab Engineering Congress, Paper No. 102) which consisted of 6″ of 1:1:6 mixture of lime, girt an *kankar* ballast, a value of Kutter's N = 0·013 was adopted. The value actually obtained was, however, repored to be 0·0145 in 1935 and 0·0153 in 1939. As all the ingredients were obtained from local *kankar* .the cost was quite low, *viz.*, Rs. 22/8/- percent sq. feet.

The joints were made V or Y shaped and later filled with bitumen in the case of the Gang

Canal. A lap type is not desirable as it breaks at the shoulder under tension. A butt Joint ¼" wide underlaid by a 1:4:8 concrete sleeper 1·0' wide, and of the same thickness as the lining, is considered to be better than a V or Y shaped joint, as this would obviate the necessity of filling with bitumen afterwards, and would simply mean leaving plain constuction joints at specific places.

To avoid cracks, it is desireable in such cases to limit the size of blocks to say 20'×20' or 15'×15'. Alternate blocks should be laid and preferably an interval of seven days allowed so that the setting contraction may take place. Alternatively, strips 5 ft. wide may be left all round the blocks, and these strips may be concreted after a week.

To avoid the subgrade absorbing moisture from the bottom portion of the concrete and thus making it spongy and permeable, it should be thoroughly moistened. According to the Buearu of Reclamation, U.S.A., the proper moisture penetration is 12" in sandy soil and 6" in other soils, except when this much moisture causes the subgrade to become muddy.

Other alteanatives are :—

(*i*) Use of oil paper. This has been found to give very good results. The cost works to about Rs. 2/- percent square feet.

(*ii*) Spreading of any crude oil on the subgrade so that the whole of the surface is fully covered. This may be usefully adopted where oil paper is not available. The cost of using linseed oil at Rs. 3/- per gallon works out to Rs. 1/10/- percent square feet.

(*iii*) 1:6 cement plaster on the subgrade. The plastering need not be very accurate, and half the labour rates normally paid for plastering should be good enough for this type of work. The cost including materials would work out to about Rs. 3/8/- percent square feet.

(*iv*) Spreading ⅛" of 1:4 cement mortar, sand slurry a couple of hours before placing the concrete. The slurry can be poured direct from the cans. This has been found to be quite successful in actual practice, and the cost works out to about Rs. 1/8/- percent square feet.

The consistency of concrete used in lining is a very important factor. The United States Bureau specifications for canal lining lay down that this consistency is critical. "The concrete must be fluid enough to compact well, especially under the reinforcement, and yet stiff enough to stay in place or the slopes. The slump must be carefully regulated. If it is too low, honeycomb is difficult to avoid, and if it is high, a bulging or wavy concrete surface may be produced. A properly designed mix, using well graded aggregate containing enough of the fine sizes to allow adequate finishing but not so much as to cause crazing of the finished surfa ce is necessary for satisfactory canal lining".

(*b*) **Cement mortar.**

This type is naturally not very durable unless suitably protected, and as such can only be used in conjnction with some other protective material. According to Etchevery, 1" thick cement mortar stops 75 percent seepage. Actually, however, it is much more effective.

(*c*) **Stone masonry.**

This has a limited application mainly on account of the cost and thus should be used only where stone is locally available. The co-efficient of rugosity is comparatively higher in this case, as stone cannot be very finely dressed.

(*d*) **Road oils.**

For using these the soil is rolled or burrowed so as to secure a penetration of 2" to 3". It is estimated that 1/16" thick layer of crude oil (at about 1/3" of a gallon per hundred square feet) stops 75 percent seepage. This is thus not as impermeable as the other types. The co-efficient of rugosity is fairly high and its effect is not durable. This does not stop burrowing by animals and will prevent weed growth for only a few seasons.

(*e*) **Sodium carbonate lining.**

This has been used in water courses and small channels and consists of 1" plaster made up of clay, *bhoosa* and sodium carbonate in the proportion of 100 cft. of clay, 6 mds., of *bhoosa* and 9 seers of sodium carbonate. The cost to the *zamindar* who provides his own labour and *bhoosa* according to the figure given by G.R. Sawhney, in his paper (lining of water-courses by G.R Sawhney, P.E.C. Paper No 240) on the subject is Rs. 25/- to Rs. 40/- per mile.

The author had an occasion to see some of this type of lining on the Lower Bari Doab Canal. The plaster was found to be damaged in most cases and could not be considered to have a useful life for more than 2 to 3 seasons. The saving of water effected by its adoption could be attributed more to the proper alignment and grading of water courses than to the plaster.

The method was also tried in the United Provinces but was not found to be a success. They had found that lining with *usar* (local salty soil containing 10% clay and 6% sodium carbonate) was more effective. A bitumen and mastic plaster would, however, be much more durable than even *Usar*.

(*f*) **Clay puddle.**

This has been in use for a long time as the only material required is suitable clay. The selected soil is allowed to weather and than pugged throughly after saturating it with water, by men marching up and down. This pugged clay is then put in position and covered with about 1′ layer of silt. This was considered to reduce seepage by about 80 percent.

Recent researches have clearly shown that there is an optimum moisture content for each soil at which the dry bulk density obtained is the maximum. We cannot, therefore hope to get the asme density by saturating the soil, as has been the practice in the past, as by controlling the moisture content and keeping it as near the optimum as practicable. This moisture content can be easily determined in the laboratory by using a compaction apparatus. An approximation can only be obtained by using the formula (paper 257 P.E.C. by K.B.S.I. Mahbub).

$$W = 25 - \cdot 14S$$

Where W = %age optimum moisture content

and S = %age of sand in the soil *i.e.* particles over 0·05 mm. diameter

The percentage of sand can be determined either by a Chain Hydrometer after taking into account the time taken by various particles to settle after dispersal, in accordance with Stoke's law, or by passing the dispersed soil through a 270 mesh sieve, with water from a wash bottle playing over it.

Rolling can than be done in 6″ layers at the optimum moisture content with toothed rollers, described in detail in Punjab Engineering Congress Paper No. 257, as these ensure a higher intensity of pressure and uniform kneading from bottom upwards.

It may be noted that the desirable placement range of moisture content is very much reduced with higher clay content and a more rigid control over moisture is thus necessary in such cases.

(*g*) **Brick Lining.**

This was used on a large scale for the first time in America in 1933 on the Southern Texas Irrigation Canals. It was also adopted with suitable modifications for the Haveli Main Line in 1937.

The Haveli Lining consisted of two layers of tiles $12'' \times 5\frac{3}{4}'' \times 2\frac{1}{2}''$ laid in cement with $\frac{1}{2}''$ layer of 1:3 cement plaster. sandwiched in between. The bottom layer of tiles was bedded on 1/2″ layer of 1 : 6 cement plaster. The massonry was reinforced with $\frac{1}{4}''$ M.S. bars laid in the 1:3 plaster, forming a grid $12\frac{1}{4}' \times 12\frac{1}{4}'$ in the sides and $24\frac{1}{2}' \times 24\frac{1}{2}'$ in the bed.

This lining failed in several reaches within a year of its construction due to water getting behind the lining from external sources and from wave action overtopping the lining. In one case the water caused a breach of the bank behind the lining, after which the lining subsided. In the other cases which were many, the pressure behind the lining caused it to fail and fall into the canal. No lining of practicable thickness could have stood under either of these circumstances. The defects that brought the failure were :—

(*a*) Inadequate compaction of the back fill.
(*b*) Lack of proper provision for the drainage of the banks.
(*c*) Insufficient free board.

Nothing was thus found wrong with the type and no similar trouble could be anticipated if the above defects were effectively removed in future designs.

5. Construction of Brick Lining.

(*a*) The following are the main precautions to be observed :—

(*i*) The earth to be used in the manufacture of bricks should not have a salt content above 0·3 percent and the quantity of calcium carbonate should not exceed 2 percent. The clay content should range from 10 to 20 percent.

(*ii*) Great care should be exercised in the moulding and burning of bricks. No *pilla* bricks should be allowed to be used under any condition.

(*iii*) The bricks should be thoroughly soaked before use. This is important as bricks which absorb water quickly lose it as quickly when taken out of water. Each mason should thus be provided with a kerosene tin of water in which his immediate requirements of bricks can be kept.

(*iv*) Sand should have a fineness modulous of preferably not less than 1·2, and should be free from organic impurities. The percentage volume of silt in it should not exceed 6 percent.

(*v*) The consistency of mortar should be properly regulated by having slump tests. The masons should not be allowed to mix water in the mortar pans at site under any conditions.

(*vi*) The plaster should be allowed to set properly for about two days before laying the masonry on top.

(*vii*) The subgrade should be properly moistened or oiled to avoid absorption of water from the bottom layer of masonry or plaster.

The bats obtained as a bye product while burning bricks can be broken into ballast passing through ½" ring and used in concrete lining, which can, for ease in execution, be restricted only to portions in bed.

This concrete can be laid in compartments, as mentioned already and levelled and compacted by means of heavy screeds or tampers fitted with handles weighing not less than 7 lbs. per linear foot. These should not be less than 3" wide and should have ¼" thick flat iron fixed on the underside. The surface can then be floated with a flat board 18' long and about 8" wide, after which it can be rolled, if necessary, by light rollers, weighing 10 to 12 lbs per ft. run.

(*b*) Brick lining in general has the following advantages over concrete lining laid in site:—

(*i*) Comparatively low cost, as bricks can be manufactured at site, thus avoiding long carriages.

(*ii*) Highly skilled or specialised labour is not required; as the work is of a simple straightforward nature.

(*iii*) No elaborate or expensive equipment is necessary as would be needed for concrete linings.

(*iv*) Due to the large number of joints, cross cracks due to contraction are greatly reduced, and buckling caused by expansion is eliminated.

(*v*) No expansion joints are necessary.

(*vi*) Brick lining can be more easily adopted to construction in circular sections.

(*vii*) There is no risk of any undetected thin areas due to poor workmanship, as the thickness is controlled in this case by the thickness of the bricks.

(*viii*) Repairs when necessary can be carried out easily and expeditiously.

6. Compactions of The Back-Fill and Banks.

(*a*) The importance of having a stable back-fill which would not settle behind the lining cannot be over-emphasized. This should be obained by compaction of the soil at the optimum moisture content. This can be easily attained by rolling the earth sufficiently in 6" layers, with a 1·3 ton toothed roller 4' wide having knobs 6"×4" and 3" high, at the optimum moisture content. Generally, 20 to 24 rollings would be good enough for the purpose. It may be remarked that the maximum dry bulk density would be obtained if the soil contains 70 percent sand and 10 to 20 percent clay.

As it is not possible to roll the ends properly, an extra width of 1½' on the inner side may also be compacted and then removed before lining.

(*b*) **Banks.**

If all the voids in a soil are filled with water, the volume of water for a given pressure is called the natutal volume of voids. The greater the pressure in the soil, the smaller is the natural volume of voids in it and *vice versa*. Settling will thus take place, if the actual volume of voids in the soil is greater than the natural volume when the soil gets wet and apparent cohesion ceases. Consequently, the more permeable the soil, the sooner will it settle. The soil in a bank should thus be rolled to the natural volume of voids corresponding to the pressures which will be in the bank to avoid future settlement.

The density of the finished bank should be checked by measuring it in a sample for every 500 cft. or so of the bank. This can be conveniently done by taking out the sample in the shape of V, so as to accommodate a wedge, 6″ square base and 8″ height, in the hole. The wedge is then placed in this hole and a measured quantity of sand poured outside it from a graduated cylinder, so as to fill all hollows. The volume of the soil would then be the volume of the wedge plus that of the sand and its dry bulk density can thus be calculated. In the case of small channels, it may be possible to run the earthen channel for six months to ensure compaction of the back fill and the consolidation of the banks before the lining of the channel is done.

7. Drainage of Storm Water.

As already mentioned, inadequate drainage facilities for the disposal of storm water were in a great measure responsible for damage to the Haveli Main Line.

The following precautions in this respect therefore seem desirable :—

(*i*) No berm should be provided in the channel as it simply forms a receptacle for water which can work its way behind the lining.

(*ii*) The dowel and bank should slope away from the channel. A slope of 1 in 80 for the bank and 1 in 20 for the dowel seems desirable.

(*iii*) A suitable drain should be provided at the toe of the bank to drain away the storm water.

(*iv*) In cases, where there is a spoil, a berm at least 5′ wide should be left on the spoil side of the drain.

(*v*) 10′ wide gaps should be left in the spoil 250′ apart to dispose of the water in the drains.

A typical cross section for a lined canal is shown in Fig. 2.

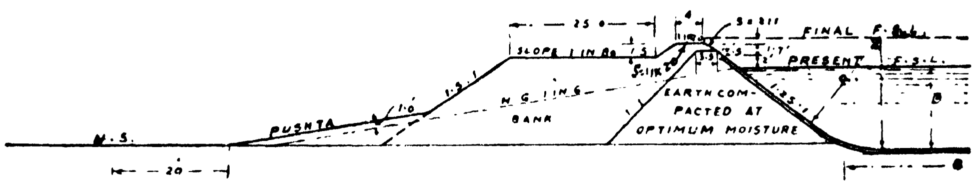


Fig. 2

8. Conversion of Existing Channels into Lined Channels.

For canals and branches it will generally be advisable to construct a new lined channel along with the old one, as the lining of the old channel cannot be carried out without interrupting the irrigation supplies.

In the case of small channels, the question as to whether the construction could be carried out in rotational closures may, however, be investigated in detail, in each case. In view of the saving which could be effected by this method, as compared with a new channel, considerable expenditure would be justified on temporary arrangements for feeding the channels during construction. For instance, banks might be strengthened to permit channels to run with larger supplies for shorter periods and the rotational programme might be modified to meet construction requirements.

9. Lining in Reaches with High Spring Level.

The lining of the head reach of a canal having high spring level will not generally be a very economical proposition due to uplift pressures under the lining in closures.

There may be circumstances, however, in which it may be desirable to line a reach in an area of a high spring level. To enable this lining to be properly maintained and inspected, it would be necessary to arrange a system of pressure relief under the lining, having a gravity or pumping outfall, depending on the conditions at site.

This pressure relief can be arranged by one of the following methods as described in a note on Confocal Conics applied to sub-soil to and from open channels, by F.F. Haigh :—

(*a*) A continuous inverted filter under the lining.
(*b*) A system of drains or porous galleries.
(*c*) A system of vertical relief pipes.

In the case of (*a*) there would be great difficulty in avoiding any leakage between the canal and the filter.

(*b*) This method also suffers from this defect to some degree. since all the length of the drains must be adjacent to the bed.

(*c*) This method suffers the least from this disadvantage and is probably the cheapest to construct.

The seepage discharge and thus the spacing and size of relief pipes or drains can be easily calculated from the formula derived by F.F. Heigh.

Whatever system is used, unless the outfall is through the lining, connection must be provided in the shape of drains or ducts from the relief elements to the outfall.

10. Losses from Lined Channels.

F. F. Haigh has suggested the following formula for the brick-lined channels :—

$K = 1{\cdot}25\ Q^{\cdot 056}$ where K = loss in cusecs per mllion sq. ft. of wetted perimeter.
Q = Discharge of the channel in cusecs.

This formula gives results generally on the safe side (if there is no drainage). This means a saving of about 75 p.c. in losses by absorption and percolation as compared with the earthen channel section.

The loss in the Bikaner Gang Canal has been estimated to be about 1·5 to 2 cusecs per million sq. feet. The tests of the losses from the Haveli Canal so far show similar figures.

11. Examination Questions.

1. Describe the advantages of brick-lining over the concrete-lining.

2. Sketch a suitable section for a lined channel in filling and describe the precautions which should be taken so that there is no settlement in the back-fill and the banks.

3. (*a*) What are the advantages of lining channels ?
(*b*) What saving in losses from canals by absorption and percolation can be expected by lining them ?

4. Describe the various types of lining usually used and say which type in your opinion is suitable for (*a*) distributaries and (*b*) main canals.

5. What additional precautions are necessary when the lining of the canal reaches with spring level higher than the bed is to be done ?

6. Draw a typical cross-section for a lined channel with a discharge of 12500 cusecs, and give a gist of formulae used by you to derive the section. What are the advantages of lining of canals. (P. U. 1952)

7. Design and sketch the section of a lined canal for a discharge of 10,500 cusecs.
The canal is liable to be subjected to heavy drawdown conditions. Devise suitable drainage arrangements for safety of the lining. (P. U. 1954)

8. Design the most economical section of a lined canal for a discharge of 12,000 cusecs with water slope of 1 in 10,000. Sketch typical cross-section of such a canal with N. S. below bed. (P. U. 1956)

9. Why are canals lined ? What are the different methods of lining irrigation canals and considerations in selection of a particular type of lining ? What are common causes of canal lining failures and how can they be prevented. (P. U. 1957)

PART II

CANAL IRRIGATION

CHAPTER IX

Cross-Drainage Works.

1. The alignments of all canals and distributing channels are selected in such a way that they run along the ridge and no drains would thus be intercepted by them. However, sometimes they have to cross the drains when the country is irregular and uneven. Works necessary to dispose of the drains are called Cross-Drainage Works. They are usually classified as follows.

(*i*) By lateral diversion, *i.e.*, by excavating a channel parrellel to the canal the stream can be thrown into another drainage line, for the disposal of which provision has been made.

(*ii*) By passing it underneath the canal, either crossing the stream on a raised aqueduct, or, if the headway is insufficient for a clear passage, the bed of the stream is depressed below normal level, and the water passing in a tunnel underneath, rising again on the further side. The latter is termed a syphon or a syphon aqueduct.

(*iii*) The drainage water can be admitted into the channel itself. This is termed an inlet.

(*iv*) The drainage can be taken into and cross the canal at the level of the bed of the latter, the inlet on one side and the exit on the opposite side. This involves one regulator across the canal and one at the further bank across the exit of the drainage. This is termed as a level crossing.

(*v*) The drainage can be taken over the canal by an aqueduct. This is called a super-passage, to distinguish it from an aqueduct proper.

(*vi*) The canal can be taken under the drainage line by a depressed syphon or syphon-superpassage.

2. Masonry Aqueduct.

(*a*) In Fig. 1. (*a*) we have an example of the usual design of a masonry aqueduct. In this work the canal bed is 25 feet above that of the drainage, giving sufficient headway to pass the highest flood, which is 18 feet deep.

Like all masonry aqueducts, the construction mainly consists of an arched bridge with platform at canal bed level, and provided with two solid parapets which retain the water flowing through.

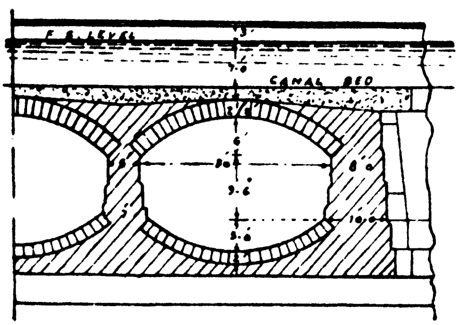


Fig. 1. (a)

To reduce expense, the waterway of an aqueduct is made narrower than the average width of the canal in earthen banks. Owing to the smoothness of the sides, the co-efficient of rugosity (N) is much less than that applicable to channels with earthen sides and bed, being ·013 in the former against ·025 or ·0225 in in the latter. This alone greatly increases the velocity, so that a considerable reduction in section can be effected, even if the original slope of the current were retained. As however, that velocity can safely be increased to 5 feet per second, if sufficient bed slope be given, a still further reduction in the width of the waterway in the aqueduct can be effected. In the example we are considering, the reduction in bed width is shown on the general plan in Fig. 1 (c).

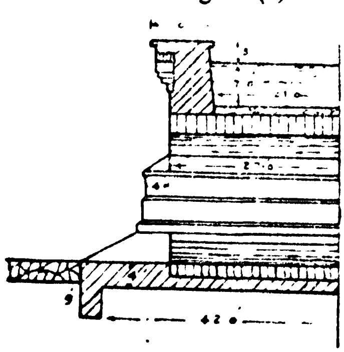


Fig. 1. (b)

(*b*) The thickness of the arch throughout is 2½ feet, the radius being 20 feet.

The value of the co-efficient in the formula, thickness $= n\sqrt{r}$ can then be approximately increased in proportion to the depth of water carried ; in ordinary cases $n = 0{\cdot}4$.

The following rule for deducing the increase to the value of n will suit in most cases (Bligh's rule). Let $d =$ depth of water, then $n = 0{\cdot}4 + 0{\cdot}02(d - 2)$. When the depth of water is 2 feet, 'n' will remain ·4 ; with 5 feet depth $n = 0{\cdot}46$; with 6 feet $n = 0{\cdot}48$; with 7 feet $n = 0{\cdot}5$; with 10 feet $n = 0{\cdot}56$. This is for large spans of over 25 feet. For smaller spans 'n' should be taken as 0·5, no matter what the depth of water is.

In the example the correct crown thickness would then be, with 7 feet water carried $= 0{\cdot}5 \times \sqrt{r}$ or $0{\cdot}5 \times 4{\cdot}5 = 2\frac{1}{4}$ feet. This would increase to $2\frac{1}{2}$ feet at the springing.

The parapets, as is usual in large aqueducts, are widened out to carry a roadway, as communication for cart traffic must be kept up along canal banks. The parapets here are 7 feet wide at base and 6 feet at top, corbelled out to provide a 10 feet roadway, with an iron rail fence on either side. The thickness of the parapet in this case is excessive. It can be made half, but should not exceed two-third of the depth of water. In the next example we shall see that it is made about $\frac{2}{3}$ D in width, D being the depth of water in channel.

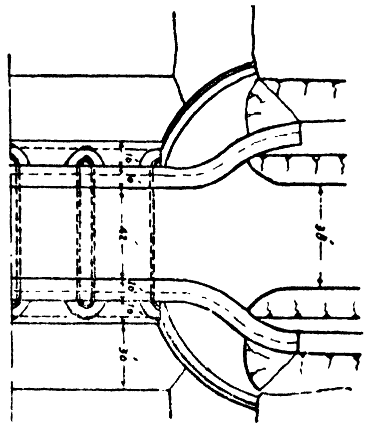


Fig. 1. (c)

(*c*) The piers are S/6 or ·167 S in thickness. They widen out by offsets to 7 feet, or 0·23S, at the base, S being the span.

There is no definite rule regarding the ratio of the thickness of piers proportional to the span in the case of large span bridges. It may be taken to vary from S/10 to S/5. For heavy works of this description the proportion S/6 as in this case, would not be excessive.

(*d*) The floor is composed of inverted arches with a versed sine of 5 feet, the thickness in the centre of span being 4 feet, and that at the spring line of the inverts 9 feet. The object of this invert is evidently to distribute the weight on the piers evenly over somewhat shallow foundation. It is very doubtful whether the inverted arch does really act in this way the use of an invert is more to prevent a floor from blowing upwards due to water pressure underneath, and it is used with advantage for this purpose in works subjected to a head of water. In cases of bridges, however, there is no appreciable head of water against the work consequently inverts are, apparently not necessary. The objection to their use is the great obstruction they offer to the free passage of water, by decreasing the effective depth to the extent of nearly 3 feet.

(*e*) Reference to Fig. 1 (c) will show the disposition of the wings always a most important point. In almost all aqueducts and super-passages double sets of wings are required, *viz.*, two long curved land wings to form the connection between the masonry aqueduct and the earthen banks of the approach channel, and two water wings connecting the face of the abutments with the river banks on either side. The land wings form really a continuation of the parapet walls, and are of the same section at the top. Being subjected to hardly any earth pressure, they can be built with vertical sides of the same width throughout as the top.

(*f*) The length, as shown in Fig. 1. (c) is determind from considerations of percolation co-efficient. The land wings and water wings should be long enough to permit the distance X to be more than the gradient 1 in 5 for the head from the water level in the canal down to the bed of the drain which may be considered dry, the distance y in the bed of the canal is paved with concrete or masonry (impervious floor) to destroy the pressure of depth of water in creep. Solani aqueduct, of Ganges Canal is shown in Fig. 2.

3. Masonry Syphon (Syphon Aqueduct).

An instructive example of a syphon passing drainage underneath a canal is given in Fig. 3.

In the transverse section it will be seen that there is a fall of 3·5 feet in the syphon. This will enable a large discharge to be passed through the work, at an increased velocity, the waterway

Type Section

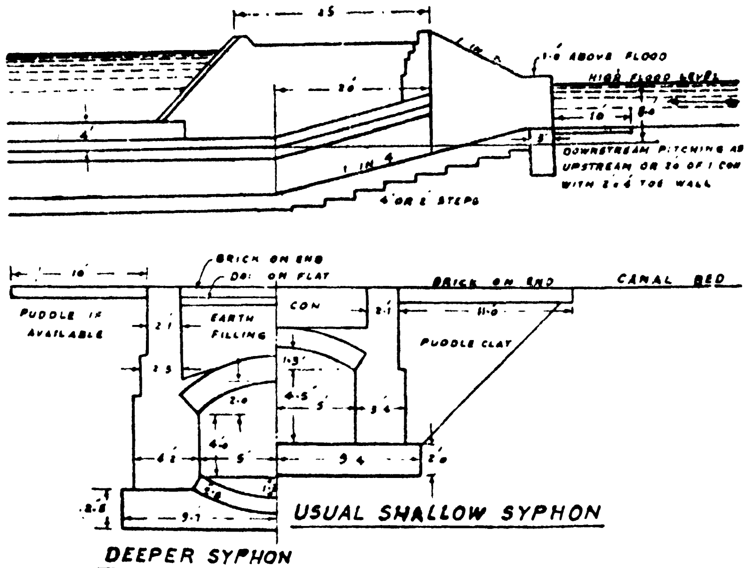


Fig. 3.

being considerably curtailed than what would otherwise be required. The spans are 9 feet, the piers 2·5 feet thick, the headway 6 feet, and the floor 3·5 feet thick. The abutment is of reasonable dimensions. The disposition of the wings is apparently good. The earth lines, so necessary to form a proper idea of the suitability of the wings, are not given.

VERTICAL WELL TYPE OF SYPHON

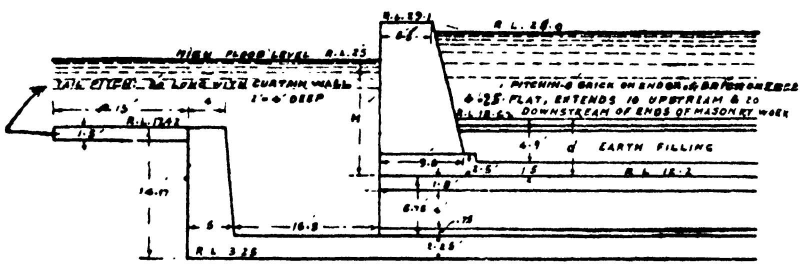


Fig. 4.

The canal is 7 feet deep and the parapets are 3 feet 6 inches wide at base and 2′-6″ at top. The roadway is formed by an outer arch-way of 6 feet in width, the piers being lengthened to carry it. This provides a roadway of 7 feet. The exit of the work has a sloping rise to the bed level of the stream, which is as it should be. The head producing upward pressure is 4 feet. The arch, with concrete covering at the crown, is 2 feet thick, and with a specific gravity of 2·0 feet will just balance this pressure. No exception can be taken to any of the details, except possibly that a thicker parapet corbelled out on both sides would provide an equally wide roadway, and thus save the lengthening of the piers and the external high level arch. A cross section of a vertical well type of syphon is shown in Fig. 4.

4. Super Passage.

Cases where the levels are such that rivers have to be taken over a canal, are comparatively rare, as they involve a very heavy work, if the stream or torrent is large, and so they are avoided so far as possible. They generally occur in the upper section of a canal, the headwork of which is situated among the lower hills high up in the course of a river. There are some very large works of this description in the old Ganges Canal. An excellent modern example is given in

Fig. 5 of the Budki Superpassage on the Sirhind Canal in the Punjab.

The length of the work is seven spans of 30 feet each, the width between parapets being 400 feet. The arches have to be raised well above the canal bed on account of the exigencies of navigation and also of the level of the bed of the torrent. The 30 feet wide spans are therefore suitable. The thickness of the arches at crown is $\cdot 45\sqrt{r}$. The thickness of the piers at springing is 6 feet or $\cdot 2S$. We have already seen that a proportion of $\sqrt{S}$ giving a thickness of 5·5 feet, is sufficient. The piers are therefore, too thick. The bottom width could be retained at 8 feet the piers having either a straight or a curved batter as has been designed. The foundations, which are probably on good soil, are admirable.

The abutment, at the look of the section, is clearly much too heavy, a very common fault among designers. To prove this actual incidence of the resultant line of pressures on the base has

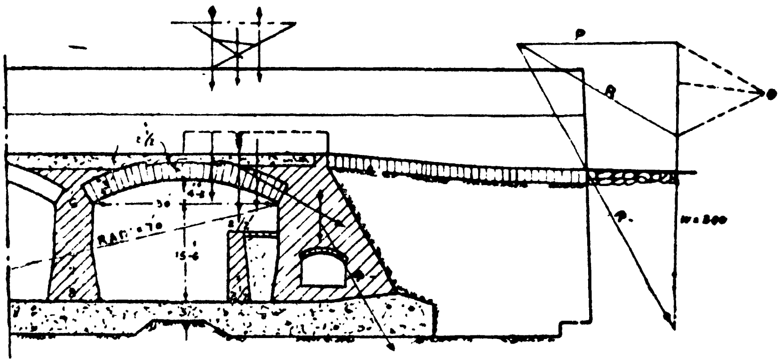
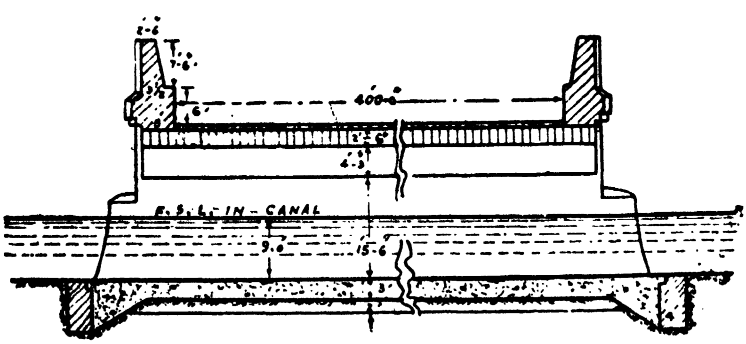


Fig. 5. (a)

been graphically found ; the method of working consists in first finding the centre of gravity of the half arch and its load of water, the latter reduced in depth, as shown by the horizontal dotted line to an equivalent mass of masonry. This process is shown in Fig. 5. (a) and the reciprocal funicular polygon above. The forces 1 and 2 are the areas of the two halves into which the half arch has been divided. Having found the centre of gravity of the half arch, a horizontal line is drawn through the centre of the arch crown to intersect the vertical through this centre of gravity, and from the point thus found the resultant line R is drawn through the centre of the arch at its springing, till it intersects a vertical line through the centre of gravity of the abutment and its water load. In the force polygon the load line, composed of the areas 1 and 2, is continued down to measure 300 square feet, the line R is then drawn from the termination of 1 and 2 parallel to its reciprocal cutting the horizontal P at a point. From this point another line, R′ joining the termination of the vertical load line. 1, 2 and 3 just obtained gives the final resultant R′. This projected on the profile of the abutment in Fig. 5. (a) from its proper starting point, *viz.*, the last intersection found cuts the base of the abutment at a point some 5 feet within its heel. As no credit has been given to the weight of the earth backing with water above it, the resultant line R′ need only just fall



within the base. This proves that the abutment is unnecessarily thick. The line P represents the horizontal thrust of the arch, and if measured, will be found to closely correspond with rt or $8 \times 30 = 240$, *i.e.*, the calculated horizontal thrust of the arch. If the abutments were made 8 feet wide at the springing and 13 feet at the base, it would probably be of sufficient section. As built, it is half as wide again as these dimensions, and besides, is provided with a large buttress of which no account has been taken. This particular analysis is due to W. G. Bligh.

It might be mentioned here that the calculations for the effect of buttresses in an abutment or retaining wall are affected as follows :—The wall should be considered as having a base equal to its normal thickness plus the length of the buttresses, but formed of two materials of different specific gravity, the solid portion being of the proper specific gravity of the material and the part behind of a lighter specific gravity, equivalent to that of a material spread over the space of the same weight as the solid buttress only. Thus, supposing a wall 6 feet thick, is provided with buttresses projecting 4 feet thick and 6 feet apart, *i.e.*, at 10 feet intervals, and let the specific gravity of the wall be 2, then the spcific gravity of the 4 feet wide space behind will be $2 \times 4/10 = 0{\cdot}8$ and the effective base width of the wall would be 10 feet, not 6 feet.

The disposition of the wings is generally similar to that usually adopted in aqueducts consisting of water wings as curved continuations of the faces of the abutments and splayed landwings which carry parapets in continuation of those in the aqueduct proper. These wings are shown in the plan over all Fig. 5. (c). As a further precaution, the ends of the land-wings are connected by a cross wall at bed level, which apparently goes down to the full depth of the foundations. The land-wings being in solid ground are stepped up in foundation, which is shown in the elevation Fig. 5.

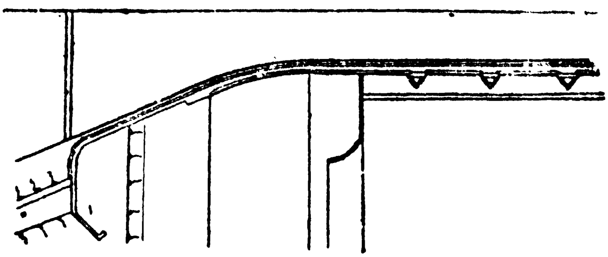


Fig. 5. (c)

Dhanaure Level crossing upstream view Ganges Canal.

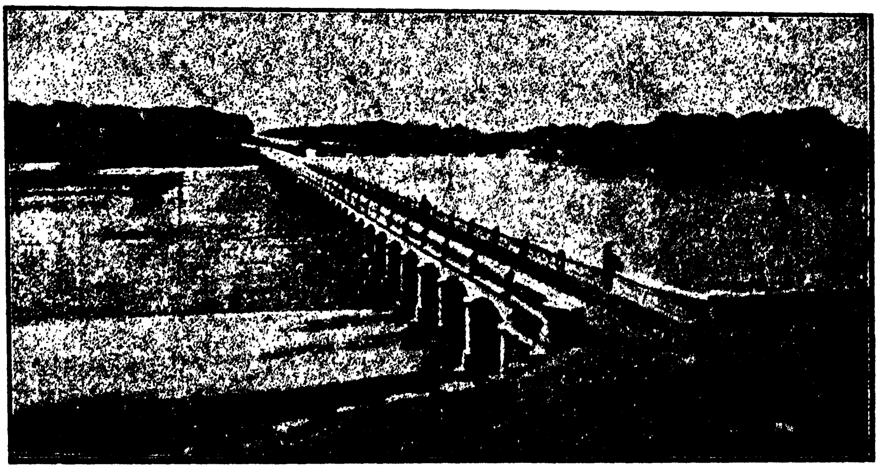


Fig. 6.

5. Level Crossing.

Dhanaure level crossing in the Upper Ganges Canal is a nice example of this type of work which is constructed when the bed levels of a drain and a canal coincide at the junction. The usual layout is shown in Fig. 6. and 7.

6. Author's Auto-Suction Weirs.

The author carried out a few experiments on the subject of cross drainage works to deal with small-seepage even-surface drains across canals. The results were published in the Indian Engineering, Calcutta, February, 1938, in an article. Three methods were suggested therein—

Solani Aqueduct, Ganges Canal.

Dhanori Level crossing Downstream view showing new protection works.

(*i*) Auto-suction Weirs. (*ii*) Syphon Spillways. (*iii*) Venturi Flume.

A brief summary of the said publication is given here.

(*a*) Our knowledge of disposal of drains across canals is limited to aqueducts, level crossings and syphons. In the aqueducts, the drain bed is required to be well above the full supply level in the canal. Level crossings are only feasible, when the drain bed and canal bed coincide. Sypnon requires that the drain bed should be sufficiently below the canal bed so that syphon barrels are not completely choked up in order to permit initial waterway for the floods in the drain. If the bed of a drain happens to be between the bed level and full supply level in the canal, none of these devices are applicable. A syphon would be completely choked up and would have no intial waterway. In a level crossing, the drain would tend to silt up the canal. Aqueducts and level crossings, are out of question when flood levels in the drains are lower than the full supply levels in the canals. Even a superpassage is not possible when a drain has got a small depth and bed level near about the bed level of the canal. The subject of drainage works across canals perhaps has not progressed beyond where the primitive hydraulicians had left it.

(*b*) Remarkable advance has no doubt been made in the design of suction hydrautomats which could lift up small drains into the canals. A suction hydrautomat requires a minimum head of at least 5·0 feet. Similarly with high heads available in the canal, the small drain could be pumped into them by the erection of compression hydrautomats or hydro-electric plants. Big falls are not generally available in the canals near the drains and all of these devices are inapplicable.

(*c*) This subject is of very great importance to the province of the Punjab where seepage-cum-surface drains are a pressing necessity to relieve waterlogging. The configuration of the country is often such that these drains are intercepted by the canals. It often happens that such drains have their bed levels above the bed of the canals and their flood levels lower than the supply levels in the canals. Pumping into the canal by oil engines has generally been tried. Pumping deals satisfactorily with seepage water, but flood water remains accumulated for a long time till it is cleared by the pumps. The gravity outfalls are not generally available except by running the drain long distances parallel to the canal where seepage from the canal itself is difficult to control.

(*d*) The disposal of small seepage-cum-surface drain across the canals with bed levels above the canal bed and flood levels below the supply level in the canal is not a subject which could be said to be of impossible solution. Such drains could easily be drawn into by the canals. In all the above mentioned three conditions, the drains should not be disturbed but the canals should be treated on the lines as described hereafter.

(*i*) **Auto-suction weirs.**

Model experiments were carried out by the writer on numerous forms of this device for drawing a natural drain into a canal. The form of the weir in Fig. 8, was found to have the maximum efficiency and comparatively low working heads. The results of two experiments were published in Appendix I of the said article. The model was 2 feet wide. The maximum depth on crest H was 1 foot. The length of the crest was 2 H. *i.e.*, 2 feet, out of which 1·5 feet was level and ·5 foot was curved with radius of 2 H. There was a drop of 0·12 foot from the crest to the point where the glacis 1 in 5 started. The slope below the orifice for the drain was tangential to the curvature of the crest. The control section of the weir, which was at a distance of H feet from the beginning of the crest, was not affected.

The drain discharged into the canal against a reduced pressure of the jet of water down the glacis on account of the pronounced effect of the centrifugal force around the curve of the crest. The design of the weir was the same in both experiments, but discharges of the drains were varied. It is clear from the results that in the first experiment the efficiency of lifting drain water into the canal is high but the working head of the weir is comparatively high with low depths on crests. In the second experiment the efficiency is about 52 percent of the depth on crest of weir which is good enough and the working head with 1′ depth on the crest is as low as 18%. If scale of the prototype be 5 times that of the model the depth on crest will be 5 feet. The working head required would be 1 foot for the weir. The floods in the drain would be drawing into the canal with level in the drain 2·5 feet lower than the full supply levels in the canals downstream of the weir. The discharge of the drain taken up by the weir would be one cusec per foot run if the height of the opening for drain in the prototype is 0·15 foot.

Auto-suction weir.

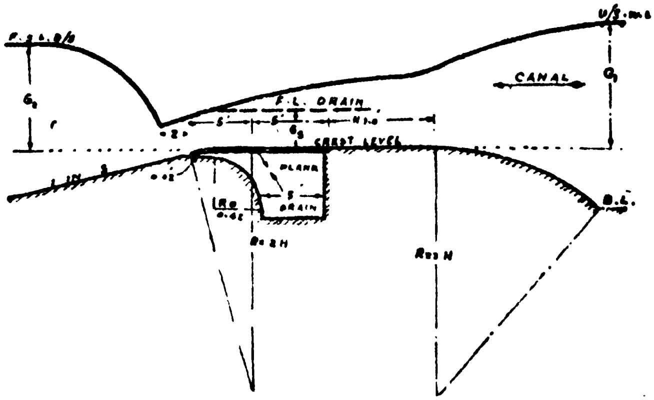


Fig. 8.

(*ii*) **Spillway syphon.**

The canal could be passed over the drain by deisgning low head spillway syphon of the form Fig. 9. which is suitable for efficient priming.

Low head spillway syphons are very suitable to pass the drains undisturbed under the canal. The spillway syphon can easily be designed with the low heads so that the pressures at the summit are equal to or greater than the vapour pressure of water. Average temperature of flowing water in the Upper Jhelum Canal has been found to be about 25 degrees C in connection with absorption experiments. The water vapour pressure for this temparature is about 13 feet head of water. The pressure at the summit of the syphon may be calculated according to the following formula :—

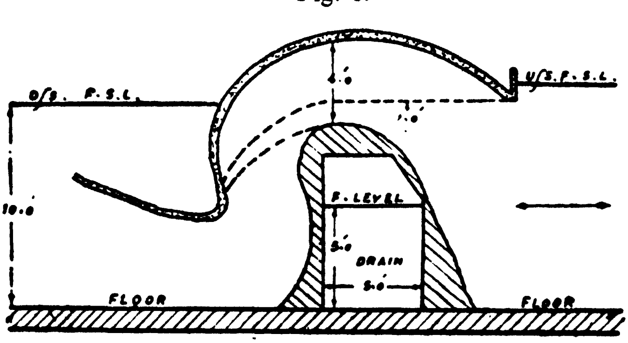


Fig. 9.

$$P=(A-H-\frac{V_s^2}{2g}-H_f)$$

P=Pressure in feet head of water.
A=Atmospheric pressure=34 feet.
H=Elevation in feet of the centre of the summit section, above water level upstream of the syphon.
V_s=Velocity at the summit of the syphon.
H_f=Losses in friction.

(*iii*) **Venturi flumes.**

The simplest solution to pass the drains over the canal is to construct a Venturi flume in the canal as in Fig. 10. Let the section of the canal be constructed to one half so that velocity of the canal is increased, say from 3·5 feet per second to 7 feet per second.

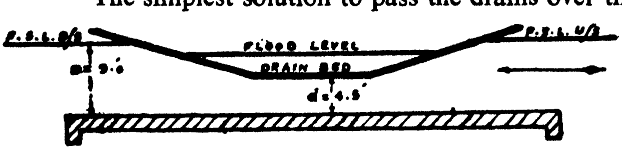


Fig. 10.

The loss of head $\frac{V_1^2}{2g}-\frac{V_2^2}{2g}=\frac{7^2}{2g}-\frac{3{\cdot}5^2}{2g}=0{\cdot}57$ feet.

The bed in the canal may not be depressed for the sake of convenient inspections.

The uplift pressures under the top slab are likely to be troublesome but they can certainly be managed at no additional cost.

(*e*) **Conclusions.**

The loss of head required in no case is more than a foot if these designs are adopted in a canal of say 4,000 cusecs discharge. Let the depth in the canal be 9 feet and the bed width 150 feet. A drain of 150 cusecs discharge could be easily disposed of by adopting any of the above mentioned designs. A fall of about a foot could generally be created because if the drain has got bed level above the bed level of the canal, the canal would be in deep digging. The main drains have hardly been constructed in this province yet. When the drains are extended to fields, there will be numerous cases where an "Auto-suction Weir," will provide a very cheap device to draw small seepage drains into the irrigation channels.

7. Example :– R. C. Trough Aqueduct.

A distributary is to cross the main channel. Design an aqueduct with the following data.

Main canal :—

Full Supply discharge	=700 cusecs.
Bed width	=60′
Bed Level	=544·5
Full Supply Level	=549·6
Slope	=1 in 6666
Main Velocity	=2·1 ft./Sec.

Distributary :—

Full Supply discharge	=409 cusecs
Bed width	=41 ft.
Bed Level	=550·7
Full supply level	=550·1
Slope	=1 in 6666
Angle of intersection	=61° 30′ 15″
Natural surface	=556·12

Branch canal flume.

Let the section be contracted to 36 ft. having 3 spans of 12 ft. with two piers of 2·3 ft. each. Distance between abutments $=36+2\times2{\cdot}3=40{\cdot}6$ ft.

Discharge per foot width $=\dfrac{700}{36}=19{\cdot}5$ cusecs.

The section of the aqueduct is less than 60 ft. as it is masonry and the co-efficient of friction is less and so the velocity is more.

By doing this, we have saved the cost by $\frac{1}{3}$

$$\text{critical depth}=D_c=\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{19{\cdot}5\times19{\cdot}5}{32{\cdot}2}}=2{\cdot}7 \text{ ft.}$$

The depth cannot be lower than this. Let depth be 5·6 ft.

Then C in the discharge formula

$Q=CB_t\,H^{3/2}$; or $C=\dfrac{Q}{B_tH^{3/2}}=\dfrac{q}{H^{3/2}}$ where q = discharge per foot width and B is 1 ft. width

$$H = \text{Head in ft.}=5{\cdot}6+\frac{V^2}{2g}=5{\cdot}6+\frac{2{\cdot}1\times2{\cdot}1}{2\times32{\cdot}2}=5{\cdot}669$$

$$\therefore\ C=\frac{19{\cdot}5}{1\times(5{\cdot}69)^{3/2}}=1{\cdot}44$$

From curve in paper No. 125 Punjab Engineering Congress. (Fane's curve Plate IV.,) $C=1{\cdot}44$ when drowning ratio $=0{\cdot}99$ ∴ Loss of head $=5{\cdot}669\times{\cdot}01={\cdot}06$ ft.

Depth has been incressed by 0·5 ft. at the flume section (5·1 ft. to 5·6 ft.). Therefore the floor will be depressed by 0·5 ft. The upstream approach in bed = 1 in 30.

The floor level is transitioned to the floor level of the downstream bed.

Discharge Flume.

Let the floor level of the distributary flume be 552·1 to ensure clearance for the branch canal. The bed is raised so that clearance of at least 1 feet is ensured, between the bottom of the slab of distributary and full supply level of the canal as it is an aqueduct.

Depth of the water $= 3 \cdot 0$ feet, Let $C = 2 \cdot 1$

Discharge per foot width $= CH^{3/2} = 2 \cdot 1 \times 3^{3/2} = 10 \cdot 91$ cusecs.

$$\text{Width required} = \frac{409}{10 \cdot 91} = 37 \cdot 5 \text{ feet}$$

Again from Fanes Curves paper No. 125 of the Punjab Engineering Congress, the drowning ratio is ·97 for $C = 2 \cdot 1$

Loss of head $= \cdot 03 \times 3 = \cdot 09$ feet, say 0·1 feet

Design of R.C. Trough of the Aqueduct shown in Fig. 11.

Calculation for the bottom slab :–

Let it be 1 feet thick

The load on the slab is that of the slab and water.

Wt. of slab 1 feet thick = 150 lbs.

Wt. of water 4′ depth $= 4 \times 62 \cdot 5 = 250$ lbs.

(for worst condition excess supply upto the top of free board)

Net load = 400 lbs /sq. feet

The slab is continuous over 3 spans of 12′ clear angle of intersection = 61°, 30′, 15″

The reinforcement is oblique and parallel to the sides.

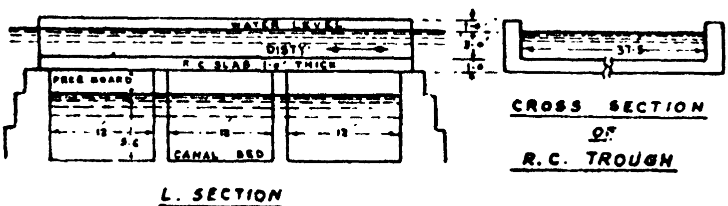


Fig. 11.

$$\text{Effective span} = \frac{12}{\sin 61°30'15''} = 13 \cdot 7 + \text{bearing of slab} = 15 \cdot 0 \text{ ft.}$$

$f_c = 750$ lbs/sq. inch ; $f_s = 18000$ lbs/sq. inch ; $n = \frac{E_s}{E_c} = 18$

$$\text{B.M.} = \frac{Wl^2}{10} = \frac{400 \times 15 \times 15}{10} = 9000 \times 12 \text{ lbs inches}$$

$$\text{Shear} = \frac{Wl}{2} = \frac{400 \times 13 \cdot 7}{2} = 2740 \text{ lbs}$$

Let d be the depth.

$$d = c\sqrt{M/b} = \cdot 085 \times \sqrt{\frac{9000 \times 12}{12}} = 8 \cdot 1 \text{ inches}$$

Let the depth be 12″ with 1·5″ covering. 10·5″ is the effective depth.

Design of Reinforcement.

$$\text{Steel} = \frac{M}{f_s\, jd} = \frac{108000}{18000 \times \cdot 878 \times 10 \cdot 5} = \cdot 65 \text{ sq. inch}$$; use 3 bars of $\frac{9}{16}$″ dia: 4″ c to c.

Bond.

$$\Sigma O = 3 \times 1 \cdot 77 = 5 \cdot 31.$$

$$U = \frac{2740}{5 \cdot 31 \times \cdot 878 \times 10 \cdot 5} = 68 \text{ lbs which is safe}$$

Temperature Steel $= A_s = p.b.d = \cdot 0025 \times 12 \times 12 = \cdot 36$ sq. inch, use 4 bars of 5/16″ dia. spacing 6″ c to c. both on top and at the bottom.

Side Walls.

They are to be designed for the water depth of 4′ including 1·5′ free board.

Overturning movement due to water pressure $= \frac{WH^3}{6} = \frac{62 \cdot 5 + 4^3}{6} = 665$ ft. lbs.

$$\text{Shear} = \frac{WH^2}{2} = \frac{62 \cdot 5 \times 4 \times 4}{2} = 500 \text{ lbs.}$$

Let d be the depth then $d = C\sqrt{\frac{M}{b}} = \cdot 085 \times \sqrt{\frac{665 \times 12}{12}} = 2 \cdot 2''$

From practical consideration it will be kept 12″ with effective width $= 10 \cdot 5''$

$A_s = \frac{M}{f_s\, jd} = \frac{665 \times 12}{18000 \times \cdot 88 \times 10 \cdot 5} = \cdot 41$; use 1/4″ bars 6″ c to c (for safety)

Temperature Steel. $A_s = p.\ b.\ d = \cdot 0025 \times 12 \times 12 = \cdot 36$ sq. inch

Use 5/16″ dia. bars 6″ c to c both inside and outside.

The side of the aqueduet is acting as a beam as the side is monolithic with the bottom slab capable of taking load so we put two bars at the top and two at the bottom to take the load of the bottom slab up to a length of 6 feet.

The bars are 5/8″ dia ; 2 number.

Expansion and contraction in Fluming.

(*a*) Let creep co-efficient be 1 in 7

Creep length required $= 7 \times 4 \cdot 4 = 30 \cdot 8$ or 30′

(*b*) We contracted the upper channel to the flume width $= 37 \cdot 5'$

Actual bed $= 41'$; and the difference $= 41 - 37 \cdot 5 = 3 \cdot 5'$

The contraction on the upstream side and expansion D.S. side be 1 in 5. Length of the *pacca* protection $= \frac{3 \cdot 5 \times 5}{2} = \frac{17 \cdot 5}{2} = 8 \cdot 75$

The flaring is designed from vertical to 1 to 1

The length of the flaring wall for depth $4 \cdot 4' = 4 \cdot 4 \times 5 = 22'$

Total length required $= 22 + 8 \cdot 75 = 30 \cdot 75'$ say $= 30'$

Difference on one side $= 1 \cdot 75'$; Flaring on each side $= 4 \cdot 4$ ft.

Total expansion and contraction $= 6 \cdot 15'$

Allowing it 1 in 5 length of pacca protection on sides $= 5 \times 6 \cdot 15 = 30 \cdot 75$ or say 30′

Sections of the walls change from vertical to sloping 1 : 1.

Percolation through Banks.

The percolation head = Distributary water level—bed of canal $= 555 \cdot 1 - 544 \cdot 5 = 10 \cdot 6$ ft.

The safe percolation co-efficient through banks is 1 in 5.

The length required $= 10 \cdot 6 \times 5 = 53 \cdot 0$ or say 55′

8. Examination Questions :—

1. Sketch a design for a brick arch syphon to carry a discharge of 40 cusecs under a disty. of bed width 6·0′, water depth 2·5′, height and width of banks 4·0 , R.L. of bed disty. 560·0′, R. L. of bed of drain 556·0′. (T. C. E. 1933)

2. (*a*) What are the different methods of disposing off cross drainage intercepted by canals ?

(*b*) Sketch a design for a syphon to carry a discharge of 100 cusecs under a disty. of bed width of 18′, discharge 100 cusecs, beds slope 1·2 ft. per mile, R.L. of bed of disty 610·0 ; R.L. bed of drain 608·0 maximum depth of water in drain = 4 ft. (T. C. E. 1934)

3. Briefly describe the various canal works required for crossing a natural drainage. (P. U. 1942)

4. An Irrigation Channel carrying 700 cusecs, with 60 ft. bed width, 5·1 ft. depth, side slope 1/2 to 1 and bed slope 1 in 6666 passes over a drain in a masonry aqueduct. Design the following if the archway has a span of 20·0 ft.

(*a*) Flumed rectaugular section for the channel with no loss of head assuming co-efficient of rugosity for brick masonry as 0·013.

(*b*) Archway for the drain.

(*c*) Masonry sides, wall and approaches for the aqueduct. (P. U. 1943)

5. Describe the points in favour and also against the following type of cross-drainage works :—

(*i*) Super-passage. (*ii*) Level crossings. (*iii*) Aqueducts. (*iv*) Syphons. For which conditions are the above type compulsory. (P. U. 1952)

6. Draw rough sketches to explain the principal features of the following cross-drainage works situated on a lined canal of capacity of 12,500 cusecs ; discharge of torrent may be taken as 2000 cusecs in each case :—

(*a*) Aqueduct. (*b*) Syphon. (*c*) Super-passage. (P. U. 1953)

7. What are the various types of cross-drainage works met with on a main canal aligned on the foot hills of a low range of hills. State which type is best used under various conditions ? (P. U. 1953)

8. Design a suitable cross drainage work for crossing of a drain with the following data :—

Canal : R. L. Bed 920 ; F. S. L. 940 ; Discharge 12000 cs ; Bed with 80·0 ft. ; Side slopes $1\frac{1}{2}$: 1
Drain : R.L. Bed 915·0 ; Highest flood level 930 ; Catchment area 10 sq. miles. Dichen's C = 1800
Bed slope 1 in 400. (P. U. 1955)

9. What are the various types of Cross-drainage works met with on a canal aligned on the foot hills. State what type is best used under various conditions. What precautions would you take in aligning a canal in order to avoid cross-drainage works as far as possible. (P. U. 1956)

10. Distinguish between Aqueduct and drainage syphon ? (P. U. 1957)

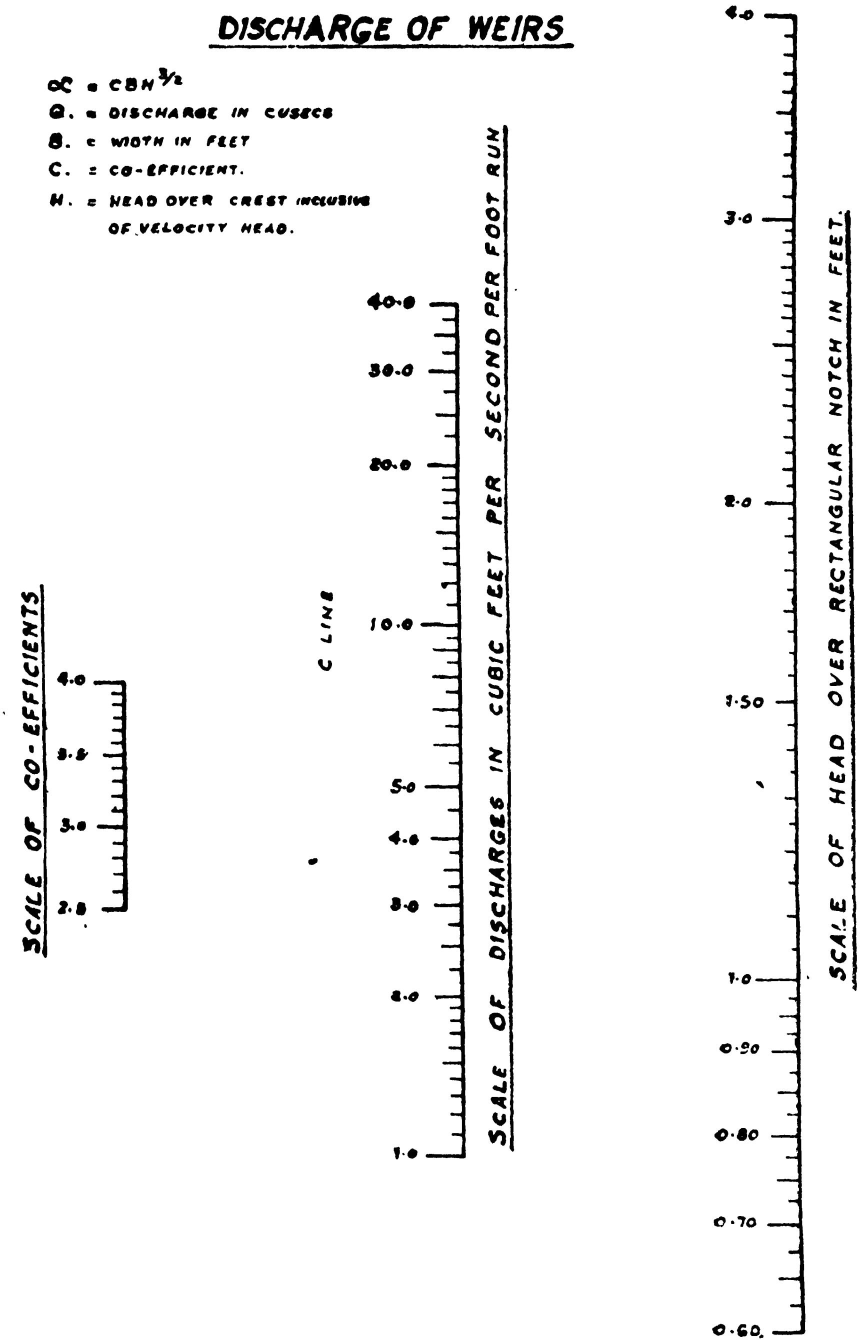

DISCHARGE OF WEIRS
Q = CBH^3/2
Q. = DISCHARGE IN CUSECS
B. = WIDTH IN FEET
C. = CO-EFFICIENT.
H. = HEAD OVER CREST INCLUSIVE OF VELOCITY HEAD.
SCALE OF CO-EFFICIENTS
4.0
3.0
C LINE
SCALE OF DISCHARGES IN CUBIC FEET PER SECOND PER FOOT RUN
40.0
30.0
20.0
10.0
5.0
3.0
1.0
SCALE OF HEAD OVER RECTANGULAR NOTCH IN FEET.
4.0
3.0
2.0
1.50
1.0
0.90
0.80
0.70
0.60

Fig. 1.

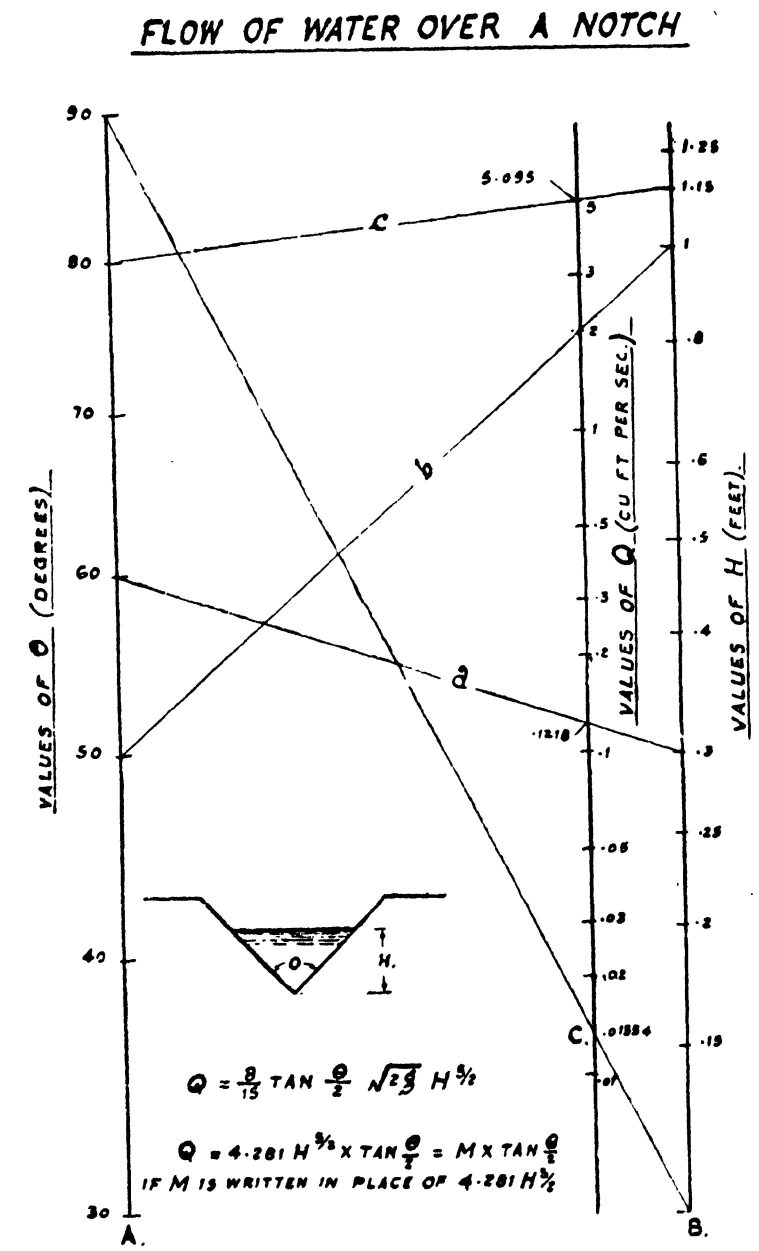

FLOW OF WATER OVER A NOTCH
VALUES OF θ (DEGREES)
VALUES OF Q (CU FT PER SEC.)
VALUES OF H (FEET)
5.095
.1218
.01884
H.
θ
Q = 8/15 TAN θ/2 √2g H^5/2
Q = 4.281 H^5/2 X TAN θ/2 = M X TAN θ/2
IF M IS WRITTEN IN PLACE OF 4.281 H^5/2
A.
B.
C.

Fig. 2.

PART II

CANAL IRRIGATION

CHAPTER X

Falls And Meter Flumes

1. Definition.

A fall may be defined as a work designed to secure the lowering of the water surface of a canal and the safe destruction of the energy so liberated. In Northern India such works are commonly named as falls but in South India and America, they ar called canal drops.

2. Necessity of Falls in Canals.

Canals are earthen irrigation channels. They require to be designed with the velocities depending on the nature of the soil of the bed and the sides so that they neither silt nor scour their beds and sides. Such a velocity is known as the critical velocity. Lacey fixes the regime slope particular for a given discharge. In every discharge formula, the bad slope is a predominant factor to determine the velocity of flow. In order that the canals run non-silting they can only be designed with the required amount of slope in the bed. If the slope of the country exceeds the slope which can be given in a canal as shown in Fig. 3 vertical falls or drops have to be given in the canal bed and the water surface.

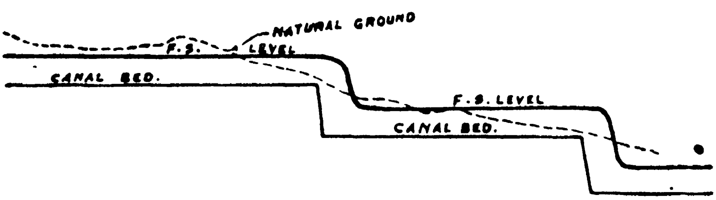


Fig. 3

3. Location of Falls.

The following factors control the selection of the suitable site for a canal fall.

(*a*) In the case of main and branch canals which do no direct irrigation, the site for the fall should be determined from considerations of economy in the digging of the canal and the making up of its banks. The most economical section of a canal is that in which the earth dug out from the bed is enough to make up the required earthwork for the banks. The depth of digging is known as the balancing depth of excavation. A suitable site for a fall would be at a place where the depth of digging drops below the balancing depth, such that the reach with digging more than the balancing depth downstream of a fall is not longer than the reach upstream of a fall with digging less than the balancing dapth.

(*b*) In the case of the distributing channels, such as distributaries and minors, the falls are located from considerations of command of the area to be irrigated. Suitable location of falls with outlets taking upstream of them results in command of extra areas.

(*c*) The combination of a bridge with a fall usully results in economy, because the floor and walls of a fall can be utilized to serve as foundations and abutments of a bridge. If there is going to be a fall near a road crossing, either the road can be diverted to the fall site or the fall may be shifted to the road site.

(*d*) Where a canal has to be bifurcated, a regulator will be required. A fall should be combined with the regulators to save the cost of the masonry works.

4. Development of the Practice of Fall Design.

When the great irrigation projects came to be constructed by the British Government in India there existed no theory of the falls to guide the designer and there were no records of the successful practice to be followed. These projects were the Western Jamuna Canal 1817, the Eastern Jamuna Canal 1823, the Ganges Canal 1842 (now Upper) and the Kistna Canal (South India) 1852.

(*a*) **Ogee Fall of Sir Probby Cantley.**

In the design of an Ogee Fall an attempt was made to avoid the destructive effect of vertical drop on the downstream floor by providing a curve from upstream bed to meet tangentially the downstream bed. The direct impact was no doubt avoided but it resulted in draw down with excesssve velocity and scour upstream, which was soon rectified by constructing the crest as shown dotted in Fig. 4. All falls on the Ganges canal are Ogee Falls. The forward velocity of water remained excessive and destructive in causing the bed and side erosion downstream which necessitated the recurring addition of loose stone pitching on the bed and on the sides Every fall possesses such protection more than a thousand feet in length.

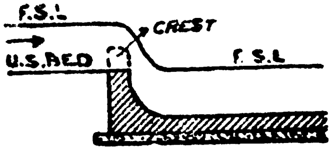


Fig. 4

(*b*) **Rapids of Lieut : Croften R.E.**

Ogee falls were followed by rapids where the drop was considerable. Rapids following the weir design are still successful falls, because they utillsed the part played by the hydraulic jump (though imperfect in destroying the energy of the fall without the designers knowing this inherent improvement in the Rapid Design). The rapids are very expensive. The western Jamuna Canal Falls were made rapids with very flat glacis sloping 1/10 to i/20 downstream to admit of timber traffic over them.

(*c*) **Vertical drop with cistern.**

Rapids were followed by falls having vertical drop with cisterns as shown in Fig. 5. The dimensions of the cisterns were put in arbitrarily in light of the experience of the designers. Another device in the form of grid was usually used in the cistern intercepting the dropping jet of water as shown in Fig. 9. Grid consisted of baulks of timber either horizontal or inclined spaced some inches apart. The grid became clogged with jungle carried by the stream and its clearance was not generally practicable except in a closure. The grid timber rotted and had to be replaced. This device has now been abandoned.

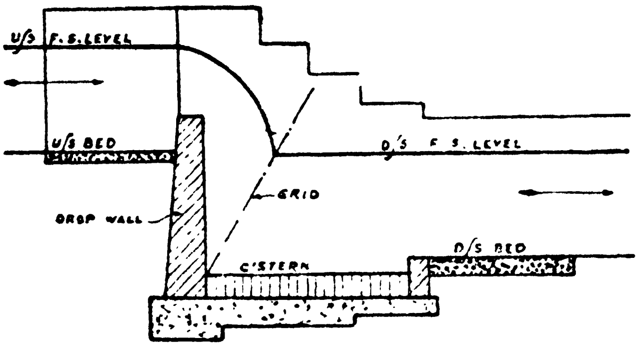


Fig. 5

(*d*) **Trapezoidal notch fall (Ried 1894).**

This design consisted of one or more trapezoidal notches in a high breast wall across a channel with smooth entrance and a flat lip projecting downstream to splay the dropping jet. The notches were calculated to give the required discharge at a Half Supply Depth and Full Supply Depth, the error in between being negligible. The discharge depth relation in the channel above the work was maintained. This relation means that the discharge in an earthen channel varies according to mean exponential formula which is expessed as $Q=k. D^{5/3}$ where Q is the discharge in cusecs, D the depth in feet and k a constant. A typical design is worked in paragraph No. 11 of this chapter and is sketched in Fig. 16. The object of the scientific design in keeping the crest at the upstream beds level was to stop silting up the upstream channel in low supplies.

The drowned falls give a lot of trouble downstream inspite of the provision of well designed cisterns while in the case of free falls there was no trouble at all. Never-the-less the trapezoidal notch fall was so sucessful that it held the field in India for many years and was copied all over the world. There was one serious defect in these falls that they could not be used as regulators in addition. In some cases grooves upstream of the notches were provided for inserting horizontal *karries* but they gave a lot of trouble by getting choked with debris.

(*e*) **Free overfall of the weir type.**

With the general improvement of irrigation practice, the more economical and accurate distribution of water became essential. Engineers found themselves obliged to measure the discharge at frequent intervals in irrigation channels. Falls had to be designed as meters of supply.

Notch falls did not admit correct calibration to give a discharge table depending on a gauge reading. Broad crestted-weirs with free overfalls supplied the necessary solution.

Further research work on brord-crested weirs proved soon that the free fall formula could be used even up to a drowning ratio of 85% to 90% when a hydraulic jump was formed downstream of the weir crest.

The efficiency of the hydraulic jump as a very potent means of destroying the energy of canal falls was brought out clearly by the research work of the Miamy Conservancy Board of Engineers in a publication by Sherman M Woodward on "Theory of Hydraulic Jump and back water curves" 1918.

This brings us to the modern designs of fall which are described in detail in this chapter.

(*f*) **Contracted falls.**

The falls of the above mentiond types were generally constructed of the full channel width. In the case of trapezoidal notch falls, the maximun construction of the width at the fall was never more than ⅛th of the bed width. But the Sutlej Valley canals in the Punjab were constructed soon after the last great war, the rates of construction were very high. and the solution for economical construction was found in cantracting or fluming the falls and combining them with bridges. The fluming was done in some cases more than half the bed width of the channel. The resulting action downstream was generally very serious and in many cases disastrous. (Paper No. 147 Punjab Engineering Congress. Lahore by E.L. Protheroe on Damage to Falls on Khadir Branch, Pakpattan Canal).

This opened up an interesting and a new field of research work indicating that there was still surplus energy even after the formation of the hydraulic jump on a glacis or the impact in a cistern which had to be destroyed further down. The development of the subject took place in two directions, firstly to invent ways and means to arrest the destructive potentiality of the high velocity jet leaving the falls near the bed in the form of baffle walls and blocks of different kinds finally devolping in the Inglis fall design, and secondly attempts were made to arrange horizontal impact in the hydraulic jump instead of a jump on a glacis leading up to latest Montagu's Fall designs. Both these designs are described in the subsequent paragraphs. The latter has proved to be the most efficient.

5. Classification of Falls.

Falls may be divided into four principal classes in accordance with the purposes of the approach.

Class I. – Falls designed to maintain the depth discharge relation.

It was believed at one time that a high-crested fall would cause silting in the channel upstream of it. This is no doubt the case when a channel is run for long periods at less than the designed full supply during the silting season. As there is great variation of the silt content over the year in the head reach of a channel, the ponding effect of a high crested fall in the head reach may have the effect of creating silt waves in the canal system below.

If the channels are run full or nil supplies as is usually done in the case of distributaries and minors, and if the period of low supplies are not protracted, modern opinion holds that the silting effect occurs only for the limited distance of the back water curve and that any such silt deposited in low supply turns will be rapidly swept out during periods of supply and equilibrium restored.

The principal types of the class are the trapezoidal notch falls and the low-crested rectangular notch falls. The trapezoidal notch falls are deseribed in detail later in this chapter. The shape of the trapezoidal notch is designed to give correct discharge at the corresponding channel depth at say, half supply and full supply. The error at intermediate discharges is negligible.

Class II—Falls designed to maintain a fixed supply level in the channel above the work.

There are usually two reasons for constructing a work of this type :—

(*a*) When a hydraulic power station is combined with a fall, the design of cuch machinery often requires a more or less fixed intake level.

(*b*) When a subsidiary channel takes off the parent some distance above work.

Such works do suffer from the defect of silting caused in the channel above the work by a raised crest fall, and the constant level is only near the work just upstream, while the levels drop off in the channel according to the back water curve in low supplies.

The principal types of the falls of this category are syphon, spillways and high crested weir falls. The syphon spillways are described in Chapter XIV of Part II. High crested weirs may be used when absolute constant level is not required, but is only necessary to maintain a high level above the work.

Class III – Falls designed to admit of variations of the surface level above the work at the will of the operator. Such falls have also to serve as regulators.

There are three methods of regulation usually resorted to ;

(*a*) Sluice gates (*b*) Horizontal stop-logs or *karries* (*c*) Vertical needles. In practice falls of this class are confined exclusively to rectangular notches.

The width of the notches depends on the size of the available gates especially when they are made of steel. Considerable economy is possible by the manufacture of such gates in standard sizes. The reduction in number of notches will increase the cost of gates. The increase in the number of notches with the corresponding reduction in the discharge intensity per foot run will result in reduced cost of the disribution of energy of the fall in the cistern.

Horizontal *karries* are usually of *deodar* wood of 5″ × 5″ section and quite strong for spans upto 10 to 12 feet. The ends are protected by iron staps. There is provided a groove at each and having metallic rod across it. The regulation staff is provided with poles with metallic hooks at one end which can fit in the grooves of the *karries* to lift them up. The water level is regulated by removing or adding *karries* according to the requirements. This is a very defective arrangment, because it is very difficult to remove the *karries* in water. Moreover, they are often choked with jungle whick has in many cases resulted in complete blockade of the falls accompanied by many breaches in the channel upstream and the outflanking of the fall ltself.

Regulation by means of vertical needles is very much superior to that by the horizontal *karries*. Their lower ends rest against an angle iron sunk in the crest of the fall and the upper end bulking against the regulating foot bridge and projects a foot or so above it. The needles can be easily pulled in and removed by a man standing on the foot bridge. Moreover, they are put near the ends of the span with free opening in the middle and, therefore, they do not choke with jungle. Where gates cannot be procured, the needle regulators should be preferred to the horizontal *kari* regulators.

Class IV – Falls under this category are desgined from special consideration without any reference to the approach conditions.

The principal types are :—

(*a*) **Cylinder Falls usually called well-falls.** They are quite suitable and economical for low discharge and high drops. Water is thrown into a well over a crest from where it escapes near its bottom. The destruction of energy is usually complete in the well.

(*b*) **Chutes or rapids.** Which conduct the stream in an open inclined trough.

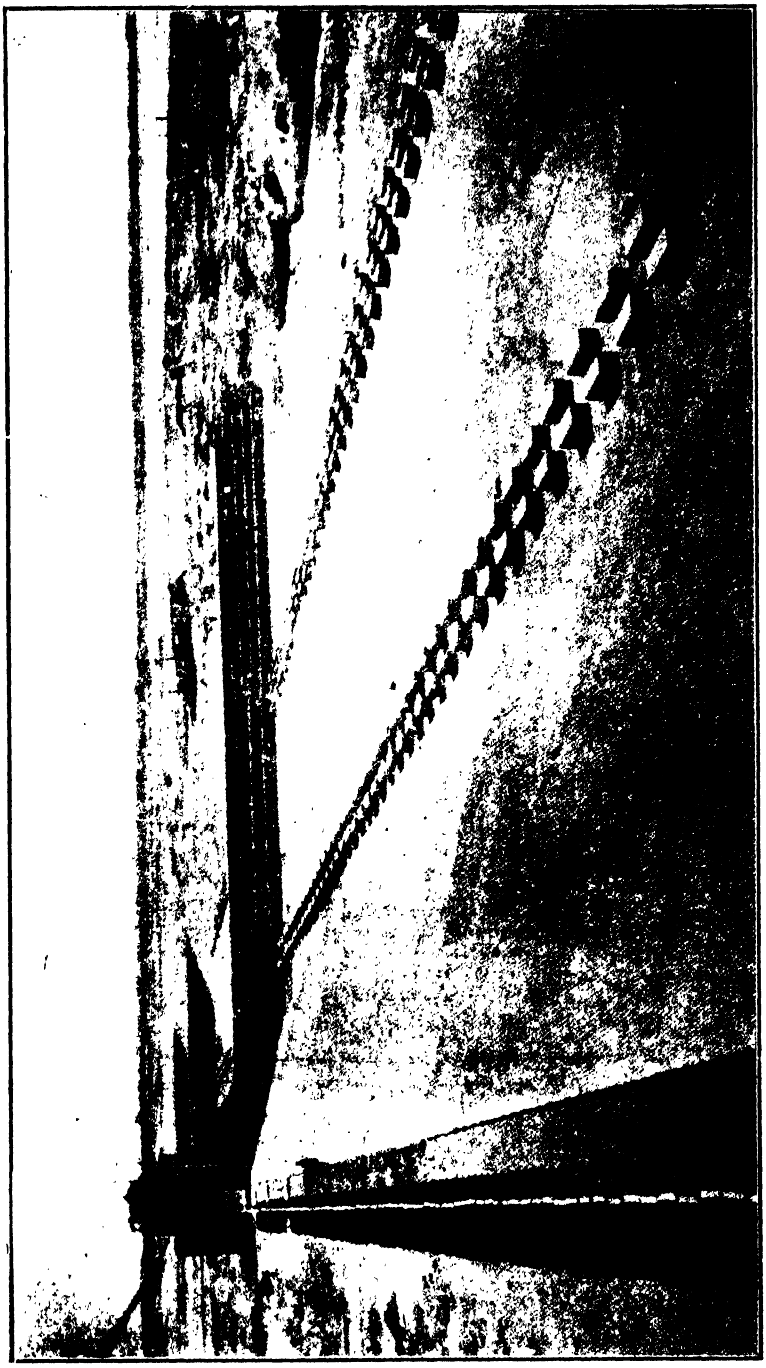
(*c*) **Pipe Falls.** Where a pipe replaces the chute.

Each of the first 3 classes of falls may be designed as.

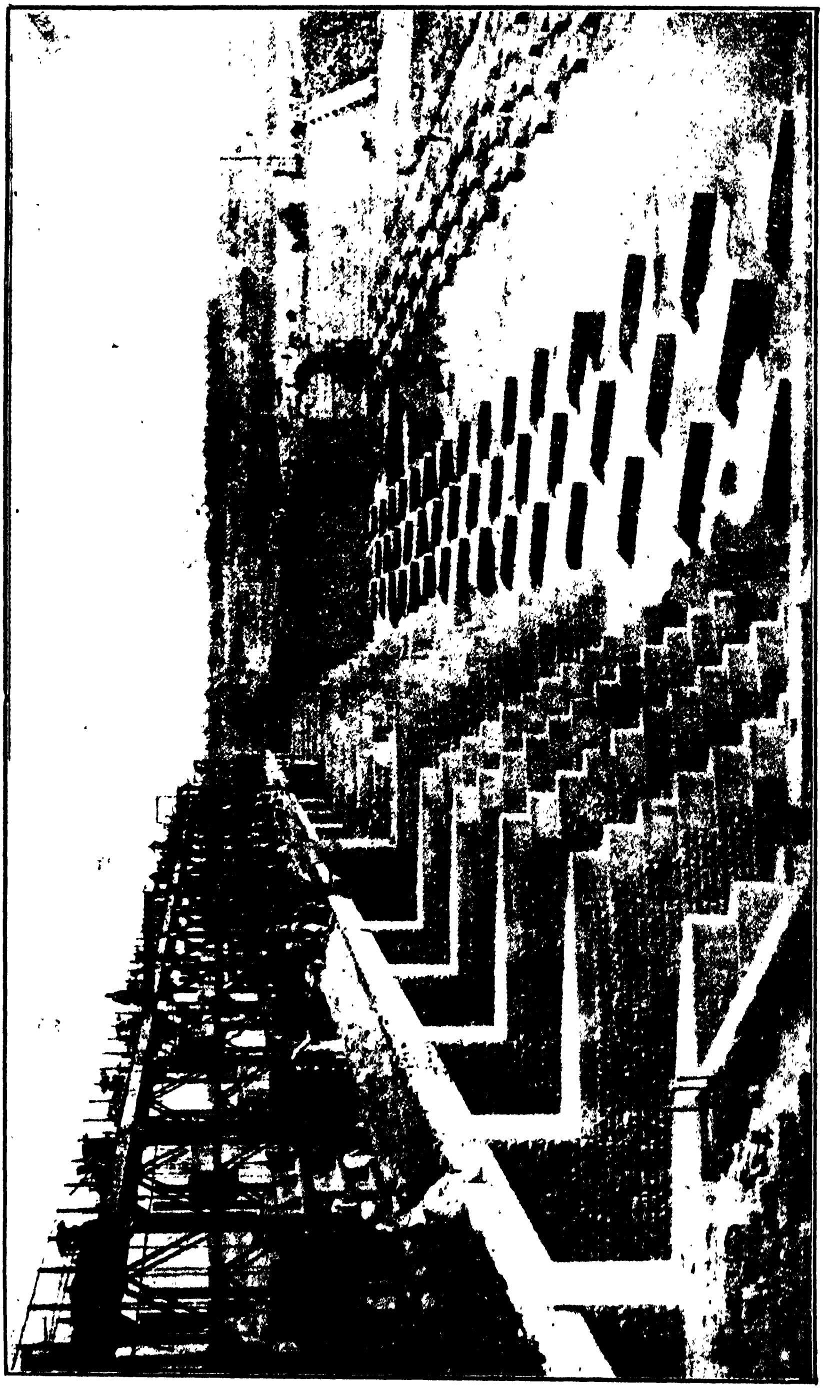
(A) Meters, (B) Non-meters.

The trapezoidal notches can be calibrated to work as meters provided the channel upstream does neither silt up nor scour. On the whole, it may be said that the trapezoidal notch fall is not a satisfactory meter.

The rectangular notch fall can satisfactorily be designed as meters. In the case of sharp-crested rectangular falls the metering is limited by the free fall condition available. Water level



Marala Weir Remodelled showing staggered blocks D. S.   Fig. 10



Protection downstream of Khanki weir showing arrangement of arrows and staggered blocks. Fig. 11

in the channel below must be lower than the crest tevel of sharp-crested rectangular notch.

It is only the broad-crested rectangular notches *viz.*, weirs which prove as successfull meters of supply. The design is described and discussed in datail later on in this chapter. They can be designed as meters with a drowning ratio of 85 to 90 percent of the depth on crest.

6. Destruction of the Energy of the Fall.

The mass of water dropping from a certain height at a fall in a canal represents the energy which must be destroyed in the fall design. The following methods are usually employed for efficient destruction of the energy thus liberated by the fall.

(*i*) **Impact.**

(*a*) Impact of falling jet over a vertical crest against the downstream floor is very efficient in destroying the energy liberated by a fall. Sometimes the dropping jet strikes against the floor at downstream bed level and often in falls with a vertical drop the downstream floor is depressed to form a cistern, which serves to add to the depth of water acting as a cushion and reduces the impact against the floor.

(*b*) Impact can also be arranged between streams two striking against each other. In a sluice regulator there may be rising and dropping gates. Water passing through the lower gates can be made to rise up meeting the dropping jet from the upper gates.

(*c*) Impact can also be arranged between the fast moving stream leaving the fall, and the slow moving stream in the channel downstream as in a hydraulic jump. Hydraulic jump is now recongnized to be the most efficient means of destroying energy liberated at falls.

(*d*) Similarly impact of hypercritical stream on a glacis downstream of a weir can be arranged against a permanent wall across the glacis known as the baffle as introduced by Inglis. Impact against staggered blocks on the glacis before formation of the jump has also been tried.

(*e*) The impact of reverse flow in a cistern has been tried with success as in the case of Colyer's Biff wall.

(*f*) Lateral impact has not yet been tried. The supply of the channel may be divided into equal parts and passed downstream from sides. High velocity jet issuing from opposite side walls should be allowed to strike against each other in a cistern or a wall.

(*ii*) **Aeration.**

The usefulness of aeration under certain conditions appears to be generally recognized by engineers in practice, but there appears to be complete absence of theoretical treatment of this subject. This has been successfully tried in America by spraying the high velocity jet downstream of the dams. A thoroughly aerated water is believed to behave as an emulsion in which stream lines cannot axist. If this be accepted, aeration provides a means of breaking of the stream line in high speed jets. The consequent destruction of energy would appear to depend on the internal impact and the work done on the wetted envelope.

(*iii*) **Super-turbulence.**

It is usually the name given to the turbulence in a hydraulic jump or that in a well designed-cistern. Formation of lot of standing waves downsteam of a drowned fall is a case of super-turbulence though inefficient in its object of energy destruction. It cannot be considered a separate means of energy destruction because it is nothing but amplification of the impact.

7. Methods of Destruction of Surplus Energy Downstream of Falls.

The various forms of impact and other methods of destruction of energy described, before aim at the destruction of the horizontal component of flow of water. Water flowing especially on an inclined floor has vertical component of flow which generates dangerous potentialities in eroding the bed and the sides of the earthen channel downstream of the falls. Energy of the fall, due to this reason, is not completely destroyed in either hydraulic jump or in impact in a cistern. There is always some surplus energy which must be destroyed before water passes on the earthen channel section. The following methods have been tried with success in practice.

(*i*) **Baffle Wall.**

A wall built transversely across the line of flow of a stream, firstly to head up water above it to such a depth that a hydraulic jump shall form and secondly to withstand the actual impact of a high speed jet of water and to dissipate energy. The height of the wall is merely a matter of judgment and sometime a serious trouble results from keeping the wall high by creating a second fall.

(*ii*) **Friction blocks or arrows.**

They are usually built on downstream floor of the falls below the glacis or the cistern. Their object is to divide the bottom high speed water laterally. They arc spaced and staggered so that there is impact between water so divided and deflected. They just serve to reduce the bottom velocity of water leaving the *pacca* downstream floor of the fall. Height of the friction blocks may be up to ¼ depth. They are spaced 1·5 to 2·0 times the height of the block. The design is shown in Fig. 6. The distance between the successive lines is equal to twice the height. Several lines may be staggered in relation to one another. The distance up to which roughening of bed is required by arrows or friction blocks is given by the following empirical formula given by Montagu.

$$L = C\frac{D_2^{3/2}\ H_l^{1/2}}{D_1}$$

where D_1 = depth of the cistern.

D_2 = depth downstream.

H_l = drop at the fall.

C = a co-efficient.

The co-efficient taken is unity for vertical impact, three for horizontal impact, four to six for inclined impact and eight for no impact.

REINFORCED CON: "FRICTIGN BLOCK"
(A) The Block (B) The Reinf.
Scale 1″ = 1 foot.
SIDE ELEVATION

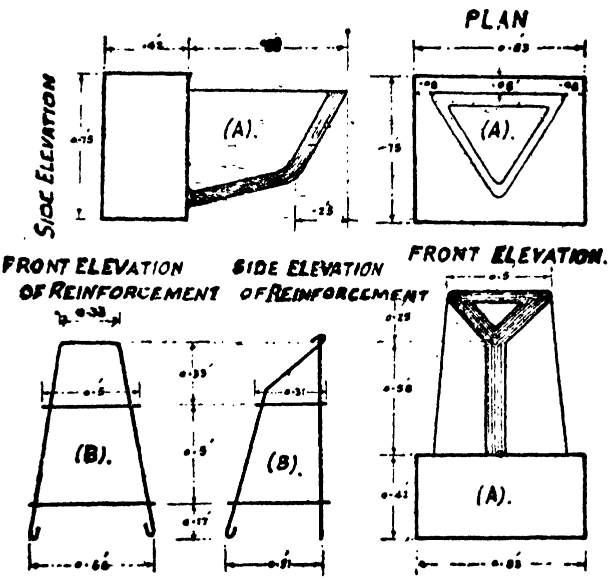


Fig. 6

(*iii*) **Dentated sill.**

A sketch of the Rehbock's dentated sill is given in Fig. 7.

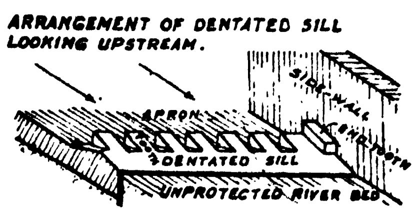


Fig. 7

The object of this sill is to deflect up the high velocity jet from near the bed. This is built at the end of *pacca* floor downstream of a fall.

(*iv*) **Deflector.** Is built on the same lines as a dentated sill. It is of uniform height unlike the dentanted sill as shown in Fig. 8. Its object is just to deflect up the high velocity jet near the bed.

Fig. 8

(*v*) **Staggered blocks.**

The staggered blocks are usually rectangular blocks as shown in Fig. 9 in plan. They are of the height not more than 1/5th of the depth. They are staggered in plan. The energy of bottom water is lost both in impact against them and in intercollision by lateral deflections. They also deflect up the high velocity jet. They are the most efficient form

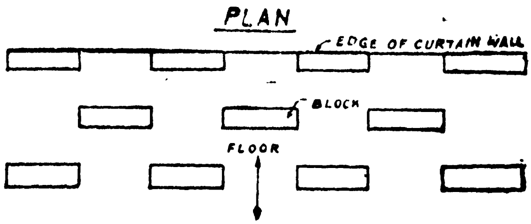


Fig. 9

of devise usually used downstream of the cistern or the hydraulic jump to destroy the surplus energy which escapes destruction in the cistern or in the jump.

(*vi*) **Cellular or ribbed pitching.**

Internal friction of water in a channel is a negligible quantity but roughening of the perimeter can successfully be developed to destroy surplus energy downstream of the fall. This has been tried in the from of Celluar to Ribbed pitching. This type of pitching is construced by putting alternate bricks on end instead of on edge as shown in Fig. 12.

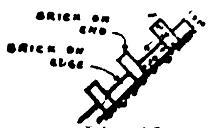


Fig. 12

(*vii*) **Biff wall.**

Colyer evolved the design of a Biff wall in the Punjab as shown in Fig. 13. This is put in at the end of the cistern. The object is to deflect back the water from cistern to create super-turbulence in it. This has been quite successful wherever it has been used.

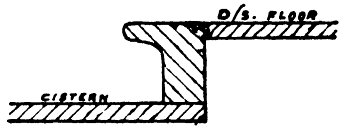


Fig. 13

8. Contracted Versus Full-width Fall.

Advantages of falls with full-width of the channel.

(*a*) Discharge intensiy is low. The energy destruction is efficient in a cistern or in a hydraulic jump energy. The cost of energy destruction devices is very low. Free falls with full width can successfully be construted without cistern and other energy destroying devices, as has been done in the case of falls constructed on the Sarda Canal in the United Provinces.

(*b*) The action in the earthen channel section is very much reduced necessitating very little elastic protection in the form of pitching.

(*c*) Such falls are most suitable for correct gauging of the depth on crest of the fall. The error due to loss of head in entry from the earthen channel to *pacca* channel between the side walls upstream of the meter is negligibly reduced.

(*d*) Wide falls with no complications of contractions and expansions serve as free fall meters up to drowning ratio as 90 percent when provided with a glacis.

Disadvantages of full channel width falls.

(*a*) Raised high crest is likely to cause silting up in the upstream channel if it runs long periods of low supplies. The silt when picked up by water in full supply or when the silt content is low as in the months of October and November, will likely cause silt movements in the channel downstream.

(*b*) When these falls are combined with bridges, they prove to be relatively more expensive than the contracted falls. Contracted falls can now safely be constructed using the modern energy-destroying devices described hitherto. They are comparatively economical when combined with bridges.

Advantages of contracted falls are stated below :—

(*a*) They serve as proportional distributors with low-set crest, causing neither afflux nor drawdown.

(*b*) They prevent excess silt entering the off-taking distributaries due to dividing the flow at the sides induced by the contracted entrance which tends to concentrate the bed silt in midstream.

(*c*) Safety of floors is increased as fluming increases the creep length.

(*d*) The cost is very much reduced especially when they have to be combined with road bridges.

9. Falls With Cistern of Glacis.

The selection of the cistern or the glacis downstream of the crest of a fall depends primarily on the prevailing levels. Theoretically there is no need of glacis when the drop available is equal to or greater than 1/3rd of the total energy depth on crest of a fall, but in actual practice the energy destruction is very imperfect in the case of falls with a cistern when the drop is about H/3 as shown in Fig. 14. below :—

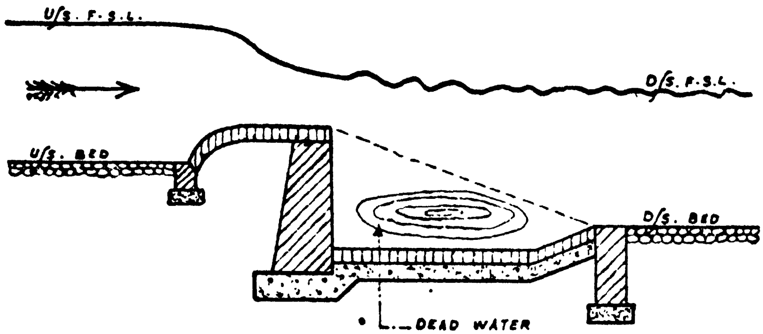


Fig. 14.

The critical jet leaving the weir crest simply skims over the cistern forming standing waves. There is neither destruction of the energy of the fall in impact in the cistern nor a hydraulic jump is formed because the critical jet is non-adherent and unstable. The cistern design should be used when the downstream water level is up to or below the crest level.

(*b*) In the case of drowned falls as shown in Fig. 14. above, the glacis design is very desirable to drop the critical depth leaving the crest to the hypercritical condition with an adherent jet which is required condition upstream of a jump.

(*c*) Glacis design is essential when the drop is not enough and it is desired to use the fall as a meter. The hydraulic jump is formed on a glacis with a fall of full channel width up to a drowning ratio of 90%, and in the case of contracted falls up to a drowning ratio of 80%. The provision of the glacis ensures the hypercritical flow downstream of the crest which does not allow the buffers of the jump to foul the control section on the weir crest.

10. Cistern Design.

Theoretical treatment to work out the dimensions of a cistern has escaped notice so far but a lot of empirical formulae proposed by the various engineers based on their experience of such works are mentioned below. The object of the cistern could be three-fold ; firstly to reduce the intensity of impact of the dropping jet against the downstream floor, secondly to provide cushion to destroy the energy of the drops, and thirdly to produce reverse flow by providing a suitable end-wall to ensure an impact in the cistern.

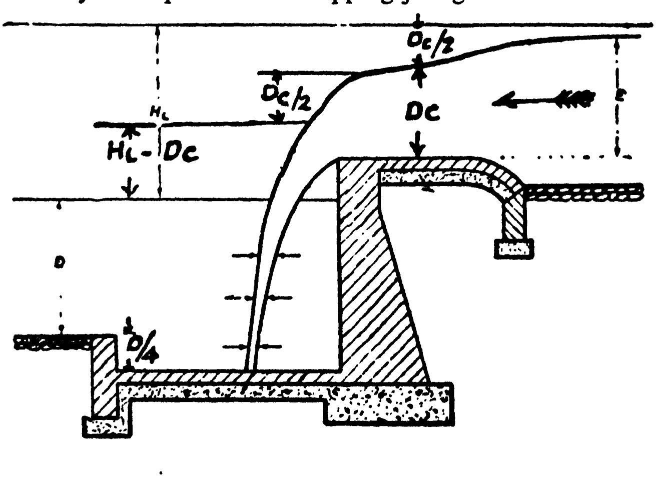


Fig. 15.

The impact against the floor depends upon the vertical drop. In Fig. 15, a fall with vertical drop is shown.

Let D/4 be the depth of the cistern.

The drop without cistern

$$=(H_l - D_c + D)$$

Drop with cistern

$$=(H_l + 1{\cdot}25D - D_c)$$

$$\text{Discharge} = g^{1/2}\,D_c^{3/2}$$
$$= 3{\cdot}09H^{3/2} \text{ cs. ft. run}$$

Impact or force against the floor is equal to the momentum destroyed per second perpendicular to the surface minus the pressure due to the depth $=M/g(V-w)D.\ a=wa.V^2/g-wD.a$

Impact per unit area $=\frac{wV^2}{g}-wD=w\left(\frac{V^2}{g}-D\right)$

Impact with no cistern $=w\left\{\frac{2g(H_l-D_c+D)}{g}-D\right\}=w\left\{2(H_l-D_c)+D\right\}$

Impact with cistern $=w\left\{\frac{2g(H_l-D_c+1\cdot25D)}{g}-1\cdot25D\right\}=w\left\{(2H_l-D_c)+1\cdot25\ D\right\}$

There is increase in impact against the floor of the cistern by an amount equal to wD/4 when the depth of the cistern is D/4.

The cushion is just the weight of volume of water displaced by the jet, as shown in Fig. 15, minus the weight of the water dropping per second. This will naturally increase if we increase depth of the cistern. If we put a bucket below a tap, water strikes against the bottom but as the depth of water in the bucket increases, the cushion increases and eventually no effect is felt at the bottom. The real advantages in a cistern for the destruction of energy is from the cushion effect and in the impact from the reverse flow while impart action against the floor increases.

(*b*) **Dimensions of a cistern.**

(*i*) Sir Proby Cautley put the depth of the cistern at one half the height of the fall without any reference to the discharge intensity.

(*ii*) Captain Dyas gives the formula.

$x=\sqrt{H_l}\times\sqrt[3]{D}$ where x = depth of cistern in feet.

H_l = drop in feet.

D = depth of water downstream in a channel.

(*iii*) Bombay Practice P.W.D. Hand Book.

$x+D=E+E^{1/3}\times H_l^{1/2}$; Length of cistern $=4/3\ E\sqrt{(H_l+D+x)}-E$

where E = total energy at crest.

(*iv*) Following is use in Bihar and Orissa, India, known as Glass formula as used by E.L. Glass, Chief Engineer.

$x+D=2\cdot25\ E^{1/2}.\ H_l^{1/3}.$

Length $=5(x+D)$

(*v*) Etchevery formula as evolved in America.

Length $=3\sqrt{H_lE}$

Depth = one sixth of its Length.

(*vi*) Montagu's formula, Punjab, India.

Depth of cistern $=\frac{1}{4}Ef_2$

When Ef_2 is the energy of flow downstream, for the discharge intensity q over the weir and the fall H_l from Plate VII, Vol. III.

Length $=l=4.Ef_2$.

The formula gives the minimum length as used in his fall design, but in the case of falls with vertical drop, it may be of the order of $6Ef_2$. In latest practice, in large falls the cistern's bed is roughened by two rows of friction blocks one at $\cdot4l$ and the other at $\cdot7l$ from the beginning of the cistern.

11. Trapezoidal Notch Falls.

(*a*) Notched Falls used to be generally constructed on canals and distributaries in the Punjab. They have many advantages. The notches are so designed that, whatever the discharge passing down the channel, the aperture of the notch is sufficient to pass that discharge at the water level of the channel at the time. The following rules were drawn up by A.G. Reid for use. (Punjab Irrigation Branch Paper No. 2, 1894.)

Notations.

l=length of cill of notch in feet ;

$n = 2 \tan \alpha$, where α = angle made by the sides of the notch with the vertical ;
$l + nx$ = width of notch at any point x above base ;
c = co-efficience of discharge = 0·78 for canal notches and 0·70 for distributary notches ;
Q = discharge in cubic feet per second.
H = depth of water in feet above cill of notch and measured to the normal surface a few feet upstream of the notch.
D = depth of water in feet for discharge Q in the reach downstream.
H_d = Height to which notch is drowned in feet.
H_l = height of fall in feet.
h_a = Head in feet due to velocily of approach = $V^2/2g$

Note : – In the following formulae Q_1 and Q_2 are the special values of Q for which the notch is calculated. Similarly H_1 and H_2 are the special values of H and, D_1 and D_2 are the special values of D corresponding to the discharge Q_1 and Q_2 respectively. Also e_1 and e_2 are the depths of drowning of the cill for upstream depths of water d_1 and d_2.

Canal notches with complete fall.

General equation is : $D = \frac{2}{3}c\sqrt{2g}(lH^{3/2} + 0{\cdot}4nH^{5/2}) = 5{\cdot}35c\,H^{3/2}(l + 0{\cdot}4\,nH)$

This expression contains two unknown quantities l and n, and for its solution two values of H with corresponding values of Q must be assumed.

Then for calculating the dimensions of canal notches, with complete or free fall, the equations are :

$$n = \frac{Q_2 H_1^{3/2} - Q_1 H_2^{3/2}}{2{\cdot}14 c H_1^{3/2}\, H_2^{3/2}(H_2 - H_1)} \qquad (1)$$

$$l = \frac{Q_1}{5{\cdot}35\ c\ H_1^{3/2}} - 0{\cdot}4\ nH_1 \qquad (2)$$

TABLE No. 1.

Table of co-efficient of formula for Discharges of Notches with complete fall.

H	$\frac{K}{l}$ $6{\cdot}0562 \times 7H_1^{5/2}$	$\frac{L}{l}$ $15{\cdot}1405 \times {\cdot}7H_1^{3/2}$	H	$\frac{K}{l}$ $6{\cdot}0562 \times {\cdot}7H_1^{5/2}$	$\frac{L}{l}$ $15{\cdot}1405 \times {\cdot}7H_1^{3/2}$
0·5	1·33447	0·26688	3·0	0·01193	0·01574
·6	0·84591	0·20302	·4	0·01107	0·01505
·7	0·57538	0·16111	·5	0·01029	0·01441
·8	0·41207	0·13189	·6	0·00959	0·013[illegible]1
·9	0·30697	0·11051	·7	0·00896	0·01326
1·0	0·23589	0·09435	·8	0·00836	0·01274
·1	0·18588	0·08178	·9	0·00785	0·01225
·2	0·14954	0·07178	4·0	0·00737	0·01179
·3	0·12242	0·06366	·1	0·00693	0·01134
·4	0·10171	0·05683	·2	0·00652	0·01096
·5	0·08560	0·05136	·3	0·00615	0·01060
·6	0·07285	0·04727	·4	0·00581	0·01022
·7	0·06260	0·04256	·5	0·00549	0·00988
·8	0·05426	0·03907	·6	0·00520	0·00956
·9	0·04740	0·03600	·7	0·00492	0·00926
2·0	0·04170	0·03336	·8	0·00467	0·00897
·1	0·03691	0·03100	·9	0·00444	0·00868
·2	0·03286	0·02892	5·0	0·00422	0·00844
·3	0·02940	0·02705	·1	0·00402	0·00820
·4	0·02644	0·02538	·2	0·00383	0·00796
·5	0·02387	0·02387	·3	0·00364	0·00773
·6	0·02164	0·02251	·4	0·00348	0·00752
·7	0·01965	0·02127	·5	0·00333	0·00732
·8	0·01793	0·02014	·6	0·00318	0·00712
·9	0·01647	0·01910	·7	0·00304	0·00693
0·3	0·01513	0·01816	·8	0·00291	0·00677
·1	0·01394	0·01729	·9	0·00279	0·00658
·2	0·01288	0·01648	6·0	0·00267	0·00642

Distributary notches with complete falls.

Take H_1 any convenient depth not less than one third or more than one half of the full supply depth in channel downstream of the notch, and let Q_1 and Q_2 be the discharges corresponding to the depths H_1 and H_2 in the channel downstream of the notch.

Then the following equations must be used :—

$$n = K\,(Q_2 - 2{\cdot}83\,Q_1) \qquad (3)$$

$$l = L\,(5{\cdot}66\,Q_1 - Q_2) \qquad (4)$$

where K and L are co-efficients the numerical values of which for different values of H_1 are given in the table.

TABLE No. 2

Table of Coefficients of formulae for Discharge of Notches with Incomplete fall

H	$\frac{M}{l}$ $6{\cdot}15\times{\cdot}7H_1^{5/2}$	$\frac{O}{l}$ $15{\cdot}38\times{\cdot}7H_1^{3/2}$	H	$\frac{M}{l}$ $6{\cdot}15\times{\cdot}7H_1^{5/2}$	$\frac{O}{l}$ $15{\cdot}38\times{\cdot}7H_1^{3/2}$
0·5	1·31407	0·26272	·3	0·01175	0·01549
·6	0·83309	0·19985	·4	0·01090	0·01482
·7	0·56661	0·15860	·5	0·01013	0·01418
·8	0·39887	0·12983	·6	0·00944	0·01360
·9	0·30229	0·10879	·7	0·00882	0·01305
1·0	0·23228	0·09288	·8	0·00825	0·01254
·1	0·18304	0·08051	·9	0·00773	0·01206
·2	0·14724	0·07066	4·0	0·00726	0·01161
·3	0·12055	0·06267	·1	0·00682	0·01119
·4	0·10016	0·05607	·2	0·00643	0·01079
·5	0·08430	0·05056	·3	0·00607	0·01041
·6	0·07173	0·04590	·4	0·00573	0·01006
·7	0·06455	0·04190	·5	0·00541	0·00973
·8	0·05344	0·03846	·6	0·00512	0·00941
·9	0·04668	0·03547	·7	0·00485	0·00912
2·0	0·04106	0·03284	·8	0·00460	0·00883
·1	0·03635	0·03055	·9	0·00437	0·00855
·2	0·03311	0·02846	5·0	0·00415	0·00830
·3	0·02895	0·02663	·1	0·00395	0·00806
·4	0·02603	0·02504	·2	0·00377	0·00783
·5	0·02351	0·02350	·3	0·00359	0·00761
·6	0·02131	0·02216	·4	0·00343	0·00740
·7	0·01939	0·02094	·5	0·00327	0·00720
·8	0·01771	0·01982	·6	0·00313	0·00701
·9	0·01622	0·01881	·7	0·00299	0·00682
3·0	0·01490	0·01788	·8	0·00287	0·00665
·1	0·01373	0·01702	·9	0·00275	0·00648
·2	0·01269	0·01626	6·0	0·00263	0·00632

Distributary notches with incomplete falls.

The following simplified formula for incomplete falls on distributaries are applicable only when the following conditions are obtained.

(*a*) The descent of the fall must not be less than one-third or more than one-half of the full supply depth of the upstream reach.

(*b*) The depth for a given discharge in the downstream reach must not be less than the depth for corresponding discharge in upstream reach.

Failing either or both of these conditions, the formulae for canal notches with incomplete falls must be used.

Let Q_1 be the discharge due to depth in the downstream channel equal to the descent of the fall and let Q_2 be the discharge due to depth of twice the descent of the fall. (The notch will thus be just free with depth $= H_l$ in the downstream reach and drowned to a depth $= H_l$ when the depth of water down-stream of it is $2H_l$).

$$n = M\,(Q_2 - 2{\cdot}5Q_1) \qquad (5)$$

$$l = O(5{\cdot}374\,Q_1 - Q_2) \qquad (6)$$

in which M and O are co-efficients, the numerical values of which for different values of H are given in table 2.

(c) Fig. 16 shows the form of notch used in the Punjab. It will be seen that the plane of the profile of the notch is set back 1·5′ from the downstream face of the notch wall. The profile itself is formed by a horizontal cill and by two equally inclined straight lines which are the traces of the crowns of two inclined surfaces of varying curvature. The limiting radii of curvature are shown in the sketch. All arcs of curvature are circular, and all centres lie in the plane of the profile. The same form may be used for large distributary notches, but in their case the plane of the profile should be set back 0·75′ only from the face wall and the unit of radius may by 0·33′ in place of 0·5′ as shown in the sketch. In all cases the upstream splay should be 45° and the downsteam splay 22°.

The height of the notches on distributaries should be in every case that of the estimated depth of full supply, and on canals it should, as a general rule, be also of that depth. There will, however, cases arise on canals in which at times the volume to be passed is conciderably greater than that due to the full supply depth which has been assumed for the purpose of calculating the profile of the notch, and in such cases the notch should be carried up to the surface level due to the maximum depth of water which will be carried by the upstream reach.

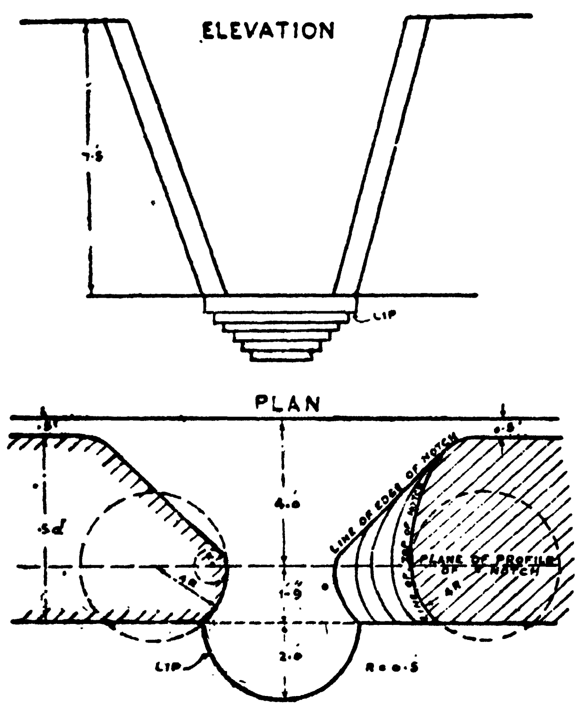


Fig. 16

The thickness of the notch wall must be sufficient to withstand the pressure to which it is subjected. Where two notches are separated by an isolated wall, it may be laid down as a rule that both faces of the wall being vertical, the thickness should not be less than half the height on canals, and 0·4 times the height on distributaries.

The lip of the notch must be corbelled out beyond the face of the crest wall of the fall. The edge of the lip should be an arc of the circle of radius such that it passes through the following three points :—

(*a*) On canal falls—At the end of the lip the two points in which the downstream splay of the notch meets the face of the notch wall ; and in the middle, a point two feet outside the face of the crest wall.

(*b*) On distributary falls—At the end of the lip the two points in which the downstream splay of the notch meets the surface of the notch wall ; and in the middle, a point one foot outside the face of the crest wall.

(*c*) **Examples**—Reid gives in his paper the following examples of the method of using these formulae.

Free Fall on Canal—Calculate the dimensions of the notches for an 8·0 feet fall on a canal under the following conditions :—

Upstream Reach

Bed width	...	...	100 feet
Inclination	...	...	1 in 6666·6
Full supply discharge	...	...	1869 cusecs

Three *rajbahas,* with an aggregate full supply of 424 cusecs to take off from this reach.

Downstream Reach.

Bed width	...	...	87 feet
Inclination	...	...	1 in 5,000
Full supply (normal)	...	...	1,445 cusecs

Minimum supply over the fall = 770 cusecs.

Assuming side slopes at $\frac{1}{2}$ to 1 the depths of full supply in the upstream and downstream reaches will be found by Higham's table to be 7·0 and 6·0 feet respectively.

We have first to consider the depth over cills of notches (H_m) at which the normal full supply of the downstream reach (1,445 cusecs) should be passed.

The depth at which a supply of 1,445 cusecs would be passed in the upstream reach is found, by Higham's tables, to be 6·0 feet, or exactly the same as in the downstream reach.

The limiting values of H_m will therefore be :—

(*i*) Depth of full supply at head of upstream reach = 6·0 feet.

(*ii*) Depth in upstream reach due to a supply of 1,445 cusecs = 6·0 feet.

If we take $H_m = 7{\cdot}0$ feet, the working of the upper reach will be perfect as long as all the *rajbahas* are running full supplies (424 cusecs), but if one or more of the *rajbahas* are closed, the notches will be very tight, and will head up the supply. If, on the other hand, we make $H_m = 6{\cdot}0$ feet or 1·0 foot less than at the head of the reach, there will be a great draw under normal conditions, *i.e.*, with all *rajbahas* open, which might cause a serious bed scour, and would lower by one foot the head available for the *rajbahas* immediately above the fall.

The value to be assumed for H_m will lie between these limits. Generally it may be said that the nearer a fall is to the head of the canal or hranch, the nearer should the value assumed approach the maximum limit. In the present case we may take an exact mean and assume $H_m = 6{\cdot}5'$.

The depth due to a supply of 1,445 cusecs is downstream reach (D_m) has been shown above to be 6·0′ and ratio of full supply depth above cills of notches to full supply depth in downstream reach is $\frac{H_m}{D_m} = \frac{6{\cdot}5}{6{\cdot}0}$

The minimum working supply is 770 cusecs, corresponding to a depth (D_0) of 4·15 feet in downstream reach, as will be found by the tables.

The supplies Q_1 and Q_2 to be taken in calculating the notch will be those due to depths D_1 and D_2 in downstream reach, and these depths may be determined thus :—

$$D_1 = D_0 + \tfrac{1}{4}(D_m - D_0) = 4{\cdot}15 + \frac{6{\cdot}0 - 4{\cdot}15}{4} = 4{\cdot}61$$

$$D_2 = D_0 + \tfrac{3}{4}(D_m - D_0) = 4{\cdot}15 + \frac{3(6{\cdot}0 - 4{\cdot}15)}{4} = 5{\cdot}54$$

The values of Q_1 and Q_2 corresponding to these depths in downstream reach may be calculated (from Higham's Tables) thus :—

Bed width = 87 feet	Incilation = 1 in 5,000
Depth $D_1 = 4{\cdot}61$	$Q_1 = 924$ cusecs
$D_2 = 5{\cdot}54$	$Q_2 = 1264$ cusecs

The corresponding depths over the notch cills H_1 and H_2 will be

$$H_1 = D_1 \frac{H_m}{D_m} = 4{\cdot}61 \times \frac{6{\cdot}5}{6{\cdot}0} = 5{\cdot}0 \text{ feet} ; \; H_2 = D_2 \frac{H_m}{D_m} = 5{\cdot}54 \times \frac{6{\cdot}5}{6{\cdot}0} = 6{\cdot}0 \text{ feet.}$$

We have therefore to calculate the dimensions of a notch which will pass 924 and 1264 cusecs, at depths over the cill 5·0 ft. and 6·0 ft. respectively.

$H_1 = 5{\cdot}0$; $Q_1 = 924$; $c = 0{\cdot}78$; $H_2 = 6{\cdot}0$; $Q_2 = 1264$.

$$n = \frac{Q_2 . H_1^{3/2} - Q_1 . H_2^{3/2}}{2{\cdot}14 c H_2^{3/2} H_1^{3/2} (H_2 - H_1)} = \frac{(1264 \times 11{\cdot}18) - (924 \times 14{\cdot}7)}{2{\cdot}14 \times 0{\cdot}78 \times 14{\cdot}7 \times 11{\cdot}18 \times (6{\cdot}0 - 5{\cdot}0)} = \frac{548{\cdot}72}{274{\cdot}47} = 2{\cdot}0 \text{ feet}$$

$$l = \frac{Q_1}{5{\cdot}35 \, c H_1^{3/2}} - 0{\cdot}4 n H_1 = \frac{924}{5{\cdot}35 \times 0{\cdot}87 \times 11{\cdot}18} - 0{\cdot}4 \times 2 \times 5{\cdot}5 = 19{\cdot}81 - 4{\cdot}0 = 15{\cdot}81.$$

The above values of n and l are for a single notch, if we design the fall with, say six notches the values for each notch will be

$$n = \frac{2{\cdot}0}{6} = \tfrac{1}{3} ; \; l = \frac{15{\cdot}81}{6} = 2{\cdot}63.$$

The width of the notch 1·0 foot above full supply or 7·5 above cill, will therefore be ⅓ of 7·5 + 2·63 = 5·13 feet and the profile of the notch will be as in. Fig. 17.

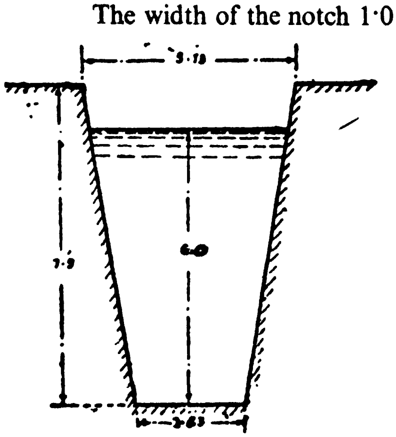


Fig. 17.

It will be found by applying the general equation $Q = 5{\cdot}35c. H^{3/2} \times (l + 0{\cdot}4 \text{ nH})$ that the full supply of 1,869 cft. of the upper reach, which may be anticipated when all upper *rajbahas* are closed, will be passed with a depth over the notch d = 7·5 feet so that the water will be just level with the top of the notch, as shown in the sketch, and there will under these extreme conditions be a heading up of 0·5 feet. If it is desirable to avoid this, or to leave a greater margin, the height of the notch walls might be reduced to that required for the normal full supply passing over them, namely 6·5 feet, and excessive supplies could pass over the whole length of the wall, but there are few circumstances in which this could be recommended.

(*d*) **Calibration of notch fall.**

Nethersol's tables. Late Sir Michael Nethersole prapared a series of tables to facilitate the calculations of the correct discharges of notches for distributry falls. Tables give the discharges through the two side triangular portions of the notch as distinct from the discharge of the rectangular portion in the centre. These tables were published by him in 1903 at the Thomason Press, Roorkee, India. The velocity of approach cannot be correctly allowed for in the calculations. Sometimes the value of the co-efficient is increased to allow for this. The calibration is nothing but approximate.

(*e*) Trapezodial notch fall is still the best type of fall in channels in which variation of supply is very much and no metering and regulation of supply is required. However, this type should be avoided when it is not a free fall.

12. Montague Type Fall.

The design of the glacis profile does not seem to have received the attention it deserves.

The whole theory of the formation of a hydraulic jump postulates an horizontal velocity and it is during the change of this horizontal velocity from the hypercritical to the subcritical stage that the destruction of energy takes place.

2. The fact that all water moving down a glacis has a vertical component of velocity seems to have been lost sight of in every publication on the subject.

It appears that the high speed jet which so often persists below a Hydraulic jump is the outcome of this vertical component of velocity, the energy of which is unaffected by the occurrence of the hydraulic jump.

3. It will be clear that a reverse slope on a glacis such that the hypercritical stream is moving horizontally, is the best solution. (Fig. 18.)

The drawback is that the Hydraulic jump is extremely sensitive on an horizontal floor, and that the level of such a floor is only correct for one ideal set of conditions. Any departure from the conditions postulated, leads to trouble which may be serious.

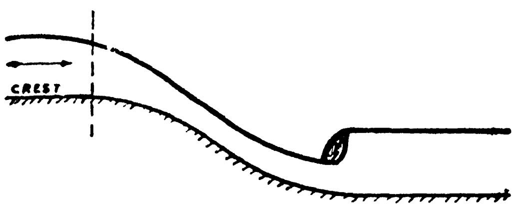


Fig. 18

But for a varying discharge, the profile of the glacis should be so designed that the maximum horizontal acceleration is imparted to the stream at all states of discharge. By imparting maximum horizontal acceleration, the maximum horizontal velocity is attained in any given length of work ; such a design is therefore conductive to economy in construction.

4. It will be clear that a stream flowing on an horizontal bed has no horizontal acceleration because the reaction is normal to the bed.

It will also be clear that a stream flowing over a parabolic bed appropriate to the ininitial horizontal velocity U, will have no horizontal acceleration because there is no reaction.

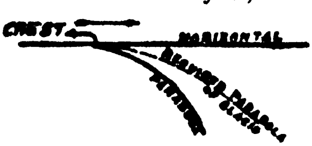


Fig. 19.

Somewhere between these two there will be a path upon which the horizontal component of the reaction imparts a maximum horizontal acceleration. It is this path which is the most efficient and economical glacis profile. (Fig. 19.)

The equation of the ideal glacis profile derived by A.M.R. Montagu is stated below and the student should refer to the original publication No. 10. Central Board of Irrigation by Montagu for its proof.

$$x=U\sqrt{\frac{4}{g}}\,.\sqrt{y}+y\ ;\ \text{where}$$

U = the intial velocity of water leaving the crest.
x = horizontal distance among the ordinate.
y = vertical distance below the horizontal.

Example. Design and sketch a fall of Montagu type for the following data—
Discharge = 207·6 cusecs ; Bed width = 31·0 feet ; F.S.D. = 3·6 ft.

$$\text{F.S.L.}=\frac{783{\cdot}94}{781{\cdot}84}\ ;\ \text{Bed level}=\frac{780{\cdot}34}{778{\cdot}34}\ ;\ \text{Fall}=2{\cdot}0\ ;\ \text{N.S.}=784{\cdot}72.$$

Calculations.

2·0 fall at R.D. 2,000 *Kokri* Distributary Sirhind Canal.
Data. Q = 207·6 cusecs ; B = 31·0 feet ; D = 3·6 feet.

$$\text{Mean velocity}=\frac{207{\cdot}6}{(31+3{\cdot}6/2)3{\cdot}6}=1{\cdot}761\ \text{ft. per second.}$$

$$\text{F.S.L.}=\frac{783{\cdot}94}{781{\cdot}84}\ ;\ \text{B.L.}=\frac{780{\cdot}34}{778{\cdot}34}\ ;\ \text{Fall } 2{\cdot}0\ \text{feet}\ ;\ \text{N.S.}=784{\cdot}72.$$

(*i*) Flume :— $h_a=\dfrac{V^2}{2g}=\dfrac{1{\cdot}76^2}{64{\cdot}4}={\cdot}05$ ft.

F.S.D. Downstream = 3·6 feet ; $Ef_2=3{\cdot}6+{\cdot}05=3{\cdot}65$ feet.
From plate XXII, Vol. III for $H_l=2{\cdot}0$ and $Ef_2=3{\cdot}65$, read $q=11{\cdot}6$ cs.

$$B_t=\frac{207{\cdot}6}{11{\cdot}6}=17{\cdot}89,\ \text{Say } 18{\cdot}0\ \text{feet.}$$

(*ii*) Crest.
$Q=207{\cdot}6$; $D_c=1{\cdot}6$ ft. (critical depth) ; $B=18{\cdot}0$ and $q=11{\cdot}4$ cs.
$Ef_1=1{\cdot}6\times 3/2 =2{\cdot}4$ feet.
R.L. of crest$=783{\cdot}99-2{\cdot}4=781{\cdot}59$; Length of crest$=2{\cdot}5$
$H=2{\cdot}5\times 2{\cdot}4=6{\cdot}0$ feet.
(*iii*) Approach.
Side expansion each side upstream$=(31-18)\frac{1}{2}=6{\cdot}5$ feet.
Expansion 2 to 1 length$=2\times 6{\cdot}5=13{\cdot}0$ feet.

5. Cistern.

Depth$=\frac{1}{2}Ef_2=\frac{3{\cdot}65}{2}=1{\cdot}83$ feet ; R.L. of cistern bed$=778{\cdot}34-1{\cdot}83=776{\cdot}51$
Length of cistern$=4\times$depth$=4\times 1{\cdot}83=7{\cdot}3$ ft.

6. Glacis.

$$x=U\sqrt{\frac{4}{g}}\times\sqrt{y}+y \qquad (1)$$

$q=\frac{207{\cdot}57}{18}=11{\cdot}53$ cusecs ; $D_c=1{\cdot}6$ ft ; $U=\frac{q}{D_c}=\frac{11{\cdot}53}{1{\cdot}6}=7{\cdot}2$ ft./sec.
Substituting value of U in formula (1) above
$x=7{\cdot}2\times{\cdot}3524\times\sqrt{y}+y=2{\cdot}54\sqrt{y}+y$
R.L. of crest$=781{\cdot}59$ and R.L. of cistern floor$=776{\cdot}51$; Difference$=5{\cdot}08$ feet.

y	$\sqrt{y}$	$2{\cdot}54\sqrt{y}$	x
1	1·0	2·54	3·54
2	1·4142	3·59	5·59
3	1·7321	4·4	7·4
4	2·0	5·08	9·08
5·08	2·252	5·72	10·8

Horizontal length of glacis is 10·8 feet.

7. Departure Downstream.

Departure on each side$=\frac{1}{2}(31-18)=6{\cdot}5$ ft. ; Splay 3 to 1.
Length$=6{\cdot}5\times 3=19{\cdot}5$ ft.

8. Exit Gradient.

For minor works founded on clay soil, the exit gradient may be 1 in 3 to 1 in 5

Exit Gradient $G_e=\frac{H}{d}\,\frac{1}{\pi\sqrt{\lambda}}$ vide Plate III, Vol. III ; where,

d=depth of cistern wall at the end of *pacca* work$=2{\cdot}15$ feet.

$$\alpha=\frac{b}{d}=\frac{56}{2{\cdot}15}=26{\cdot}5$$

Head=H=worst when water level is upto crest and downstream bed dry$=781{\cdot}59-778{\cdot}34=3{\cdot}25$ feet

$\therefore\ \frac{1}{\pi\sqrt{\lambda}}={\cdot}086$; From plate III, Vol. III ; $G_e=\frac{3{\cdot}25}{2{\cdot}15}\times{\cdot}086={\cdot}13$ which is safe.

9. Upstream Pressure.

(1) Upstream curtain wall.

$d=1{\cdot}4$ feet, $b=56{\cdot}0'$, $\alpha={\cdot}025$

$\phi_c=85\%$ from Plate II

Corrections of thickness ; total drop$=15\%$

$\phi_c=\frac{15\times 40}{100}=4{\cdot}5\%$

Proportional to thickness$=\frac{4{\cdot}5\times{\cdot}9}{1{\cdot}4}=2{\cdot}86\%$

ϕ_c corrected $= 85+2{\cdot}86+{\cdot}6$

$= 88{\cdot}46\%$

Interference of Glacis toe wall.

$= 19\sqrt{\frac{4{\cdot}33}{29{\cdot}8}} \times \frac{4{\cdot}83}{56} = 0{\cdot}6\%$

(2) Glacis toe wall. d$=1{\cdot}4$; b$=56'$. Correction for slope of Glacis.

$\frac{b_1}{b} = \frac{29{\cdot}3}{56} = {\cdot}53$; $\alpha = \frac{56}{1{\cdot}4} = 40$;

For slope 1 in 2 read from Fig. 14, Chapter V.

$\phi_c = 48\%$; $\phi_d = 48\%$; $\phi_e = 48\%$.

$= \frac{10{\cdot}8}{29{\cdot}1} \times 6{\cdot}3 = 2{\cdot}3\%$

Corrected $\phi_c = 48\%$

(3) Cocrected $\phi_e = \phi_d = 48+2{\cdot}3 = 50{\cdot}3\%$

Downstream curtain wall.

d$=2{\cdot}15$ feet

$\frac{b_1}{b} = 0$

$\frac{1}{\alpha} = \frac{2{\cdot}15}{56} = {\cdot}0384$

$\phi_e = 18\%$

Corrections

Thickness of floor ; drop$=18\%$

$\phi_e = 18 \times \frac{30}{100} = 5{\cdot}4\%$

Proportional to thickness

$= 5{\cdot}4 \times \frac{{\cdot}9}{2{\cdot}15} = 2{\cdot}27\%$

Interference of glacis toe wall

$19\sqrt{\frac{2{\cdot}33}{25}} \times \frac{3{\cdot}58}{56} = {\cdot}37\%$

Corrected $\phi_e = 18 - 2{\cdot}27 - {\cdot}37 = 15{\cdot}36\%$

Thickness of floor.

Soil below the floor is sandy mixed with *kankar* and therefore pervious. Worst H$=3{\cdot}25$ ft.

Table 3.

Description.	Percentage.	Full seepage flow pressure	Thickness of floor calculated	Thickness provided
Under cistern	50·3%	1·64	1·64	1·9
10·25 from toe of glacis		1·4	1·4	1·4
19·8 do		0·9	0·9	1·15

10. Thickness of the Glacis.

The thickness of the downstream glacis has been designed for the worst conditions of the trough of the hydraulic jump and it occurs when the supply in the distridutary is minimum.

Area of trough at the jump$= \frac{8\times4}{2} = 16$ sft. per foot width.

Area of *pacca* floor under trough of the jump$=8{\cdot}5\times$average thickness$=8{\cdot}5\times1{\cdot}9=16{\cdot}15$ sft. per foot width. Hence thickness is sufficient.

11. Curtain Walls.

Upstream curtain wall$=1/3$; F.S.D.$=1/3\times3{\cdot}6=1{\cdot}2$ feet ; thickness$=1{\cdot}4$ feet.

Downstream curtain wall$=\frac{1}{2}$ of downstream depth$=\frac{3{\cdot}6}{2}=1{\cdot}8$ feet.

Thickness provided at the axis of the channel$=2{\cdot}15$ feet as the bed is bowed, as per Central Board of Irrigation publication No. 6, Fluming, Page 10.

12. Bowing of Floor Downstream of Cistern.

It is raised up 1 in 10 to R.L. 777·57 permitting a bowing of B/30 to B/40 below the designed bed. End curtain wall is also bowed. Let it be B/30 in this case = 1·0 feet

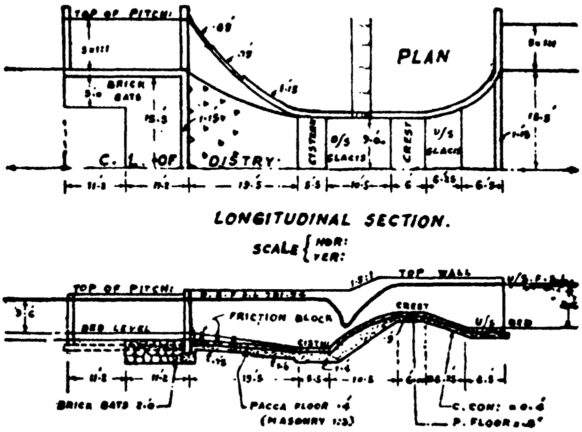


Fig. 20

Downstream bed protection. 2 ft. thick loose bats far a length of 4 to D, 3·6 × 54 = 14·4 say 15′ bed and 10′ on sides in bed.

Roughening of the cistern bed. Cistern bed is roughened by two rows of friction blocks.

Side walls. Side walls are flared from the vertical at the end of the crest to a slope 1 in 1 at the end of the cistern Fig. 20.

13. Inglis Type Fall.

The inventor of this type of contracted fall (C.C. Inglis M.I.C.E, Director Research Station, Bombay) described the essential features of his design in paper No. 44. Technical Publication P.W.D. Bombay and paper No : 170 Punjab Engineering Congress 1933. A lucid description of the necessary points is given in Appendix V of the Central Board of Irrigation Publication No : 10. A summary is presented here.

(*a*) **Notations.**

Q = discharge in cft. per second (cusecs).

q = Discharge per foot width of a channel or flume.

q_o = Discharge per foot run of overall throat width of flume *i.e.*, including piers.

V = velocity in ft. per second at any selected point.

d_1 = Depth of water in upstream section of a channel (*e.g.*, above a fall or contraction).

d_2 = Depth of water on baffle pavement at toe of fall (assuming no standing wave has formed).

d_3 = Depth of the water in the downstream section of a channel or flume (*e.g.*, below standing wave.)

d_x = Depth of water downstream of standing wave above baffle pavement level in a parallel sided channel.

d_y = Depth of water downstream of standing wave above baffle pavement level in expanding flume.

d_z = Water depth in cistern, downstream of the baffle pavement.

D_1 = Depth of water upstream, above cill level.

D = Effective depth of water above cill upstream $= D_1 + ch_x$

h_v = Head causing velocity

H = Fall in water level *i.e.*, difference of water levels of upstream and downstream of fall (below standing wave).

H_x = Fall in water levels with parallel sides.

H_y = Fall in water levels with expanding sides.

h_2 = Difference of level of water upstream of and on baffle pavement at the toe of the fall, assuming no standing wave *i.e.*, the effective head for downstream velocity V_2

h_b = Height of baffle

B = Bed width of a channel

B_2 = Bed width of flume at the contracted section (excluding piers).

B_o = Overall bed width of flume at the contrated section including piers.

L_b = Distance of the upstream face of baffle from toe of fall.

(*b*) **Description of the Design.**

A section and a plan of the fall is given in Fig. 21

Design consists of ;

(*i*) A standard long throated weir flume followed by a glacis slope and a pavement on which a baffle is fixed to dissipate energy. A cistern downstream of the baffle with a deflector at the downstream end of the cistern is provided. *Pacca* pavement ends at the deflector. There is a second cistern downstream of the deflector the section of which need only be pitched.

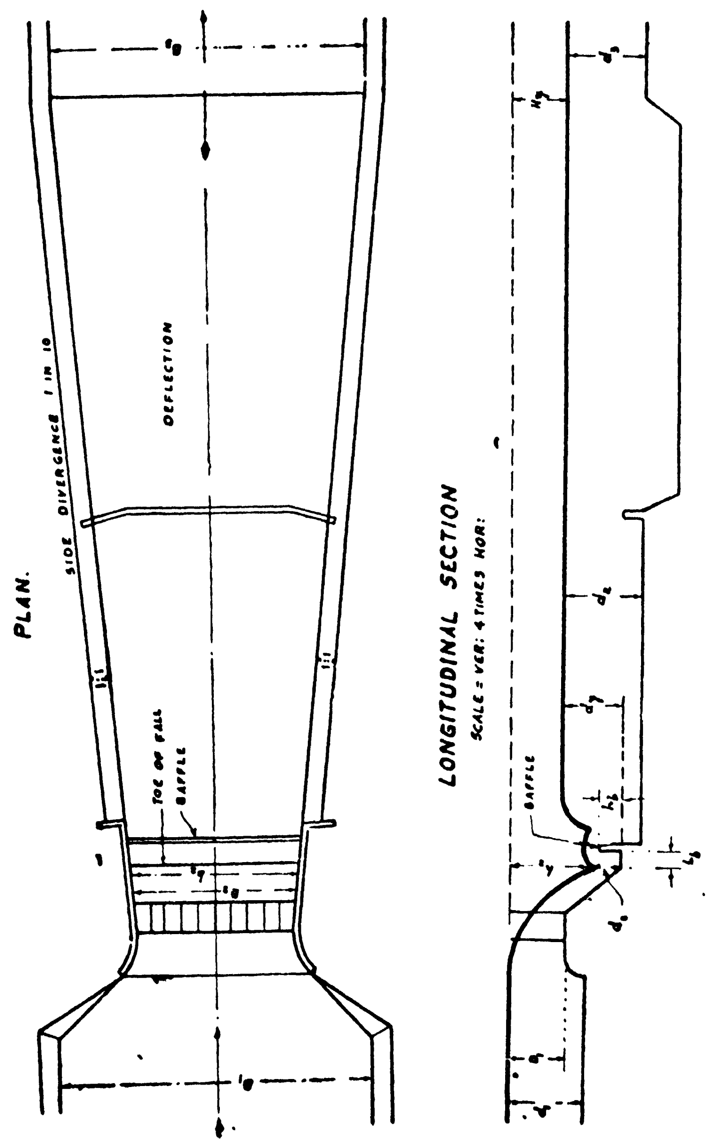


Fig. 21

(*i*) At the toe of the glacis, very high velocities are generated, and the energy in excess of that required for normal flow has to be dissipated. Under such conditions energy is usually dissipated by means of a hydraulic jump which forms at the point where there is a balance between the total energy due to pressure plus the rate of change of momentum of the high velocity flow and that of the low velocity flow in the downstream channel.

(*ii*) Though a hydraulic jump dissipates a considerable amount of excess energy in the form of heat, yet residual eddies and turbulence persist and the distribution of velocities downstream of a fall does not approximate to the distribution in a normal channel. It follows that when the materials of the channel bed are erodable, heavy scour results, and this is accentuated where the sides diverge sharply below a fall, because cork screw eddies are then generated. Where the height of fall exceeds 1/3rd the depth of the channel, conditions are markedly improved by adding a properly designed baffle.

(*c*) The maximum dissipation of energy by a hydraulic jump occurs when the jump forms at the toe of a glacis. In practice, if there is a sloping glacis, the hydraulic jump may form.

(*i*) On the glacis, (*ii*) at the toe, or (*iii*) downstream of the toe.

(*i*) If the hydraulic jump would form on the glacis, where there is no baffle ; *i.e.*, if d_3 ; the normal depth of water in the channel downstream ; would be greater than d_x (or d_y) ; the optimum depth, the baffle should be fixed on a platform at a higher tevel than the canal bed level, so that the natural wave would then form at the toe on the platform.

(*ii*) If the hydraulic jump would form at the toe of the fall *i.e.*, when $d_3 = d_x$ (or d_y); the baffle should be fixed at pavement level.

(*iii*) If the hydraulic jump would form downstream of the toe, *i.e.*, if d_3 would be less than d_r or (d_y) the glacis should be extended and a cistern provided of such depth as to bring the wave to the toe and the baffle fixed on the bed of the cistern.

Plate XV enables us to determine the level of the platform on which the baffle should be fixed for an overall discharge of q_o per foot run between abutments for a fall H_r feet when the channel has parallel sides and when the sides diverge. The height of fall H_y can be turned into equivalent height for parallel sides H_r; vide Plate XV and knowing H_r we can find d_r from Plate XV and as $H_y+d_y=H_r+d_r$, we thus get d_y, the depth of the baffle pavement below the water level downstream of the fall.

The height of the baffle should be equal to $1{\cdot}3d_2$; d_2 being given for any q_o and H_r on graph of Plate XV and the distance of the baffle from the toe of the fall $=5{\cdot}25\ d_2$.

It is important and interesting to note that as the position of the standing wave depends only on the discharge per foot run and the head available, it is immaterial whether there are piers or gates in the throat, provided the piers do not cause afflux and the co-efficient of gates is unity ; but if the co-efficient is less than unity, than the head available is the difference of head upstream and downstream (H_g) less the energy head destroyed by the gates or piers ; which can be calculated.

The baffle ceases to be effective when D_3/D_1 exceeds about 0·60, because then it begins to become "drowned".

The graphs shown in Plate XV, Vol. III, allow for normal (10%) losses due to friction and the graphs in plate XV allow for the part of the pressure of standing wave which is balanced by the side walls.

(*d*) Even though the energy is effectivly dissipated by baffle near the toe of a fall, the distribution of velocities is not normal, hence a cistern and deflector are provided. At the point were the downstream flume width is equal to 3/4 bed width of the channel, the depth of water which is necessary to give the mean velocity of $V=c\times 0{\cdot}84d^{0{\cdot}64}$ is calculated. The difference between this depth and the normal depth downstream gives the amount by which the pavement is sunk below the downstream bed level. Plate XV shows the values of L in terms of B_o for various values of B_3/B_o and divergence of 1 in 4 to 1 in 10. A deflector is fixed at the end of the cistern *i.e.*, at a distance L from where divergence starts, the height of which is 1·12 of (F.S.D. +depth of the cistern below the diagonal canal bed level downstream).

(*e*) **Side divergence.**

The side divergences should not be sharper than those shown in Plate XV otherwise a return flow occurs. A cistern with a semi-elliptical cross section gives a much better distribution of velocities than a cistern with a flat bed. The side slope deflectors are beneficial by causing the bed roller to form over the full width, thus preventing of lock screw eddies.

(*f*) **Scour downstream of bed deflector.**

Experiments have shown that where a deflector Fig. 22 was fixed, the material tended to bank up behind the deflector to a slope of about 1 in 3 to 1 in 4 and for the deflector to be fully effective this scour should be allowed to occur because flat pitching prevents the formation of the beneficial horizontal bed roller. Protection should be laid to the natural slope of 1 in 3.

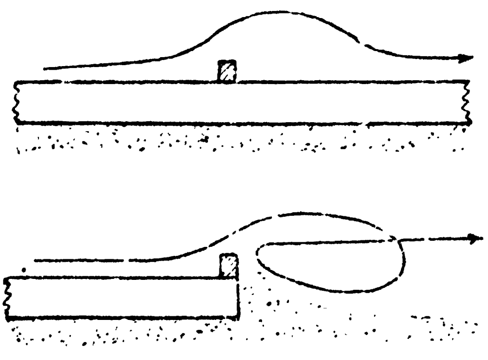


Fig. 22

(*g*) **Examples.**

Design an Inglis Type flumed fall in a channel with the following data :—

Discharge = 700 cusecs ; Drop = 6·63 feet

Bed width = 60 feet

Depth 5·1 ft. F.S.L. $= \dfrac{\text{upstream}}{\text{downstream}} = \dfrac{627{\cdot}69}{621{\cdot}06}$;

Bed levels $= \dfrac{622{\cdot}59}{615{\cdot}96}$; Mean velocity = 2·19 ft./sec.

N.S. = 624·0 ; Side slope $\frac{1}{2}$ to 1.

(*i*) **Design of flume :—**

Area of the channel section $=5{\cdot}1\left(60+\frac{5{\cdot}1}{2}\right)=319$ sft.

$P_w=60+2\times5{\cdot}1\sqrt{1+{\cdot}5^2}=71{\cdot}4$ ft. ; $R=\frac{A}{P_w}=\frac{319}{71{\cdot}4}=4{\cdot}47$ ft.

f_l (from $V=1{\cdot}15\sqrt{f_lR}$) $=0{\cdot}81$

Height of the hump in throat $=\frac{d_3}{8}=\frac{5{\cdot}1}{8}={\cdot}64$ ft.

$D_1=\frac{7}{8}\,d_3=\frac{7}{8}\times5{\cdot}1=4{\cdot}46$ feet.
d_1 = height of the hump $+D_1={\cdot}64+4{\cdot}46=5{\cdot}1$ ft.
V_1 = velocity of approach $=2{\cdot}19$ feet per second.

h_v = head due to velocity of approach $=\frac{V_1^2}{50}={\cdot}086$

Note :—Instead of 2g, C.C. Inglis adopted 50 to calculate h_v.
$D=D_1+h_v=4{\cdot}46+{\cdot}09=4{\cdot}55$ ft.
$Q=3{\cdot}09\,BD^{1{\cdot}5}=700$ ∴ $B=23{\cdot}4$ ft. say $24{\cdot}0$ ft.

It is desirable in practice to design the throat in an exact multiple of feet so that the span width is about $1{\cdot}5\,D_1$; let there be 3 spans with two piers of $2{\cdot}5$ ft. thick each, the discharge co-efficient reduces in that case by $1{\cdot}5\%$.

∴ The new $D=\left(\frac{700}{24\times3{\cdot}01}\right)^{2/3}=4{\cdot}52$ feet.

This will give $D_1=4{\cdot}43$ and the height of the hump $={\cdot}67$

(*ii*) **Bell mouth upstream approach.**
Radius $=2D^{1{\cdot}5}=2\times4{\cdot}52^{1{\cdot}5}=19{\cdot}2$ feet.

The curve should subtend an angle of 60° and then continue tangentially till the wall penetrated the side slopes above F.S.L.

The sill of throat may be formed to upstream bed level by a sloping pavement.

(*iii*) **Position of gauge chamber.**
The position of the upstream gauge chamber from the end upstream of the throat $=4D^{1{\cdot}5}$ $=38{\cdot}4$ feet say $38{\cdot}0$ ft.

(*iv*) **Throat length** $=2{\cdot}5D=2{\cdot}5\times5{\cdot}1=11{\cdot}3$ feet.

(*v*) **Throat width** $=3\times8+2\times2{\cdot}5=29$ feet.

$q_o=\frac{700}{29}=24{\cdot}14$ cusecs. The glacis slope will be $2{\cdot}5$ to 1 and will be joined to the throat and the baffle platform by radius $2D=9{\cdot}04$ feet.

(*vi*) **Side divergence ;** $\frac{B_3}{d_3}=\frac{60}{5{\cdot}0}=11{\cdot}76$, from plate XV, for $\frac{B}{D}=11{\cdot}76$ the side divergence should be 1 in $9{\cdot}2$. In the design, it may be adopted as 1 in 8.

(*vii*) **Baffle and Baffle platform.**
H_y = U/S waterlevel—water level D/S = crest level $+d_1-621{\cdot}06=522{\cdot}59+5{\cdot}1-521{\cdot}00$ $=6{\cdot}63$ ft. From plate XV for $q_e=24{\cdot}14$ and $H_y=6{\cdot}63$.
or from equation $d_2={\cdot}11\,q_o\,H_x^{\cdot35}$ (A)
and $x=73\,q_o^{\cdot52}\,H_x^{\cdot21}$ (B)

and from $\frac{B_2}{B_o}=\frac{60}{29}=2{\cdot}07=K$; $C=K^{\cdot152}=2{\cdot}07^{\cdot152}=1{\cdot}12$

$\frac{H_y}{C}=H_x=\frac{6{\cdot}63}{1{\cdot}12}=5{\cdot}94$ feet ; d_2 from equation (A) above $=1{\cdot}0$ ft.

d_x form equation (B) above $=5{\cdot}56$ feet.

$D_c=\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{24{\cdot}14^2}{g}}=2{\cdot}63$ ft.

h_b = height of the baffle $=1{\cdot}2(D_c-D_2)=1{\cdot}96$ feet.

The baffle should be 1·96 ft. high in ⅗ length and should have its top sloping down at the ends. The distance of the baffle from the toe of the fall $=5h_b=5\times1{\cdot}96=9{\cdot}8$ feet ; $H_y+d_y=H_x+d_x$; $d_y={\cdot}87$.

R.L. of the baffle platform = downstream water level—$d_y=621{\cdot}06-4{\cdot}87=515{\cdot}19$

(*viii*) **Cistern downstream of baffle.**

Minimum length of the cistern is given by $L=6{\cdot}3\ Q^{{\cdot}3}=45$ feet.

The bed width at the end of the cistern expanding 1 in 8 = 48·78 feet ; q per foot width at the end of the cistern = 14·36 cusecs.

Depth of cistern is given $V=\frac{q}{d_2}={\cdot}84\ cd_z^{{\cdot}64}$

if $f_l={\cdot}81$ and then $C={\cdot}9$ $\therefore d_z=6{\cdot}02$ feet.

Actual length of the cistern is usually kept rather liberal. Let it be 76 feet in this case, 41 feet with *pacca* sides and bed and 35 feet downstream of the deflector with pitched bed.

The cross-section of the bed of the cistern is bowed. The amount of bowing in midstream equals to 25% R, where R is the hydraulic mean depth. The actual bowing of cistern bed can best be done by letting a string held across the section touch the three points, two ends and the middle. The curve obtained is catenary.

The depth of water in midstream at the end of cistern = 621·06—614·82 = 6·24 feet.

The bed deflector will be $=\frac{6{\cdot}24}{12}={\cdot}52$ feet say 6″ high. Side deflector $=\frac{624}{6}=1{\cdot}04$ feet say 1·0 feet.

(*ix*) **Vanes.**

It is usual to provide vanes in the continuation of piers to stabilize the hypercritical flow. The vanes may be 6″ R.C. walls with 6″ free board.

Sometimes in addition to the deflectors, staggered rows of rectilinear blooks say 4 in number are provided to stabilize flow downstream of the pavement and to reduce scour. In this case the second cistern can be omitted and only one cistern of about 41 ft. length at the worked out level will do.

(*h*) This type of fall is considered to be very efficient for distribution of energy but the cistern element is unnecessarily elaborate and very expensive. The position of the gauging site is defective because it will be opposite an earthen bed which is liable to silt in low supplies. In this the correction due to velocity of approach cannot be correctly applied as will be explained later.

14. Meter Flume.

The correct design of meter flume received the first impetus from the experiments carried out by E.S. Crump I.S.E. Punjab Irrigation, described in Papers No. 26 and 30-A of Punjab Irrigation Branch Publications, Class A 1923-25. These experiments brought out three points very clearly.

(*i*) The discharge formula $Q=CB_tH^{3/2}$ was applicable to the long crested weirs and that the value of the discharge co-efficient C was 3·09 which is its theoritical value.

(*ii*) The length of crest should be 2H.

(*iii*) The value of the co-efficient was constant and independent of H in a long-throated weir flume upto a drowning ratio of 91% (model L of Crump contracted flume with length of crest 2H and Expansion 1 in 10 downstream on both sides). In case of flumes with vertical drop it was constant upto 66% drowning ratio.

(*b*) **Discharge formula.**

Let q be the discharge per foot run ; D be the depth upstream.

Initial Energy $E=D+V^2/2g$

Depth on crest=Initial energy—Rise of crest. $H=E-X$

Let y be the depth of water on the crest and the motion be in a frictionless channel. Fig. 23.

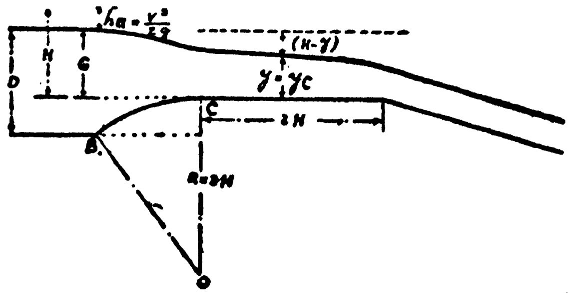


Fig. 23

The discharge per unit width of the weir,

$q=A.V=y\sqrt{2g}\ \overline{(H-y)}.=\sqrt{2g}\ .y(H-y)^{1/2}$ differentiating with respect to y.

$$\frac{dq}{dy}=\sqrt{2g}\left\{(H-y)^{1/2}-\tfrac{1}{2}y\frac{1}{(H-y)^{1/2}}\right\}$$

for the maximum value of q for the values of y ; $\frac{dq}{dy}=0$

$$(H-y)^{1/2}-\tfrac{1}{2}y\ \frac{1}{(H-y)^{1/2}}=0,\ \text{or}\ H-y=\tfrac{1}{2}y\ \therefore\ H=\tfrac{3}{2}y\ \text{or}\ y=\tfrac{2}{3}H$$

This is just the necessary condition which gives the maximum discharge for the known value of H. Therefore discharge.

$$q=\tfrac{2}{3}H.\sqrt{2g\frac{H}{3}}=\tfrac{2}{3}\sqrt{\frac{2g}{3}}\,.\,H^{3/2}=3{\cdot}088\ H^{3/2}=3{\cdot}09\ H^{3/2} \qquad \text{(A)}$$

we obtain critical depth from the equation

$$y_c=\sqrt[3]{\frac{q^2}{g}}=\left\{\frac{(\tfrac{2}{3}\sqrt{\frac{2g}{3}}\ H^{3/2})^2}{g}\right\}^{1/3}=(\frac{8}{27}H^3)^{1/3}=\tfrac{2}{3}\ H\ \text{or}\ H=\tfrac{3}{2}y_c$$

Substituting y_c in terms of H in equation (A)

$$q=\sqrt{\frac{2g}{3}}.\ \tfrac{2}{3}[\tfrac{3}{2}y_c]^{3/2}=g^{1/2}\ y_c^{3/2}=\sqrt{g y_c^3} \qquad \text{(B)}$$

It is therefore that the critical depth is the depth of water required at the crest to give the maximum discharge or the co-efficient 3·09, provided the flow is frictionless in a rectangular channel. It has been shown that the water surface profile in an open channel, is parallel to the bed profile in the condition of critical flow. This is the section of weir, where parallelism of flow is attained. It is the section which gives the maximum discharge so long as the parallelism is not fouled by the conditions of flow upstream and downstream. Any disturbance of this control section results in the reduction of discharge of weir. It is to attain this condition of parallel flow that the crest is kept level.

(*c*) **Length of crest.**

Advantages of long length of crest.

(*i*) A long length of crest as usually adopted in weir flumes ensures a constant co-efficient of discharge with all depths on crest.

(*ii*) A long throat provides a considerable increase in the modular limits. Free fall formula is applicable to as high a drowning ratio as 90%.

(*iii*) In long throated weirs, the cessation of modularity and the disappearance of the hydraulic jump occur simultaneously, so that they offer this great advantage over their short-necked brothers that one can tell at a glance whether a flume or a weir is delivering its full modular discharge or not.

The water surface profile shown in Fig. 23 is convex in the beginning, then attains parrallelism in the critical conditions of flow (usually called the point of inflextion) and lastly drops below the critical conditions in a concave curve. The convexity connotes a curvature of filaments which is attended by an increase in the velocity of successive filaments from the surface to the bed. This will mean that the actual pressure against the bed will be less than the actual depth in a convex flow. Such a flow is easily liable to pressure inflations by a high pressure water available below it as in a jump. The crest, should, therefore, be long enough so that the convex flow dies out on the crest followed by the critical conditions of flow where parallelism is attained. The parallelism connotes that the velocity is the same from the surface to the bed and that the pressure against the bed will be equal to the actual depth. For constancy of the discharge even this is not enough but the crest must be longer still to ensure hypercritical flow for sufficient length downstream of the critical section so that the buffer of the hydraulic jump in the case of meters with a glacis and the convexity of the dropping jet in the case of falls with a vertical drop do not foul the critical section. Hypercritical jet is adherent and not liable to pressure inflations. It has been observed that the control section or the point of inflexion occurs in the middle of the crest when the length of crest is 2H. There is thus a level length of H feet downstream of control section which is enough in the case of flumes with a glacis because the height of the jump at the beginning of a glacis shall never be more than $^1/_3$ H and therefore its splash shall always be less than H. In the case of meters with vertical drop, the convexity introduced by the drop may be serious requiring a length of crest greater than 2H. The author worked out the length of level crest from considerations of a stable and an adherent hypercritical jet dropping in depth by 5% from the critical depth in his Article in Indian Engineering, Calcutta, December 1936. The length of level crest worked out to be equal to H downstream of the control section with f=·0066 and if the crest was of cement concrete or plastered masonry it should be about 2H with f=·0044. The cength should, therefore be 2H in the case of brick-masonry flumes with a glacis and 3H in the lase of falls with vertical drop or with crest surface cement-plastered.

In the above-mentioned publication of the author, effect of the circular approach curve to the crest was also mathematically worked out. This will tend to increase the convexity of surface flow upstream of the control section and, therefore, a relatively greater length of level crest will be required upstream of the control section or the point of inflexion. This effect increases with the discharge and therefore the lengths of the crest were suggested therein as given below :—

Discharge in cusecs per foot run.	Length of crest.
Below 15	2 H
15 to 30	2·5 H
30 to 50	3·0 H

15. Requisites of a Meter Flumes.

The requisites of a correctly designed meter flumes are :—

(*i*) The length of crest from 2H to 3H.

(*ii*) Available working head or drop or the fall more than the minimum modecular head.

The minimum modular head is the difference between the water level upstream at the gauging site and the downstream water level, when the buffer of the jump just starts fouling the control section and the discharge drops by not more than one percent.

Type	M.M.H. in terms of H
Free overfall	33 percent
Contracted flumes (Crump's L type)	9 percent
Contracted meter flume (1 in 5 downstream expansion)	20 percent
Full channel width meter with 1 in 5 glacis.	10 percent

A safety margin of half a foot at least should be allowed for the likely silting up of the channel.

(*iii*) Streamline approach to the crest both on sides and the bed. The object is to minimise the losses due to adrupt entry and shock. A bell mouth approach as already described in case of the Inglis Fall design is the best in the case of contracted falls. In the cass of meters with full width of channel downstream, the side approach will be straight with vertical walls and it is usual to provide a circular approach curve in bed with radius 2H (Crump's original design). The author avoided the corner at the beginning of the curve by giving an S curve. The corner was always found to be filled up with irrgular debris and was not, therefor, desirable. (Fig. 24.)

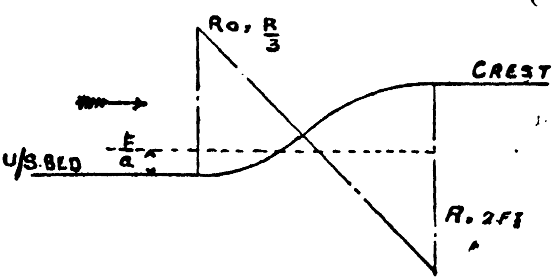


Fig. 24

(*iv*) **Correct Gauging of the depth on crest.**

It would be very accurate if the depth at the control section where parallelism is attained could be accurately measured, because there the discharge will then be simply $\sqrt{gD_c^3}$ and no correction for the velocity of approach will be required. The control section is simply a point of inflexion on the surface curve and it is very difficult to locate it correctly. The depth on crest has, therfore, to be measured upstream of the rest and the correction for the velocity of approah has to be applied. The essential features for the correct Gauging of meter Gauge would be :—

(*a*) The surface profile should be very nearly horizontal for an appreciable distance upstream and downstream of the Gauge hole. This means that there should be no change in the section in this length. The change in section shall produce a slope in the water surface. A Gauge well in the bell mouth approach to crest is, therefore, out of question.

(*b*) The velocity of approch should be correctly calculated for the section opposite the Gauge hole. This is only possible when the section opposite Gauge hole would neither silt not scour for the range of the variation of the discharge over the meter.

In the case of contracted meter flumes with bell mouth approach up to the crest, the gauging has to be done in the earthen channel upstream where the bed level changes by silting in low supply and the velocity of approach cannot be calculated correctly. Such falls serves as meters only in the full supply conditions and even then the results are approximate, as they neglect the energy losses from gauge well the control section which is a considerable distance.

In the standard meter flume design described by the author in his paper No. 154 Punjab Engineeing Congress, 1932 the floor level opposite gauge well was designd in such a way that it

Plan of Meter Flume

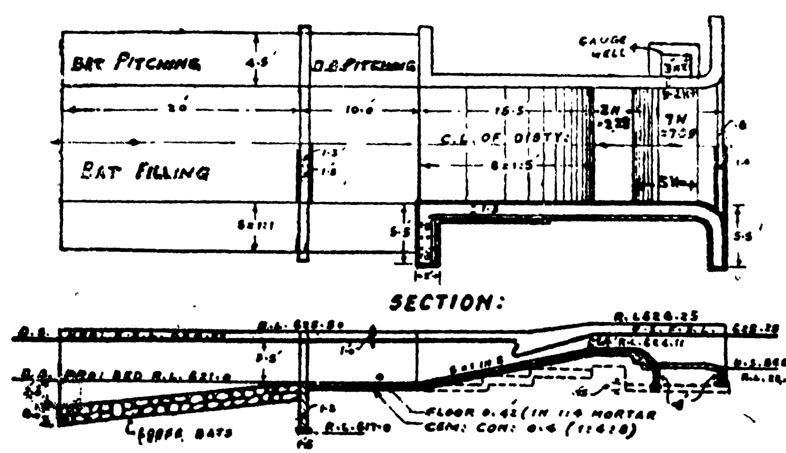


Fig. 5.

remained free of silt even with half supply. The method is illustrated by an example in this chapter. The type is shown in Fig. 25.

(*c*) The design of gauge well and gauge hole should be such that the surface pulsations in the channel are not conveyed to the water surface in the gauge well. This suggests the use of a single gauge hole located below the crest level. In the standard meter flume desgin mentioned above, it was provided to a circular hole in a metallic plate fixed flush with the wall outside, a 3″×3″ hole in the well as shown in Fig. 26, and the area of the hole, in the middle of the iron plate, from 1/2000 to 1/5000 of the gauge well erea.

The position of the gauge hole from the beginning of the crest in the standard meter flume design is kept 3H while there is provided a straight reach 2H at least upstream of it. Thus horizontal surface profile opposite gauge well is ensured. The pulsations on the water surface of the channel are produced by action of the wind on the surface. It is advisable to divide the large flumes into compartents by constructing thin R.B. walls 9″ thick projecting ·5 ft. above F.S.L. and to construct gauge wells on both sides to take the average gauge reading for accurate gauging.

(*d*) The water surface in the gauge well should be accurately measured. It should be measured by means of a Hook Gauge which is not usually practicable because educated gauge readers capable of handling such gauges are not available. The accurate record of gauge is now made posssible by installing automatic weter level recorders. Legget's water level recorder is usually used in the Punjab. This instrument which gives a chart of the water surface level in the gauge well is manufactured in the Central Irrigation Workshops at Amritsar, Punjab, but it is a costly instrument and cannot be installed everywhere. It is usual to put in sloping gauges in the gauge wells divided accurately to the second place of decimal of a foot or showing cusecs. A sloping gauge provides enough length for small sub-divisions.

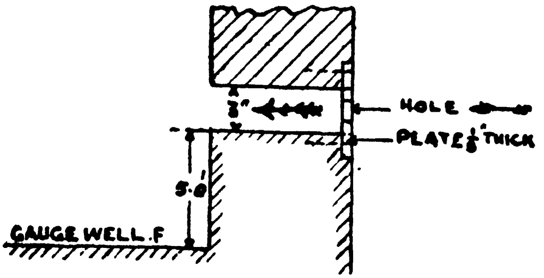


Fig. 26

16. Calibration of Meter Flumes.

The formula of discharge as worked out for a long-crested weir flume in paragraph 14 of this chapter is based on the assumption that the channel was frictionless. Let h_f be the loss of head from the gauge site to the control section on the weir crest.

In the case of raised crested meter flumes, the approach curve in bed is usually circular as described before. Even if it is omitted or if it is a short slope, the stream lines are curved from the bed upstream to the crest. The curvature in the stream lines cannot be avoided except by giving a very flat slope, say 1 in 10, which in not possible in the practical designs. The effect of circular approach in bending the stream lines and thereby producing centrifugal force, was worked out by the author in the his article December, 1936 Page 199 Indian Engineering, Culcutta. Let it be denoted by F., so that F=SH. The discharge formula $q=3{\cdot}09H^{3/2}$, can be written in the form $q=C\,(G+h_a-h_f+F)^{3/2}$ per poot run of the weir, where G is the gauge reading and h_a is velocity of approach head$=V_a^2/2g$ and C is the co-efficient of discharge. If $h_f=F$, C will be equal to its theoretical value 3·09 but if h_f is greater than F as in the case of small chennels, the value of C is less than 3·09 and in case of large channels, the effect of the centrifugal force is very pronounced, the value of C will be more than 3·09.

The author had to carry out large scale observations on Lower Jhelum Canal on meter flumes with the Standard Crump-Sharma approaches with silt-free floor levels which were opposite gauge hole site up to half supply conditions. One interesting point in these observations was that discharges were observed opposite the gauge hole by running of traveller on two ropes stretched across the side walls in the section which was always free of silt and level. The velocities were taken with current meter. The value of actual co-efficient was sensibly constant for every meter flume with respect to H but varied with the size of the channel, *viz.*, the discharge for the different meter flumes as plotted in Fig. 27.

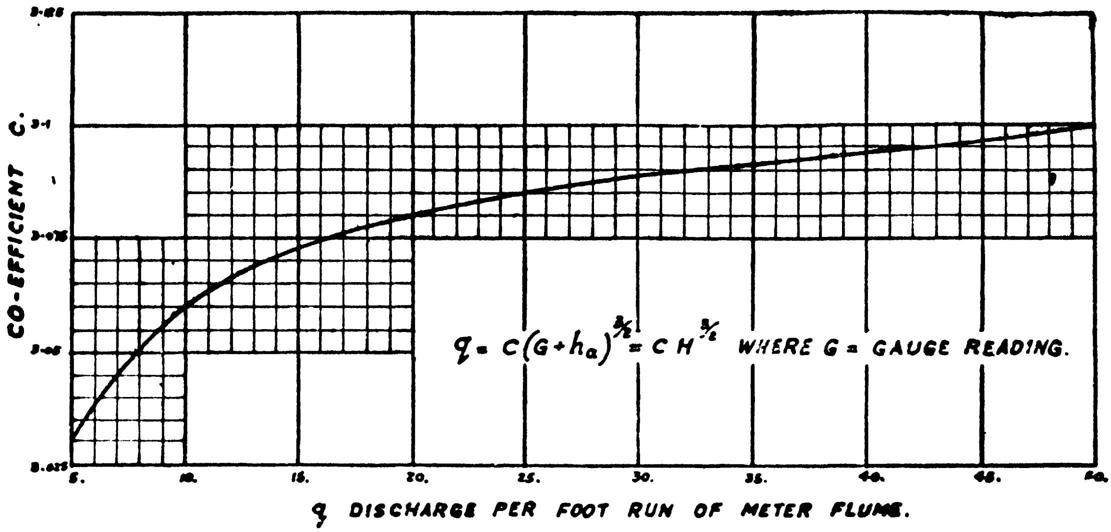


Fig. 27.

(*b*) After the value of C has been actually observed for a meter flume or taken from the graph above, the discharge table for the meter flume can be worked out with the help of the table No. 3 of the chapter or the graph in Plate IV (B), Vol. III.

The object of the table is merely to provide a ready means of performing the process usually referred to as "correcting for velocity of approach," in other words, of applying the above formula $Q=CB\,(G+h_a)^{3/2}$ (1) in view of the difficulty arising from the fact that the term h_a is itself dependent on the discharge Q. We have $h_a=\frac{V_a^2}{2g}=\frac{1}{2g}\times\frac{Q^2}{A^2}$.

so that the formula for discharge may be written

$$Q=CB_t\left(G+\frac{1}{2g}\times\frac{Q^2}{A^2}\right)^{3/2} \quad \text{where } B_t=\text{width of meter at crest} \qquad (2)$$

The value of C, B, G and A being known, the value of Q can, of course, be determined by trial and error. It is, however, a tedious and lengthy process from which the practical engineer would be glad to be saved. In the attached table No. 3 and curve, in 'Plate IV B, corresponding values of $\frac{C'}{C}$ and $\frac{CB_t\,G}{A}$ are given which enable the discharge Q to be calculated directly and rapidly from the formula, $Q=C'B_t\,G^{3/2}$ (3)

in which the value of C′ is obtained by multiplying the known value of C by the value of $\frac{C'}{C}$ taken from the table or curve.

The calculation of corresponding values of $\frac{C'}{C}$ and $\frac{CB_t\,G}{A}$ have been made as follows :—

From equations 1 and 3 above we have $Q=CB_t\,(G+h_a)^{3/2}=C'B_t\,H^{3/2}$

$$\text{whence } \frac{C'}{C}=\left(\frac{G+h_a}{G}\right)^{3/2} \text{ or } \left(\frac{C'}{C}\right)^{2/3}=1+\frac{h_a}{G} \qquad (4)$$

$$\text{But } h_a=\frac{1}{2g}\times\frac{Q^2}{A^2}=\frac{1}{2g}\left(\frac{C'B_t\,G^{3/2}}{A^2}\right)^2$$

$$\therefore \frac{h_a}{G}=\frac{1}{2g}\left(\frac{C'B_tG}{A}\right)^2=\left(\frac{C'}{C}\right)^2\frac{1}{2g}\left(\frac{CB_tG}{A}\right)^2$$

whence from (4)

$$\left(\frac{C'}{C}\right)^{2/3}=1+\left(\frac{C'}{C}\right)^2\frac{1}{2g}\left(\frac{CB_tG}{A}\right)^2 \qquad (5)$$

Putting $\frac{C'}{C}=y$ and $\frac{CB_tG}{A}=x=\frac{CG}{G+Z}$; if Z=Height of crest above U/S floor.

Equation (5) becomes ; $y^{\frac{2}{3}}=1+y^2\frac{x^2}{2g}$; or $x=\frac{1}{y}\sqrt{2g\,(y^{2/3}-1)}$ (6)

which enables us to calculate values of $x=\frac{CB_tG}{A}$ for various values of $y=\frac{C'}{C}$.

Corresponding values of these two quantities are exhibited in table 3 and curve attached in Plate IV (B).

By way of example we will take the case of a weir where :—
$G=16{\cdot}0$ feet ; $C=3{\cdot}40$; $B_t=240$ feet ; $A=4500$ square feet.

We have $x=\frac{CB_tG}{A}=\frac{3{\cdot}40\times240\times16{\cdot}0}{4500}=2{\cdot}90$

From the table or curve Plate IV (B) we find the corresponding value of $y=\frac{C'}{C}$ to be 1·42.

whence $C'=1{\cdot}42$; $C=1{\cdot}42\times3{\cdot}40=4{\cdot}83$
and $Q=C'B_tG^{3/2}=4{\cdot}83\times240\times(16{\cdot}0)^{3/2}=74{\cdot}200$ cusecs.

TABLE No. 3

Table of corresponding values of $y=\frac{C'}{C}$ and $x=\frac{CB_tG}{A}=\frac{1}{y}\sqrt{2g(y^{2/3}-1)}$.

$\frac{C'}{C}$	$\frac{CB_tG}{A}$	$\frac{C'}{C}$	$\frac{CB_tG}{A}$	$\frac{C'}{C}$	$\frac{CB_tG}{A}$	$\frac{C'}{C}$	$\frac{CB_tG}{A}$
1·020	0·907	1·240	2·541	1·460	2·944	1·680	3·071
1·030	1·099	1·250	2·571	1·470	2·954	1·690	3·073
1·040	1·256	1·260	2·600	1·480	2·963	1·700	3·075
1·050	1·390	1·270	2·626	1·490	2·972	1·710	3·077
1·060	1·507	1·280	2·652	1·500	2·981	1·720	3·079
1·070	1·611	1·290	2·676	1·510	2·988	1·730	3·081
1·080	1·705	1·300	2·699	1·520	2·996	1·740	3·082
1·090	1·790	1·310	2·721	1·530	3·003	1·750	3·084
1·100	1·869	1·320	2·741	1·540	3·010	1·760	3·085
1·110	1·941	1·330	2·761	1·550	3·016	1·770	3·086
1·120	2·007	1·340	2·780	1·560	3·022	1·780	3·087
1·130	2·069	1·350	2·798	1·570	3·028	1·790	3·087
1·140	2·127	1·360	2·815	1·580	3·033	1·800	3·088
1·150	2·181	1·370	2·831	1·590	3·038	1·810	3·088
1·160	2·231	1·380	2·846	1·600	3·043	1·820	3·089
1·170	2·278	1·390	2·861	1·610	3·047	1·830	3·089
1·180	2·323	1·400	2·874	1·620	3·051		
1·190	2·365	1·410	2·888	1·630	3·055		
1·200	2·404	1·420	2·900	1·640	3·059		
1·210	2·441	1·430	2·912	1·650	3·062		
1·220	2·477	1·440	2·923	1·660	3·065		
1·230	2·510	1·450	2·934	1·670	3·068		

17. Meter Flume Design Examples (Anthor's Development of Crump's Design)

[A] Design a meter flume in the channel with the following data :—
Discharge=150 cusecs. Depth=3·8 feet ; side slope=½ to 1 ; Bed width=20 feet

Drop $=1{\cdot}5$ feet. Range from full to half supply.

(*a*) **Crest level** :—Width of the flume $B_t=20$ feet.

Discharge per foot run $=\dfrac{150}{20}=7{\cdot}5$ cusecs.

The value of C from Fig. 27 $=3{\cdot}045$.
Let total energy depth above crest $=$ H

$\therefore\ H=\left(\dfrac{q}{C}\right)^{2/3}=\left(\dfrac{7{\cdot}5}{3{\cdot}043}\right)^{2/3}=1{\cdot}825$; Depth in channel $=3{\cdot}8$ feet.

velocity of approach $=\dfrac{7{\cdot}5}{3{\cdot}8}=1{\cdot}97$ feet per second.

$h_a=\dfrac{V^2}{2g}=\dfrac{1{\cdot}97^2}{64{\cdot}4}={\cdot}0652$ feet.

The depth on crest $=1{\cdot}825-{\cdot}065=1{\cdot}76$ feet.
Let upstream water level $=500{\cdot}0$
The crest level $=500-1{\cdot}76=498{\cdot}24$

(*b*) **Floor level opposite gauge hole.**
Half supply discharge $=75$ cusecs.

Discharge per foot run $=\dfrac{75}{20}=3{\cdot}75$ cusecs per foot run.

Scour depth or non-silting depth

$D=1{\cdot}11\ q^{{\cdot}61}=1{\cdot}11\times3{\cdot}75^{{\cdot}61}=2{\cdot}5$ feet.

Total energy depth on crest for half supply

$$H=\left(\frac{3{\cdot}75}{3{\cdot}043}\right)^{2/3}=1{\cdot}15\text{ feet.}$$

Velocity of approach $=\dfrac{3{\cdot}75}{2{\cdot}5}=1{\cdot}5$ feet per second.

Approach head $=\dfrac{1{\cdot}5^2}{2g}={\cdot}035$ feet.

Water level in half supply $=498{\cdot}24+1{\cdot}15-{\cdot}035=499{\cdot}36$
The floor level opposite gauge well $=499{\cdot}36-2{\cdot}5=496{\cdot}86$

(*c*) Length of crest $=2H=2\times1{\cdot}82=3{\cdot}64$ say $3{\cdot}6$ ft.
Radius of upstream approach curve $=2H=3{\cdot}6$ feet.
Distance of gauge hole $=3H=3\times1{\cdot}83=5{\cdot}4$ feet.
Straight distance beyond gauge hole $=2H=3{\cdot}6$ feet.
Radius of the curve of the side walls is 2H. Keep $5{\cdot}0$ ft. in this case.

(*d*) **Glacis.** The glacis slope 1 in 5
Critical depth $=\frac{2}{3}\times1{\cdot}83=1{\cdot}22$ feet

Crump's $L=1{\cdot}5$ ft. and $\dfrac{L}{C}=\dfrac{1{\cdot}5}{1{\cdot}22}=1{\cdot}24$

From plate V, for $\dfrac{L}{C}=1{\cdot}24$; $\dfrac{K+F}{C}=3{\cdot}4$; $\dfrac{x}{c}={\cdot}42$ and $\dfrac{y}{c}=2{\cdot}02$

$F=3{\cdot}4\times1{\cdot}22-1{\cdot}83=4{\cdot}14-1{\cdot}83=2{\cdot}31$
Jump will take place at a distance from the beginning of glacis

$=5\times2{\cdot}31=11{\cdot}55$ say $11{\cdot}6$ ft.

Depth upstream of jump $={\cdot}42\times1{\cdot}22=0{\cdot}51$ feet
Depth downstream of jump $=2{\cdot}02\times1{\cdot}22=2{\cdot}46$ feet
Downstream water level $=500-1{\cdot}5=498{\cdot}5$

The bed level downstream$=498\cdot5-3\cdot8=494\cdot7$
Drop from crest to downstream floor level ; $498\cdot24-494\cdot7=3\cdot54$
Length of glacis$=3\cdot54\times5=17\cdot70$ feet
The jump is at a distance of 11·6 feet and, therefore, well within the glacis.

(*e*) Length of downstream protection$=5D=5\times3\cdot8=19\cdot0$ say 20 feet.

This may be partly *pacca* if staggered blocks or friction blocks are added, but in the case of small channel this may by only ·65 feet thick dry pitching 10 feet long.

The level floor will be followed by loose bats protection 1·5 feet deep in bed in a slope of 1 in 10 for a length of 10 feet and sides bat-filling 1·5′ slope 1 to 1.

(B) Design a meter flume in a channel with the following data :
Discharge 700 cusecs. Drop 6·63 feet. Bed width 60·0. Depth 5·1.

$$\text{Full supply levels}=\frac{\text{Upstream}}{\text{downstream}}=\frac{627\cdot69}{621\cdot06}$$

Bed levels$=\dfrac{\text{Upstream}}{\text{downstream}}=\dfrac{622\cdot59}{615\cdot96}$; sides$-\frac{1}{2}$ to, 1; N.S.$=624\cdot0$ feet.

Calculation. The drop in this case is more than the depth. The meter flume shall be with a vertical drop.

Bed width downstream$=60$ feet
The width between the side walls downstream of the crest shall be 60 feet.
The width at the weir crest$=60-1=59$ feet.

When water drops over the weir crest, the drop causes convexity in the stream lines. The convexity results in the distribution of the pressure. The jet leaves the crest with pressure at the bottom below atmospheric pressure and the surface at atmospheric pressure. The actual water level below the jet is at C′D′ instead of ABCD, Fig. 28. The difference has been observed to be about 0·5 foot in the falls of the size as in this example. As the jet is being discharged against a pressure varying from about 0·5 foot below atmospheric to at mospheric, the discharge co-efficient of meter flume varies and increases more than the theoretical. This introduces another uncertain factor and the remedy is to keep the cistern width 6 inches more on either side than the crest width so the free air can get below the jet.

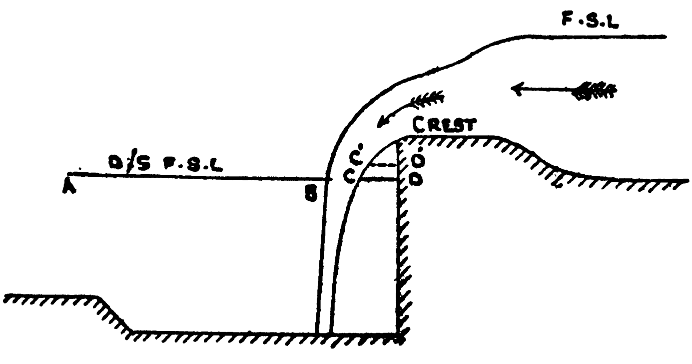


Fig 28

The discharge per foot run of the crest width ; $q=\dfrac{700}{59}=11\cdot9$ cusecs

The value of C from Fig. 27 is 3·065 for $q=11\cdot9$

Total energy depth above crest $H=\left(\dfrac{11\cdot9}{3\cdot065}\right)^{2/3}=2\cdot47$ feet

Velocity of approach (approximate as the floor level is not yet fixed)$=\dfrac{11\cdot9}{5\cdot0}=2\cdot4$ ft./sec.

$\therefore\ h_a=\dfrac{V^2}{2g}=\dfrac{2\cdot4^2}{2g}=\cdot09$ feet.

The Gauge Reading$=G=2\cdot47-\cdot09=2\cdot38$ feet.
Crest level$=$F.S.L.$-G=627\cdot69-2\cdot38=625\cdot31$

(*ii*) **Upstream floor level.**

Half supply $=\frac{11 \cdot 9}{2}=5 \cdot 95$ cusecs per foot run.

According to Kennedy's formula, non-silting depth $=D=1 \cdot 11 \times q^{\cdot 61}=1 \cdot 11 \times 5 \cdot 95^{\cdot 61}=3 \cdot 3$ ft.

$H_1=\left(\frac{5 \cdot 95}{3 \cdot 065}\right)^{2/3}=1 \cdot 585$ feet

Velocity of approach $=\frac{5 \cdot 95}{3 \cdot 3}=1 \cdot 8$ ft./sec.; $\therefore\ h_a=\frac{\cdot 1 \cdot 8^2}{2g}=\cdot 05$ ft.

Gauge reading $=G_1=1 \cdot 585-\cdot 05=1 \cdot 535$ feet
Water level in half supply = crest level $+G_1=625 \cdot 31+1 \cdot 54=626 \cdot 85$
The floor level opposite gauge well $=626 \cdot 85-3 \cdot 3=623 \cdot 55$
(*iii*) Length of crest $=2H=2 \times 2 \cdot 47=4 \cdot 94$ say 5·0 feet
The distance of gauge well hole from beginning of crest $=3H=3 \times 2 \cdot 5=7 \cdot 5$ feet
The straight portion upstream Gauge hole $=2H=5$ feet.
The rise in bed to floor $=622 \cdot 59-621 \cdot 06=1 \cdot 53$ ft. Allowing a slope of 1 in 5 $=1 \cdot 53 \times 5=7 \cdot 6$ feet.
The radius of the side wall at upstream $=7 \cdot 6$ ft.

(*iv*) **Cistern.** Etchevery formula :—
Length $=3\sqrt{H_l}$. $H=3\sqrt{6 \cdot 63} \times 2 \cdot 47=12 \cdot 15$ ft. say 12·2 feet
Depth = one sixth $=1/6 \times 12 \cdot 2=2 \cdot 0$ feet.
Montagu formula :—
From plate VII, for $H_l=6 \cdot 63$ and $q=11 \cdot 9$, the value of
$Ef_2=4 \cdot 25$; Depth $=4 \cdot 25 \times \frac{1}{2}=2 \cdot 12$ say 2·0 feet
Length $=4Ef_2=4 \times 4 \cdot 25=17 \cdot 0$ ft.

(*v*) The protection downstream of cistern :—
The total length 4D or $5D=5 \times 5 \cdot 1=25 \cdot 5$, say 26 ft.
This consists of 10 ft. level floor with a curtain wall at the end and then followed by 15 ft. of loose bats filling 2 feet deep in slope 1 in 10 in bed and sides 1·5 feet deep in slope 1 to 10.

(*vi*) Floor design (Bligh's method), Fig. 29 :
The design is simple, using Bligh's theory. A creep co-efficient of 1 in 7 is enough for the Punjab clay soil, and in some cases it has been kept 1 in 6 with success. The worst

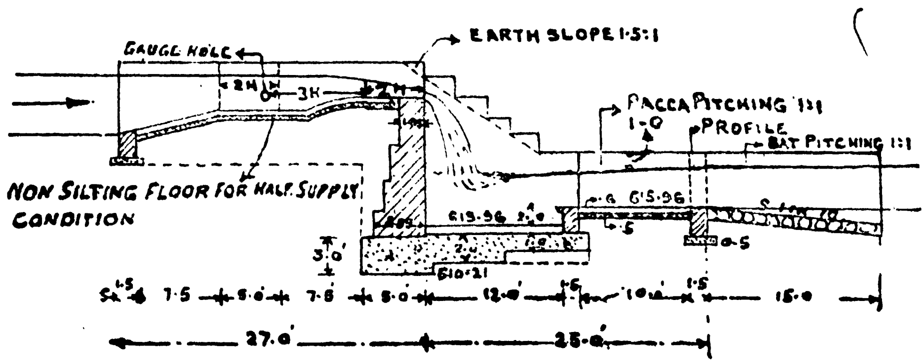


Fig. 29

condition for the creep is when water level is headed upto crest level and the downstream bed is dry.
Creep Head = crest level—downstream bed level $=625 \cdot 31-615 \cdot 96=9 \cdot 35$ feet.
Creep length required $=7 \times 9 \cdot 35=66$.
Actual creep length $=89 \cdot 9$, which is enough, Fig. 30.
The upstream floor opposite guage could be cheapened by making it in dry brick pitching 6 feet

deep and need not be designed in this case. The actual creep up to beginning of cistern=54 feet. Head lost=54/7=7·7 feet.

$$\text{The thickness of floor}=\frac{4}{3}\times\frac{9{\cdot}35-7{\cdot}7}{\rho-1}=\frac{4}{3}\times\frac{1{\cdot}65}{2{\cdot}0-1{\cdot}0}=2{\cdot}20 \text{ ft.}$$

Keep 2·75 for half length and 1·75 feet consisting of ·75 feet masonry of brick on one foot of conerete for the remaining length of cistern.

(*vii*) Cistern floor Design ; Khosla's method.
Depth of downstream cut off=3·5 feet.
Total length of impervious floor=52 feet.

$$\alpha=\frac{52}{3{\cdot}5}=14{\cdot}3 \text{ and } \frac{H}{d}=\frac{9{\cdot}35}{3{\cdot}5}=2{\cdot}67$$

$$\frac{1}{\pi\sqrt{\lambda}}=\cdot115 \quad \therefore\ G_e=\frac{H}{d}\times\frac{1}{\pi\sqrt{\lambda}}=2{\cdot}67\times\cdot115=\cdot318$$

Increase the masonary depth to 3·25 and concrete=·75 for the end curtain wall then

$$\alpha=\frac{52}{4}=13\ ;\ \therefore\ \frac{1}{\pi\sqrt{\lambda}} \text{from plate III}=\cdot12$$

$$G_e=\frac{9{\cdot}35}{4}\times\cdot12=\cdot28 \text{ (the safe value is } 1/3)$$

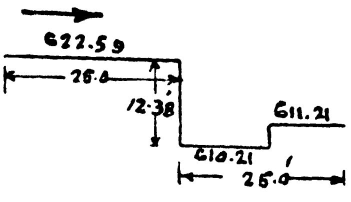


Fig. 30.

This is safe enough for the Punjab clay soil. Note that on account of clay in the soil the pressure calculated from Khosla's theory are reduced to 70%.
Pressure at the beginning of the cistern.
b=27+25=52 d=12·38 feet

$\frac{b_1}{b}=\frac{27}{52}=\cdot52$ and $\alpha=52/12{\cdot}38=4{\cdot}2$

$\phi_e=64\%$, $\phi_c=35\%$, $\phi_d=49\%$

The correction for depth=drop (49—35)=14×11·38/12·38=12·85%
$\phi_c=35+12{\cdot}85=47{\cdot}85\%$
The other corrections are negligble.
Pressure=47·85×9·35=4·46%
The soil being a mixture of clay and sand, the effective pressure =·7×4·46=3·122 ft.
Actual depth of floor=3·75 ft. is therefore safe.

18. Examination Questions.

1. What do you know about the standing wave meter flume. Explain the use of it. (*T.C.E. 1933*)

2. Discuss the advantages and disadvantages of a V notch versus plain crest in the case of canal falls. Show by a sketch the effects of extension of impermeable platform in a weir (a) upstream of the drop wall (b) downstream of the wall. (*T.C.E. 1934*)

3. (i) Discuss the advantages and disadvantages of a plain crest versus trapezoidal notches in the case of falls in (i) Main Canal (ii) Distributaries (iii) Drains and escape channel
(ii) Calculate the height of crest required for a 2 ft. fall in a Disty, with full supply discharge of 150 cusecs, bed width 25 ft., bed slope 1/5000, length of crest being 20 ft. (*T.C. 1935*)

4. At a certain point on a Disty, the channel dimensions are as given below :—
Discharge=110 cs. Bed width=20 feet.
F.S. depth=3 ft. Top width of bank 5 feet.
Top level of bank 1 ft. 6 inch above F.S.L. Full supply level at natural surface.
It is desired to raise the full Supply Level upstream of this point by 1 ft. 6 inch. Design a broad crested meter flume fall allowing for the possibility of some slight silting occuring downstream.
Give dimensioned sketches not necessarily to scale from which a Draftsman could prepare working drawings. The thickness of wingwalls will be fixed by the Draftsman. Give reasons for the particular ratio of width of flume to bed width of channel and of depth on crest to depth of channel. (*P.I.B. 1940*)

5. Describe a method to check heavy bed scour immediately blow a fall in main canal. (*P.I.B. 1939*)

6. Design a broad crested meter for a distributary carrying a discharge of 19·5 cusecs with full supply depth 3·3 ft. and bed width 13·75 ft. Difference in upstream and downstream level may be taken as

·75 ft. velocity of approach should also be taken into account, also draw its dimensioned sketch (which need not be to scale) showing plan of the meter with position of the guage well and its L. section with downstream protection etc. All dimensions relating to the design should be clearly shown. *(P.I.B. 1938)*

7. Prepare the preliminary free hand dimensioned sketches and calculations for designing a broad crested meter flume for the head reach for channel with A.F.S 300 cusecs in sun dry loam soil. *(P I.B. 1937)*

8 Design and sketch a broad crested measuring weir for a channel with the following data :—

(1) Discharge=100 cusecs (2) C=3·1 (3) Difference between upstream and downsream water levels=0·8 ft. (4) Depth of water in channel 3·1 ft. (5) Width of channel=17·0 (6) sides=$\frac{1}{2}$ to 1. *(P. U. 1942)*

9. Describe briefly with help of sketches the various types of the canal falls. *(P. U. 1942)*

10. Describe the various methods of destroying surplus energy downstream of the canal falls with sketches and give reasons why you consider a particular device superior to others. *(P. U. 1943)*

11 Describe the significance of long crest in a long crested meter flume and explain the essential requisites in a good meter flume design.

12. Describe the advantages and disadvantages of contracted falls. Explain with sketches the chief characteristic of Montagu Type fall as used in the Punjab.

13. Deduce an expression for determining the discharge over a weir with a clear overfall. How would the above be modified if the weir is partiallv submerged during floods ? *(F.S.C. 1935)*

14. The upper and lower surface water are 6·0 ft. and 2·0 ft. respectively above the crest of submerged weir 70 ft. long. Calculate the discharge ; take C=0·8 for the drowned portion and C=·557 for undrowned portion. *(F.S.C. 1937)*

15. Explain the phenomenon of standing wave and state the condition necessary for its formation. Draw a sketch of standing wave flume carrying out its essential parts and explain the function of each. Deduce an expression for determining the discharge of a channel on which a standing wave flume is installed. State assumptions made. *(F.S.C. 1939)*

16. Explain the phenomenon of standing wave and state the conditions necessary for its formation. At what place on a weir profile, should the standing wave form to ensure maximum safety. *(P.U. 1952)*

17. Design the principal elements in a fall in a channel of 5000 cusecs discharge having a surface drop of 8 ft. Rough aimentioned sketch is required to show the cistern and other features needed for prevention of scour. *(P.U. 1953)*

18. What are the functions of regulators and falls on a main canal ? How are these works made safe against failure by (a) Piping. (b) Bed scour. *(P.U 1953)*

19. Design and sketch the principal hydraulic elements of a fall for 10,000 cusecs canal with the following data :—

F. S. L. U/S=971·7 ; F. S. L. D/S=961·0 ; Fluming Ratio=2. *(P.U. 1954)*

20. What are the various methods used for dissipation of energy in fall and sketch a fall which can be used as a meter. *(P.U. 1955)*

21 Design and sketch a non regulating fall on a lined canal with the following data :—

Discharge 10500 cs. ; Drop 10 ft. ; Bed width u/s and downstream 75 ft. and 75 ft. respectively. Full supply depth U/S and D/S 17·5 ft. and 17·5 ft. Full supply level upstream/dowustream 1000·00 /990·00 ; mean velocity U/S and D/S ; 6/6 ft./Sec. Check the foundation for safe exit gradient. Only rough dimensions for thickness of floor may be shown on the sketch. *(P.U. 1956)*

22. What is the necessity of falls and how are they located ?

Illustrate with sketches various methods of destruction of surplus energy D/S of a fall. Sketch an Inglis type of fall and describe the features of its design. *(P.U. 1957)*

23. Design an unflumed baffle fall for a branch with the following data :

F. S. discharge	upstream/downstream		=246 14 cusecs
F S. L.	,,	,,	=81·78/77 73
F. S. depth	,,	,,	=3·9/3 9
Bed level	,,	,,	=77·88/73·88
Bed width	,,	,,	=20/20
H_l (drop)	,,	,,	=4 ft.

(P.U. 1958)

PART II

CANAL IRRIGATION

CHAPTER XI

Silt Excluders And Ejectors

1. Introduction.

The idea of silt excluders originated with the late H.V. Elsdon, Executive Engineer Punjab in his paper No. 25, P.W.D. Irrigation Branch Publication 1922. Three ejectors or extractors and two silt excluders were designed and constructed in 1934-35 on the upper Jhelum Canal by E.S. Crump, Superintending Engineer. Salampur Extractor on the Upper Bari Doab Canal followed soon after. Khanki and Dadupur Silt excluders were constructed recently, and the Haveli main line has been provided with a silt ejector. A very comprehensive paper No. 211 was written by F.F. Haigh, Superintending Engineer, on silt excluders in the Punjab Engineering Congress 1938. A brief summary is given here.

2. Definition.

The silt excluding device built in conjuction with the head regulator of a canal on the upstream side in the river is called a silt excluder.

When silt excluding structure is constructed across a canal, it is usually called a silt extractor or ejector.

The basic principle on which both are designed lies in the fact that in a flowing stream carrying silt in suspension, the concentration of the silt charge in the lower layers is greater than in the upper ones Fig 9 Chapter VI. Consequently, if we can escape the lower supply without interfering with the silt distribution, the remaining water will have less silt in it per unit volume than the water upstream of escape.

(*i*) The essential features of the design should, therefore, be to provide concentration of silt charge near bed by some such devices as reduction of friction by paving or by plastering the bed and the sides, or by increasing the section and reducing velocity, thereby lowering the bed just upstream of the excluding device. This can easily be arranged in the case of a canal section flowing straight with uniform flow. In the case of a river the conditions of approach to head are usually curved and disturbed. Silt ejectors in the canal are, therefore, more efficient than the silt excluders at the head of a canal.

(*ii*) The other most important feature of design should be to provide seperation of the bottom water charged with concentrated silt from the upper water wtithout any disturbance. This requires that water should enter the approach to the chamber below the Diaphragm slab with the same velocity as it is flowing in the canal approaching the work without disturbance.

(*iii*) Lastly the high silt-laden water thus stealthily taken away from the canal should be escaped through the tunnels connected with approach with a velocity which does not disturb the entry of water at the approach.

3. Approach Channel to a Silt Extractor.

Undoubtedly the object should be to provide a long straight approach channel. in which the silt can settle into the lower layers of the water, and thus increase the efficiency of the extraction. Anything after the nature of curve, which will displace the silt concentration to the side of the channel as well as reduce it, or any obstruction on the sides or bed which will set up turbulence and hence destroy the bed concentration, should be avoided.

The great advantage that an extractor has over an excluder is, that the approach channel can generally be secured without difficulty in the case of the former, while in case of the latter it is usually necessary to turn the water through a right angle bend before separation is effected and in any case, the approach channel consists of a natural river bed, will probably have curves in it and will certainly have very irregular boudaries.

(i) The approach channel should be designed with the flattest slope which will carry the heviest grade of silt likely to approach the work.

(ii) It should be pitched or lined up to 3 to 4 D (depth in the channel) upstream of the work.

(iii) If possible, the bed may be depressed upto 1·0 ft. by a gradual transition curve in the bed, there should be no change of section on the sides.

(iv) The minimum length of approach channel below a head regulator to ensure the silting of the turbulence downstream of the head and then the restoration of the normal silt distribution may be taken about 1000 ft. for a canal with about 6000 cusecs discharge.

4. Escape Discharge above Extractor.

The efficiency of an excluder may be defined as the reduction per unit of the silt intensity in the canal supply when compared with that of the water approaching the work. This, though the only practical standard, is a false criterion. The true measure of efficiency of an excluder is unity minus the ratio of the silt entering the canal to that which would enter it if the excluder was not working. The point of this distinction is that the addition of the escapage discharge to the canal discharge increases the silt approaching in the canal and increases it in a proportion greater than that of the discharge, because in Kennedy's theory, silt transported varies as $D^{5/2}$ and discharge varies as $D^{5/3}$. We must not, therefore, blindly accept the idea that the greater the escapage the greater the efficiency. Research is necessary to determine what the optimum proportion is 20% or below is suggested as reasonable.

5. Approach Design of the Extractor.

The separation of the escape water from the canal supply at the edge of the diaphragm should obviously be arranged without disturbing the silt distribution. It is easy enough to arrange this for fixed canal and escapage discharges by placing the diaphragm at a height such that it divides the normal stream into the correct proportion. In practice however, it is always necessary to vary both the canal supply and the escapage and if the height of the diaphragm is fixed it will generally not suit the varying proportion of the two. In the ideal case, water near the bed should enter the ejector with the same velocity with which it is approaching it and in the full supply condition. the stream lines will be horizontal for the water passing over the diaphragm. If, however, it is required to run the canal supply full, while sufficient water is not available for the discharge escapage or if the escapage is run full, when canal supply is low, we shall have stream lines some what as Fig. 3.

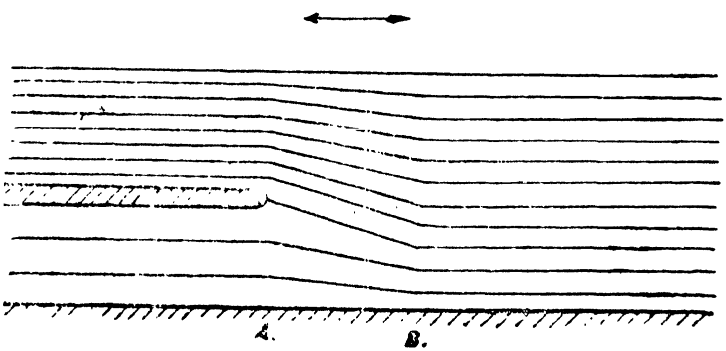


Fig. 3.

In the region AB there may be a certain amount of turbulence set up which will cause a less favourable silt distribution. The question is, is this disturbance serious ? According to Elsdon it would be as shown by his proposal to have the height of the edge of the diaphragm variable. In practice this device has not been made use of and little attention has been paid to the point in the design. This point needs investigations in models.

6. Tunnel Entrances.

For economy and to avert any danger of silting the velocity in the tunnels must be high.

We have, therefore, to transform the comparatively low velocity of the escapage at the approach entry to the high tunnel velocity. This must be done without allowing the draw of the tunnels to affect the velocity distribution upstream of the separation and in certain cases when escapage head is valuable, a gentle transformation is necessary to avoid loss. The former point can always be secured by placing the entrances a sufficient distance downstream from the edge of the diaphragm, but this may involve an expensive cantilevered slab. The methods by which this is usually arranged is shown in Fig. 4.

(*b*) **Tunnel design.**

The tunnels themselves must be arranged to evacuate the escapage at high velocity, say not less than 10 ft. per second. They must also provide control of the discharge so that the same velocity

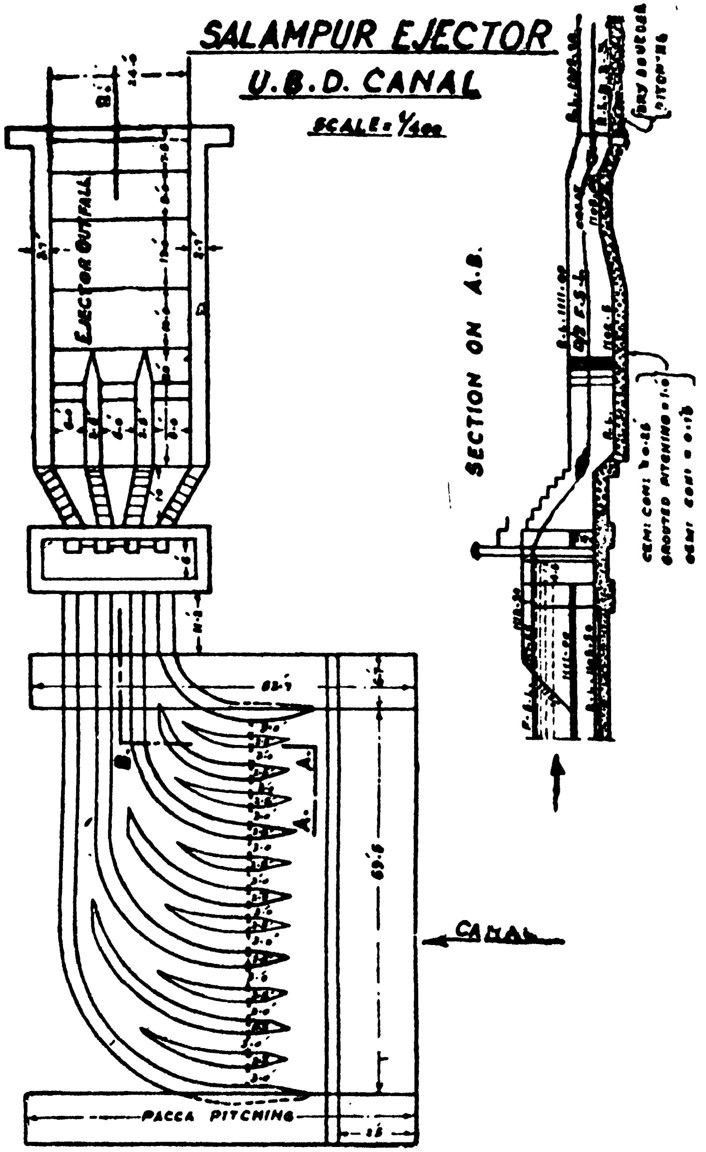
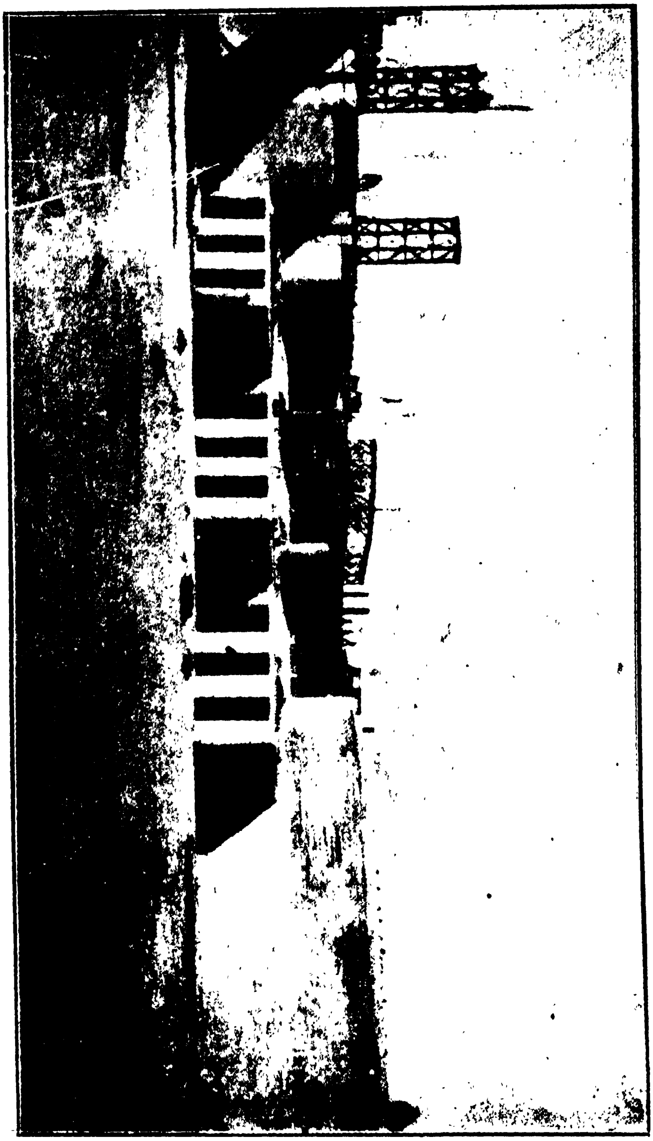
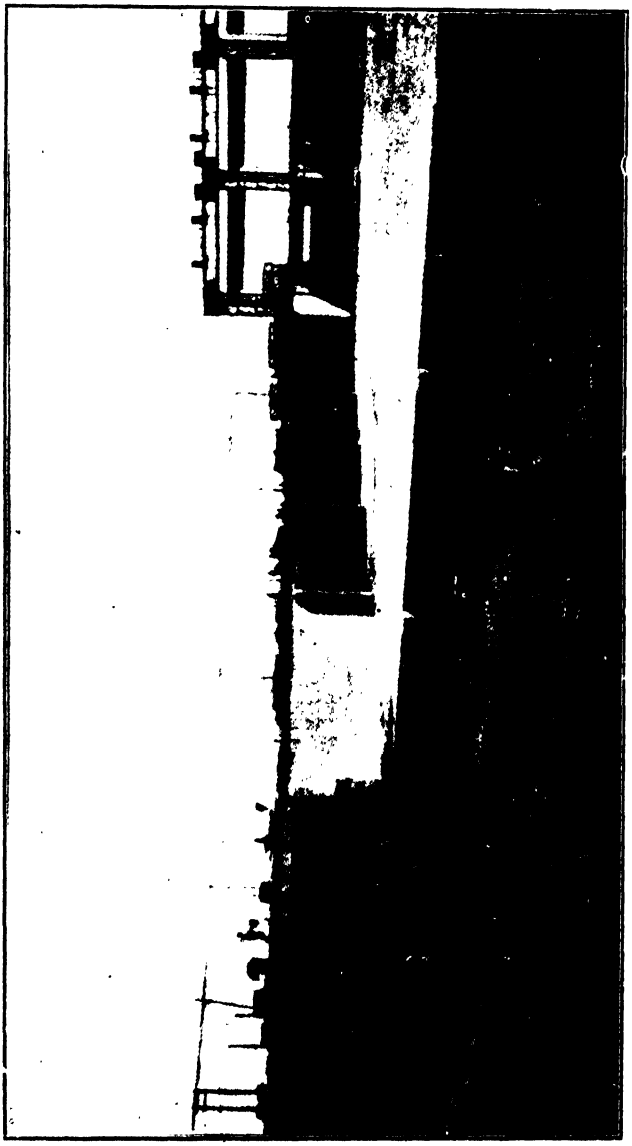


Fig 4.

is secured at the entrance to each tunnel. This may be done either by keeping the same tunnel dimensions and varying the width of canal served by each tunnel, or by keeping the widths served



Silt Excluder in right under-sluices at Trimmu. Fig. 1



Silt Excluder Tunnels in left under-sluices, view from upstream at Trimmu. Fig. 2

and hence the discharge the same, and varying the tunnel sectional dimensions to secure the same discharge with the varying tunnel lengths. The loss of head in each tunnel may be calculated by Manning's formula. At Jaba, the widths served and the tunnel dimensions are the same and the discharges are equalized by a varying degree of contraction at the tunnel exits. At Dadupur, the discharge of each tunnel is the same and the tunnel dimensions are varied. There is a control weir at the end of each tunnel, the crest levels of which are varied to suit variations in width. At Khanki and Madhopur there appear to be no devices for equalizing the tunnel discharges. The tunnel roof should be designed to take the full water pressure above it with the minimum pressure which may occur inside it, assuming the entrance to be blocked. If the tunnels act as weir for the canal supply, the possibility of uplift occuring with the tunnels closed at the downstream end, and a velocity depression over the roof should be studied. The escapage, if less than full supply, is regulated by gates on the downstream end of the tunnel. At Madhopur, a surge chamber is provided at the downstream end of the tunnels which serves to obviate any danger of water hammer. This arrangement also permits the regulating gates to be placed with the chamber and enables grooves with only one face machined, to be used. At Dadupur, the gates are placed on the outer ends of the tunnels and the grooves are machined on both faces, the gates against the upstream face. At Khanki, Bhong and Jaba the existing undersluice gates were used to control the tunnel discharge.

(*c*) **Tail race.**

The channel from the ejector to the river for escaping its discharge should be designed with somewhat steeper slope and relatively higher value of C. V. R. as compared to the canal.

7. Extractors Combined with a Meter.

At the head of a canal a silt ejector can conveniently be combined with a meter flume with crest higher than the diaphragm level. For accurate gauging it would still be better if the floor level opposite gauge wells be kept higher than the diaphragm level. For ideal conditions, it should be the top level of the diaphragm slab. The meter flume should be with full width of the channel without bringing in complication of the contracted approaches.

There has not been any serious trouble with the existing silt extractors resulting in their choking due to jungle getting into them. There is usually no arrangement in the form of trash-racks to remove jungle. Jaba had a slight trouble to begin with.

8. Effect of Excluder or Extractor on the Canal Regime.

So far the silt excluders have been built where the silt entry has been affecting the regime of the Canal to a dangerous extent. In the case of the Upper Jhelum Canal, the capacity had been reduced considerably and was still diminishing, as a result of silt entry. In the case of the Lower Chenab Canal, excessive slit entry at the head had caused the bed of the main line to rise, and was giving great trouble in the branches and distributaries. In all these cases, therefore, it was obviously necessary to exclude or extract silt from the canal, as much as possible, and the excluders were constructed as a method of affecting this, but in doing so, no attempt was made to consider what grade or quantity of silt was suited to the canal regime, and to regulate the silt entry accordingly. The excluder has always been designed to be as efficient as local conditions would permit at reasonable cost. It is evident, of course, that if the excluder was too effective and resulted in too rapid retrogression of the canal, it could be disused or worked intermittently and this had actually to be done in the case of the Bhong excluder, where the retrogression endangered masonry works in the canal.

On the Upper Jhelum Canal the effect of the excluders has been so far to stop the progressive silting up of the canal which was in progress, and to restore its capicity to that designed. On this canal the capacity is ruled by a maximum gauge of 11·3 at Jaggu. The discharges given by this gauge in different years are, 1934 – 7695; 1935 7875; 1936 – 8160. In 1937 the gauge of 11·3 was never attained. In January of that year a gauge of 11·25 gave a discharge of 8385 cusecs which appears to have been the maximum run.

On the Lower Chenab Canal, the progress of silt erosin is about 1·56 ft. in the head reach upto R. D. 40,000 and ·93 ft. upto R.D. 140,000. The effect of silt excluders in the case of the Western Jamna Canal and the Upper Bari Doab Canal has not yet been felt.

The amount of silt exclusion in all cases was arbitrary. No attempt has, however, so far been made to regulate the silt at entry to a grade and quantity suited to the slopes for which the canal has been designed, but such regulation must be the aim in the case of an established canal system. How this aim has to be achieved, however, is by no means clear. To regulate silt, we must have some standard for measuring it which can be readily observed. The most generally accepted standard for silt grading is Lacey's 'f_l' which cannot be defined more clearly than to say that it is a characteristic which relates silt grade to discharge and slope in a regime channel. The difficulty of the problem is increased by the fact that 'f_l' varies throughout a canal system, under the influence of such factors as attrition and the unequal silt extraction of off-takes, and before we could regulate to a given 'f_l' at the head we would have to relate that 'f_l' to those of the canal as a whole. There does not, therefore, seem to be much prospect of silt regulation in the immediate future, in terms of 'f_l'.

However, the author's formula $R_c = r_1^{1/3}.\ r_2^{\cdot 3}.\lambda^{1/6}$ giving the change of C.V.R. with respect to the silt charge and silt grade can successfully be used even in the case of silt excluders as has already been done in the case of the silt-selective distributary head-regulator, Chapter XV.

9. Efficiency of Silt Excluders or Extractors.

There has been some doubt in the past as to the best method of calculating the efficiency of excluders. The one in general use gives only the reduction of silt intensity in the canal water as compared with that of the approach flume. On the Upper Jhelum Canal, however, it was formerly customary to take the ratio of the total silt escaped to that of the approach flume as the efficiency, and this method is also employed at Madhopur as well.

If Q, I and S are the discharge, intensity and total silt contents the suffixes, f, c and x denote the approach flume, the canal and the escape respectively, the efficiency is given by Haigh's formula.

$$E = \frac{I_f - I_c}{I_f} = 1 - \frac{I_c}{I_f} \qquad \text{(A)}$$

If observations of the approach flume are not available, the efficiency may be obtained from the canal and escape observations by the formula :—

$$E = \frac{Q_x(I_x - I_c)}{I_c\,Q_c + I_x Q_x} \qquad \text{(B)}$$

According to the second method referred to above the efficiency is given by : —

$$E' = \frac{S_x}{S_f} = \frac{Q_x I_x}{Q_f I_f} = \frac{Q_x I_x}{Q_c I_c + Q_x I_x} = \frac{Q_f I_f - Q_c I_c}{Q_f I_f}$$

$$\text{whence } E' = E + \frac{Q_x I_x}{Q_f I_f} \qquad \text{(B')}$$

It is clear from this that the latter method gives a greater value of efficiency than that calculated by the former. It may also be noted that if the excluded efficiency is nil, *i.e.*, if the intensities in the two downstream channels are the same as that upstream, the latter formula would still show high efficiency Q_x/Q_f, while the former would correctly give a nil value.

In comparing the efficiencies of excluders it must be remembered that the greater the proportion of the supply escaped, the greater the efficiency. As has been pointed out above, the efficiency will not vary directly with the escapage. Since the intensity decreases rapidly with depth, additional escapage will increase the efficiency, but slowly.

Another point to be borne in mind in this connection is that the efficiency must be affected by the grade of material carried by water. Since that grade has been found to cause trouble in practice, efforts are made to exclude everything larger than 0·2 mm. diameter. The proportion of the total silt which is greater than 0·2 mm. and the relation of the coarsest silt carried to this grade, will, however, vary at different sites.

The same excluder may be expected to work more efficiently where the proportion of coarser silt is grater than where it is small, but on the other hand, the coarser the grade of silt carried, the greater the slope and velocity, and consequently the less the concentration of silt in

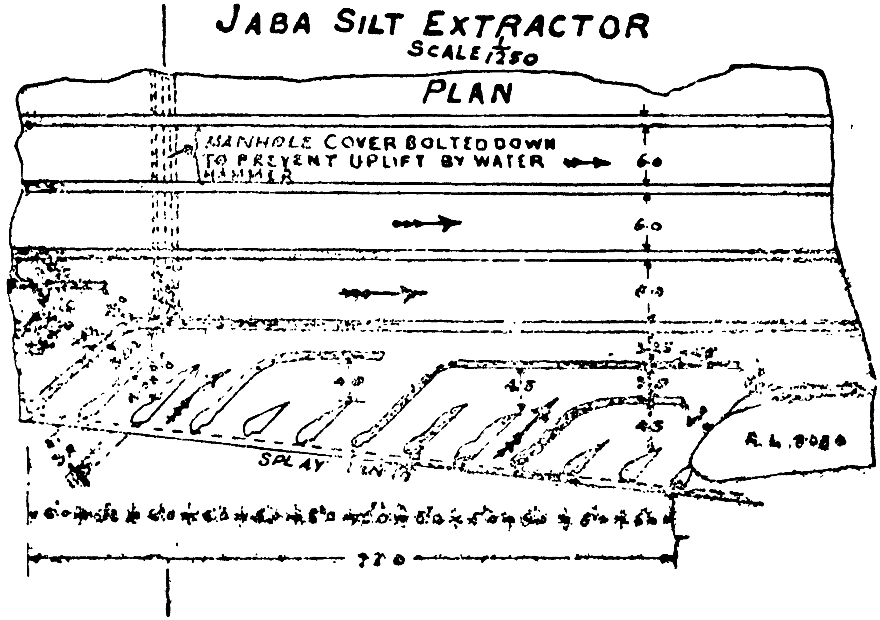

JABA SILT EXTRACTOR
SCALE 1/250
PLAN
MANHOLE COVER BOLTED DOWN
TO PREVENT UPLIFT BY WATER
HAMMER
6.0
6.0
SPLAY

Fig. 5.

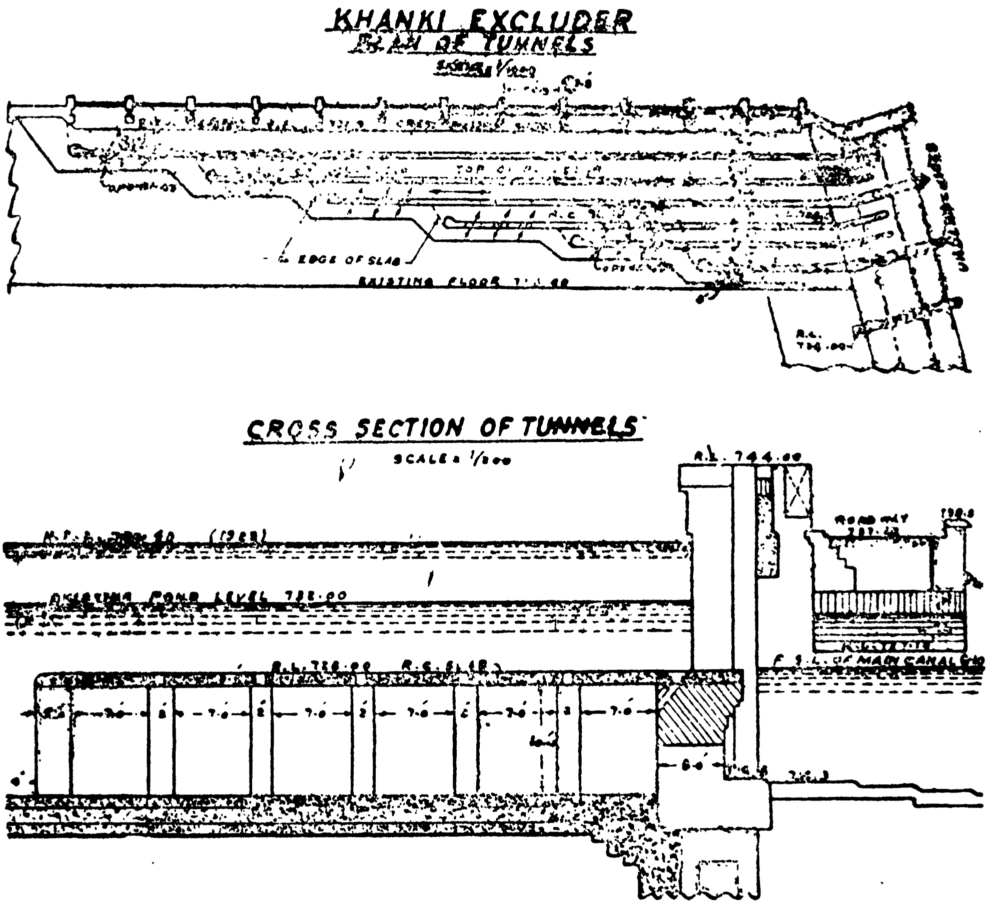

KHANKI EXCLUDER
PLAN OF TUNNELS
EDGE OF SLAB
CROSS SECTION OF TUNNELS

Fig. 6.

the lower layers. It also seems probable that the coarser the silt present, the greater the efficiency when based on the same grade, but this is by no means proved by our present knowledge of the subject.

10. Typical Designs.

The typical design of a silt ejector is shown in Fig. 5. as used at Jaba, the Upper Jhelum Canal, and a different design of a silt excluder is given in Fig. 6. as used at Khanki.

11. Automatic Silt Extractor Without Loss of Water.

The author described a design of such extractor in the Indian Engineering, Calcutta, January 1938. A brief summary is given here.

The silt movements take place after the Monsoons in the months of October and November and the silt ejectors would be most effective then, but on account of the *Rabi* showing period, they have to be kept closed as not a drop can then be wasted.

(*a*) Different types of silt extractors or ejectors have already been tried. The silt ejector at R.D. 1,00,000 of Jhang Branch has simply got slits a couple of feet below the silted bed level. The depth of the channel is about 10 feet. The depth of silt above the slits stands by mutual cohesion of particles and no silt or water flows out. If the silt above the slits is disturbed, the canal loses more than one-third supply under pressure head of 12 feet. Such ejectors worked by the pressure of the depth are useless on account of loss of a great amount of water. Improved type of silt ejectors have been constructed on the Upper Jhelum Canal where the discharge lost through them is controlled by means of gates on sluice valves. Ghak - Sikandar ejector is a nice example of a very efficient type. The maximum discharge is restricted to the discharge of the canal up to 1 foot depth. Even the controlled discharge is as much as 250 cusecs which cannot be spared for escapage throughout the year.

(*b*) The principles of silt exclusion for distributary head regulators, described by the author in his Paper "A Silt Selective Distributary Head Regulators" published by the Punjab Engineering Congress 1936, are equally applicable to the silt extractions from the canals. This paper describes clearly the detailed experiments which were carried out on full size models made of glass to determine the effect of streamlines in each case. About twice the normal silt charge in a cubic foot of water could be extracted if the bed water of the canal be stealthily extracted without the slightest disturbance. It is very essential that no disturbance should be generated in the act of stealing the bed silt laden water. Water should be taken away with the same velocity with which it is approaching the silt extracting device, which should not in itself be an obstruction or a disturbing factor.

(*c*) If the supply which escapes in such a case is, say 10 percent, the silt extracted by weight will be one-fifth of the total silt charge of the channel. This is an appreciable exclusion as it mostly consists of the objectionable course silt. In order to further increase the efficiency of a silt extractor, the efforts should be directed to concentrate the silt charge near the bed of a canal. This could be attained by reducing the roughness of the opproach canal section. The reducing in roughness means elimination of vertical silt carrying eddies. Cement plastered sides and bed will do, upstream of silt ejecting aperture in bed of a canal up to a length of say 5 times the depth of the channel. The concentrated silt charge near the bed will thus be led to the apertures without disturbance. The degree of concentration of silt charge near the bed depends upon the reduction in roughness in the approach channel. Glazing or even covering the perimeter with mica or celluloid sheets will be more efficient than cement plaster. The author thinks that such expensive measures are not in fact necessary. Even cement plaster will reduce the roughness of the channel section to one half in comparison to an earthen section and consequently is likely to double the efficiency of a silt ejector which is sure to meet the requirements in all cases. The scientists having resources of Hydraulic Research Institutes should help the engineers by evolving some cheap methods of hydrogen ionisation of water reaching a silt extractor to concentrate all silt charge near its bed.

(*d*) After providing suitable approaches for efficient silt ejection, the design of the ejector should involve as little loss of water as possible. The aim should be that there is no loss of water as there is hardly enough water in winter for the crops and no water could be spared from any canal for the purposes of silt-ejection. There is bound to be some loss of water if the silt-ejecting device is worked by the hydraulic pressure of depth. Unfortunately, all existing ejectors are worked by pressure. They invariably entail a considerable loss of water. We cannot afford to waste a single drop of water in winter and they have to be closed down for six months. We should therefore, think of some other property of silt or water to work our silt ejectors. Laboratory experiments could be usefully employed to discover some device to eject silt from canals adopting some simple principal such as variation of viscosity due to different silt charges in water. As described below, the writer has worked out a design for silt ejection from a canal, utilizing the difference of specific gravity of silt and water. This device essentially aims at little or no loss of water in the process of silt ejection.

(*e*) **Suitable design.**

A suitable design of an automic silt extractor without loss of water is given in Fig. 7. (a). It is drawn to scale for construction at R. D. 70 of Mancher Distributary, Lower Chenab Canal, the discharge of distributary being 70 cusecs. Regime slope cannot be given and the available slope is 1 in 5,714. The designed depth is 2·5 feet. This distributary takes off from one bay of Chenawan escape which has got a very low bed level as shown in the drawing and which can easily take away the silt extracted from the Mancher Distributary. Smooth approach in cement plaster is provided upstream of silt extracting apertures. The width of the channel is not restricted. The reduced roughness of the approach channel will thus concentrate the silt charge near the bed.

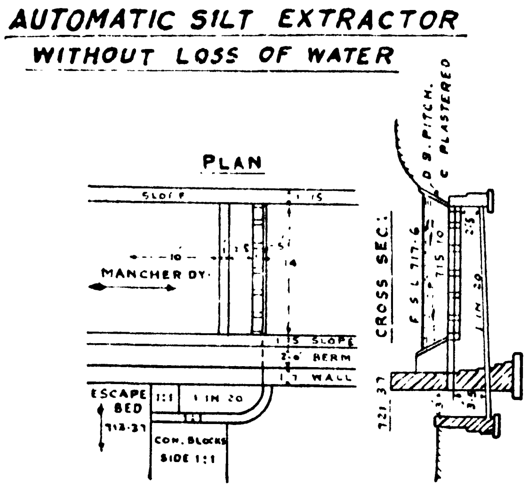


Fig. 7. (a)

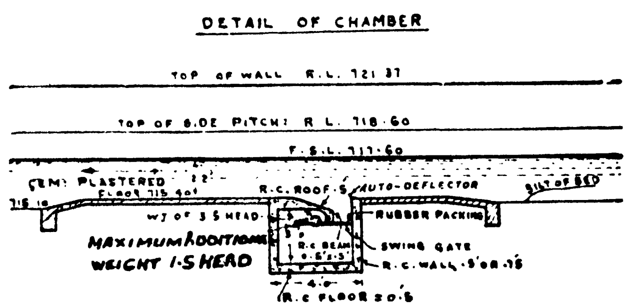


Fig. 7. (b)

Water will leave its silt cargo above the aperture as a suitable expansion is provided for reduction in velocity. A guaze deflector is provided as an additional factor to catch the silt. The automatic device consists of a swing gate which is permanently loaded on the other side for the head of water above the gate which is 3·5 feet in this case. Additional loads are provided for the weight of silt on the gate. The additional weights could be adjusted in such a way that the swing gate should open when the silt is say 9 inches over it. Provision is made that the gate opens just enough to drop away the silt. The gate will then automatically close the aperture till it is again loaded with silt up to a stipulated height. Very little water will flow out when the silt is dropped. Inspection Chamber is provided as shown in section. A very steep slope of 1 in 20 is given in the tunnel so that the silt moves out into the escape with the least amount of water. There are no sensitive moving parts needing special attention. The ball-bearing of the swing gate shall require to be oiled occasionally. This device provides a suitable design which could be installed on canals which have got syphons with good outfalls. The working of this silt ejecting contrivance is automatic and without much loss of water.

12. Example of Design. (*silt Extractor at R. D. 600 Thal Miani line*)

(*i*) **Tunnels.**

Each tunnel serves 80 feet of the bed width. To ensure correct distribution over this width there is proposed a diaphragm separating it into two orifices, spread 6′ apart, each taking 12 cusecs under a head of 0·5 foot. The area required for this is given by $q=7A\sqrt{H}$; whence $A=2{\cdot}4$ sq. ft. *i.e.*, $2{\cdot}4' \times 1'$: the orifices have been streamlined. The tunnel of the left of the diaphragm is $4' \times 4'$ for which $a=16$, $f=10{\cdot}6$ and $m=1$. The discharge in this channel as compared with that in the right channel looses additional head in a length of 30′. This is calculated as below.

From Manning taking $h=0{\cdot}0165$; $V=0{\cdot}9m^{2/3}\ s^{1/2}$ where 's' is the drop in 10,000 ft.

$s=\frac{V^2}{0{\cdot}81.\ m^{4/3}}$; now $V=3{\cdot}75$; $m=1$; whence $s=0{\cdot}174\%$.

Loss of head in $30'=\frac{30}{100}\times 0{\cdot}174=0{\cdot}0522'$. This may be compensated by raising the velocity in the right channel to V′ so that $\frac{(V'-3{\cdot}75)^2}{64}=0{\cdot}0522.$

$V=1{\cdot}83+3{\cdot}75=5{\cdot}58$ ft. Area of tunnel at the narrowest section

$=\frac{60}{5{\cdot}58}=10{\cdot}75$ sq. ft. $=2{\cdot}69$ ft. $\times 4$ ft.

The length of tunnel downstream of the end of diaphragm is $76+170=246'$. For the tunnel, $a=32$; $P_w=24$; $m=1{\cdot}33$ ft. Loss of head per % foot $=0{\cdot}115$ ft. ∴ loss of head in 246 length $=0{\cdot}29$ ft. Between each tunnel there is a difference of about 60′ in length. Loss of head in 60 ft. $=0{\cdot}118\times\frac{60}{100}=0{\cdot}0708$ ft.

Difference in length.	Difference in Loss of head.	Maximum velocity required in tunnel at the point of sudden expansion.	Area required.	Depth.	Width.
2nd tunnel 60 ft.	0·0708	5 89	20 4	4	5·1
3rd tunnel 120 ft.	0·1416	6·77	17·7	4	4·43
4th tunnel 160 ft.	0·2124	7·45	16·1	4	4·0

Total loss of head in any tunnel, therefore, is

Loss at entry	=0·500
Loss in 4′ tunnel	=0·052
Loss in 8′ tunnel	=0·290
Loss at exit	=0·219
	1·061 ft.

(*ii*) **Out fall.**

(*a*) **Surface flow :**—The minimum river level with which it will be necessary to work the extractors may be taken as 675·0.

The normal discharge per foot of tunnel is 15 cusecs which is reduced to 12 cusecs in the cistern. Assuming the final pond at R.L. 694·0 and a T.E.L. at the tunnel exit of 693·0.

$H=693-675=18$ feet.

$H_{f_2}=5{\cdot}2$. Hence it will suffice to put cistern floor at R.L. 670·0.

With a pond level of 690 and tunnel soffit level of 682 at the downstream end, the head will be 8 ft. and the discharge 1400 cusecs.

∴ Discharge per foot in cistern=34 cusecs. Downstream T.E.L.=683·9

The minimum downstream levels required to keep the standing wave on the cistern floor is 677·4. Keep cistern roof at R.L. 678·0 to provide for the depth.

With a pond level of 692, the tunnel intensity will be 49·5 and the maximum downstream level consistent with the discharge about 682·0.

With a downstream level of 685 corresponding with about 1·5 lakhs in the river, the extractor will take 1300 cs.

(*iii*) **Sub-soil flow and pressures.**

With the extractor closed and the minimum downstream water level of 673·0, it is estimated that spring level will not exceed R.L. 682·0. Pressure at the back of pile line will be governed by spring level and may be taken as 3/4 of the head from spring level to outfall *i.e.*, 3/4×9 or say 7′.

The depth of pile line required for an exit gradient of 1/4 is given by :— $G_e=\frac{1}{4}=\frac{H}{\pi d}\times\frac{1}{\sqrt{\lambda}}$

whence d=9·3 say a 10 ft. pipe line.

As regards pressure, the alternative, of an open cistern involving heavy wing walls and roofing the cistern has been worked out and the later is found to be considerably cheaper. The maximum pressure under the cistern floor will be about 9′. It is necessary, however, to provide for silt depositing on top of the roof for which a level of 690·0 is suggested. The pressures resulting from this are greater than the sub-soil pressures and hence rule out the design. The roof will be loaded to the level *ab initio* to take the sub-soil pressure.

The thickness of the barrels, walls and roofs will be approximately as shown in the drawing Fig. 8.

(*iv*) **An economical thickness of a slab.**

Thickness of a slab does not appreciably affect the cost of form work, hence in working out an economical thickness of a slab the cost of shuttering and centring is not considered.

Let M=Bending moment.
t=effective thickness of slab.
c=cost of concrete per cft.
s=cost of steel per cwt.
1·5″=Concrete covering for steel.

Further assume that distribution steel is 25 percent of the main steel and the area of main steel in square inches$=\frac{M}{14000t}$

Cost of slab per sq. ft. :—

$$\text{Steel}=\frac{12\cdot5\text{ M}}{14000t}\times\frac{490s}{144\times112}=\frac{Ms}{368000t}\text{ ; Concrete}=\frac{(t+1\cdot5)c}{12}$$

$$\text{Total cost per sq. ft.}=K=\frac{Ms}{368000t}+\frac{(t+1\cdot5)c}{12}$$

$$\text{For K to be minimum }\frac{dk}{dt}=0\text{ ; } i.e., -\frac{1}{368000}\times\frac{Ms}{t^2}+\frac{1}{12}c=0$$

$$\text{or }t^2=\frac{1}{30670}\times\frac{Ms}{c}\text{ ; }t=0\cdot0057\sqrt{\frac{Ms}{c}}$$

At Kalabagh Headworks, s=15·25 and c=0·48,

$$\therefore t=0\cdot0057\times\sqrt{\frac{15\cdot25}{0\cdot48}}\times\sqrt{M}=0\cdot0822\times\sqrt{M}$$

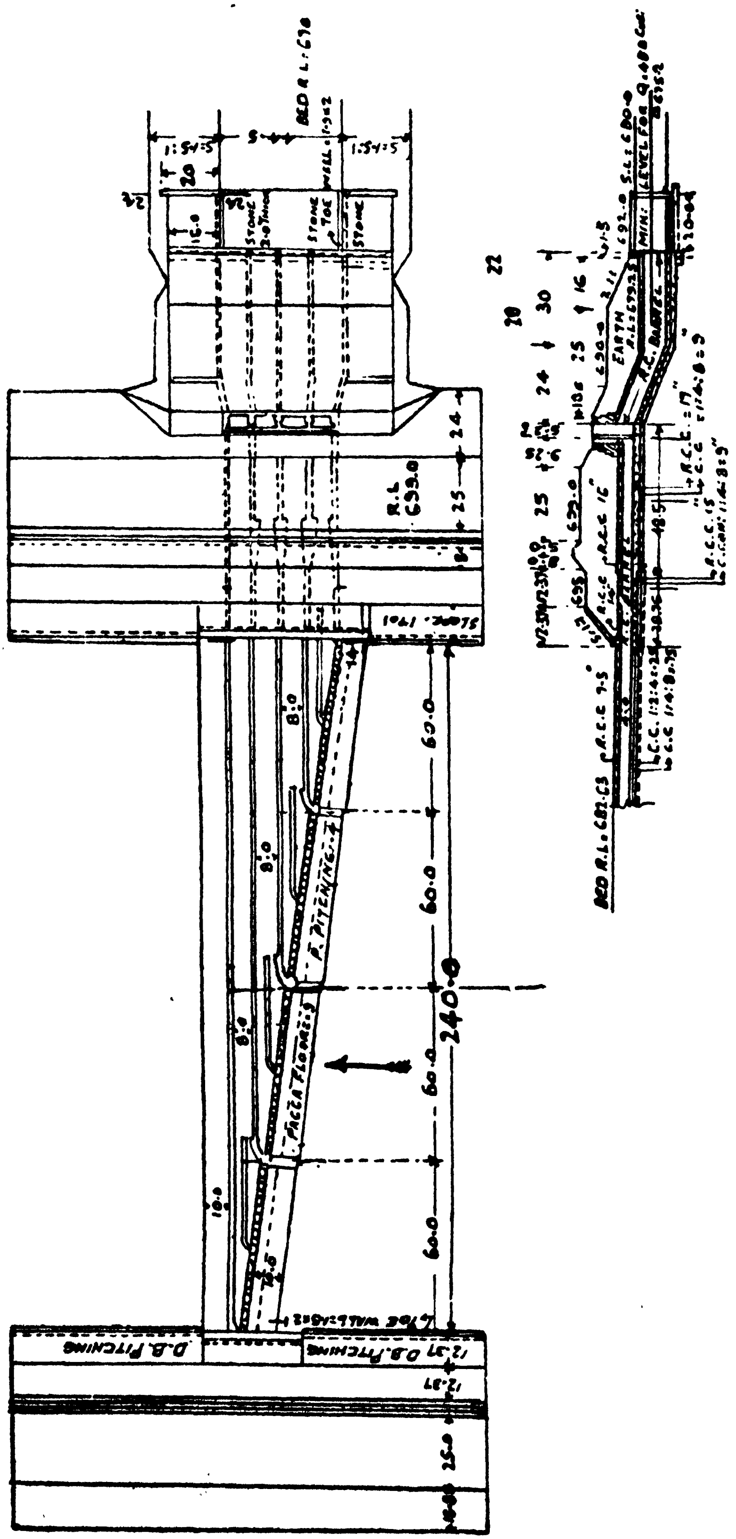
13. Examination Questions.

1. Describe briefly and explain the working of any existing work, which should be named, if possible, for excluding silt at,
(*i*) Canal Head Works ; (*ii*) Branch Regulator ; (*iii*) A Distributary Regulator. (*P. I.B. 1941*)

2. Show by dimensional sketches a design for silt control at a Distributary Regulator taking off 300 cusecs at right angles above a fall discharging 200 cusecs. Explain how your design can be altered to reduce proportion of silt taken by the Disty ; offtake. (*P.I.B. 1941*)

3. Explain any device for reducing the in-draft of silt into the head of a canal or disty. Also explain any from of siltometer. (*P.I.B. 1938*)

4. A channel is found to be silting badly in its head reach. Describe how the defect would be studied and remedied if due to a defective head. (*P.I.B. 1935*)



5. What are the various ways of controlling excessive silt entry in (*i*) Canals (*ii*) Branches (*iii*) Distributaries (*iv*) Minors (*v*) Outlets. What would be an ideal irrigatian channel in respect of silt transportation and to what particulars it should conform. (*P.U. 1952*)

6. Write short notes on silt excluder and silt ejector. (*P.E. 1952-56*)

7. What is silt excluder. Sketch an excluder at the head of a main canal and indicate on a cross section the principles of its working. River discharge in the pocket 50,000 cusecs canal discharge 12,500 cs., excludes discharge 2,000 cs. (*P.U. 1953*)

8. Distinguish between silt excluder and silt ejectors. (*P.U. 1957*)

9. Differentiate between silt excluders and silt ejectors. Sketch neatly a canal ejector showing its various components clearly. Give the complete basis of design for the ejector. (*P.U. 1958*)

Fig. 8.

PART II

CANAL IRRIGATION

CHAPTER XII

Fluming and Headless Meters

1. Definition.

A flumed work is one, built in a stream, the waterway of which is reduced below the normal. It is necessarily accompanied by an increase in velocity.

This result may be obtained in a variety of ways.

(*a*) **A reduction of width.**

The discharge per foot of the width necessarily increases. This may be accompaied by an increase or even reduction of the depth. Examples of this type of flumes are lined channel section, contraction at the aqueducts and the flumed bridges.

(*b*) **A reduction of depth.**

This may be accompanied by a constant width. Works falling in this category are weirs, flumed or contracted falls and headless meter flumes.

A few examples of the design are given here and the student should refer to A.M.R, Montagu's paper No. 6, Central Board of Irrigation, Simla and F.H. Burkett's Paper No. 125. Punjab Engineering, Congress 1929, for a detailed study.

2. Classification of Flumes.

Flumes may be divided primarily into two main classes :—

Class 1. Flumes with free water surface *i.e.*, a water surface open to the atmosphere. In such cases the pressure on the free surface remains constant.

Class II. Flumes with "sealed" water surface. *i.e.*, of which a roof forms part of the desigen

The essential hydraulic difference between these two types is explained below :—

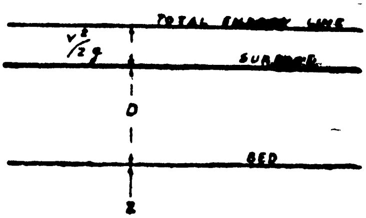


Fig. 1

In class I, the surface is open to the atmosphere and the pressure on the suaface remains constant. Also the area of the cross section is free to alter with the change is velocity, Fig. 1. Consequently, Bernouilli's equation in the special form $Z+D+V^2/2g=C$ remains sensibly true if the effects of curvature are neglected.

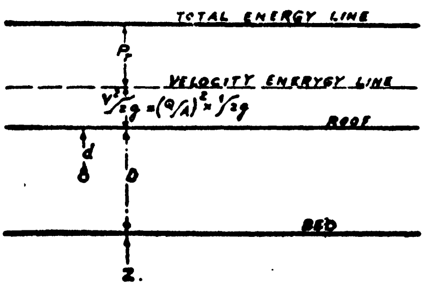


Fig. 2

But in the case of Class II the surface is not free. Provided the surface pressure does not fall below zero (when cavitation takes place) the above equation has to be modified. (Fig. 2)

Consider a particle of unit mass at a depth D below the surface moving with velocity V.

The potential energy$=Z+D-d$

Kinetic energy$=V^2/2g$

Let P_r be the pressure on the roof. Then the pressure on the particle$=P_r\times d$; $\therefore$ the total energy per lb. of the particle, measured in "feet head of water"

$$=Z+D-d+\frac{V^2}{2g}+P_r+d=Z+D+\frac{V^2}{2g}+P_r$$

Since total energy remains constant (save for the reduction by friction, etc., represented by the slope of total energy line), it follows that :—

If the velocity energy in "feet head of water" be plotted above the roof, the pressure thereon is represented by the intercept between the velocity energy line and the total energy line.

3. Bed Profile.

(A) **General Conditions of flow.**

Water has inertia. It follows that any sudden application of any external force will bring the laws of impact into play.

All forces tending to alter the direction of flow must therefore be applied gradually if impact and the consequent loss of energy are to be avoided. The gradual application of a force is followed by a change of velocity or direction—the stream lines are curved.

It would appear that this curvature may be increased by increasing the external impressed force upto a limit at present unknown.

If this limit be exceeded, the nature of flowing water exhibits the property of trying to correct the improper curvature, by the interposition of a layer of minute eddies, in the form of horizontal or vertical rollers. The eddies consume energy and the total energy of the stream is reduced.

If the curvature be still further increased, the eddies are on longer effective in correcting the curvature of the boundary, the stream lines break down and pure impact results.

A vertical obstruction across a channel in the from of a wall will produce horizontal rollers both upstream and downstream of the obstruction. If the obstruction is excessive, there will be pure impact against the wall and also between water passing over it and that moving slowly ahead of it. It implies that all changes in the profile must be gradual.

(B) The forces acting upon a stream of flowing water are :—

(*a*) Gravity, which is constant in value, direction and sense.

(*b*) Friction, the value of which varies with the area of wetted envelope, the velocity of the stream always acts against the direction of flow.

(*c*) The reaction of the envelope, which (when separated from friction) acts normally to the surface of the envelope at the point. Its value depends on a number of factors.

It is the variation in the reaction of the envelope that causes curvature.

(C) **Ideal bed profile.**

The change in section should be such that the jet remains adherent so that vertical component of the reaction at any point is equal to the pressure due to the depth at that point. Although the qualitative effects of curvature in the stream lines are well known, our knowledge of the quantitative effects ; that is the equations defining the fundamental laws, is practically nil.

Consider a rectangular open channel of which the bed rises and falls while the width remains constant. Ideal change in section would be in the form sketched in Fig. 3. It is combination of double S curve to avoid all corners. The stream line upstream of the crest shall be curved upwards connoting vertical component of flow due to centrifugal effect. The reaction shall be less than that due to the depth of water at any point. The water surface profile shall be convex. The convexity connotes that the jet is not adherent and is liable to pressure inflations. A very easy approach upstream of the crest is, therefore, required. On the down stream side the stream lines are concave and, therefore, adherent.

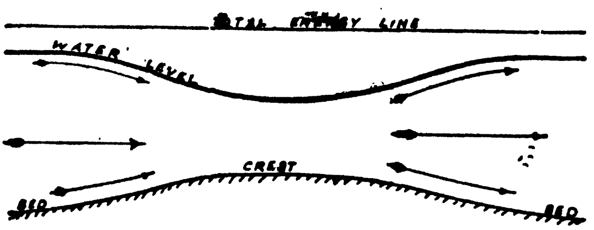


Fig. 3

The convexity upstream of the crest, can be removed (or rather reduced if friction be

Fig. 4

taken into account) by providing an easy uniform negative slope. From experience one would suggest a suitable slope from 1 in 3 for low velocities to 1 in 5 for high velocities. In the case of the uniform slope, the corners need to be transitioned by introducing short circular curve tangential to both planes as shown in Fig. 4.

The upstream curve to crest with radius 2H as introduced by Crump in meter flume design looks, no doubt graceful, but the effect of curvature in producing centrifugal effect is very pronounced as calculated by the author in his article in Indian Engineering Calcutta, December 1935. On the downstream side S curve shall be ideal and the increase in depth may be arranged even in a relatively shorter distance for sub-critical stream and hypercritical stream the Montagu's design of bed profile is ideal. (Para 12 Chapter XI)

4. Approach or Contraction Upstream on Sides.

In this case, a curvature on the side, will no doubt introduce the centrifugal effect which usually results in heaving up of the surface about the middle of the stream and which also accounts for the concentration of the silt charge about its middle, but the jet is perfectly adherent to the sides due to the inertia or the momentum of the approaching water. There is no chance of the reaction against the side-wall being less than that due to the depth against it. There is no possibility of the forming of vertical rollers which usually occur in the expansion downstream of the crest.

The design of the approach should aim at the following requirements of a suitable approach :—

(*a*) There should be no sudden change in section to produce the loss of head in entry.
(*b*) The change in section should be gradual to avoid direct impact against the wall.

There is no mathematical solution yet possible to work out the conditions necessary to fulfill the above-mentioned requirements. One could only say that to avoid impact, the forward component of the incoming water should be equal to the centrifugal effect so that the reaction against the wall is not changed. However, from experience it can be suggested that a splay of 1:2 for low velocities and 1 : 3 for high velocities will do for the approach on sides. In plan it shall be of the form as sketched in Fig. 5.

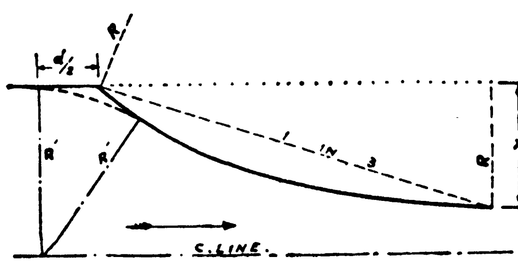


Fig. 5

An ideal curve would be with infinite radius in the beginning and the end as shown in Fig. 6.

However, it is not possible to design such a curve which may be laid out with precision by an average Engineering Subordinate. It will be enough to design a circular curve as shown in Fig. 6.

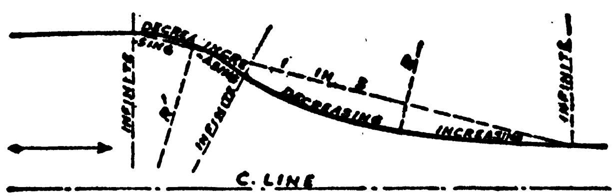


Fig. 6

The radius of a circular curve can easily be worked out assuming it to be tangential at the point where the cantraction is complete. The curve needs to be transitioned in the beginning for a short distance, as shown above, to avoid loss of head in entry.

5. Departure or Expansion on Sides Downstream of the Contraction.

(A) Expansion and dispertions of a stream of water is another Hydraulic Phenomenon which has received the scantiest consideration of mathematicians. Even experimentally little work appears to have been done on the subject.

An ideal expansion would be when the stream is just adherent on the sides. If it is less than the ideal, the jet will no doubt be adherent on the sides ; but the work will be expensive. If it is more rapid than needed, the jet will not be adherent and low pressure pockets near the side will cause the formation of vertical rollers with the consequent loss of energy and with the eddy-motion causing scour on sides in the earthen section as the stream leaves the *pacca* work. The reverse flow on sides is the clearest indication of a too rapid expansion.

(B) Forces acting.

The forces acting on a stream flowing in a section are as given below :—

(*i*) **Reaction.** The reaction of bed and the sides is opposed by the internal pressure of the water which can easily be computed at any point of the perimeter. In a rectangular section, the reaction against the bed will be equal to the pressure due to depth, and against the sides it will be variable according to the depth. In an ideal expansion the jet should remain adherent to the side. Any factor tending to increase the bed reaction will help the lateral dispersion.

(*ii*) **Friction.** The effect of friction can be depicted by giving a downward slope to the total energy line. The effect of friction on total energy line can easily be calculated. Friction always acts against the direction of flow. It may be considered as a component (resolved along the bed) of the reaction. Its effect is to deflect the reaction against the direction of flow and the total value of reaction. The net effect of friction is to increase the rate of lateral expansion. In a frictionless channel, lateral expansion of the jet or dispersion will be nill on this account.

(*iii*) **Gravity.** A mass of water (M) moving with a velocity V has a momentum $\frac{MV}{g}$. The change of momentum per second is a measure of the force which a stream is capable of exerting. Any structural change in a channel section tending to retard the inertia shall produce a change in the internal pressures and the consequent dispersion of the jet. The inertia acts in the line of the direction of the motion of the mass. Velocity on the line of flow has no bearing on the inertia transverse to the line of flow unless there is something to change the line of flow with the consequent change in the internal pressures.

In a hypercritial jet leaving a crest of a free fall weir, forward momentum is predominent, and there is no lateral expansion, even though the side be suddenly removed. There is no lateral expansion of the jet, if it follows its natural parabolic path, but if the bed profile is flatter than the parabola of the dropping jet, there is lateral depression because the line of the velocity has been changed which results in redistribution of internal pressures. The dispersion in hypercritical jet will be very slow.

If the stream leaving the crest is subcritical, the parabola of drop will be steeper still with this low velocity. The departure introduced by a slope will be more pronounced resulting in the relatively greater depression. Moreover, the forward force due to momentum is in this case less than the side pressure, the jet will expand adherent to the side. Much more rapid expansion or dispersion of a subcritical stream is, therefore, possible.

The water surface profiles are governed by the conditions of flow as descrided in Chapter IV Part VI. The effect of friction and the bed slope flatter than the theoretical parabola of dropping water is essentially to disperse the jet laterally. The condition of flow may be subcritical or hypercritical with level, downward (positive) slope and upward (negative) slope ; but the expansion against negative slope cannot in practice be arranged unless the surface is curved. The dispersion necessarily means the reduction of the discharge intensity with the consequent reduction in the critical depth.

(C) Conclusions.

(*a*) The greater the discharge intensity, the higher the velocity and the less the dispersion in both subcritical and hypercritical conditions. A stream flowing with lower initial velocity will expand its sross-sectional area by a given amount in a shorter length of the work than the stream flowing with higher velocity, other things being equal.

(*b*) The flatter the bed slope in the outfall, the greater the dispersion in both the subcritical and hypercritical flow.

(*c*) With hypercritical stream leaving the crest, the radius of curvature in the outfall should be infinite at the beginning of the lateral expansion. The radius of curvature may decrease as the expansion proceeds depending on the first two conclusions.

A straight line expansion, however gentle, is not satisfactory, because a stream will not adhere both in the beginning and at the end. In the case of subcritical flow, a circular splay tangential at the contraction is likely to serve the purpose, if a decreasing radius connot be arranged.

(*d*) In the case of flumed falls, the expansion should be nil or very slow upto the jump of the glacis, but may be rapid after the jump in the subcritical flow. The roughening of the bed by artificial device such as friction blocks will help the rapid dispersion.

(D) Length of Expansion.

(*a*) The factor that remains unknown is the length. Owing to the existing ignorance of the manner in which the varying pressures act on the inertia of the mass, the acceleration of the lateral expansion cannot be predicted at present. Experience alone must be the guide in determining the important factor of length. The departure splay is of the order, varying from 1 : 5 to 1 : 10 as usually allowed in practical designs.

(*b*) F. H. Burkett dealt with the question of expansion from consideration of pressures alone in his paper No. 125 Punjab Engineering Congress, Lahore, on headless meters as given below :—

"Now it appears obvious that a change in the direction of flow along the side wall is caused by water pressure at right angles to the latter. It should, therefore, be our aim to keep this pressure constant and as the velocity of water drops, the radius of curvature of the side wall should decrease. As shown in diagram (Fig. 7) RA^2 should remain constant, where R is the radius of curvature of the side walls and A is the area of the waterway."

"If the floor downstream followed a constant slope, a differential equation could be obtained connecting distances downstream of the crest with off-sets to the side-walls, but in practice, this is not possible as the question of critical velocity upsets the uniformity of the floor gradient. Fig. 7 shows how these walls may be laid out for any initial radius."

$$\sin \phi_1 = \frac{X}{R_1}$$

$$\sin \phi_2 = \frac{2X - (R_1 - R_2)\sin \phi_1}{R_2}$$

$$\sin \phi_N = \frac{NX - (R_1 - R_2)\sin \phi_1 - (R_2 - R_3)\sin \phi_2 - \cdots - (R_{N-1} - R_N)\sin \phi_{N-1}}{R_N}$$

$$Y_1 = X \tan \frac{\phi_1}{2}$$

$$Y_2 = X \tan \frac{\phi_1 + \phi_2}{2}$$

$$Y_N = X \tan \frac{\phi_{N-1} + \phi_N}{2}$$

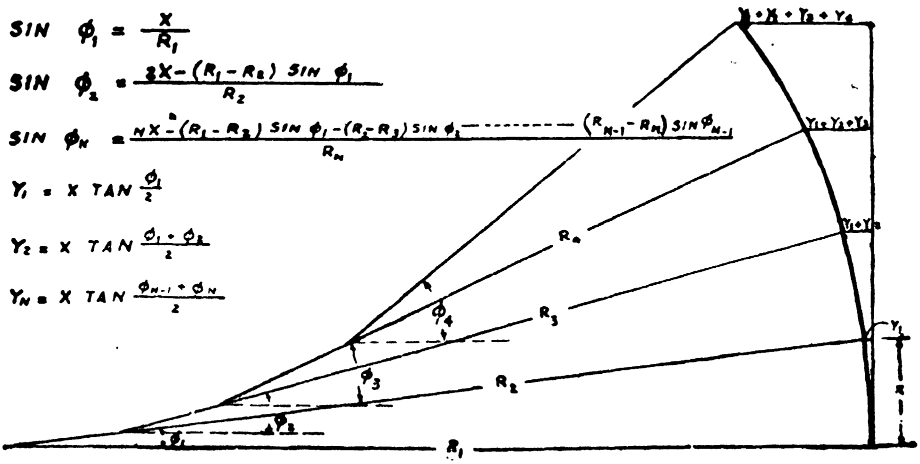


Fig. 7.

(E) The Author's Method.

(*i*) The rate of expansion is to be determined from the garph given in Fig. 8. This will fix the length of the outfall.

In the case of subcritical flow downstream of the contraction, as usually available in the case of flumed bridges or headless meters the expansion may be circular, tangential to the contraction.

In the case of the falls when water leaves the crest with hypercritial flow, is then followed by a jump on the glacis and lastly the subcritical flow, an ideal shape of an expansion is Bernauillis Lemmiscade as used in the design of the roof block of the A. P. M. outlets. It has got an infinite radius in the beginning and then it decreases towards end of the expansion as shown in Fig. 9.

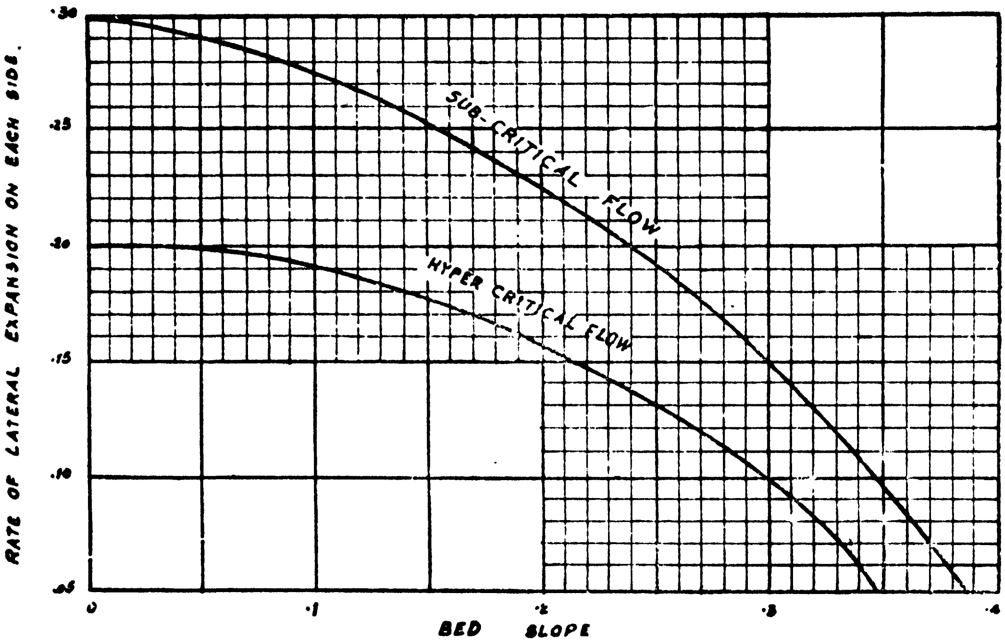


Fig. 8.

The curve recommended is the one formed by bending a uniform flexible strip of some homogeneous material (such as steel or celluloid) in manner indicated above. Knowing the length of expansion as determined by its rate of change as fixed beforehand, the curve could be drawn in a plan to a reduced scale and the percentage departure read at suitable distances from the beginning and then accordingly laid at site.

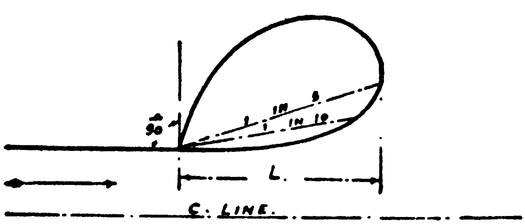


Fig. 9

(*ii*) **Flaring.** In flumes, long vertical walls are not only very expensive, but also they cause considerable reduction in waterway even when they are built at the toe of the side slope.

The usual side slope allowed in pitching is ½ to 1 in distributaries and minors and 1 to 1 in Branch and Main Canals. The change of vertical section to the normal means considerable contraction upstream and expansion downstream. It is, therefore, usual to flare the side wall from vertical at the crest to the permissible side slope at the beginning of the contraction upsteam and that at the end of expansion downstream. The typical section of flared walls are shown in plate XIX, Vol. III.

The section (*b*), (*c*), (*d*) as compared with section (*a*) show considerable saving in the section of masonry and concrete. The saving of cost in a long wall is about 40%.

7. *Example No : 1.* **Flumed Aqeduct.**

A channel with the following data has to pass in a masonry aqueduct flume with vertical sides with N=·013 with no loss of head (with the slope of the channel). Design the fluming.

F. S. Discharge 700 cusecs; Full Supply level 549·6 ; Bed width 60 feet ; Slope 1 in 6666 ; Bed level 594·5 ; Side slope ½ to 1 ; Depth 5·1 ft.

(*a*) Assume width at the contracted section as 42 feet: Sectional area=A=5·1×42 =214·2 sft. ; Perimeter=42+2×5·1=52·2 feet.

$R = H.\ M.\ D. = \frac{A}{P} = \frac{214 \cdot 2}{52 \cdot 2} = 4 \cdot 1$ feet.

Using Manning's formula :—

$V = \frac{1 \cdot 485}{N} R^{2/3} S^{1/2} = \frac{1 \cdot 485}{\cdot 013} \times 4 \cdot 1^{2/3} \times (\frac{1}{6666})^{1/2} = 3 \cdot 3$ ft./sec.

Discharge $= 42 \times 5 \cdot 1 \times 3 \cdot 3 = 706$ cusecs, which is nearly the same as required, 700.

(b) Length of contraction upstream allowing 1 : 3

$= 3 \times \frac{60-42}{2} = 27$ feet. Radius of circular contraction $R = (2R-9)9 = 27^2 \therefore R = 394$ feet.

(c) The departure downstream 1 in $5 = 5 \times \frac{60-42}{2} = 45$ ft.

Radius of circular expansion $= (2R-9)9 = 45^2 \therefore R = 117$ ft.

(d) The walls shall be flared both upstream and downstrearm to $\frac{1}{2}$ to 1 side slopes in Fig. 10.

8. *Example II.* **Flumed Bridges.** (Montagu)

It is required to flume a channel with the data given below without exceeding the critical velocity to a width of 10 ft. which is fixed for an existing railway bridge. Find the level of the bottom of girders allowing a freeboard of 1·5 ft. with 100 as bed level of the earthen channel.

Discharge 250 cusecs; Slope 1 in 4000; Bed 25 feet; Depth 4·1 feet; Velocity 2·3 ft. per second; sides $\frac{1}{2}$ to 1.

Calculations.

(a) Throat under the bridge with vertical sides. The discharge intensity $= 250/10 = 25$ cs. Critical depth $= \sqrt[3]{(25)^2/g} = 2 \cdot 69$. The actual depth must remain well above the critical by at least 10% say not less than 3·0 feet.

(b) Upstream side contraction 3 :1 ft. Let it be 1 in 3 at bed level.

The length of upstream approach $= 3 \times \frac{25-10}{2} = 22 \cdot 5$ ft.

Radius $= (2R - 7 \cdot 5)\ 7 \cdot 5 = 22 \cdot 5^2$
$\therefore R = 37 \cdot 33$ feet.

The wall shall be flared to $\frac{1}{2}$ to 1 slope with 1 foot freeboard.

The radius of top of wall ; contraction each side $= 7 \cdot 5 + \frac{5 \cdot 1}{2} = 10 \cdot 05$ feet.

$R = (2R - 10 \cdot 05)\ 10 \cdot 05 = 22 \cdot 5^2 \therefore R = \frac{22 \cdot 5^2 + 110 \cdot 4}{20 \cdot 1} = \frac{615}{21} = 30 \cdot 0$ feet nearly.

(c) Downstream Departure.
Let the side splay be 1 in 5 at bed level;

Length of the outfall $= 5 \times \frac{25-10}{2} = 37 \cdot 5$ ft.

Radius at bed level $= (2R - 7 \cdot 5)\ 7 \cdot 5 = 37 \cdot 5^2 \therefore R = 97 \cdot 73$ feet.
The wall shall be flared to half to one slop with 1·0 ft. freeboard. The expansion at top level $= 7 \cdot 5 + \frac{4 \cdot 1 + 1}{2} = 10 \cdot 05$.

The radius at top level $= (2R - 10 \cdot 05)\ 10 \cdot 05 = 37 \cdot 5^2 \therefore R = 75 \cdot 2$ feet.

(d) Throat length.
This depends upon the straight portion required for the bridge, let it be 10 feet.

Three methods of solution are possible in this example, only one of which will be illustrated here.

(*i*) The bed may be sketched in, and wing walls designed therefrom.

(*ii*) The wing walls may be fixed and bed designed therefrom.

(*iii*) Sketch in the water surface and design the bed and the sides.

It is proposed to illustrate the second method here.

(*e*) From Ef_2 Diagram of Montagu ; Plate VII, Vol. III.

Discharge 25 cusecs; Depth of 3·0 ; Ef_2 4·08 ft. Total energy level in the open channel

$= 100 + 4{\cdot}1 + V^2/2g = 104{\cdot}1 + 2{\cdot}3^2/2g = 104{\cdot}18$

Bed level in the throat $= 104{\cdot}18 - 4{\cdot}08 = 100{\cdot}10$;

Full Supply Level $= 100{\cdot}10 + 3 = 103{\cdot}1$

The bed need to be raised by 0·1 ft. only which is trifling.

Now work out the design with no change in bed level.

Read depth for $Ff_2 = 4{\cdot}18$; Depth $= 3{\cdot}28$; Water level $= 100 + 3{\cdot}28 = 103{\cdot}28$.

The bottom of girder $= 103{\cdot}28 + 1{\cdot}5 = 104{\cdot}78$

(*f*) Friction loss.

Velocity in contracted sec. $= \dfrac{25}{3{\cdot}28} = 7{\cdot}64$ ft./sec.

Velocity in the beginning $= 2{\cdot}3$;

Average velocity $= \dfrac{7{\cdot}64 + 2{\cdot}3}{2} = 5{\cdot}0$ ft. nearly

H.M.D. in the beginning ; area $= (25 + 4{\cdot}1/2)4{\cdot}1$ $= 110{\cdot}9$ say $111{\cdot}0$

Perimeter $= 25 + 4{\cdot}1 \times 2\sqrt{1 + (\cdot 5)^2} = 34{\cdot}17$;

$R = \dfrac{111{\cdot}0}{34{\cdot}17} = 3{\cdot}26$ ft.

H.M.D. in the contracted section

$= \dfrac{10 \times 3{\cdot}28}{10 + 2 \times 3{\cdot}28} = \dfrac{32{\cdot}8}{16{\cdot}56} = 1{\cdot}97$ ft.

Average H.M.D. $= \dfrac{3{\cdot}26 + 1{\cdot}97}{2} = \dfrac{5{\cdot}22}{2} = 2{\cdot}61$ ft.;

For $N = {\cdot}015$ in Manning's formula with $R = 2{\cdot}61$ and $V = 2{\cdot}0$ from Plate XIV; Vol. III.

$S = \dfrac{1}{1400}$

Flumed Bridge Scale 1/200

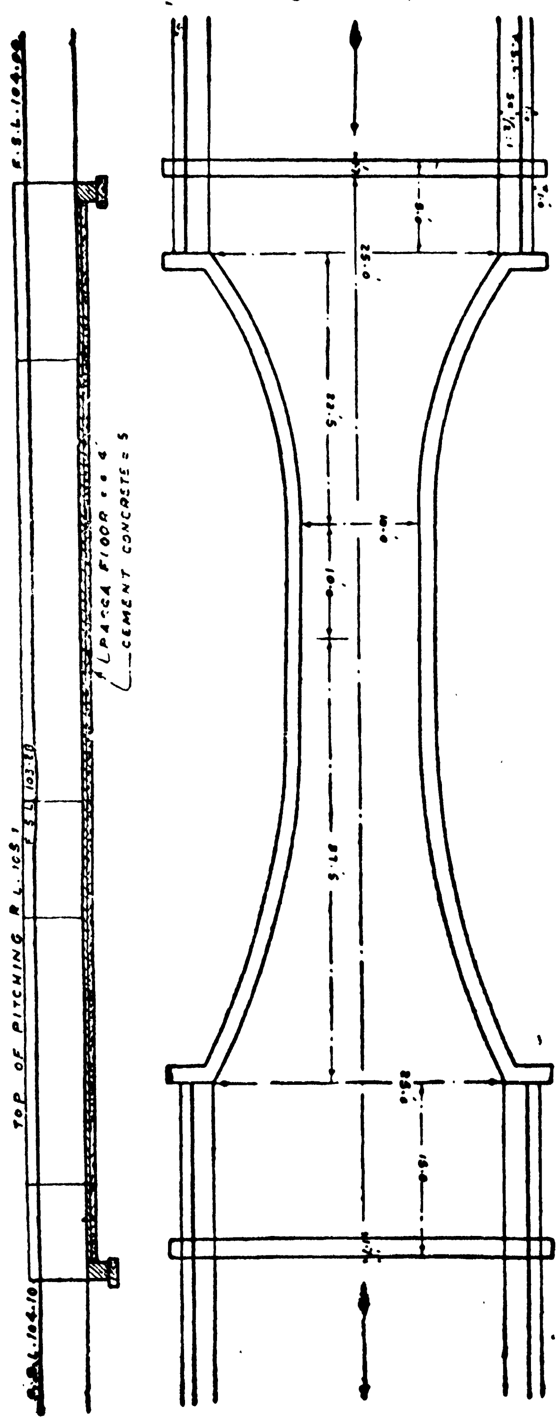


Fig. 10.

Loss in approach $=\frac{22\cdot5}{1400}=\cdot016$ ft.

Loss in outfall $=\frac{37\cdot5}{1400}=\cdot027$; Rate of loss in throat for $R=1\cdot97$ and $V=7\cdot64$

$S=\frac{1}{600}\times10=\cdot016$; Net Loss $=\cdot06$ ft. Say 0·1 ft.

The downstream water level may be shown 0·1 ft. lower. The total energy line will have slope of 1/1400 in approach and outfall and of 1/600 in the throat.

This slope is rather very small and there will be no change in the design.

9. *Example III.* **Contraction with Depressed Floor.** (Montagu)

In the above example, contract to 5 ft. width with vertical side walls (A.M.R. Montagu's Solution C.B.I. publication No. 6). Then mean velocity energy $E_v=V^2/2g=0\cdot082$.

Mean value of $E_t=4\cdot182$. Assuming the wing walls of the approach and departure to be vertical, *i.e.*, the section rectangular, then on the section at entrance to approach transition. Fig. 11.
$Q_o=250$ cusecs ; $B_o=25$ feet ; ∴ discharge per foot width $q=10$ cusecs ;
$E_t=4\cdot182$; ∴ $D_o=4\cdot9$ feet ; and $V_o=2\cdot44$ feet per second.

The throat. Assume a fluming ratio of one-fifth width of throat, $B_t=5'$; Discharge per foot of width, $q=50$ cusecs.

The velocity is not to exceed the critical velocity. Experience dictates that the designer should keep well clear of the critical point.

Examine the curve appropriate to the value of $q=50$ cusecs. Select a point thereon, well clear of the locus of the critical point. Let the point selected have the co-ordinates.
$D_t=7\cdot0$; $E_{tt}=7\cdot8$

Then the line of the bed may be set off downwards from the total-energy line, 7·8 ft. and the line of the surface set up from the bed, 7 feet ; $V_t=q_t/D_t=7\cdot14$. This is satisfactory.

The transitions :—Begin by assuming an approach transition of 1 : 2 splay in plan. Its length will be :—$L_a=B_o-B_t/2\times\frac{2}{1}=20$ feet.

∴ The slope of the bed from level of normal channel to level of the throat will be :—
$(E_{tt}-E_{to})/L_a=18\%$. The combination of approach splay and bed slope appear satisfactory and may be adopted.

Now assume a departure transition of 1 : 3 splay in plan its length will be,

$L_a=\frac{B_o-B_t}{2}\times\frac{3}{1}=30$ feet. and the slopefrom level of the throat to level of normal

channel will be :— $\frac{E_{tt}-E_{to}}{L_d}=12\%$.

Let the length of throat $L_t=10$ feet.

The design (resumed)—Refer now to Fig. 11. Assume any number of sections XYZ etc. on the plan and elevatoin and proceed as follows, to construct the tabular statement shown at the bottom of the plate. The total discharge Q remains constant.

Measure the successive bed-widths B on sections XYZ etc., and record them in the table.

The discharge per foot width q at the successive sections equals Q divided by B at each section. Calculate and record.

The value of E_t appropriate to each section is determined by the intersection of the appropriate q curve with the "curve of transition characteristics". Record this.

The depth at the point is read off the Hydraulic Diagrams at the same time as E_t and recorded.

The R. L. of the bed is determined by substracting the value of E_t from the R. L. of the total energy line at the section. Fig. 11.

The R. L. of the surface is Determined by adding the value of the Depth to the R. L. of the bed at the sections. Fig. 11.

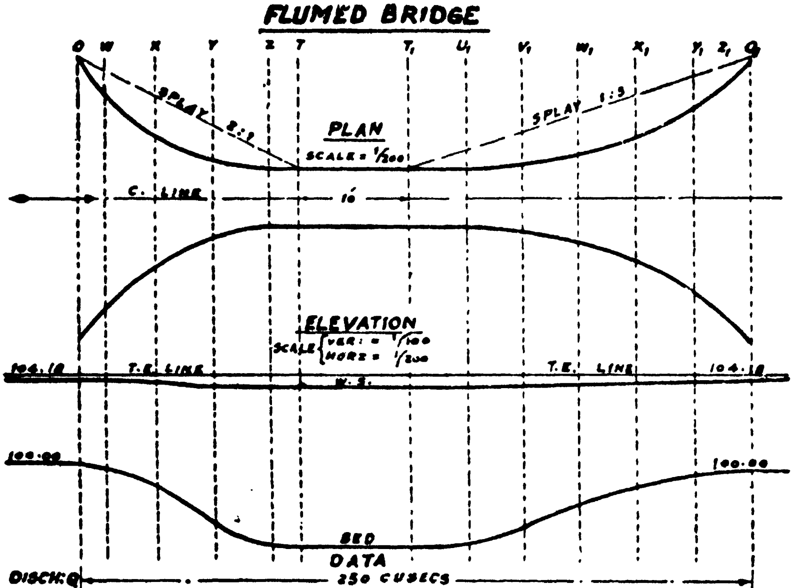


Fig. 11.

	O	W	X	Y	Z	T	T1	U1	V1	W1	X1	Y1	Q
BED WIDTH	25.0	20.0	11.4	6.80	5.10	5.00	5.00	5.20	6.35	8.70	12.2	17.5	25.0
q	10.0	12.5	21.9	36.8	49.0	50.0	50.0	48.0	40.0	28.7	20.5	14.3	10.0
Ef	4.18	4.38	5.29	6.76	7.84	7.88	7.88	7.84	6.90	5.89	5.15	4.54	4.18
DEPTH	4.09	4.27	5.00	6.15	6.90	7.00	7.00	6.90	6.27	5.46	4.88	4.40	4.09
R.L. W.S.	104.09	104.07	103.89	103.57	103.46	103.38	103.38	103.40	103.55	103.75	103.91	104.04	104.09
R.L. BED	100.00	99.80	98.89	97.42	96.56	96.38	96.38	96.50	97.28	98.29	99.03	99.64	100.00

10. *Example IV.* **Headless Meter Flume.** (Burkitt's paper No. 125 Punjab Engineering Congress)

(*a*) The theory is simple. If at two section in a stream close togather, where there are no sudden changes to upset a stream line flow, the breadths are b_0 and b_1 and the depths y_0 and y_1, the depression in water surface between the two sections being h, the discharge Q is given by the equation :—

$$Q=8{\cdot}025\, b_0.\, y_0.\, y_1. \left\{\frac{h}{b_0^2/b_1^2(y_0^2-y_1^2)}\right\}^{\frac{1}{2}}$$

$$\text{For :— } V_1^2-V_0^2=2gh\ ;\ \therefore \frac{Q^2}{A_1^2}-\frac{Q^2}{A_0^2}=2gh\ ;\ \therefore Q^2=\frac{2gh\, A_0^2A_1^2}{A_0^2-A_1^2}$$

$$\therefore Q=8{\cdot}025\, b_0.y_0.b_1.y_1. \left\{\frac{h}{b_0^2y_0^2-b_1^2y_1^2}\right\}^{\frac{1}{2}}=8{\cdot}025\, b_0.y_0.y_1 \left\{\frac{h}{b_0^2/b_1^2\ (y_0^2-y_1^2)}\right\}^{\frac{1}{2}}$$

(*b*) Headless meter at R. D. 103676 Mithalak Distributary.

F. S. Discharge$=38{\cdot}1$ cusecs ; Bed level$=\frac{622{\cdot}15}{622{\cdot}05}$

F. S. L.= $\frac{624 \cdot 85}{624 \cdot 75}$. Allow a loss of 0·1 ft.
Bed width=10 ft. ; Sides—½ to 1 ; N. S.—622.74
Velocity 1·3 ft. per second ; Depth 2·7 ft.

(*c*) **Calculations :—**

Velocity of approach=1·3 ft. per second

Velocity of approach head $=h_a=\frac{V^2}{2g}=\frac{1 \cdot 3^2}{64 \cdot 4}=\cdot 03$ ft.

let the crest be level with the downstream bed ; Depth upstream =2·8 ft.

Depth downstream=2·7 ft. ; Drowninng ratio= $\frac{2 \cdot 7}{2 \cdot 8}=\cdot 964$

The co-efficient C for a drowned weir, from Fane's curve Plate IV, for this drowning ratio=2·32. Let B be the width at the maximum contraction.

$$B=\frac{Q}{CH^{3/2}}=\frac{38 \cdot 1}{2 \cdot 32 \times 2 \cdot 8^{3/2}}=3 \cdot 52 \text{ ft.}$$

(*d*) Width opposit upstream gague for nonsilting conditions ,—
Depth=2·8 ft. $V_0=\cdot 84\, d^{\cdot 64}=\cdot 84 \times 2 \cdot 8^{\cdot 64}=1 \cdot 63$ say 1·7 ft./sec.

$$\text{Width}=\frac{Q}{DV}=\frac{38 \cdot 1}{2 \cdot 8 \times 1 \cdot 7}=8 \cdot 0 \text{ ft.}$$

There will be one gauge where width is 8·0 ft. and one where width is 3·52 ft.

(*e*) Draw-down to upstream gauge.
Total energy depth=2·8+·03=2·83 ft.=H ; Let depth at the first gauge well=Hp.
The drop $h_1=(1-p)H$; ∴ Velocity $=\sqrt{2gh_1}$;
Discharge per foot $=pH \times \sqrt{2g}\ \sqrt{(1-p).\ H}=p\sqrt{1-p}.\ \sqrt{2g}\, H^{3/2}$;

$p\sqrt{1-p}=\frac{38 \cdot 1}{8} \times \frac{1}{\sqrt{2g} \times (2 \cdot 83)^{3/2}}=\cdot 1265$. The solution of this equation is got from Brown curve Plate IV in Punjab Engineering Congress Paper 125, 1929 ; but it can easily be got by the use of slide rule.

"Set the reversed slide to ·1265 on the D scale and then read under the curser on the A and C scale so that total of p and 1—p is one."
∴ 1—p=·017. Draw down $=h_1=\cdot 017 \times 2 \cdot 83=\cdot 0476$ ft.

(*f*) Draw-down to second gauge.
Discharge intensity= $\frac{38 \cdot 1}{8 \cdot 52}$ cusecs per ft. Draw-down $=h_2 \doteqdot (1-p)H$

where $H=d+h_a=2 \cdot 83$ feet ; $p\sqrt{1-p}=\frac{38 \cdot 1}{3 \cdot 52} \times \frac{1}{\sqrt{2g}} \times \frac{1}{2 \cdot 83^{3/2}}$, solving with slide rule, 1—p=·103 ∴ draw-down $+h_a=2 \cdot 83+\cdot 103=2 \cdot 94$.

(*g*) Depression or difference between the two gauge reading
$h=h_1-h_2$; Depression=·294—·0470=247

This will be the difference in the full supply conditions the discharge for other observed values of depression 'h' being according to formula in (*a*) above. The design is shown in Fig. 12.

11. *Example V.* **Raised Crest Headless Meter Flume.** (Author's)

In the last example, the width at both gauge-wells should be the same but the crest be raised at the second gauge-well.

(*a*) Let width=8 ft. opposite upstream gauge.
(*b*) Height of crest

The depth upstream approach $=2{\cdot}80$ ft.
Let crest height be R
Depth on crest $=H-R=2{\cdot}80-R$
Depth downstream $=2{\cdot}7-R$

Drowning ratio $=\dfrac{2{\cdot}7-R}{2{\cdot}8-R}$; Let $R=1{\cdot}4$ ft.

Drowning ratio $=\dfrac{2{\cdot}7-1{\cdot}4}{2{\cdot}8-1{\cdot}4}={\cdot}928$

C from Fane's curve $=2{\cdot}9$
Depth on crest $=1{\cdot}4$ ft. ; $h_a={\cdot}03$
$\therefore$ Discharge $=2{\cdot}9\times8\times1{\cdot}43=38{\cdot}3$ cs. The actual is $38{\cdot}1$ cs. and then $R=1{\cdot}4$ ft. is O. K.

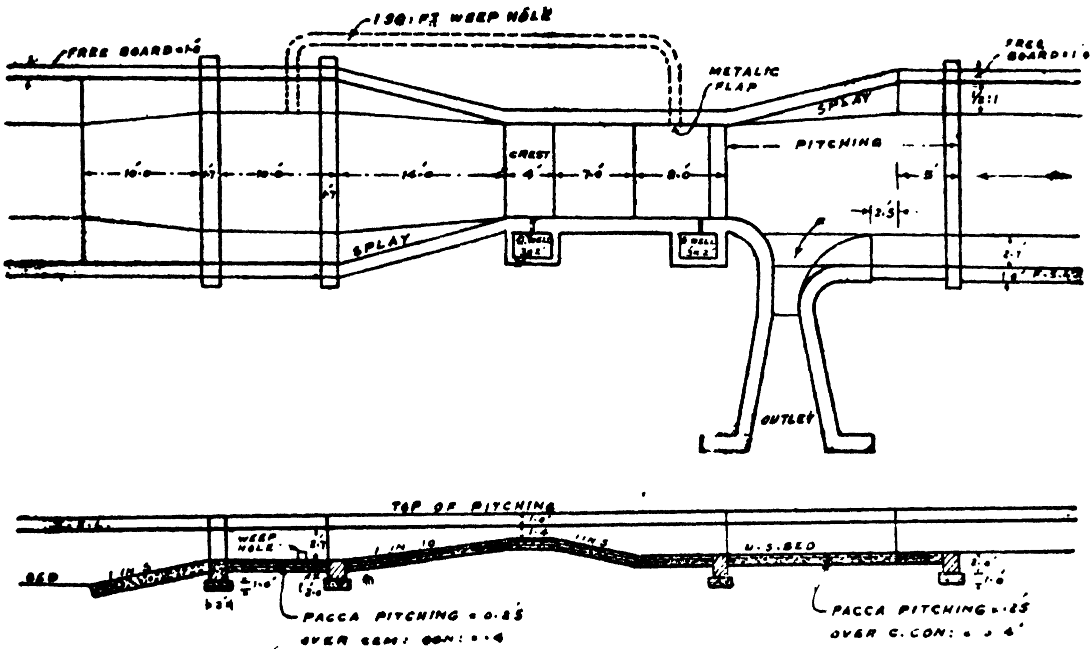


Fig. 12

(*c*) Draw-down to first gauge as before $={\cdot}0476$. Let depth opposite the second gauge be pH, where $H=1{\cdot}43$ ft. $\therefore\ p\sqrt{1-p}=\dfrac{38{\cdot}1}{8{\cdot}025\times8}\times\dfrac{1}{1{\cdot}43^{3/2}}$

Solving with a slide rule $1-p={\cdot}2$
Draw-down to second gauge $={\cdot}2\times1{\cdot}43={\cdot}286$ feet.

(*d*) Depression in the design of full supply $h={\cdot}286-{\cdot}0476={\cdot}2384$ feet.

12. *Example VI.* **Authors's Improved Headless Meter Flume.**

(A) Defects in Burkitt's Headless Meter Flumes

(*i*) Burkitt's Headless Meter Flume applies to vertical sections and therefore, entails expenditure of heavy wall sections.

(*ii*) The correction for velocity of approach is rather very cumbersome, only possible by trial and error from the gauge readings of the first gauge section.

(*iii*) Expensive gauge-wells and inaccurate gauging.

(B) Improvement in Author's Improved Headless Flume.

A flume of this type was designed and constructed by the author at R.D. 120635 of Ghudinala Drain, Upper Jhelum Canal comprising the following special features :—

(*a*) The hydraulics is developed to suit the trapezoidal sections so that the sides need not be vertica'.

(*b*) The design is very cheep with only pitching for the side walls.

(*c*) The gauge wells are suitably located in the central pier divided in two parts. One records the gauge at the contracted section, and the other upstream one is so designed as to record the depth and velocity of approach head serving as an open Pitot gauge for the apstream section.

(*d*) The correction for velocity of approach is automatic.

(*e*) A differential gauge was installed in the form of inverted U tube and water was sucked up from both gauge wells to give a direct reading of the differences that is, 'H' head causing flow between two sections. A vernier scale was provided to give the reading correct to third place of decimal.

(*f*) The formula of Discharge is very simple. If H is the reading of differential gauge, the discharge $Q=A\sqrt{2gH}$; where A is the sectional area at the contracted section.

(*g*) **Data.**

F.S. Channel Data.

$b=19\cdot0$ ft. ; $d=7\cdot0$ ft. ; $s=1\cdot5$ to 1 ; $Q=700$ cusecs.

$A=7\cdot0\ (19\cdot0+10\cdot5)=7\cdot0\times29\cdot5=206\cdot5$ sq. ft. ; $V=\frac{700}{206\cdot5}=3\cdot39$ ft./sec.

$$h_a=\frac{V^2}{2g}=\frac{3\cdot39^2}{64\cdot4}=0\cdot1785 \text{ ft.}$$

Flumed section :—Design for F.S. afflux of 0·3 ft. Try $F=1\cdot0$ (Rise in bed) and side slope 1 to 1

$A=x\ (B+x)$; and $w=B+2x$; $h=\frac{A}{2w}=\frac{x(B+x)}{2B+4x}$

$V=\sqrt{2gh}=8\cdot025\sqrt{h}$; $H=x+h$;

$H+1\cdot0=7\cdot3+0\cdot178=7\cdot478$; $x+h=6\cdot478$

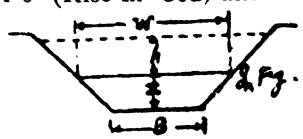


Fig. 13

$$i.e.,\ x+x\frac{B+x}{2B+4x}=6\cdot478\ ;\ x\left(1+\frac{B+x}{2B+4x}\right)=6\cdot478$$

$$x\frac{3B+5x}{2B+4x}=6\cdot478\ (1).\quad \frac{x^{3/2}(B+x)^{3/2}}{\sqrt{2B+4x}}=\frac{700}{8\cdot025}\quad(2)$$

From (1) $3Bx+5x^2=12\cdot956\,B+25\cdot912x$; $B(3x-12\cdot956)=x\,(25\cdot912-5x)$

$$B=x\frac{25\cdot912-5x}{3x-12\cdot956}\quad(3).\ \text{Put (3) in (2) then;}\ \frac{x^2\,(12\cdot956-2x)^{3/2}}{3x-12\cdot926}=123\cdot4$$

(*i*) Let $x=4\cdot6$; Left hand expression $=\frac{4\cdot6^2\,(12\cdot956-2\times4\cdot6)^{3/2}}{3\times2\cdot6-12\cdot956}=182\cdot5$

(*ii*) $x=4\cdot7$; Left hand expression $=\frac{4\cdot7^2\,(12\cdot956-2\times4\cdot7)^{3/2}}{3\times4\cdot7-12\cdot956}=129\cdot4$

(*iii*) $x=4\cdot8$; Left hand expression $=\frac{4\cdot8^2(12\cdot956-2\times4\cdot8)^{3/2}}{3\times4\cdot8-12\cdot956}=98\cdot1$

(*iv*) $x=4\cdot716$; Left hand expression $=\frac{4\cdot716^2(12\cdot956-2\times4\cdot716)^{3/2}}{3\times4\cdot716-12\cdot956}=123\cdot4$

$$B=x\frac{25\cdot912-5x}{3x-12\cdot956}=4\cdot716\ \frac{2\cdot332}{1\cdot192}=9\cdot24 \text{ ft.}$$

(C) **Solution.** $x=4\cdot716$; $B=9\cdot24$

$A=x(B+x)=4\cdot7\ (9\cdot240+4\cdot716)=4\cdot716\times13\cdot956=65\cdot816$

$w=B+2x=9\cdot420+9\cdot432=18\cdot672$; $h=\frac{A}{2w}=\frac{65\cdot816}{37\cdot344}=1\cdot762$

$V=\sqrt{2gh}=8\cdot025\times\sqrt{1\cdot762}=10\cdot65$; $Q=VA=10\cdot65\times65\cdot82=701$ cusecs.

$H=x+h=4\cdot716+1\cdot762=6\cdot478$ correct.

Variation of Working Head. Assume Manning for Channel.

i. e., $V=\frac{1\cdot436}{N}R^{2/3}S^{1/2}$; *i.e.*, $V\times R^{2/3}$; $V=K.R^{2/3}$

$Q=700$; $A=206\cdot5$; $V=3\cdot39$; $P=19\cdot0+3\cdot006\times7\cdot0=19\cdot0\times25\cdot242=44\cdot24$

$R=\frac{A}{P}=\frac{206\cdot5}{44\cdot24}=4\cdot67$ $R^{2/3}=2\cdot794$; $K=\frac{3\cdot39}{2\cdot794}=1\cdot213$; $V=1\cdot213R^{2/3}$

TABLE I

d	A	P	R	$R^{2/3}$	V	Q
1	2	3	4	5	6	7
1·0	20·50	22·61	0·907	0·9366	1·136	23·3
2·0	44·00	26·21	1·679	1·413	1.714	75·4
3·0	70·50	29·82	2·364	1·775	2·150	151·6
4·0	100·0	33.42	2·992	2·079	2·250	252·0
5·0	132·5	37·03	3·580	2·343	2·840	376·4
6·0	168·0	40·64	4·130	2·575	3·124	525·0
7·0	206·5	44·24	4·670	2·794	3·390	700·0

(D) **Section oppositc U. S. Gauge-Well**

The section is arbitrarily selected to be 15 feet wide with slope 1 to 1 and the floor level, *viz.*, one foot raised above earthen channel section upstream with the object that the floor remains clear of slit under all conditions of flow. Actually the discharge is obtained independent of the measurements of the sectional area opposite upstream gauge-well unlike Burkitt's headless Meter flume Fig. 14.

(E) **Flume conditions.**

$A=x(9\cdot24+x)$; $W=9\cdot24+2x$; $h=\frac{A}{2w}$; $V=8\cdot025\sqrt{h}$; $Q=VA$

TABLE II

x	w	9·24+x	A	$h=\frac{A}{2w}$	H=x+H	$V=\sqrt{2gh}$	Q
1	2	3	4	5	6	7	8
·5	10·24	9·74	4·87	0·238	0·738	3·914	19·0
1·0	11·24	10·24	10·24	0·456	1·456	5·42	55·5
1·5	12.24	10·74	16·11	0·658	2·158	6·51	104·9
2·0	13·24	11·24	22·48	0·849	2·849	7·39	166·3
2·5	14·24	11·74	29·35	1·032	3·532	8·15	239·2
3·0	15·24	12·24	36·72	1·205	4·205	8·81	323·2
3·5	16·24	12·74	44·59	1·373	4·873	9·41	419
4·0	17·24	13·24	52·96	1·536	5·536	9·95	527
4·5	18·24	13·74	61·83	1·696	6·196	10·45	646
4·716	18·67	13·96	65·82	1·762	6·478	10·65	701

Working head=floor level+Column 6 of table II—(D.S. bed level+Column 1 of table I)

The flume will work in free fall conditions upto a discharge of 323 giving a working head of 16%.

Calibration :—

Head H from the differential gauge installed in open Pitot Gauge Well which registers the water level at the first section plus the velocity of approach head and also the Depth "x" from the gauge reading in the Gauge Well B. Discharge$=(9\cdot26+x)\sqrt{2gH}$ cusecs.

A difference of gauge reading C and B is denoted by G ; the formula for discharge becomes with $H=G+h_a$

$$Q=\frac{R_1A_2\sqrt{2gH}}{\sqrt{A_1^2-A_2^2}}$$

13. *Example VII.* **Weir Spillway Syphon Flumes.** – Fig. 15.

Conversion of R. Culvert @ R.D. 54,600 of P. R. K. drain into a weir spilway syphon to carry 200 cusecs.

Data.

Section of the existing culvert is as shown in the sketch.
Bed width of drain = 18·0 ft. ; F.S. depth = 4·2 feet ; Bed level = 732·07 ; F. S. L. = 733·27 ; Leagth of culvert is 4·47 feet ; floor level = 736·27 ; Depth above floor level = 736·27—732·23 = 4·04 say 4 ft.

Upstream side.

$Q = 3{\cdot}0 \times 6 \times H^{3/2}$; $154 = 18H^{3/2}$; $H = 4{\cdot}18'$.
At the lowest point of the roof p = 12·5 ; assume depth = x

$$A = 12{\cdot}5x \; ; \; V = \frac{154}{12{\cdot}5x} = \frac{12{\cdot}32}{x} \; ;$$

$$\frac{V^2}{2g} = \frac{12{\cdot}32^2}{x} \times \frac{1}{2g} = \frac{2{\cdot}355}{x^2}$$

But for a free surface;

$$x = \text{depth} = 4{\cdot}18 - \frac{2{\cdot}355}{x^2}$$

Let x = 3·9; R.H. = 4·025; x = 4·04 ; R. H. = 4·031. Position of wave with 154 cusecs.

Q = 154 cusecs ; L = 4·18 + ·16 − 3·59 = 4·34 − 3·59 = ·75 ft.

K + F = 4·18 + ·16 = 4·34

1st. Try W = 11·0 ; $q = \frac{154}{11}$

$$= 14 \; ; \; C = \sqrt[3]{\frac{314^2}{g}} = 1{\cdot}826 \; \frac{L}{C} = \frac{\cdot 75}{1{\cdot}836}$$

$$= \cdot 411 ; \; \frac{K+F}{C} = 2{\cdot}267$$

$$\text{Actual } \frac{K+F}{1{\cdot}826} = 2{\cdot}378$$

which is more.

2nd. Try W = 12

$$q = \frac{154}{12} = 12{\cdot}83$$

$$C = \sqrt[3]{\frac{12{\cdot}83^2}{g}} = 1{\cdot}723$$

$$\frac{L}{C} = \frac{\cdot 75}{1{\cdot}723} = \cdot 436$$

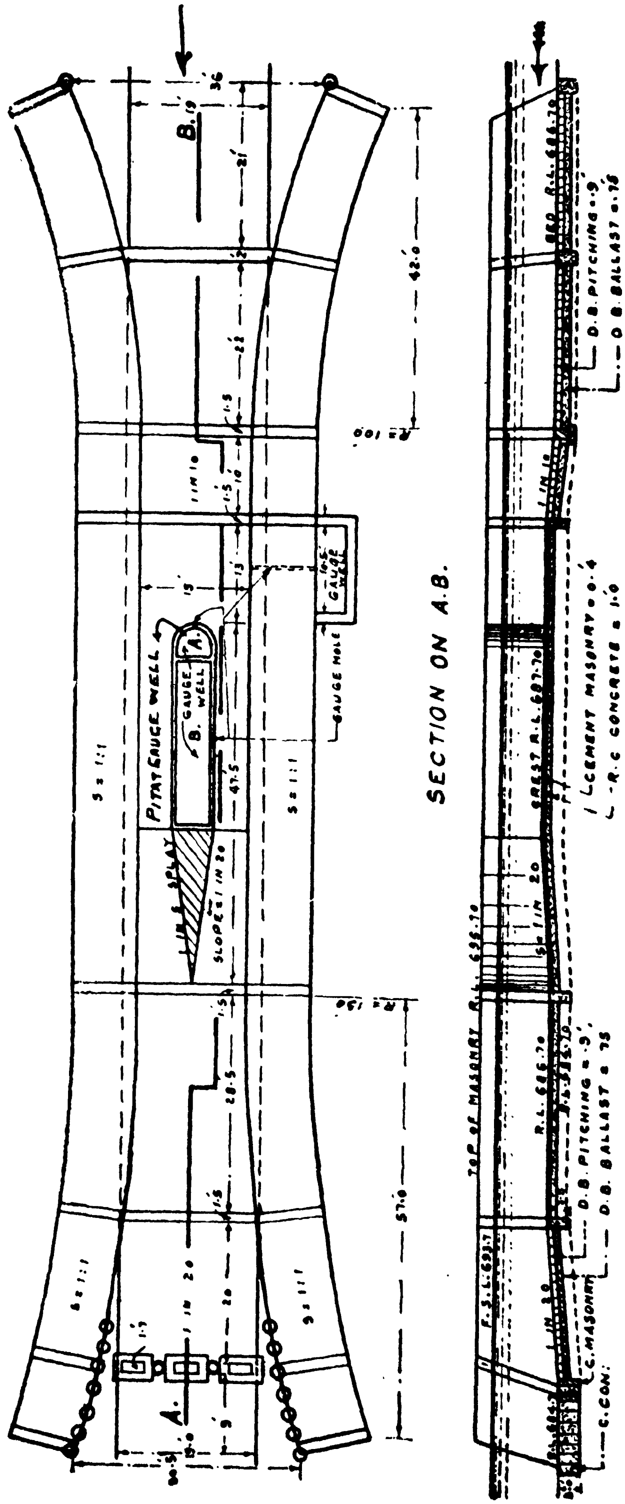


Fig. 15.

$\frac{K+F}{C}=2\cdot303$; Actual$=\frac{4\cdot39}{1\cdot723}=2\cdot52$ which is also more.

3rd Try $w=10$ feet

$q=\frac{154}{10}=15\cdot4$; $C=\sqrt[3]{\frac{15\cdot4^2}{g}}=1\cdot948$; $\frac{L}{C}=\frac{\cdot75}{1\cdot94}=0\cdot385$; $\therefore \frac{K+F}{C}=2\cdot228$;

against $\frac{4\cdot34}{1\cdot946}=2\cdot228$; which is equal.

Wave shall form when
$W=10'$; $x/C=\cdot545$; $x=1\cdot06$ ft. ; Barrel depth$=2\cdot8$ ft.

Reinforcement.

Maximum vacuum potential$=5\cdot9—4\cdot54=1\cdot36$ ft. ; $\frac{V^2}{2g}=\frac{5\cdot65^2}{2g}=\frac{\cdot5}{1\cdot86}$

Pressure due to vacuum$=1\cdot86\times62\cdot5=116$ lbs.
Add weight of cowl$=$ 70 lbs.
Crowd load$=110$ lbs.
Total 296 lbs.
say 300 lbs.

Max. span$=7\cdot0$ feet ; $M=\frac{Wl^2}{8}=300\times\frac{7^2}{8}=1840$ lbs.

$R=\cdot555\%_o=1840$ lbs. ; $d=\sqrt{\frac{M}{74}}=\sqrt{25}=5''$ max. $6''$ max.

and use this throughout ; Iron$=\frac{12\times6\times\cdot55}{100}=\cdot4$
4 bars of $\frac{3}{8}''$ dia : max: $\cdot44$ which is ample.

Neglecting the effects of curvature we may arrange for the upstream lip at R.L.$=736\cdot23$ *i.e.*, $4\cdot0$ feet above the floor of the culvert. Regarding the latter as an open flume weir we have for the priming discharge ; $Q=3\times6\cdot0\times4\cdot0^{3/2}=144$ cusecs.

R. L. of D. S. lip.

Depth of flow in the earthern channel for 144 cusecs ; $d=4\cdot2\frac{(144)^{3/5}}{200}=3\cdot45$ ft.

The downstream lip of syphon must dip into the level to ensure sealing ; surface level in drain for depth $3\cdot45=732\cdot07+3\cdot45=735\cdot52$.

We will therefore keep the downstream lip at R.L.$=735\cdot50$
Position of wave at point of priming with discharge 144 cs.
We have L=drop$=36\cdot23—35\cdot52=\cdot71$ ft.

1st. Assume width$=18\cdot0'$; $q=\frac{144}{18}=8$ cusecs.

$C=\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{8^2}{g}}=1\cdot258$; $\frac{L}{C}=\frac{\cdot71}{1\cdot258}=\cdot565$;

from Crump's diagram$=\frac{K+F}{C}=2\cdot48$; $K+F=2\cdot488+1\cdot258=3\cdot13$ ft.

Giving a floor level (limiting)$=736\cdot27-3\cdot13=733\cdot14$
The design bed$=733\cdot00$

This shows that the wave shall form U.S. of $18\cdot0$ feet width of the flume (the end of the syphon).

No special precaution to form a wave is necessary as the floor is already low enough. We could on the contrary reduce the width to some extent in order to bring the wave nearer to the lowest point of the cowl and then extend the cowl at an upward slope to downstream F.S. expanding at the same time. This expedient would give us a higher co-efficient for the syphon.

2nd. Try a width of 15 ft.

$$q=\frac{144}{15}=9{\cdot}6\text{ cs.};\ C=\sqrt[3]{\frac{9{\cdot}6^2}{g}}=1{\cdot}421;$$

$$L={\cdot}71\ \frac{L}{C}=\frac{{\cdot}71}{1{\cdot}421}={\cdot}50;\ \therefore\ \frac{K+F}{C}=2{\cdot}395$$

$$\therefore\quad K+F=2{\cdot}395\times1{\cdot}421=3{\cdot}4\text{ ft.}$$

This gives a limiting floor level $=736{\cdot}23-3{\cdot}4=732{\cdot}83$ against $732{\cdot}07$ *i.e.*, R.L. of bed of drain.

3rd. Try a width of 12 feet.

$$q=\frac{144}{12}=12{\cdot}0\text{ cusecs.};\ C=\sqrt[3]{\frac{12^2}{g}}=1{\cdot}647$$

$$\frac{L}{C}=\frac{{\cdot}71}{1{\cdot}647};\ \therefore\text{ from Crump's diagram}$$

$$\frac{K+F}{C}=2{\cdot}295;\ K+F=3{\cdot}78$$

Giving a limiting floor level $=736{\cdot}23-3{\cdot}78=732{\cdot}45$

We might go further in this direction, but to do so would involve a risk of wave forming too far downstream. By adopting a width of 12 feet of the lowest point of the cowl we shall be on the safe side with a nicely balanced design. Dotted water line shows conditions of flow for a discharge of 144 cusecs when the priming just starts.

Downstream priming conditions.

Downstream end will become sealed when the water surface touches the lowest point of the roof slab at R.L. 739·5 at 16 feet from downstream face wall.

The cross section at this point

$=(735{\cdot}5-732{\cdot}07)\times11=3{\cdot}43\times11=37{\cdot}73.$

$$V=\frac{Q}{A}=\frac{Q}{37{\cdot}73}\qquad(1)$$

$$h_a=\frac{V_1^2}{2g}=\frac{Q^2}{(37{\cdot}73)^2}\times\frac{1}{2g}=\frac{Q^2}{91{\cdot}700}$$

At crest, the width $=17$ feet

$$\text{section}=17d;\ \text{velocity}=\frac{Q}{17d}$$

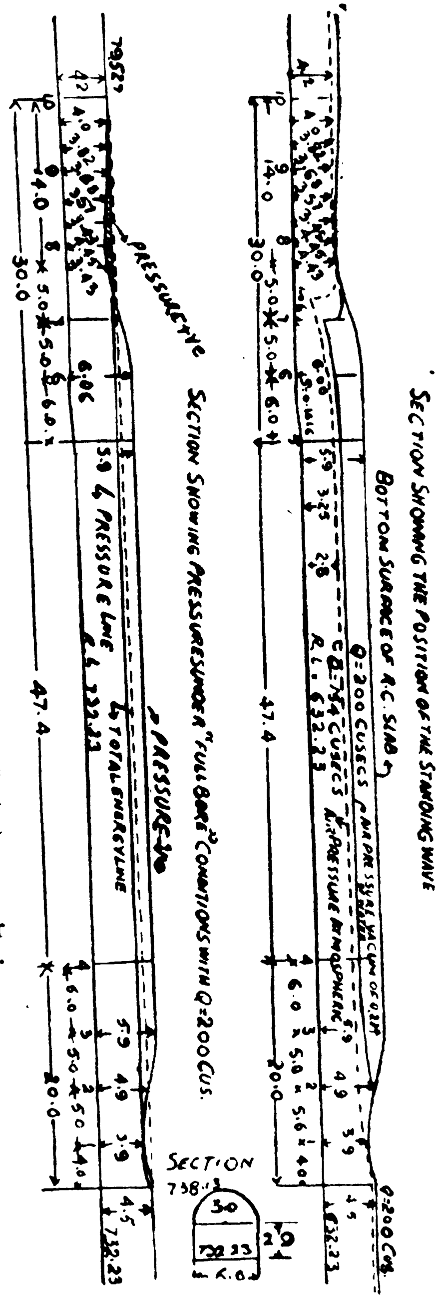


Fig. 15.

$$h_a=\frac{V_2^2}{2g}=\frac{Q^2}{18600d^2};\ \text{Difference in surface level}$$

$$\frac{V_1^2}{2g}-\frac{V_2^2}{2g}=\frac{Q^2}{91700}-\frac{Q^2}{18600d^2}=Q^2\left(\frac{1}{91700}-\frac{1}{18600d^2}\right)\text{ in depth at}$$

end $d=3{\cdot}43+Q^2\left(\frac{1}{91700}-\frac{1}{18600d^2}\right)$ (2)

For the drain in full supply we have
$Q=200$ cusecs ; $d=4{\cdot}2$ ft.

assume $\frac{Q}{200}=\left(\frac{d}{4{\cdot}2}\right)^{5/3}$ (3)

Try $d=3{\cdot}5$ feet ; from (3) $Q=147{\cdot}6$

R.H. from equation (2)

$d'=3{\cdot}43+143{\cdot}6^2\left(\frac{1}{91700}-\frac{1}{18600\times3{\cdot}5^2}\right)=3{\cdot}572$; $d'-d={\cdot}072$

Try, $d=3{\cdot}45$; $Q=144{\cdot}3$; $d'=3{\cdot}563$; $d'-d={\cdot}113$; Try, $d=3{\cdot}59$; $Q=154$

$d'=3{\cdot}45+154^2\left(\frac{1}{91700}-\frac{1}{18600\times3{\cdot}59^2}\right)=3.59$ which is the same as assumed.

14. *Example VIII.* **Surge Tanks and Chambers.**

(*a*) **Surge tanks in water supply.**

To cushion all parts of the water conduit against the full effect of forces due to water it is variously equipped with relief valves, bursting plates, and surge tanks. A surge tank is doubly valuable to a penstock because it will not only absorb energy during deceleration but it will also provide a ready reservior from which the turbines can draw temporarily, as when they are started, or, during normal operation, when a sudden heavy demand causes rapid opening of the turbine gates. Surge tank types are illustrated in. Fig. 16.

The simple surge tank often lives upto its name when a resonant condition causes succeeding surges to become ever greater until the tank spills over.

(*b*) **Surge chamber for controlled hydraulic jump (Crump).**

At Dubbiwala culvert, Upper Jhelum Canal a silt ejector was built and the escape supply of 200 cusecs had to be dropped in the adjoining Jhelum River in which H.F.L. was 75 feet lower than canal F.S.L. Water was carried in a 6 feet wide flume with 1 in 10 slope and the hydraulic jump was allowed to occur in a chamber covered by a reinforced concrete slab which was anchored and weight-down as shown in Fig. 17.

The idea of the chamber was to destroy the energy of fall of 70 feet with a depth of 4·00 feet in a chamber downstream of the hypercritical jet.

Type of Surge Tanks

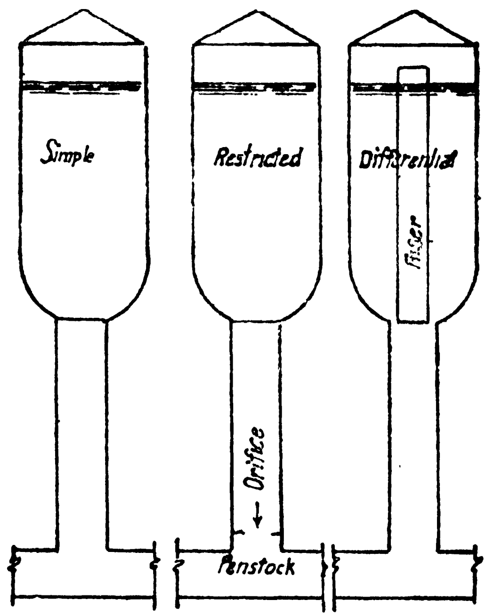


Fig. 16.

15. *Example IX.* **Rigid Top Flumes.**

(Hypercritical to sub- critical without jump) Fig. 18. Suppose the original velocity is V_1 and depth D_1. If the channel is open at the top, the static head at the point K is D_1 and the velocity head is $V_1^2/2g$.

Suppose the channel to be of uniform width and the bottom level. In order to secure the gradual smooth enlargement of cross section, suppose a rigid top to be added to the channel as shown from E to H, rising along a straight line. We know by experience that if the angle is not made too big the water will cling to the inclined plane and as the cross section increases the velocity will be smoothly and gradually reduced, as examplified in the expanding tube of the Venturimeter.

At point B, where the depth has increased to D and the velocity has decreased to V, the velocity head will have diminished to $V^2/2g$.

By Bernoulli's theorem the static pressure at B then equales

$$D_1 + \frac{V_1^2}{2g} - \frac{V^2}{2g} = D_1 + \frac{V_1^2}{2g}\left(1 - \frac{D_1^2}{D^2}\right)$$

In general, this pressure at B would not equal D exactly, but would be something greater, as BF. By means of the above expression, the static pressure corresponding to each depth can be easily calculated for given conditions.

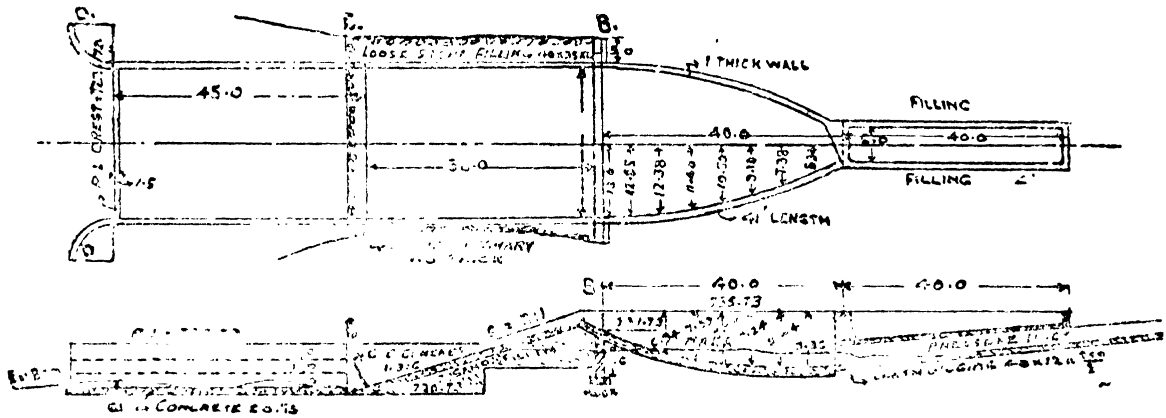


Fig. 17.

The resulting static pressure, shown in Fig. 18 gives the curve EFGH. This curve which crosses the surface of the water at E and H indicates relations of great fundamental importance which should be carefully noted. At E the static pressure on the botton is exactly that due to the depath of water AE. As soon as the velocity is reduced and part of the velocity head is thus converted into pressure, the pressure on the bottom is greater than that due to the depth of the water along. Thus at P the pressure is that caused by a head GP. This means that at the point N there is a pressure against the upper bounding surface of the water equal to that caused by a head GN. If a small piezometer tube should be inserted through the upper surface at N, the water would rise in it to the level of G. This pressure tends to burst the cover of the conduit. The curve EFGH is really the hydraulic grade line through the expanding section.

The interior bursting pressure against the upper surface begins with zero at E and increases very rapidly at first, and then more slowly, until it reaches a maximum value such as GN at N. From this point onward it gradually decreases untill it again reaches zero at H. The pressure on the bottom at M is again exactly that caused dy the depth of the water.

As the velocity decreases between E and N, the corresponding dimunition of velocity head is more than enough to raise the water along the rigid, slanting, upper boundry surface, and hence there is an accumulation of excess static pressure. As the velocity decreases from N to H, the gradual conversation of velocity head into the pressure is not sufficient to raise the water, and hence the previous accumulation of pressure is gradually drawn upon in raising the surface to higher and higher elevations.

If the whole apparatus is open to air, the water at H would no longer follow the upper

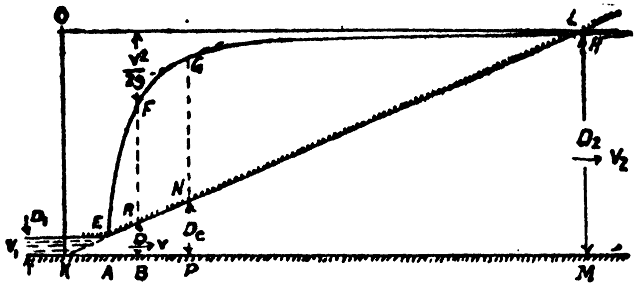


Fig. 18

slanting surface but would break loose and flow away with level surface from this point. If the air were excluded by suitable means beyond the point H, it would be under a vacuum increasing in intensity with the continued expansion of the cross section.

The cross section at which the upward pressure on the surface E H is the maximum at NP is of a particular interest. At this place a tangent to the curve at G is parallel to the water surface at N, and the change of velocity head is just sufficient to produce the change in elevation of water surface so that there is neither accumulation nor reduction of pressure. On account of this particular balance between velocity, head and depth, this particular section may be said to mark a critical point or a condition of critical flow.

It can prove that; $V_c = gD_c = gQ$; and $D_c = \frac{V_c^2}{g} = \frac{Q^2}{g}$

The curve EFGH. if extended according to its mathematical equation, has as asymptotes the straight lines OL and OK. The curve EH need not follow a straight line. Any smooth curve would do and Bernaul's Lemnscate provides the ideal surface expansion. The curve EFGH would change correspondingly in shape but would have the same properties as enumerated above, and would, for each depth of flow, be at the same distance above the top.

The depths at E and H always have a definite relation to each other because at these two points the sum of the depth and the velocity head is the same. Depths so related are called alternate depths.

Let D_1=depth at E ; D_2=depth at H ; $Q - V_1D_1 = V_2D_2$

$$\frac{V_1^2}{2g} + D_1 = \frac{V_2^2}{2g} + D_2$$

Substituting for velocities their values in terms of Q and transposing.

$$\frac{Q^2}{2gD_1^2} - \frac{Q^2}{2gD_2^2} = D_2 - D_1 \ ; \ \frac{Q^2}{2g} \cdot \frac{D_2^2 - D_1^2}{D_2^2 \cdot D_1^2} = D_2 - D_1$$

Dividing through by $D_2 - D_1$ and substituting, D_c^3 for Q^2/g ; $D_c^3 = \frac{2D_1^2 \times D_2^2}{D_1 + D_2}$

If the values of any two of the depths are given the value of the third can readily be abtained. This device in the simple from was used by the author to reduce M. M. H. of the Orifice semi module as published in paper No. 237, Punjab Engineering Congress 1940 and discribed in paragraph 5 (e) chapter XII, part II.

PART II

CANAL IRRIGATION

CHAPTER XIII

Siphon Spillways and Hydratomats

1. Introduction.

Instead of allowing water to spill over the crest of a dam or weir the surplus water may be dealt-with by a siphon spillway. This may comprise one or more siphon units. A siphon, (not syphon, as it is sometimes incorrectly spelt) is a pipe or tube bent to form two legs of unequal effective length, by which a liquid can be transferred to a lower level, over an intermediate elevation, by the pressure of the atmosphere in forcing the liquid up the shorter leg of the pipe immersed in it, while the excess weight of the water in the longer leg (when once filled) causes a continuous flow. The flow takes place only when the discharging extremity is lower than the liquid surface, and when no part of the pipe is higher above that surface then the same liquid will rise by atmospheric pressure (about 34 feet for water near sea-level). As applied to a dam the siphon is usually formed monolithically with the dam. Such a Siphon is illustrated diagrammatically in Fig. 1, wherein are shown the name usually given to the various parts. As the reservoir overflows, water flows over the crest of the siphon as in a plain spillway, but the design is such that the air which is contained in the upper bend is shut off from the outer atmosphere either by a water seal below the downstream leg, or by a curtain of falling water. If now the air is exhausted from the upper bend the siphon will run full and is said to have "primed". The discharge is now very much greater than could be obtained with the same reservoir level and a plain spillway of equal length to the siphon spillway.

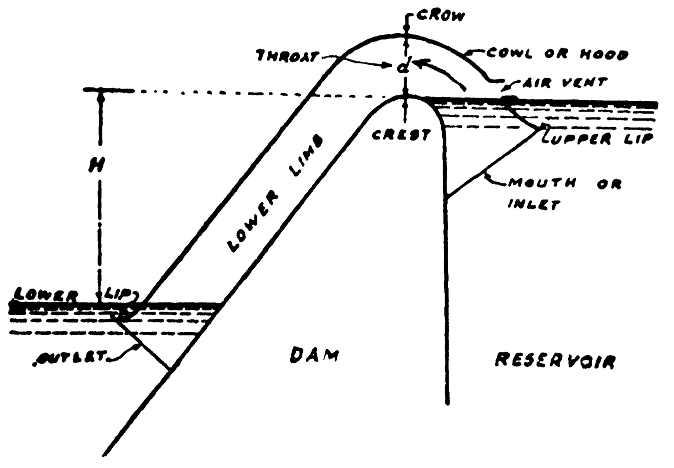


Fig. 1.

The air may be drawn from the upper bend by an air pump, but it is usual to arrange that priming shall be automatic, the siphon being so designed that the water flowing over the crest itself carries away the air from the upper bend.

In all siphons the inlet is bell-mouthed or funnel-shaped so as to reduce loss at entry and minimize surface draw-down above the siphon mouth. Usually the upper lip of the mouth of the siphon is carried down below crest level and in this case it is necessary to provide air-vents in the hood of siphon just below the level at which it is desired that siphonic action shall cease, Fig. 1. If these were not provided, a siphon would continue to function untill the reservoir level was reduced to level of the siphon lip and much storage would be lost.

2. Discharge Formula.

The usual siphon formula is :

$Q = C_d . A. \sqrt{2gH}$, where C_d=Co-efficient Head ; A=Throat area.

The value of C_d has been taken to be ·8 in some of the low head designs adopted in the Punjab for plain siphons used to keep a constant level in the channel.

A.H. Naylor, who built the Laggan and Dunalastair Dams in Scotland, considers in his book on ' Siphon Spillways, 1935" that the head should not be introduced into the expression for the characteristics of a siphon. His opinion is that in view of the lack of knowledge of the priming level before construction of the actual siphon, and in view of its variability according as external conditions vary, the performance of a siphon should be assessed as the ratio of the discharge to the discharge of a "perfect" siphon of the same throat area. By a "perfect" siphon is meant the purely theoretical conception of a siphon with a perfect vacuum across the throat. This ratio, it is suggested, should be called the "Efficiency" of a siphon.

Efficiency $\eta = \frac{Q}{A\sqrt{2ga}}$; , where Q is the discharge of the siphon ; A is the throat area ; a is the atmospheric pressure in feet head of water units ; feet and second.

At sea level 'a'=34′ and the expression then becomes : $\eta = \frac{Q}{47\,A}$

Q can be calculated with a fair degree of accuracy or may be measured by means of a scale model. In a few cases it is possible to check its value against the performance of the actual siphon, thoug this is usually rather difficult to measure accurately. This is a true efficiency and can never attain a value of 100 percent. It is a measure of the intensity of discharge over the crest. The only shortcoming is that, like the co-efficient of discharge, it can take no account of priming level. But it has the advantage of definiteness for this very reason. A further disadvantage is the disappearance from consideration of the somewhat indefinite H. The "co-efficient of discharge" diverts attention from the true desiderata of a siphon. It suggests that the discharge is proportional to the square root of the head available in any particular design, whereas it depends rather upon the vacuum attainable at the throat action.

3. Type of Siphon Spillway.

Siphons may be classified according to Head as High, Medium or Low Head siphons.

High head. Consider a siphon whose area of cross section is constant from the throat downwards. If the lower limb be imagined to be lengthened, then, as the operating head is increased, the degree of vacuum at the throat will increase. When the head becomes greater than 30′ (the actual figure depends upon the design, berometric height and the amount of air in solution in the water) the lower part of the siphon will not flow full, or alternatively violent pulsations of flow will begin to take place. It is desirable to avoid this, and therefore for greater operating heads the outlet area should be reduced by a nozzle, or by tapering the lower part of the siphon, or in some other manner increasing the resistance to flow. Such convergent siphon are conveiently designated High Head siphons.

Medium head. With somewhat smaller head a siphon of constant cross sectional area is suitable. Such siphons are often used where a greater discharge could be obtained with a divergent lower leg. They have the advantage that priming occurs at a lower level and that from the constructional point of view a uniform cross section is the cheapest. Siphons in which a lower limb is of constant area may be designated Medium Head siphons.

Low head. There remain the siphons with divergent lower limbs. These are classed as Low Head siphons.

The classification into high, medium, or low head siphon according as the outlet area is less, equal, or greater than the area at the throat, does not allow definite ranges of head being assigned to each type. There is a certain amount of overlapping. It is suggested that the classification according to head should be, over 20 feet, 20 to 10 feet, upto 10 feet. This is purely arbitrary. However, for heads less than about 20 feet (depending upon the design) the discharge may, in general, be increased by the use of the low-head type of siphon. In all siphons the maximum discharge is primarily limited by the design of the upper bend. Unless the avaiable head is very low, it is usually possible to design the remainder of the siphon so as to utilize the full capacity of the upper bend. With a low head, there are two main reasons why it is difficult to obtain the full discharge of which the throat design is capable. These are the difficulties of ensuring priming at the desired water level and the loss of head in a divergent outlet.

4. Priming Methods.

No two siphons are alike. It is usual to classify them according to the methods adopted o ensure priming.

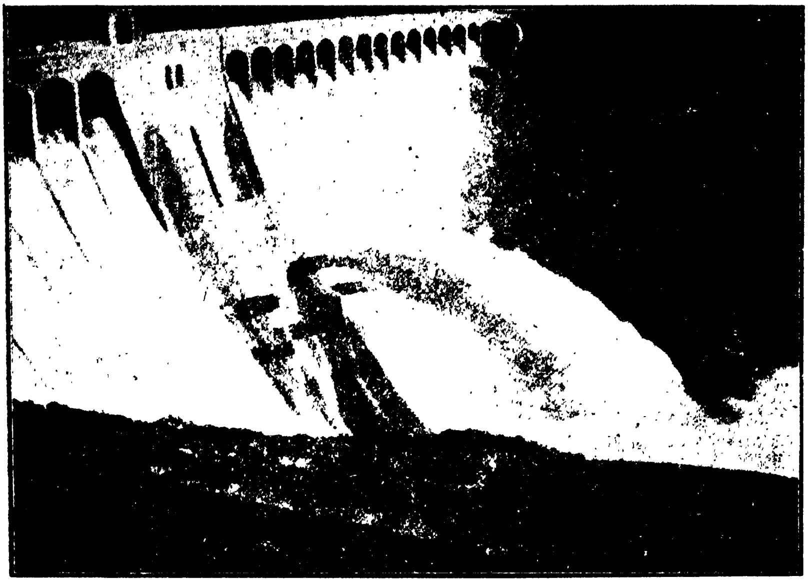


Fig. 2 Laggan Siphon showing the outlet and end siphon in action

Fig. 3 Laggan Siphon showing the effect of the Jet dispersers in the outlet.

(*a*) **Air pump.**

The air pump or ejector is the suitable device by means of which priming may be initiated at will with the water level below the crest. Provided that the mouth of the siphon is situated sufficiently low to remain immersed, the siphon can be primed with the water-level many feet below the crest, the criterion being that the vacuum at crest must not exeed about 24 feet of water (a complete vacuum is unattainable owing to liberation of dissolved air at low pressures). The air pump can be applied to any shape of siphon provided that there is throughout the priming a water seal to the lower limb. The great disadvantage of this type is, that, should anything happen to impair the proper functioning of the air pump or ejector, the dam would be endangered. The air passages being small, the possibility of choking by debris or freezing cannot be ruled out.

An interesting application of the ejector principle is to a battery of siphons. To avoid shock and undue vibaration it is usually arranged that the siphon units should prime at different times. This is easily ensured by the ejector principle. In the partition between adjacent siphons a small inclined passage is formed above the crest. After the first siphon is primed, the vacuum above the crest causes air to be drawn from the next siphon untill this primes and so on successively throughout the battery. The time lag between successive priming can be increased by reducing size of air passages which must be small, as otherwise the priming of the first siphon will be hindered. This system cannot be recommended, as it makes the priming of the whole battery and the safety of the dam dependent on the priming of a single siphon.

This method was used at Mittersheim and also for the Oswego lock on the New York State Barge Canal U.S.A. In the latter case it is controlled and priming depth on the crest is d/3 where d is the depth at the throat.

(*b*) **Auxiliary (Baby) siphons.**

In some siphons a small auxiliary or "baby" siphon is formed just below the crest. This will, of course, flow full by the time the water reaches crest level. The sheet of water issuing from the baby siphon is arranged to shoot obliquely across the lower limb of the siphon so as to seal it and prevent air entering from below. The enclosed air is gradually carried away by the surface of the falling water untill priming takes place.

A good example is the Maramsilli siphon installation. The baby siphon is one foot high and 8 feet wide at the throat against 8 feet by 8 feet in case of the main siphon. Priming of the baby siphon occurs before the water level rises to the crest of the main siphon. With the better design of the two types of baby siphon used. Fig. 4. priming of the main siphon followed in 15 seconds.

While the baby siphon obviates the need for a sealing basin below the lower limb, it has many disadvantages and it is probable that equally good results can be attained by one of the methods described later. The effective top water-level for the consideration of storage of water is not the crest of the main siphon, but the crest of the auxiliary siphon. So that from the point of view of the maximum discharge the auxiliary siphon must be looked upon as an obstruction placed inside the siphon. The minimum size for the auxiliary siphon is fixed by the consideration that throughout priming the sealing jet must function as such. As the flow over the main siphon crest increases, the jet from the baby siphon is increasingly depressed and the combined jet will strike even lower down on the outer cover. The

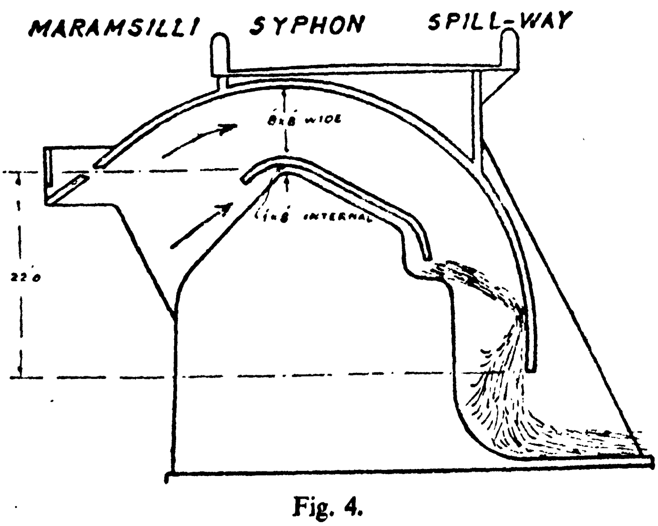


Fig. 4.

baby siphon must be sufficiently large to ensure that the combined jet never falls below the lip of the lower leg. This size can be easily calculated from consideration of the momenta of the two streams of water. From the Maramsilli tests it would appear that with good design priming can be relied upon to take place with a rise of water level of less than d/8, where d is the depth at the throat. The baby siphon is expensive and difficult to construct, as it must extend unbroken, the full width of the siphon. Any partition would cause a break in the issuing jet of water. It is a source of weakness. Its cowl must, if necessary, be thin at the crest section and yet the outside will be subject to a partial vacuum with possibly cavitation and destructive vibration. There is a danger of choking of the relatively small air passages.

(*c*) **Priming weir.**

The priming weir was introduced to ensure priming where for some reason it was desirable to form the lower limb of a siphon at a flat slope Fig. 5. In this device a weir is constructed at crest level from the crest backwards along the side and across the back of the siphon so that when overflow commences, the falling nappes completely seal the air in the crown of the siphon. Air is then gradually carried away by the falling water until priming ensues.

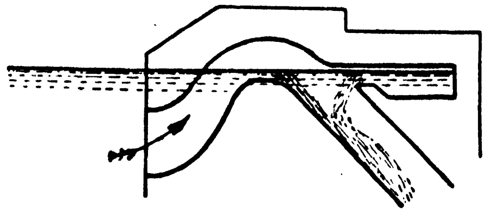


Fig. 5.

Tests on the Bear River Siphons of the Mokelumna Project indicated that the priming weir by itself could not be relied upon to produce priming with the permissible head above the crest of 0·3d. The outfall of this siphon was then modified so as to ensure priming independently of the priming weir. It was then found that by its effect of doubling the length of weir the priming weir caused the siphon to prime with a little more than half the rise of water-level necessary, when the priming weir was boarded off. The turbulence and reduction of discharge caused by this device probably neutralizes its advantages. For the same reason the priming weir cannot be regarded as a satisfactory device.

(*d*) **Heyn's flexible tongue priming device.**

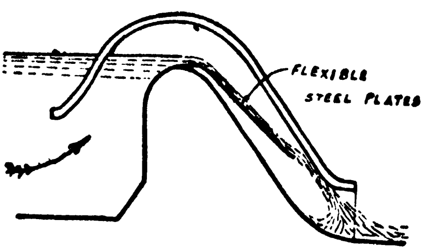


Fig. 6.

An ingenious application is the system suggested by Heyn in which a flexible steel tongue directs the over-flow across the siphon until priming begins when the weight of water presses down the steel tongue against the inner wall so that an unobstructed flow ensues, Fig. 6. But it is well to avoid moving parts.

(*e*) **Priming step or tooth.**

If the sloping lower limb is not too steep then, by suddenly turning it down near the lower end, priming may be brought about without auxiliary device. When overflow commences the sheet of running water, owing to its momentum, is unable to follow the surface of the lower limb and shoots forward across the siphon, thus forming a seal.

The priming efficiency of a sheet of water shooting across and sealing the lower limb depends upon the amount of air which can be carried away. This will be proportional to the velocity of the falling nappe rather than the amount of over-flow. The more acute the angle at which the nappe strikes the outer cover, the more effectively will air be carried away by the water. The step or changes of slope should, therefore, be situated sufficiently far down the lower limb to allow the water to attain its maximum possible velocity, yet it must be sufficiently high up to ensure striking the outer cover well above the lip. There must be no danger of the nappe clinging to the lower surface. This can be avoided by a definite discontinuity such as a small step Fig. 7.

This device is known to ensure priming with depth d/6 as used in Germany.

The lower limb is often terminated in a sealing basin, the cill of which is level with the bottom edge of the siphon cover. Examples of this type have already been mentioned. The lower portion of the siphon should be steep, but must not in this case be quite vertical, so that any air carried down into the sealing pool will bubble up clear of the siphon. Thus the sealing pool acts as a non return valve for the air and ensures the steady evacuation of the siphon, Fig. 8.

An evil of submerged outlet may be noted. Consider the upstream water level to be at the level of the mouth of the siphon, for air-vents are provided with the level of the top of the air vents, any rise of the U/S level will now compress the enclosed air so that the water surface inside the mouth will rise by a smaller amount and therefore, priming will be delayed. A rise in the downstream water-level will likewise compress the air and may even depress the upstream water-level inside the siphon sufficiently to prevent priming. In such cases an air relief valve should be provided in the crown of the siphon. A simple flap valve is all that is required.

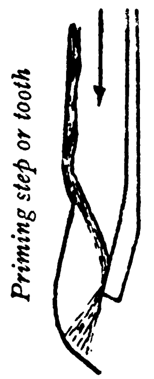


Fig. 7.
Sealing Basin

(*f*) Vertical or overhanging crest to ensure priming.

Water flowing over a weir the vertical profile of which is an arc of a circle will spring clear when it reaches a point about 60° from the vertical except in the case of a very thin adherent nappe. Fig. 9.

In the Verona siphons constructed by Gregotti, the crest is an overhanging sharp-edged weir. This has, of course, excellent priming qualities, a rise of water level above the crest of 2 inches to 3 inches, *i.e.*, d/12 to d/8, producing priming in 1 to 2 minutes. There are heavy hydraulic loses in such a siphon and the co-efficient of discharge is only 0·4.

Other instances are the Canberra West Lake Siphon, which primed with 2 inches over the crest, or d/27, some of the siphons at Huntington Lake and the Leaburg siphons. The last-named were of various sizes, but all primed with head from d/7 to d/4 above the crest level.

The good priming qualities of siphons with overhanging crest are obtained at the expense of the discharge, which is some what reduced by the large changes of direction of flow.

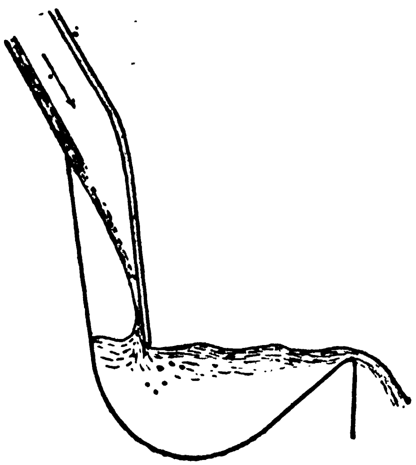


Fig. 8.

(*g*) Comparison of priming devices.

The air pump is only suitable where manual control is desired. Priming by this means takes place at a steady rate dependent upon the capacity of the air pump (except when a vacuum tank is provided) so that, it is suitable where the rise of water level may be very rapid.

A well designed auxiliary siphon can induce priming with a head above crest level of d/8. The priming weir is not very reliable as a priming device, but in conjunction with other priming devices allows of priming taking place with a reduced size of water-level. The capacity of the siphon, is, however, reduced by the increase of resistance due to the presence of the weir. A water seal reduces the discharge, but used in conjunction with a priming device it makes priming more certain. A few inches overflow usually suffices to bring about priming. If, however, the lip terminating the lower limb of the siphon is deeply immersed in the water seal, priming will be retarded and may even be prevented. Provision must be made to prevent compression of the air enclosed in the siphon or overflow at the crest may be reduced or even stopped.

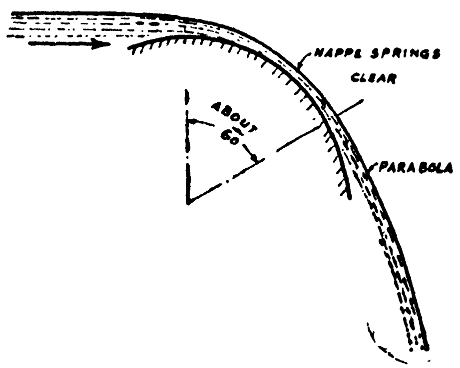


Fig. 9

A joggle or step in the lower limb of a siphon can be arranged to cause priming with a height of the water surface above crest level of d/3. In conjunction with a water seal to the lower limb this may be reduced to d/5 or less.

With the lower limb steep, vertical or with over-hanging crest, provided that the design is sound, priming can be relied upon with a head of water above crest level d/3 and will generally take place with a head of d/4 or less.

For good self-priming qualities there must be sharp curves, large changes of direction or projections in the siphon. The requirements for a high discharge are easy curves and small changes of direction, so as to minimize hydraulic losses. A compromise must be struck between these opposites for good design. Priming must first be ensured with the permitted rise of water above crest level and than the shape improved as much as is compatible with this consideration.

4. Design of Siphon Spillway for High Dams.

(*a*) **Throat depth.**

The crest of a siphon may be sharp as in the Verona siphons designed by Gregotti or it may be wide. In the former case, the priming depth would be lessened, and the crest being sharp, would not take much space, a large outer radius could be adopted, at the same time containing the whole siphon top including its inlet within the limited top width of the dam. But it has been found that a sharp crest reduces efficiency and therefore discharge, and also causes cavitation and vibration to the structure and is therefore undesirable, though welcome for a reduced priming depth.

Experience has shown that in a siphon design, the minimum permissible vacuum should not exceed 24 ft. of water, because with lower pressures too much air in solution is released from the water passing through, which causes vibration, cavitation and parting of the water from the crest. If we thus limit the vacuum to 24 feet, and this at the crest of the siphon, evidently if V_o is the velocity of the stream lines at the crest, then ; $V_o = \sqrt{2g \times 24} = 8 \times 2\sqrt{6} = 39$ ft. per second.

Now the flow at the throat of a siphon, as proved by professor Gibson, Davies and others in the papers contributed by them to the Institute of Engineers, London, approximates to a free vortex flow, which has the well-known property, *viz.*, $V \times r$ or the product of velocity and the radius at any point in the stream equals a constant. Fig. 10. Therefore at any section, where the radius $= r$ and velocity $= V$;

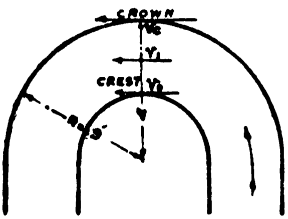


Fig. 10

$V \times r = V_o \times r_o$ (at crest) $= 39 \times r_o$

$\therefore\ V = 39 \frac{r_o}{r}$

$\therefore$ the discharge through the slit of height dr and width 1 foot

$= d_q = V \times d\ A = \frac{39 r_o}{r} \times dr \times 1$

$$Q = \text{Total discharge through the throat } R = \int_{r_o}^{R} \frac{39 r_o}{r}\, dr = 39\ r_o \log_e \frac{R}{r_o}$$

$\therefore$ the discharge q per foot width of siphon throat is $= 39\ r_o \log_e \frac{R}{r_o}$

Efficieney ; $\eta = \frac{Q}{A\sqrt{2ga}} = \frac{Q}{47A}$

$A = (R - r_o) \times 1$ per foot width ; $\therefore\ \eta = \frac{39\ r_o \log_e(R/r_o)}{47\ (R - r_o)}$

Discharge per foot width of siphon tube $q = \eta \times 47\ (R - r_o)$

Negative pressure at the crest $= 24$ feet ; Velocity at the crown $= 39 \frac{r_o}{R}$

Pressure at the crown$=(R-r_o)-\frac{(39\,r_o/R)^2}{2g}$ft.

(*b*) **Width of siphon.**

The throat setcion must be rectangular in order to get a maximum area in a given heignt. The width of each unit is determined chiefly from structural considerations. There is usually a high vacuum pressure acting on the outer cover. However, if the cowl is curved, a fairly wide siphon is permissible without making this unduly high. The effect of vibration must be borne in mind. During priming there is a likelihood of violent vibrations of the water column and if the siphon cover is thin and too wide, it may vibrate in sympathy. No rule can be laid down, but width of about 2d gives good proportions. If the width is must less, the hydraulic mean radius is sensibly reduced and friction losses and the cost of construction are increased.

The discharge per foot width is worked out and now the discharge of each siphon is got by multiplying it with the throat width.

The number of siphon tubes can thus be worked out by dividing total discharge capacity by the capacity of each siphon.

(*c*) **Level of the outlet.**

High head siphons are obviously indicated when the drop exceeds 34 feet over the dam. Water should be shot clear of the toe of the dam in the form of jets. Since the discharge is limited by the vacuum at the crest and since the lower the orifice the greater the jet velocity, the orifice must be reduced in area all the more. A low orifice with its greater velocity and its proximity to the toe of the dam can shoot a jet further downstream, so that the possibility of its affectiong the dam is reduced. A low orifice has many disadvantages, howevers. The extra length of siphons means extra cost. The greater construction of the passage increases the posibility of obstruction. The downstream toe is weakened in the region of greatest compressive stress. The high velocity about 700 feet per second with 120 feet head might damage the surface of the concrete. In this connection it should be pointed out that there is no evidence of deterioration of smooth, dense concrete under velocities as high as 60 feet per second, if there are no irregularities to upset the smoothness of flow and if the water is free from grit.

Let it be limited to 50 feet per second in the case of concrete and assuming co-efficient discharge$=\cdot8$; $V=\cdot8\sqrt{2gH}$; $\therefore$ $H=60$ feet.

The level of the outlet could be 60 feet lower than crest.

In order that the jet should strike the river bed at a maximum distance from the dam, the jet must be inclined upwards.

Let θ=inclination of nozzle : x=height above river bed ; l=horizontol travel of jet ; V=initial jet velocity.

Neglecting air friction, which will not be important far a large jet, it is easily shown that

for l to be a maximum : $\operatorname{Cosec}\theta=\sqrt{\frac{2gx}{V^2}+2}$ and

then $l=\frac{V^2}{g}\cos\theta\left\{\sin\theta+\sqrt{\sin^2\theta\times\frac{2gx}{V^2}}\right\}$

The area of the orifice at the outlet ; $A_o=\frac{Q}{50}$sq. feet.

(*d*) **Vertical limb.**

(*i*) The cross section area at throat is A sq. ft. and that of the outlet is A_o sq ft. It is desirable to reduce A to A_o as quickly as possible from considerations of the strength to give the maximum thickness between the siphon units.

If pressure is kept constant, the rate of increase of Kinetic energy will equal the rate of loss of potential energy ; $d\left(\frac{V^2}{2g}\right)=-dh$.

Integrating within limits h_1 and h_2 ; $h_1-h_2=\frac{Q^2}{2g}\left(\frac{1}{A_0^2}-\frac{1}{A^2}\right)$

This gives the length of the transition for the change of the sectional area.

(*ii*) **Shape of verticel limb.** From the hydraulic point of view a circular shape of the vertical limp is ideal as having the least wetted perimeter. From the point of view of stress in the dam the concrete between the siphons should be as thick as possible, and free from sharp corners. This would suggest an oval section or a rectangular section with rounded corners. But the extra expense is not justified and a circular section is usually adopted.

(*iii*) **Bend on the vertical limb.** "As investigated by Professor Gibson and Davies, there would be some loss of head at the lower bend and it has been found that, in a pipe, the minimum loss of head occurs when the ratio of twice the radius of the bend to that of the pipe is=5.".

(*e*) **Inlet cowl.**

We shall now consider the design of the inlet. In the early siphons, the lip of the inlet was kept at the same level as the crest of the siphon, so that, as soon as the water level went down to the crest, siphonic action ceased, air rushing in as soon as the lip was uncovered. But this arrangement has obviously great disadvantage that the surging of the water due to waves would prime and deprime the siphon, causing intermittent working. Also, even when the siphon primed,the shallow screen of water over the lip would tend towards the formation of a vortex. Sometimes even a hollow vortex, resulting in air being sucked into the siphon and preventing its priming even when the water level was for above the lip level. This was, therefore, remedied by carrying the siphon lip well below the crest level and by providing air-vents at a suitable height to deprime the siphon. Now the usual practice is to give an area for the widest cross section of the inlet, twice that provided for the throat of the siphon.

(*f*) **Air vents.**

As already stated above, when the lip of a siphon is carried so deep below the crest of the siphon, for purposes of depriming, an air-vent of a suitable size must be provided in the hood with its top at a level where it is desired to stop the siphonic action. The air-vent is usually made rectangular with its width equal to the width of the siphon and its height not very large, because the gradual uncovering of the vent by the gradual fall of the flood level would result in partial operation of the siphon over a long period, causing noise and vibration. With a high vent there is also tendency of vortex formation with the rising water level and this would either prevent or delay the priming. Information available regarding the size of an air-vent necessary to deprime a siphon is very meagre. We have got some experiments conducted by Inglis at Poona but better than this we have the experience of the siphons actually working in different parts of the world. The area of the air vent is generally expressed in terms of the throat-area. Stickney, the grandfather of siphons and who long back designed and constructed siphons in different parts of the world, says that the ratio should be 1/24. The ratio in case of the Marmasilli siphons has been kept at 1/16 by Davies. In the Lake Fife siphon, however, it is as small as 1/28 and this has been found to reduce the flow. The largest ratio is in the case of the Wood Creek siphon and this is 1/4·3. The Hetch Hetchy siphons have a ratio of 1/10, the Dunalastair larg siphons have a ratio of 1/16 and the Laggan siphons recently constructed in England have a ratio of 1/11·5. Except in the case of two the ratio in case of most of the siphons of the world ranges from maximum of 1/4·3 to a minimum of 1/28. The general practice is however to err on the safer side. Scalemodel experiments may also be conducted but the results though indicative, cannot be correctly representative in this respect, on account of viscosity and surface tension etc. We normally might keep the ratio at about 1/12 the throat area.

The shape of the lip is also important. If it has a sharp edge, then, under a strong draught of air, as the rising water reaches the lip, the draught will depress the water surface below the lip and it would require several inches of rise of water level above the lip before the air-vent is actually sealed and thus priming would be delayed, though, of course, for the very reasons explained above, depriming would be fairly quick. If, on the other hand, the lip surface is broad and rounded, then as the rising water reaches the lip, air will be cut off, as there will be no water depression now and the priming will be satisfactory. But as the water level falls below the lip the air rushing under will be at sub-atmospheric pressure and this would suck up the water, which would throttle the air flow and the siphon will not stop working until the water level falls some distance below the lip. The depriming is thus delayed and the water in the reservoir would be wasted. Thus, with no draught of air, a sharp-edged air lip is ideal; but where the rise of water level is to be strictly limited, a broad lip is better and safer ; but we must on the other hand, be prepared to lose some storage, due to the delayed depriming. The actual slope to be used, therfore, depends upon the condition of the reservoir working. A compromise is always desirable. To prevent the interference in priming and depriming, due to the surge of waves, it would be convenient to provide an exterior air lip as shown in Fig. 11, though this would involve an extra cost.

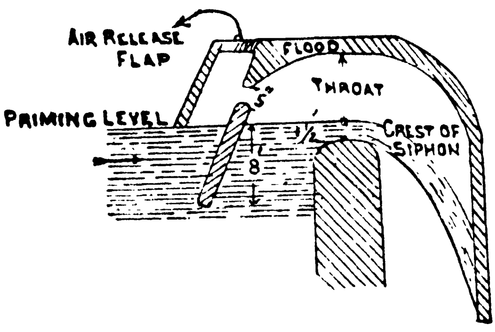


Fig. 11

(*g*) **Jet dispersers.**

The rock in the bed of the river downtream of the dam may not be very hard and may be intersected with a number of cleavage planes. The jets of water with an initial velocity of about 50 feet per second are likely to disintegrate or remove the rock in the bed by their impact. The present day designers, who have to deal with such high velocities and such large masses of water, use, what are called, "Jet Dispersers." These have been used for the sluices in the Mettur dam and also, in the new Lloyd dam. Such dispersers have been used, also, in the laggat siphons, Eng., these dispersers have been patented by Messrs Glenfield and Kennedy and they "consist essentially of a set of radial vanes of radially varying pitch, so that, the core of the issuing jet receives a higher speed of rotation than the surface, and therefore, owing to the resulting centrifugal force being greater at the centre than at the circumference, the jet on leaving the dispersers literally explodes, breaking up into a fine spray the energy of which is soon dissipated by friction with the air." Due to the friction, the path of the particles of water will no longer be parabolic and it will spread itself over a large area, thus reducing impact. These dispersers, since they offer some resistance to the free flow from the siphon outlet, naturally cause a certain amount of back-pressure. Hence, it would be better to design for a slightly larger outlet vent than usually necessary by about half a foot diameter.

6. Graphical Method of Design.

"According to usual hydrodynamic theory for two dimensional flow of an irritational fluid, such flow can be expressed by the equation.

$\phi+\iota\psi=f(x+\iota y)$; where ψ is the flux ; ϕ is the velocity potential

This is usually written ; $\omega=f(z)$

If in the plane of ω, ϕ is taken as the abscissa and ψ as the ordinate, and straight lines $\phi=1, 2, 3$ ..... and $\psi=1, 2, 3$, are drawn the ω plane will be divided into a net work of equal squares. This represents a straight, uniform, parallel flow from left to right.

Now suppose$=\omega\equiv\phi=\iota\ \psi=F\ (t)\equiv F\ (\xi+\iota\eta)$.

where F is any function. The ξ and η lines in the t plane corresponding to the ϕ and ψ lines in the ω plane will intersect orthogonally to from squares, but in general, the sides will be curved and the squares unequal in size. To every point in the ω plane there is a corresponding point in the t plane and η lines represent a possible distribution of stream lines. By successive intermediate transformations of this nature, the relationship, $\phi+\iota\psi=f(x+\iota y)$ is finally reached and it is sometimes possible to obtain a distribution of stream lines and velocity-potential lines which will fit the boundary surfaces of the flow under consideration. Schwarz and Christoffel have shown how such transformations may be mathematically performed, but a solution is only possible in a few simple cases where the solid boundaries are straight lines or circular arcs.

By virtue of the properties of the ϕ and ψ lines of intersecting orthogonally and of dividing region of flow into curvilinear squares ; it is always possible to obtain a graphical solution provided that the boundary conditions are known.

Consider the case of a siphon the cross-section of which is throughout rectangular and of constant bredth so that the flow is in two dimensions only. In order that the flow into the mouth of the siphon may be two-dimensional also, it will considered to be a unit in a battery of siphons all in operation at the same time. The inner boundary forms the stream line $\psi=0$ and the outer boundary the stream line $\psi=\eta$, where η is an arbitrary number of stream tubes. The difficulty lies in finding two known lines of constant velocity-potential, or what amounts to the same thing, the direction of the stream lines over the remaining two boundaries. If the reservoir is deep as compared with the diameter of the siphon then at a distance of several diameter the ψ lines will be radial to the mouth of the siphon and the ϕ lines will be concentric circles as BB. It is necessary to fix the remaining boundary before attempting to trace the stream lines. If the siphon includes a long straight, parallel leg there is little error in assuming that the flow is parallel to the leg at the center of its length *i. e.*, that the ϕ lines at the center are normal to the sides. as AA, Fig 12.

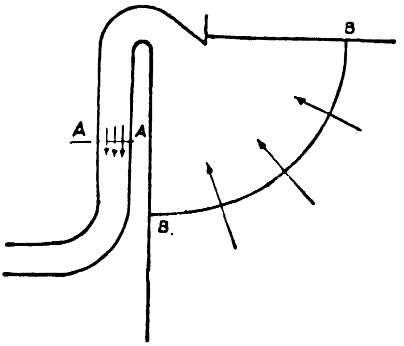


Fig. 12

Now divide the area ABBA into stream tubes of equal flow. Four is a convenient number These are sketched in by guesswork. At AA the lines will be equally spaced. At the crest section, where the flow will approximate to a free vortex, the lines will be more closely spaced at the crest then at the crown. Finally at BB the spacing will again be approximately uniform. Sketch in the ϕ lines so as to intersect the ψ lines at right angles. It saves time if these are spaced at intervals along the stream tube, which is nearest to the center of the siphon, equal to the thickness of the tube. It will probably be found that most of the elemental areas are by no mean square, the mean distance between the ϕ sides being unequal to the mean distance between the ψ sides. The ψ lines should now be adjusted so as to make the elements between adjacent ϕ lines as nearly square as possible. Then new ϕ lines can be sketched in crossing these ψ lines orthogonally, and the process should be continued until the accuracy desired has been attained.

It is advisable to sketch in the whole flow network in the first place, however approximately, as the terminal conditions effect every ψ and ϕ line. The flow in the upper bend never quite attains a simple, free vortex flow, but unless the angle of the bend is small, the approximation is very close at the center of the bend and the central ϕ line may with little error be assumed to be a straight radial line.

7. Effect of Siphons on the Dams.

The flowing water exerts a centrifugal force at all bends, a longitudinal force where-ever an acceleration of the stream takes place, and in addition tangential friction forces. The effect of these and the lightening of the dam by the replacement of masonry or concrete by air or water throughout the siphon should be calculated when considering the stability of the dam. The stability of each section of the dam between contraction joints should be considered independently. The general effect of these forces is to increase the overturning moment on the dam.

The centrifugal force at a bend of angle θ is in a direction disecting the angle of the bend and has a magnitude of $2\frac{w}{g}QV\sin\frac{\theta}{2}$; where V is the mean velocity. The accelerating forces excerted by the siphon walls are given by $\frac{wQ}{2g}\,\delta V$, where δV is the change of velocity. Note that the total accelerating force acting on the water is $\frac{wQ}{g}\,\delta V$, but only one half of this is exerted by the siphon walls. The longitudinal friction is given by wAh_f, where h_f is the head lost in friction in the part under consideration.

In considering the stress in the siphon cowl the centrifugal force cánnot be used, but rather the actual distribution of pressure on the underside of the cowl.

Where the siphon is carried through the dam, the possibility of the maximum shear or compressive stresses being increased must be remembered.

Where the water from siphons flow down the face of a dam, there will be an increased pressure on the downward curved toe. Where the impact is below the dam, the nature of the rock must be considered.

Not least important is the possibility of vibration and its effect on the dam. Thin reinforced concrete structures are particularly susceptible. Vibration only results when siphon is not flowing full and free from air, so that its elimination lies in designing to avoid sharp radii with consequenent vacua and vortices and in arranging for priming and beaking to be completed in a short time. The arranging of the siphon units to prime at different levels and use of mechanically-operated valves are of particular advantage in this respect. A typical pressure diagram got in experiments by R. H. Freeman is shown in Fig. 13.

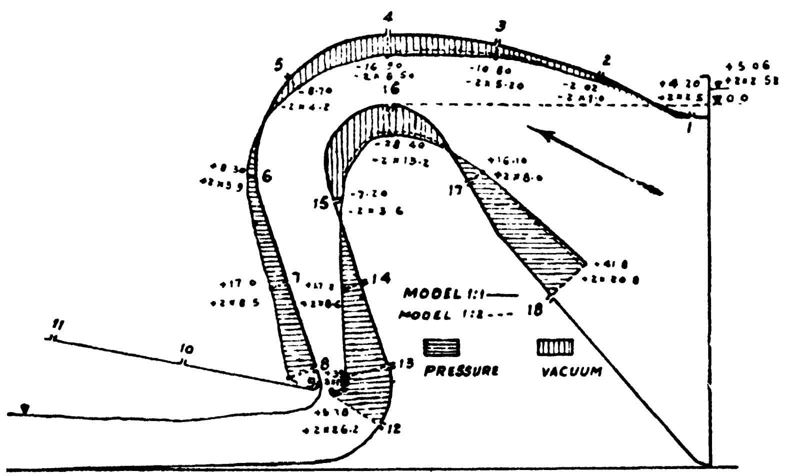


Fig. 13.

8. Ice Trouble in Siphons.

One of the greatest fears in connection with siphons has been the danger of freezing. There should be no horizontal projection under which an ice sheet might catch.

A spell of hard frost is always accompanied by reduced run-off, which, in the case of a hydro-electric scheme means a fall in water level. Before a rise of water level can take place a thaw must have set in, and long before there is any appreciable overflow, any ice will have disappeared. The pipes supplying the still-water wells would likewise thaw out before any appreciable rise had taken place.

It is highly improbable that the interior of the dam will ever reach a temperature approaching freezing, but the horizontol limbs of the siphons are drained as a precaution against ice formation.

9. Hydratomats.

The Hydratomat in its many forms converts the energy from low falls of water in canal systems and rivers to air powers, either compressed air or vacuum and such air power is transmitted to lifting units some distance apart, principally for the purpose of raising water.

From a cursory study of a following, it will be appreciated that the system from beginning to end is free from any working parts and therefore, superietendence, other than that required to start and stop, is eliminated. Moreover it can be safely assumed that the initial cost of construction being mainly simple reinforced concrete is low.

In the installation of the Upper Chenab Canal Punjab, the system may be considered in two parts, where it has been used to lift a seepage drain into canal.

(*a*) Rarefier or vacuum power unit, which is situated at the fall (*b*) Lifting or pumping unit, which in this case is raising water from a drain about 1,500 feet away.

The Rarefier as illustrated in Fig. 14, comprises a siphon which is raised to a height slightly above the vacuum required at the Lifter. In this instance the siphon Rarefier is built into existing masonry piers and is 8 feet 6 inches wide, while the enterance waterway is 6 feet deep.

In the central or partition wall "B" are fixed three Priming ports "F" which, when starting up, are used to prime the siphon. When swung open by levers from the outside, these ports give a free seepage of water from upstream to downstream. Taking the lowest part, which is always submerged, water passes from the upstream leg and on splashing on the downstream, traps much air from the inside of the siphon. The air passes out of the siphon and is freed to atmosphere. This has the effect of creating a partial vacuum inside the siphon and the water level rises untill the next higher port comes into action and so on untill the siphon commences running when all priming gates are closed. For quite a large siphon of this description and to a vacuum of 16 feet with a fall of about 5 feet, the priming is accomplished in a period of 20 minutes.

Flow can take place like an ordinary siphon upwards in passage "A" over the partition wall "B" and thence downwards in passage "C" and so to downstream. Aspirating fins "D" which are perforated, are arranged in the throat of the siphon at intervals across the width. These fins communicate with an air duct "E" built into the top of the partition walls or crest "B" which communicates with the air-main outside, connecting the Lifting unit.

Now the water in passing these perforated fins draws-in air from the air-main which enters the water in the form of small air buobles. This emulsion of air and water passes downwards in passages "C" and out of the siphon where the air is freed to atmosphere.

Thus a partial vacuum is created in any chamber connected to the air-main.

It is when air is admitted to the siphon in increasing volume, that the discharge slows down until a point is reached when the siphon becomes unstable and finally breaks down. To ensure that the siphon runs steadily with the maximum air capacity and at the same time is free from any possibility of breaking down, a special Governor-gear is arranged which effectively and simply controls the volume of air admitted under all conditions. It will be understood that an air-main connects the Rarefiere to the Lifter and thus the Rarefier is continuously extracting air from the Lifter and maintaining a partial vacuum.

Referring now to Fig. 15, it is observed that unit consists of a concrete chamber having an inlet at the lower level "A" communicating with three lift pipes "B" and an outlet from chamber "D" to the higher level "C".

Inside each lift pipes "B" is placed an air inlet "E" communicating with atmosphere by air pipes "F", rising through the roof of the Lifter.

Assuming that the air is shut off from the air inlets and the main air valve on the vacuum main is opened, the water level inside chamber "D" will rise up and similarly inside the lift pipes "B". The air pipes "F" are now opened to atmosphere with the result that air enters the water through the air inlets "E" which are placed slightly above the lower water level. Aerated columns are thus formed inside pipes "B" which having a lower specific gravity than the solid water column inside chamber "D", rise and overflow into the chamber "D" and so to the higher level "C".

Hydratomats Rarefier

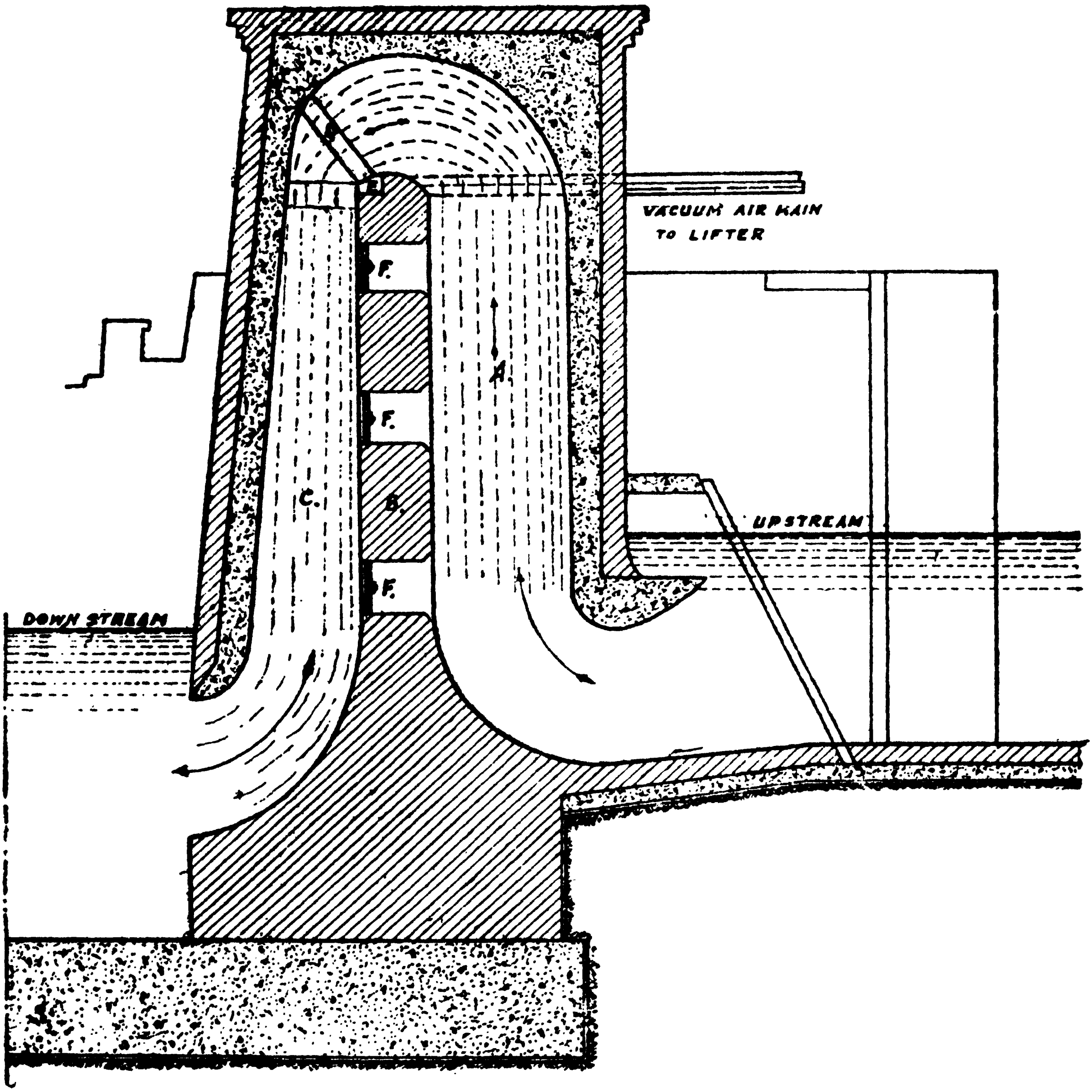


Fig. 11.

The air, which was contained in the water, is liberated in chamber "D" and is continuously extracted by the Rarefier.

Flow in the lifter is continuous and wide variations in head and quantity can be given.

In this instance pumping is from drains and the lift is variable. In fact, the plant is arranged for pumping from 0 to 10 feet and in order that reasonably efficient conditions will prevail over most of the range the air inlets can be raised or lowered to follow the lower water level although it is not necessary to attend strictly to this adjustment.

Hydratomats Lifter

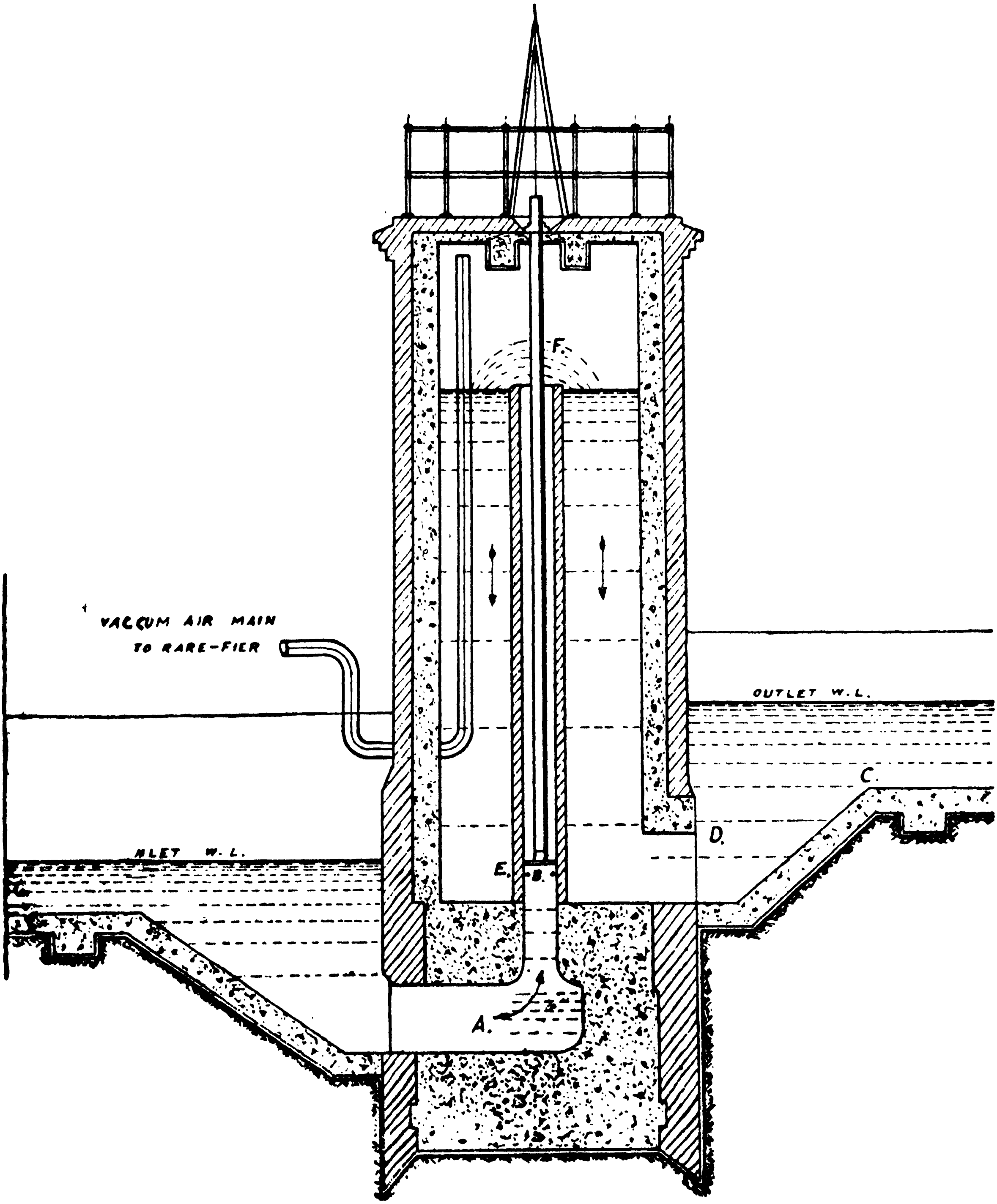


Fig. 15

This type of Hydratomat may be used in irrigation for pumping water into higher level. distributaries, pumping water from drains etc., and in works connected with water supply.

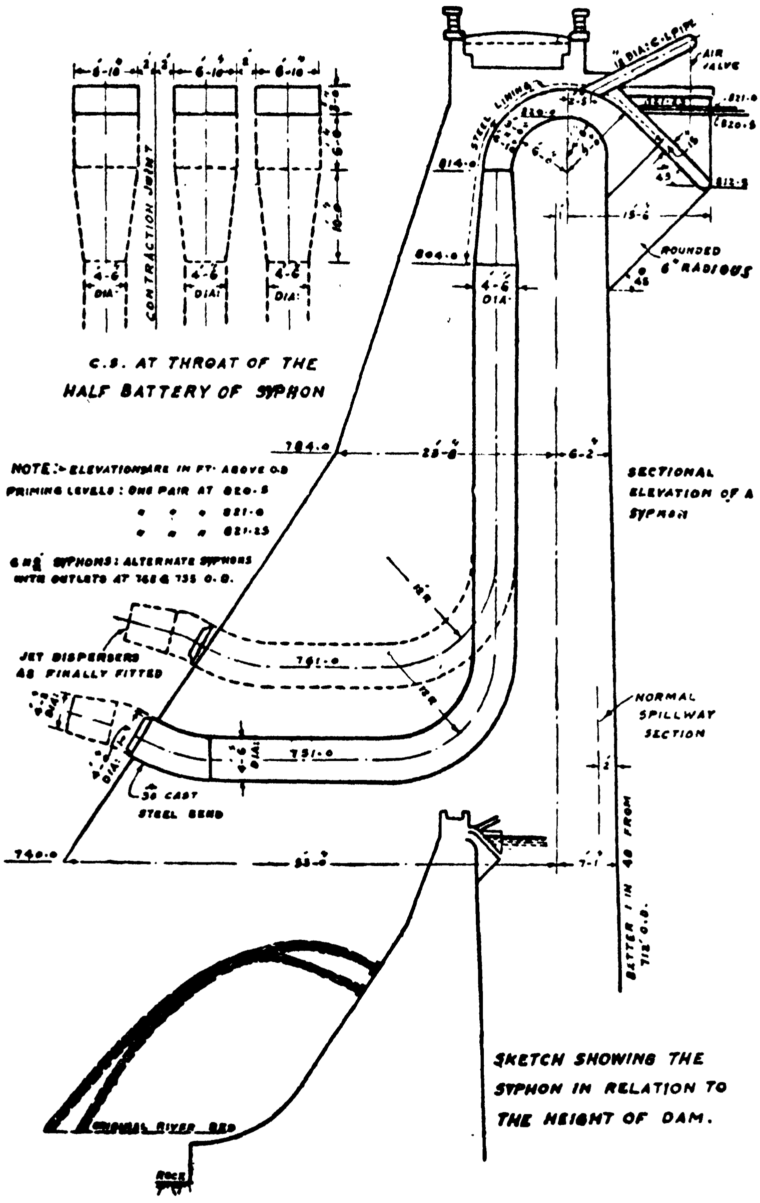


Fig. 16.

On the other hand there are systems which compress air in similarly simple manner and the air power is conveyed to batteries of tube-wells where pumping is carried out by air lift pump Hydratomat Limited, of Victoria Station House, London S. W. 1., are responsible for these developments.

10. Efficieney of Hydratomats.

The efficiency of Hydratomats is rather extremely poor usually from 15 to 20%.

11. *Example I.* **Laggan Dam Siphon.** (by A.H. Naylor)

(*a*) **Data.**

Flood discharge 3600 cusecs with priming head 1·0 ft. Dam section given in Fig. 17. Height 120 ft. ; Base 53·0 ft. ; Top width=15 ft.

(*b*) **Throat depth.**

In an actual model test it was found that the priming depth required was d/4 to d/5. Keeping it d/3, the throat depth will be 3·0 ft. Fig. 16.

(*c*) **Radius of crest.**

This depends on the space available. Keep it 6 feet for the crest and 9 feet for the crown. (Fig. (16).

(*d*) Discharge velocity at the crest for permissible 25 ft. negative pressure $=\sqrt{2g\times 24}=39$ ft. per sec.

Efficiency $\eta=\frac{Q}{47A}=\frac{39\ r_o \log_e (R/r_o)}{47(R-r_o)}=68\%$; where $r_o=6$ and $R=9$

Discharge per ft. width$=\cdot 68\times 47(9-6)\times 1=96$ cusecs ; Velocity at the crown$=39\times \frac{6}{9}$ $=26$ ft. per second.

Pressure at the crown$=-3-\frac{V^2}{2g}=-3-\frac{26^2}{2g}=-13$ feet head of water

(*e*) **Width and number of units.**

The width of siphon tubes at crest is usually 2d. In this case it is kept 6 ft. 10 inches. Discharge of each unit$=6\cdot 83\times 96=650$ cusecs. To give 3600 cusecs 6 units are required.

(*f*) **Outlet.**

(*i*) **Level of outlet.** Permissible velocity in concrete$=50$ ft. per sec. ; $V=C_d\sqrt{2gH}$ where $C_d=\cdot 8$; $\therefore$ Head$=\left(\frac{50}{\cdot 8\sqrt{2g}}\right)^2=60$ feet.

The outlet should be at 60 ft. below crest level.

(*ii*) **Inclination of the outlet.** $\operatorname{cosec}\theta=\sqrt{\frac{2gx}{V^2}+2}$; where x=height above river bed=64 ft. say.

$V=50$ ft. per second ; $\therefore\ \theta=31\cdot 6^\circ$ say 30°

$$l=\frac{V^2}{g}\cos\theta\left\{\sin\theta+\sqrt{\sin^2\theta+\frac{2gx}{V^2}}\right\}=128 \text{ ft.}$$

(*iii*) **Area of outlet.** Area$=Q/V=650/50=13$ sq. ft. ; Keep it 4′—6″ dia (16 sq. ft. arc) on account of bend immediately behind.

(*g*) **Vertieal limb of the outlet and for fixing jet dispersers.**

(*i*) **Transition.** The area at crest=20 sq. ft., Area of the outlet=16 sq. ft.

Length of transition$=H_1-H_2=\frac{Q^2}{2g}\left(\frac{1}{A_o^2}-\frac{1}{A^2}\right)=\frac{650^2}{64\cdot 4}\left(\frac{1}{16^2}-\frac{1}{20^2}\right)=9$ ft.

It is kept 10 feet in original design.

(*h*) **Inlet cowl.**

The area of mouth equals twice the area of the throat. The velocity of entry$=650/40$ $=16$ ft. per second. This is rather high and has caused vortex in the actual. The arcs and entry may be increased from 2·5 to 3 times the area of throat.

(*i*) **Losses of head.** The most important losses of head are due to bends. There is no satisfactory information on the subject and it largely for this reason that scale model experiments are so advisable. The loss is not a property of the bend itself but of the whole assmblage of bends and straight pipes. There is a certain amount of loss due to the redistribution of velocity on entering a bend until an approximation to free vortex flow is established. Then there will be eddy losses on the inside of the bend, additonal to ordinary friction loss, especially when the bend is of a small radius, and finally there will be loss due to the readjustment of velocities in the straight pipe succeeding the bend. It would not be expected, therefore that the loss would be simply proportional to the angle of bend. Bouchayar and Viallet found the relationship between loss and angle of bend to be very irregular, the loss being less for a 60° than that for a 30° bend.

After a careful consideration of available information the following values have been chosen :

Upper bend, angle about 135°; $\frac{2R}{d}=5$; $l=\text{loss}=0{\cdot}35\frac{V^2}{2g}$. Lower bend ; angle 90° ; $\frac{2R}{d}=5$;

$\text{loss}=0{\cdot}30\ \frac{V^2}{2g}$. Outlet bend, angle 30° ; $\frac{2R}{d}=6$; $\text{loss}=0{\cdot}15\ \frac{V^2}{2g}$.

The distribution of velocity across a pipe is not uniform even in the case of a straight pipe. Therefore, the kinetic energy of the issuing jet will be greater than the kinetic energy corresponding to the mean velocity. Owing to the high value of c, the kinetic energy is taken as

$1{\cdot}05\frac{V_2^2}{2g}$ instead of $1{\cdot}12\frac{V^2}{2g}$ as recommended by Professor F.C. Lea for normal velocities.

It is now possible to proceed to the calculations, we have ;

Total Head H = Loss at entrance, (h_e) + loss at bends, (h_b) + friction loss, (h_f) + K.E. of jet, (h_v).

	(a) H=56′	(b) H=66′
$h_e=0{\cdot}5\frac{V_o^2}{2g}=0{\cdot}5\times0{\cdot}04\ \frac{V_2^2}{2g}$	$=0{\cdot}02\frac{V_2^2}{2g}$	$=0{\cdot}02\frac{V_2^2}{2g}$
$h_b=$ Upper Bend, $0{\cdot}35\frac{V_2^2}{2g}$	$=0{\cdot}35\frac{V_2^2}{2g}$	$=0{\cdot}35\frac{V_2^2}{2g}$
Lower Band, $0{\cdot}30\frac{V_2^2}{2g}=0{\cdot}30\times1{\cdot}62\ \frac{V_2^2}{2g}$	$=0{\cdot}49\frac{V_2^2}{2g}$	$=0{\cdot}49\frac{V_2^2}{2g}$
30° Bend, $0{\cdot}15\frac{V_8^2}{2g}=0{\cdot}15\times1{\cdot}62\frac{V_2^2}{2g}$	$=0{\cdot}24\frac{V_2^2}{2g}$	$=0.24\frac{V_2^2}{2g}$
h_f (Q being 125) Mouth 0 to 1 ; $\left.\begin{matrix} l=12 \\ V^2=0{\cdot}11V_2^2 \\ m=1{\cdot}94\end{matrix}\right\}$		
$\frac{lV^2}{c^2m}=\frac{12\times0{\cdot}11}{145^2\times1{\cdot}94}\times64{\cdot}4\ \frac{V_2^2}{2g}$	$=0{\cdot}00\frac{V_2^2}{2g}$	$=0{\cdot}00\frac{V_2^2}{2g}$
Upper Bent 1 to 3		
$\left.\begin{matrix} l=18 \\ V^2=V_2^2 \\ m=1{\cdot}04\end{matrix}\right\}$; $\frac{lV^2}{c^2m}=\frac{18}{145^2\times1{\cdot}04}\times64{\cdot}4\ \frac{V_2^2}{2g}$	$=0{\cdot}05\frac{V_2^2}{2g}$	$=0{\cdot}05\frac{V_2^2}{2g}$
Trasition 3 to 4		
$\left.\begin{matrix} l=10 \\ V^2=1{\cdot}26V_2^2 \\ m=1{\cdot}07\end{matrix}\right\}$; $\frac{lV^2}{c^2m}=\frac{10\times1{\cdot}26}{145^2\times1{\cdot}07}\times64{\cdot}4\ \frac{V_2^2}{2g}$	$=0{\cdot}04\frac{V_2^2}{2g}$	$=0{\cdot}04\frac{V_2^2}{2g}$
Remainder 4 to 8		
(a) $\left.\begin{matrix} l=70 \\ V^2=1{\cdot}62V_2^2 \\ m=1{\cdot}12\end{matrix}\right\}$; $\frac{lV^2}{c^2m}=\frac{70\times1{\cdot}62}{145^2\times1{\cdot}12}\times64{\cdot}4\ \frac{V_2^2}{2g}$	$=0{\cdot}31\frac{V_2^2}{2g}$	

(*b*) $l=87\,;\dfrac{lV^2}{c^2m}=\dfrac{87\times 1{\cdot}62}{145^2\times 1{\cdot}12}\times 64{\cdot}4\;\dfrac{V_2^2}{2g}$ $\qquad=0{\cdot}39\;\dfrac{V_2^2}{2g}$

$h_v=1{\cdot}05\dfrac{V_2^2}{2g}=1{\cdot}05\times 2{\cdot}63\;\dfrac{V_2^2}{2g}$ $\qquad=2{\cdot}76\;\dfrac{V_2^2}{2g}$ $\qquad=2{\cdot}76\;\dfrac{V_2^2}{2g}$

(*a*) $\text{Total}=4{\cdot}26\dfrac{V_2^2}{2g}$ (*b*) $\text{Total}=4{\cdot}34\;\dfrac{V_2^2}{2g}$

where H is measured from the upstream water level at priming to the centre of the nozzle. Hence,

$$\left.\begin{aligned}&(a)\quad V_2=29{\cdot}1,\ \therefore\ Q=a_2V_2=20{\cdot}3\times 29{\cdot}1=590\text{ cusecs}\\&(b)\quad V_2=31{\cdot}3,\ \therefore\ Q=a_2V_2=20{\cdot}3\times 31{\cdot}3=640\text{ cusecs}\end{aligned}\right\}$$

The efficiencies are (*a*) $\dfrac{590}{20{\cdot}3\times 47}=62\%$; (*b*) $\dfrac{640}{20{\cdot}3\times 47}=67\%$.

These compare with a permissible efficiency for this design of upper bend of 68%.

(*i*) Effect of the siphon on the dam. The effect is shown in Fig. 17.

12. *Example II.* **Marikanve Dam, India** (Prof. K.D. Joshi, Mysore)

Design a spillway siphon battery from a flood discharge of 20,000 cusecs for the Marikanve Dam of the section given in Fig. 18. The priming head not to exceed 15 ft.

(*a*) **Throat depth.**

$d=3\times 1{\cdot}5=4{\cdot}5$; Top width is only 16 feet ; The radius of crest$=4{\cdot}5$ ft. Radius of the Crown$=9{\cdot}0$ ft. The crest shall be a full semi-circle.

(*b*) **Discharge.**

For 24 ft. negative pressure, the velocity at crest$=39$ ft./sec.

$$\eta=\text{efficiency}=\frac{Q}{47A}=\frac{39r\log_e(R/r)}{47(R-r)}=58\%\ ;\ \text{where } r=4{\cdot}5\text{ ft. ; } R=9{\cdot}0\text{ ft.}$$

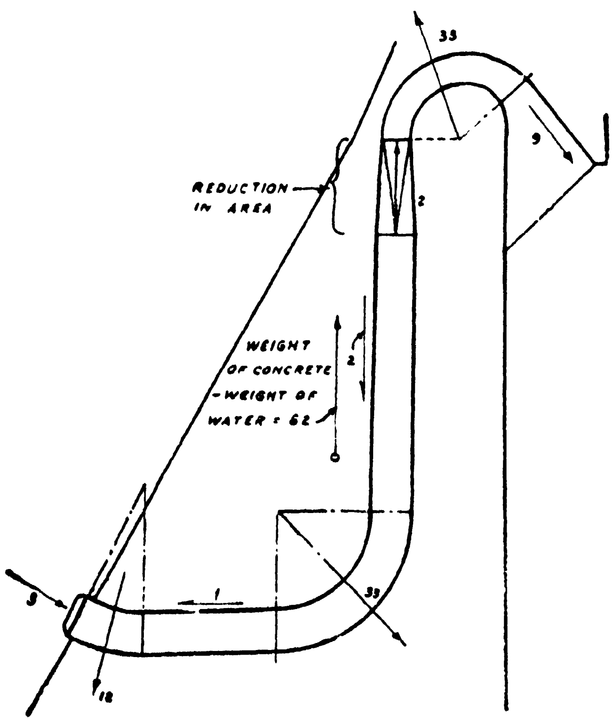


Fig. 17

Discharge$=\cdot 58\times 47\times 4{\cdot}5\times 1=123$ cs.

Velocity at the crown$=V=39\times\dfrac{4{\cdot}5}{9}=19{\cdot}5$ ft. per second. Pressure at the crown$=$

$$-4{\cdot}5-\frac{V^2}{2g}=-4{\cdot}5-\frac{19{\cdot}5^2}{2g}=-10{\cdot}5\text{ ft.}$$

(*c*) **Width and number of units.**

Let width$=2d=2\times 4{\cdot}5=9$ feet. Discharge for unit$=9\times 123=1107$ cs. No. of units for flood discharge of 20,000 cusecs is 19.

(*d*) **Outlet.**

(*i*) **Outlet cowl.** The co-efficient of discharge as given by efficiency$=\cdot 58$; permissible velocity in concrete section $=50$ feet per second. Velocity with 100 ft. head.$=\cdot 58\times\sqrt{2g\times 100}=46{\cdot}4$ ft. per second or say 50 ft./sec.

(*ii*) **Angle of exite.** Cos θ

$$=\sqrt{\frac{2gx}{V^2}+2}\ ;\ x=32\text{ height of outlet}$$

above bed.

$\theta=37°$; $V=50$ ft. per second.

(*iii*) Horizontal distance from

outlet where jet sheet meets the bed.

$$l=\frac{V^2}{g}\cos\theta\left\{\sin\theta+\sqrt{\sin^2\theta+\frac{2gx}{V^2}}\right\};\ V=50 \text{ and } x=32\ \therefore\ l=132 \text{ ft.}$$

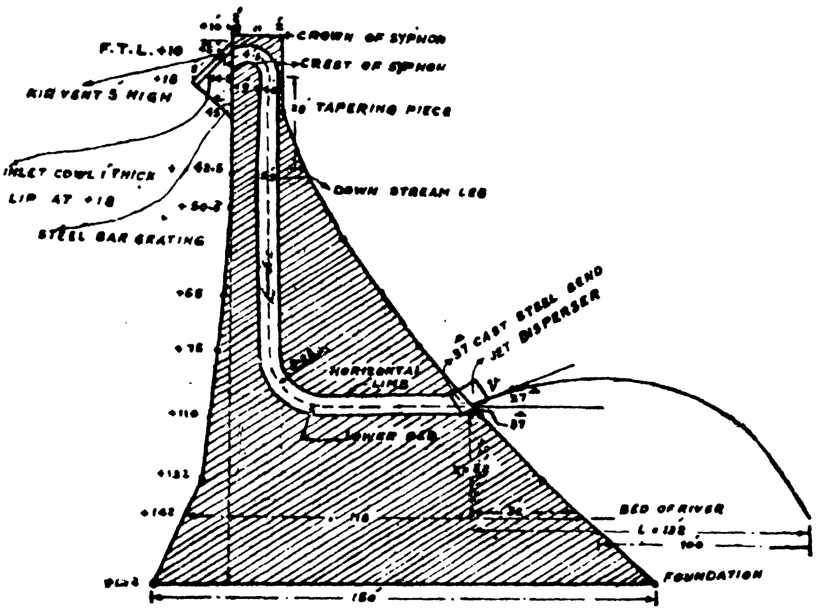


Fig. 18.

(*iv*) Area of the outlet $A_o=\frac{Q}{V}=\frac{1107}{50}=22$ sq. ft.

(*e*) **Vertical leg.**

Length of transition for area change from 40·5 sft. at crest to 22 sft. in outlet.

$$h_2-h_1=\frac{Q^2}{2g}\left(\frac{1}{A_o^2}-\frac{1}{A^2}\right)=\frac{1107^2}{64\cdot4}\left(\frac{1}{22^2}-\frac{1}{40\cdot5^2}\right)=28 \text{ ft.}$$

The diameter of the outlet : $D=\sqrt{\frac{4}{\pi}\times22}=5\cdot3$

(*f*) **Inlet cowl.**

The area at entry=twice the area at the crown$=2\times9\times4\cdot5=81$ sft.

Velocity of entry$=\frac{1100}{81}=12\cdot5$ ft. per second. The head required for this velocity over the top$=\frac{V^2}{2g}=\frac{12\cdot5^2}{2g}=2\cdot5$ ft. nearly

(*g*) **Air vent.**

Area of air vent $\frac{1}{12}$ of area at throat$=\frac{1}{12}\times9\times4\cdot5=3\cdot484$ ft.

Height of vent$=\frac{3\cdot4}{9}=\cdot4$ ft. say$=5$ inches

Note —The crest need not be a complete semicircle but only upto 60° arc on the reservoir side. This will give 1·6 ft. additional thickness behind the tube instead of 1·5 ft. only. The entry area at the entry should be increased to 2·5 times to avoid vortex formation.

13. *Example III.* **Plain Siphons.**

Fig. 19, represents a siphon in nature. The velocity of the water in the siphon in meters per second is $V=\sqrt{\dfrac{2gH}{(1+\zeta)+\lambda(L/d)}}$ in which H is the difference in water levels in meters, "ζ is equal to 0·4 for a plain sharpedged circular orifice and is equal to 0·1 for a well-rounded mouthpiece." λ is a friction factor dependent on the nature of the material, L is the length of the siphon in meters, d its dia. in meters. Also p_s, (the pressure at the summit) is $p_s=\gamma(h_0-H_1-\dfrac{V_s^2}{2g}-\Sigma\omega)$ in which $h_0=10{\cdot}3$ meters, the pressure head due to the atmosphere ; H_1 is the height of the summit of the siphon above the upper water surface, measured in meters ; V_s is the velocity of the water at the summit ; and $\Sigma\omega$ is the total loss in pressure head upto the summit due to the resistance of the pipe. P_s must be equal to or greater than e, the pressure of the water vapour if the siphon is to operate.

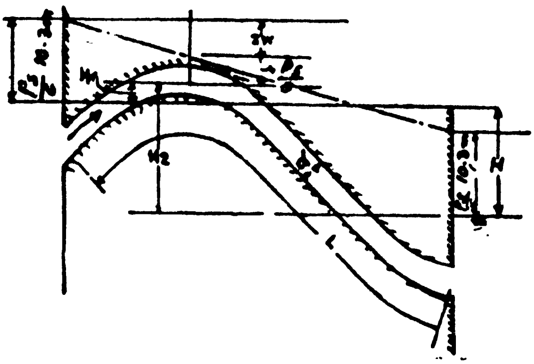


Fig. 19

This is, $P_s \geqq e$ for the siphon to operate or

$$\frac{P_s}{\gamma} \geqq \frac{e}{\gamma} \geqq Z_e \text{ for the siphon to operate}$$

Values of e and for Z_e for water are to be found in the following table.

Pressure Temperature Table for water Vapour.

Temperature in degrees Celsius (T)	0°	20°	40°	60°	80°	100°
e in atmospheres	0·006	0·023	0·072	0·196	0·462	1·000
Z_e in meters	0·06	0·24	0·74	0·02	4·81	1·03

Thus, if water at 40° is used, $\dfrac{P_s}{\gamma}$ must be equal to or greater than 0·74 meters if the siphon is to operate.

In the model, $V_m=\sqrt{\dfrac{2gH_m}{(1+\zeta)+\lambda_m(L_m/d_m)}}$

Where $H_m=\dfrac{H}{n}$; $L_m=\dfrac{L}{n}$; $d_m=\dfrac{d}{n}$

ζ, as before, lies between 0·4 and 0·1 and should be the same as in nature. λ_m will probaly not be the same as λ since the frictional resistance to flow for the model and for nature will not necessarily be the same.

$$\text{the model is } (p_s)_m=\gamma\left(h_o-(H_1)_m-\frac{(V_s)_m^2}{2g}-(\Sigma\omega)_m\right)$$

$(P_s)_m$ must be equal to or greater than $\dfrac{e}{n}$ if the model siphon is to operate. $(P_s)_m \geqq \dfrac{e}{n}$ for the model to operate. or $\dfrac{(p_s)_m}{\gamma} \geqq \dfrac{Z_e}{n}$ for the model to operate.

Example. If the temperature of the water in nature is 40° and n is 10, will the model operate ?
For water at 40°, $Z_e=0{\cdot}74$ meters (see table).

Hence, $(Z_e)_m = \frac{0{\cdot}74}{10} = 0{\cdot}074$ meters which would indicate a temperature for the water less than 5°C (see table) which is impracticable.

Example. Assume the following data for the model of a siphon :—
$n=10$, $H_m=0{\cdot}6_m$, $d_m=0{\cdot}08_m$, $L_m=8{\cdot}0_m$, $(H_1)_m=0{\cdot}4_m$, $T=T_m=20°C$, $\zeta=0{\cdot}4$ and $\lambda=\lambda_m=0{\cdot}025$, and $\Sigma\omega=(\Sigma\omega)_m=\lambda_m \frac{L_m}{d_m} \cdot \frac{(V_s)_m}{2g}$; Determine if the model siphon, as well as, the siphon in nature will operate.

$$V_m=\sqrt{\frac{2gH_m}{(1+\zeta)+\lambda_m(L_m/d_m)}}=\sqrt{\frac{19{\cdot}62\times0{\cdot}6}{(1+0{\cdot}4)+0{\cdot}25(8{\cdot}0/0{\cdot}08)}}=\sqrt{3{\cdot}02}=1{\cdot}74 \text{ m/s}$$

Now $\frac{(p_s)_m}{\gamma}=h_0-(H_1)_m-\frac{(V_s)_m^2}{2g}-(\Sigma\omega)_m$; or $\frac{(p_s)_m}{\gamma}=10{\cdot}3-0{\cdot}4-\frac{1{\cdot}74^2}{19{\cdot}62}-0{\cdot}025\frac{8{\cdot}0}{0{\cdot}08}\times\frac{1{\cdot}74^2}{19{\cdot}62}$
$=9{\cdot}36_m$. From the table, page 305, for $T=20°C$, $Z_e=0{\cdot}74_m$. Since the value for $\frac{(p_s)_m}{\gamma}$ of $9{\cdot}36_m$ if greater than the value of Z_e of $0{\cdot}74_m$, the model siphon will operate.

In nature, $V=\sqrt{\frac{2gH}{(1+\zeta)+\lambda\,(L/d)}}$; . Also $H=H_m\times n=0{\cdot}6\times10=6{\cdot}0_m$;
$H_1=(H_1)_m\times n=0{\cdot}4\times10=4{\cdot}0_m$; $L=L_m\times n=8{\cdot}0\times10=80{\cdot}0_m$ and $d=d_m\times n=0{\cdot}08\times10=0{\cdot}8_m$.

Hence $V=\sqrt{\frac{19{\cdot}62\times6{\cdot}0}{(1+0{\cdot}4)+0{\cdot}025\times(80{\cdot}0/0{\cdot}8)}}=\sqrt{30{\cdot}2}=5{\cdot}50$ m/s

Now $\frac{p_s}{\gamma}=h_o-H_1-\frac{V_s^2}{2g}-\Sigma\omega.$

Then $\frac{p_s}{\gamma}=10{\cdot}3-4{\cdot}0-\frac{5{\cdot}50^2}{19{\cdot}62}-0{\cdot}025\times\frac{80{\cdot}0}{0{\cdot}8}\times\frac{5{\cdot}50^2}{19{\cdot}62}=10{\cdot}3-4{\cdot}0-1{\cdot}5-3{\cdot}9=0{\cdot}9_m$ from the table, page 305, for $T=20°C$; $Z_e=0{\cdot}74_m$. Since $\frac{p_s}{\gamma}$ has a value of $0{\cdot}9_m$. which is greater than the value for Z_e=of $0{\cdot}74=_m$, the siphon in nature will operate.

14. *Example IV.* **Siphon to Maintain a Constant Level in the Parent Channel.** (Crump).

These are plain siphons as described above. They are constructed to maintain a constant level in the parent channel either to feed a high level off-take with fluctuating supplies in the parent as a siphon at R.D. 67,000 Gujrat Branch, Upper Jhelum Canal Punjab, or to feed the turbine plants constructed for Hydro-electric Installation at a constant head with varying supply conditions in the canal such as at Renala Khurd, Lower Bari Doab Canal, Punjab.

15. *Example V.* **Gulping Siphon** (Crump).

Rasul Siphon of Upper Jhelum Canal.

The Upper Jhelum Canal runs in the reach along the steep slopes of pabbi hills on the one side and the river Jhelum on the other, about 75 feet below. There are a lot of barrel-type siphons under the canal to pass the storm water from these hill sides. The storm water comes down the hill side which is sun dry and usually silts up the siphon barrels when the flood is dropping inspite of steep slopes of the order 1/300 available on the outfall. If the outfall and barrel are not closed before the next rain, the masonry barrels are blown on account of the blockade of water-way as the silt in the barrels gets consolidated and strengthened with grass growth in a month or so.

E.S. Crump, I.S.E., cured the trouble by constructing a Gulping siphon (spillway siphon). The intermittent flow in low supplies served to keep a defined channel upstream and a clear channel downstream in the outfall as shown in plan and section in Fig. 20.

Data.

Siphon under canal 12 ft. wide barrel ; 320 ft. long with segmental invert arch in floor and semicircular arch above top. Flood discharge $=1400$ cusecs ; Flood level upstream of siphon $=786{\cdot}25$; Crest level of 34 ft. wide ; weir $=781{\cdot}75$; Siphon width $=6{\cdot}0$ ft. Siphon depth $=6{\cdot}0$; Crest level of siphon $=781{\cdot}0$

A. **Calculations.** Discharge $=3{\cdot}09\times 34\times 4{\cdot}5^{3/2}+8\times\sqrt{2g}\times 6\times 6\sqrt{5{\cdot}25}=1000+420$ $=1420$ cusecs.

Calculations for siphon reinforced concrete.

(B) **Roof of Siphon.** This is at all times subject to pressures less than atmospheric. The greatest negative pressures occcur when the upstream water-level is lowest, namely at R.L. 776·50. With this level the maximum working head is about 5·0 feet. Assuming the siphon formula see Fig. 20.

$Q=5A\sqrt{h}$; the velocity $V=5\sqrt{h}$; and the kinetic energy $\frac{V^2}{2g}=\frac{25h}{64{\cdot}4}=\frac{25\times 5}{64{\cdot}4}=2$ feet say ; the static pressure at the crown is that due to a height of $787{\cdot}0-776{\cdot}5=10{\cdot}5$ ft. of water.

The total negative pressure is, therefore, that due to $10{\cdot}5+2{\cdot}0=12{\cdot}5$ feet of water or $12{\cdot}5\times 62{\cdot}5=782$ lbs/sq. ft. Add for weight of slab $=118$ lbs/sq. ft. Total $=900$ lbs/sq. ft.

(C) **Sides of siphons.** The average pressure on the sides at the crown of the siphon is that due to $12{\cdot}5-\frac{\quad}{2}=9{\cdot}5$ ft. of water *i.e.*, $W=9{\cdot}5\times 62{\cdot}5=593$ lbs per foot.

(D) **Side of delivery pit.** These will be subjected to earth pressure from the earth below the existing floor. The pressure intensity may be taken as 30 d lbs/sq. ft. at depth d. The resultant per ft. run for the depth d will, therefore, be 15 d^2. Instead of building a retaining wall it is proposed to reinforce the sides and support the vertical slabs by R.C. beams placed along the upper edges of the pit and normal to the siphon roof at the siphon exit. The floor beam will be 15 ft. long and loaded at intervals of one foot measured from the downstream end 1, 2, 5, 10, 25, 50, 90, 150, 180, 220, 250, 280, 320, and 320 lbs. respectively.

Total moment $=20885$ ft. lbs.

The reduction R at the upstream end is thus given by $R\times 15{\cdot}0=20885$ ∴ $R=1392$ lbs. At 5·0 ft. from the upstream end the bending moment is $1392\times 5{\cdot}0-(320\times 4+320\times 3+280\times 2+250\times 1)=6960-3050=3910$ ft. lbs.

At 6·0 ft. from the end it is $3910+1392-1390=3912$ ft. lbs. These being practically equal, indicate that they are near the maximum value which may be taken as 3912 ft. lbs. For a beam of 9″ in width this gives an intensity of $\frac{3912}{0{\cdot}75}=5213$ ft. lbs per foot $=5213$ in lbs per inch.

Giving : $h=\sqrt{\frac{M}{74}}=\sqrt{\frac{5213}{74}}=8{\cdot}4$ inches

The beam will be 9″ × 9″ with 0·555 percent reinforcement *i.e.*, $a=\cdot00555\times 9\times 8{\cdot}4=0{\cdot}42$ sq. in. Allow 3 No 1/2″ dia bars. For the short beams of 6·0 span, we may assume these uniformly loaded with a load of 300 lbs per ft.

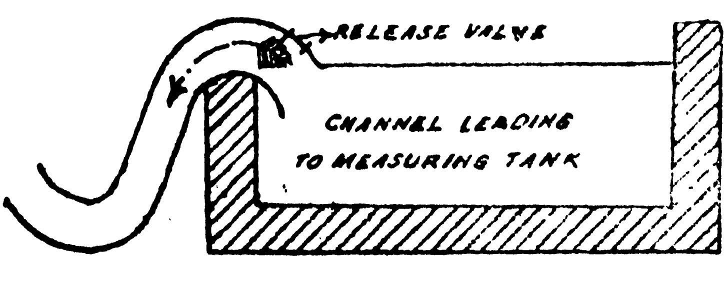


Fig: 21

As this is less than the loading on the sides of the siphon at the crown, the reinforcement already provided is ample to take this load, provided the vertical reinforcements already provided are also ample to take this load and are made to overlap the last three of the inclined rods. The latter rods are placed on 5″ centres, so that we may assume that the terminal reactions

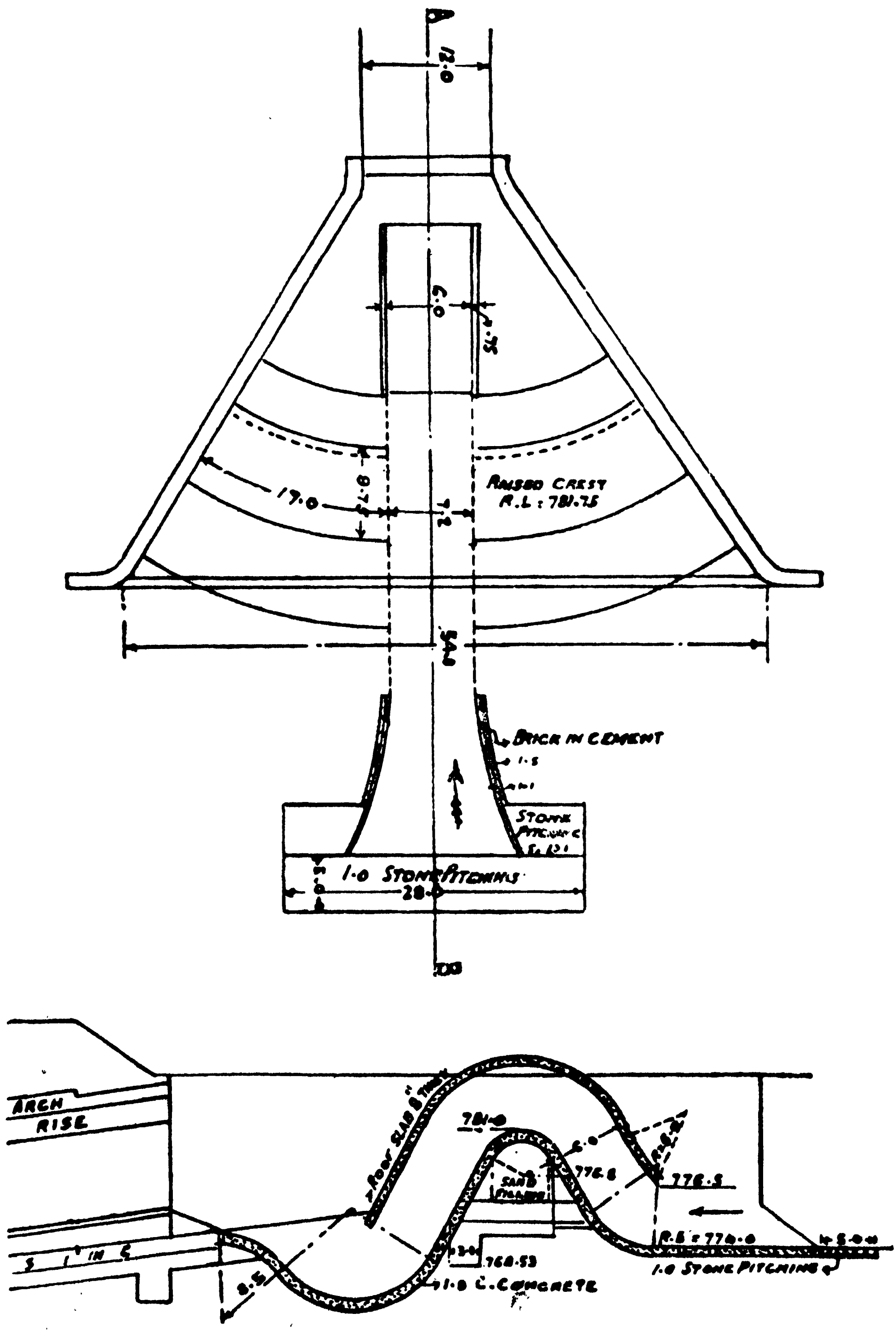

12·0
6·0
RAISED CREST
R.L : 781·75
17·0
BRICK IN CEMENT
STONE PITCHING
1·0 STONE PITCHING
28·0
ARCH
RISE
781·0
SAND FILLING
776·5
R.L : 774·0
1·0 STONE PITCHING
1·0 C.CONCRETE
768·53

Fig. 20

from the two beams are spread over the lower one ft. length of the 8″ roof slab. The total compression on this strut amounts ; 1392+900=2292 lbs. The cross section of the strut is $8\times12=96$ sq. inches so that neglecting the assistance given by reinforcement the compressive stress is only $\frac{2292}{96}=24$ lbs sq. ft. which is very small and calls for no further reinforcement than that already provided for the roof slab.

16. *Example VI.* **Use of Siphon as Automatic Make-and-Break Arrangement.**

Plain siphon designed as shown in the sketch Fig. 21 was used by the author to eliminate the errors in tank measurements of the outlet discharge due to varying levels in the supply channel as described in the Author's paper No. 237, Punjab Engineering Congress Lahore, 1940.

The release valve admitting air at atmospheric pressure is opened and water from the channel let into the tank and the stop-watch started.

PART II

CANAL IRRIGATION

CHAPTER XIV

Distributary Head Regulators and Distributors

1. Introduction.

A distributary head regulator is an important feature of the problems of distribution of supplies. It is moreover, a necessary link between an earthen parent channel and an off-taking distributary. A distributary head is a regulator a meter of supply and a silt selective structure.

A distributary head was designed in the beginning simply as a regulator. It used to have a floor or a crest at bed level of parent channel with gates to regulate supply. The regulator was undershot. The distridutries were boosted with excessive silt charge and their discharge tables, based on gauges fixed in their head reaches, had to be revised occasionally.

To begin with, silt exclusion was the first to receive attention. A silt wall in the parent channel away from the gates was the first device tried in practice. It very soon proved to be an inefficient remedy.

Gibb did pioneer work in successful exclusion of silt (as described in Paper No. 28 of the Punjab Engineering Congress, 1916). Construction of a Gibb wall did meet with considerable success in some cases. (Work on similar lines by Elsdon was also discribed in Paper No. 30 of the Punjab Engineering Congress, 1916.)

Woods described his work in paper No. 29 of the Punjab Engineering Congress in 1916. It was provided to exclude silt by making the regulation overshot, using rising cill gates. Metering of supply was arranged by making a head regulator to work as a free fall over the gates with overshot regulation. It was the first attempt to correlate the silt selective function of a distributary head regulatcr with the wcrking of an off-taking channel. The device met with a little success only in a few cases. Later on Walton designed a useful rising gate on similar lines.

King developed a silt excluding device in the form of silt vanes (as described by him in Paper Nos. 41, 62 and 169 of the Punjab Engineering Congress). Gibb walls as used by their author on the Lower chenab Canal, East Circle were soon provided with R. C. slabs with a tunnel below them. This developed the design of the skimming platform as a device to exclude silt. Skimming platform and silt vanes met with considerable success in some cases, but they failed badly in cases where supply in the parent channel was disturbed before the platforms or the vanes could function. Gibbs silt wall, Wood's rising gates, King's silt vanes and Routh's skimming platforms were useful silt excluding devices. They served their purpose very well. but they did not approach the ideal *i.e.*, where to exclude silt, and how much of it. The author in his paper No. 189 on a silt selective Distributory Head Regulator, Punjab Engineering Congress 1936 made up this deficiency by producing a design which would pass as much silt as the off-taking distributary could carry according to the C.V.R. allowed in it with the available slope.

Metering of supplies at the distributary head regulators was the last to receive attention. This was arranged in the form of open flumes as described by Crump in Irrigation Branch Paper No. 26 Class A and as improved upon *vide* paragraph 15 Chapter X of this part.

2. Old Under-Shot Distributary Head-Regulator.

A typcial cross-section is shown in Fig. 1. The regulation was done in this case on the crest which used to be at the bed-level of the parent channel and in some cases a foot higher.

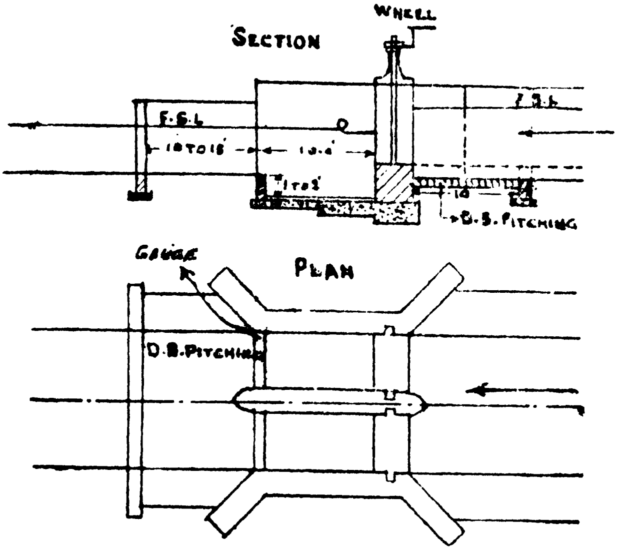


Fig. 1

The crest was followed by a cistern 1 to 2 ft. deep and about 10 feet long. There used to be a dry brick (D.B.) pitching downstream of the cistern. The upstream wings were sloping 1 in 1 and splayed at 45°. In some cases D.B. pitching in bed in parent channel was put in as shown in plan and section. The iron gates used, were lifted by winches rising in grooves about 6" deep as shown in the plan. The regulation was done on a gauge usually fixed some 50 ft. downstream of the head and in some case fixed against the downstream return wall at G in the plan, Fig. 1.

There are no experiments on record which would give the silt conductive power of the head but it can easily be imagined that it would be of the order of 125 to 130 percent by weight of the silt charge in the parent channel. The silt trouble was most acute in the off-taking channels fitted with such regulators.

These head regulators were provided with silt walls as shown dotted in the cross section but served no useful purpose because the draw towards the head was so great that the silt could jump over these walls.

3. Wood's Rising Cill Gate (Overshot Regulators).

A regulator of the type which existed at the head of Salam distributary, Lower Jhelum Canal is shown in Fig. 2. The distributary takes off at R. D. 24700L of Northern Branch. It had a raised crest with a rising gate behind it after the design of Woods. The basic principle in this design is, that top water in the parent channel contains relatively low silt charge and should be taken into the distributary.

Results of an experiment on this type of head were published by the author in paper No. 189 P. E. C. Lahore. Silt taken by the Head regularor is 110·4% by weight on the average and sieve analysis shows that the distributary took at least as much coarse silt as that in the parent channel.

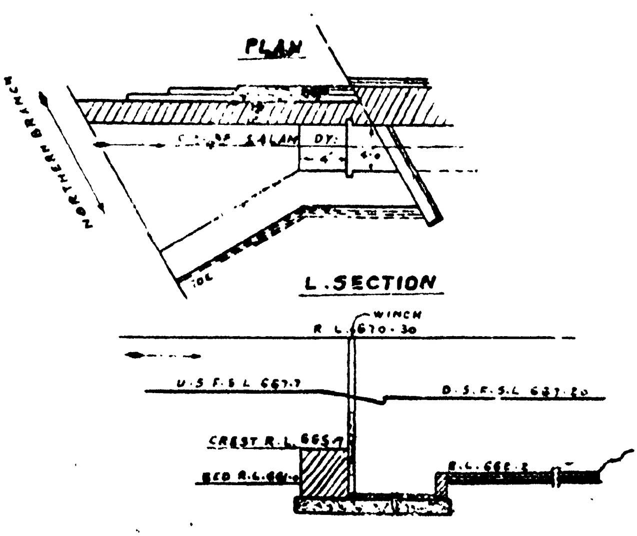


Fig. 2

Although it took very nearly surface water of the parent chennel, the silt conductive power of the head was very high. It was due to disturbance and eddies upstream of the gates caused by unsuitable approaches. Even coarse silt easily jumped over the vertical crest and the gates. The Salam distributary was, therefore, a bad silting channel. It was 18·0 ft. wide with 1·0 foot depth for a discharge of 30 cusecs and had a slope of 1 in 2080. This expereiment showed that in order to exclude silt it was not sufficient to take simply surface water from a parent the channel, but also it was imperative to provide suitable approaches upstream, so that there was no disturbance upstream of the crest.

4. Gibb's Distributary Head Regulator.

The existing design of Fatehpur Distributary, Lower Jhelum Canal head regulator is a Gibb design given in Fig. 3.

The head is an open flume and is provided with Gibb's groyne wall on lines of Elsdon's suggestions described in paper No. 30 of the Punjab Engineering Congress. 1916

Head regulator of Fathepur Distributary off-taking R. D. 83 300 of Northern Branch

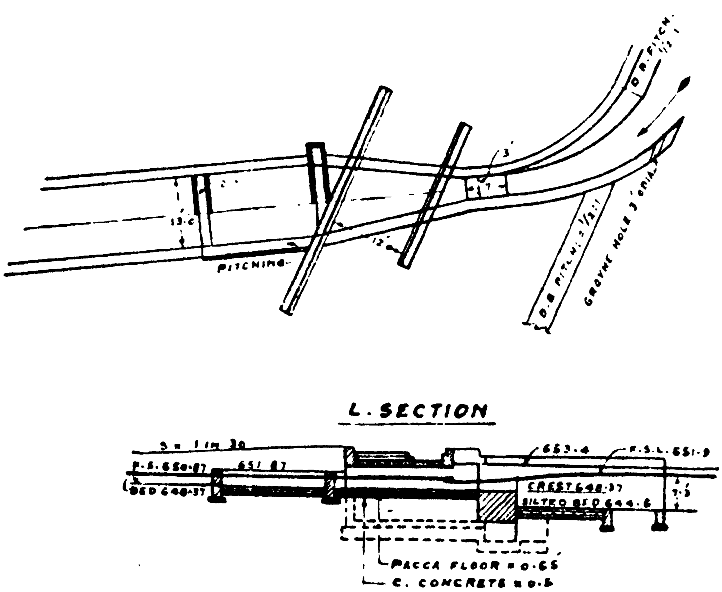


Fig. 3

The basic principle underlying this design is that the cross waves of silt should not to allowed to enter the distributary by constructing a Gibb groyne wall and the amplitude of these waves should be increased by providing pitching on the side upstream of the head in the parent channel. The waterway at the entrance to the chamber, enclosed by the groyne, is allowed considering the effective depth as two-third. A hole of three feet diameter is provided in the Gibb wall to escape the additional supply from the approach chamber. This hole is at the bed-level of the parent channel. There is a dry brick side-pitching 30 ft. long upstream of the head in the parent channel.

The velocity is increased on the sides on account of the presence of the side pitching this reduces the difference between the side velocity and the mean velocity of the parent channel, as water approaches the head ; but increases the amplitude of the waves of cross flow of silt near the bed, from the middle of the channel towards the sides. Cross waves of the silt, thus miss the entrance of the head and are intercepted by the groyne wall (Para 15 (d) of Chapter VI Part II),

Silt observations were carried out by the author as published in paper No. 189 Punjab Engineering Congress. Silt taken by the head regulator is about 98·75% of the silt passing downstream of the fall at R. D. 64000 Northern Branch, but the silt taken is somewhat finer than that in the Northern Branch. Mithalak Distributary is situated between the Fatehpur Distributary head and the fall. Silt exclusion at the head regulator of this distributary influences to some extent the results of this experiment. It is concluded that the design of the Fatehpur distributary head is just a suitable design to give a proportionate share of silt to an off-take.

5. King's Silt Vanes.

These have been used with success in some cases in conjunction with design of the distributary head regulator as discribed in paragraph 2 above. They are shown in plan in Fig. 4. The basic principle on which they are designed is that the water near the bed of the parent channel contains retatively a high silt charge, Fig. 9 Chapter II Part VI, and should, therefore, be deflected away without disturbance. Water entering the distributary would thus contain a relatively low silt charge and grade. The principle is very sound, but it failed in some cases where there was a violent approach with a strong draw towards the head picking silt over the vanes.

The silt vanes are usually thin R. B. or R. C. walls 3" thick constructed on the *pacca* platform as shown in Fig. 4. They are sometimes metallic as in Fig. 4 (b). The radius of vanes is kept more than 25 or 30 ft. usually, they are cut short when they are inclined to the straight flow at an angle of 30°.

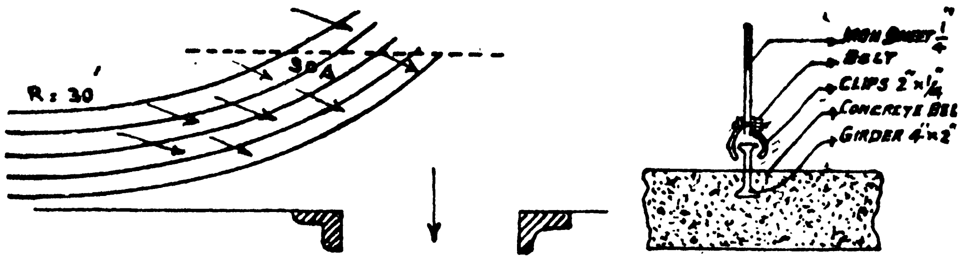


Fig. 4.

The height is ⅓ to ¼ depth. The spacing between them is 1½ times the height. The number of the vanes depends on the discharge of the channel to off-take relative to that in the parent channel. The proportional bed should be covered by vanes with a minimum number of 3.

There are no actual observations on record to give the silt conductive power of heads provided with this device.

Head Regulator of the Satghara distributary off-taking R. D. 76000 of Northern Branch.

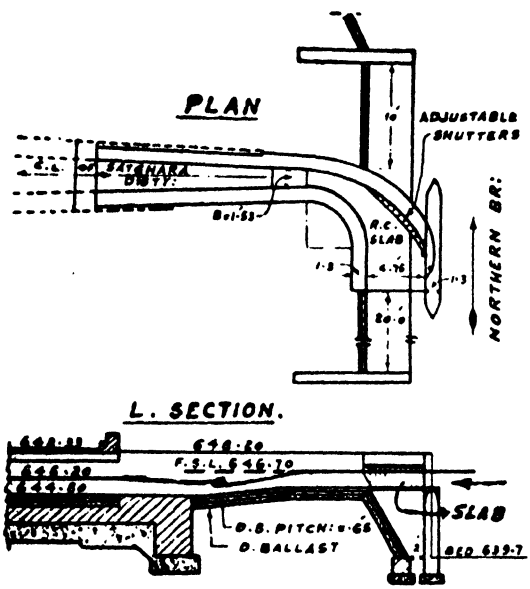


Fig. 5

6. Head With a Skimming Platform.

An extract from experiments described by the author on this type in paper number 189 Punjab Engineering Congress, is given below :—

(*a*) This type is constructed at the Satghara Distributary head regulator taking off at R. D. 76000 L Northern Branch. The design of the head is given in Fig. 5. It is an open flume head regulator, provided with a Gibb wall and a skimming platform. A circular groyne wall is provided with adjustable regulating shutters above the reinforced concrete platform. The depth on the platform is 1·5 feet against normal supply depth in the parent channel of 7·0 feet. Shutters are adjusted in such a way that there is no draw-down at the nose of the groyne wall. A surface float in the parent channel put in line with the nose of the groyne wall comes straight to the nose. There is no disturbance in the supply approaching the approach chamber above the platform. There is a further clarification of silt on the platform by escaping the supply straight below the shutters. In this case the head has the maximum efficiency as a silt-excluding device and the tunnel below the platform is never choked.

The experiment was carried out in two parts. The first part consists of three observations with shutters working, and the second part with the shutters down, as if the groyne wall above the platform was a solid one. In these experiments the silt of the Northern Branch was measured at the fall at R. D. 64000, Northern Branch and the silt in the distributary was measured from the hydraulic jump downstream of the head of the distributary.

Results of the first part of the experiment as published in paper No. 189 P. E. C. show that the silt conduction of a head regulator with a skimming platform was as low as 72·9% by weight and that it took distinctly fine silt in comparsion with silt in the parent channel upstream of it.

Results of the second part show that even a skimming platform might fail to achieve high silt exclusion by its defective working.

The water way at the entrance above the platform is an important factor controlling the entry of silt into the chamber enclosed by the groyne. From the following practical considrations it has generally to be more then that required.

1. To suite the average supply conditions in the parent channel,
2. to allow for *rabi* and *Kharif* supply levels in the parent channel,
3. to allow sufficient width between the pier and the toe of the sides so that the tunnel is not choked.

Supply of the distributary is limited to the authorised discharge at the meter flume downstream of it, and the excess supply turns round the nose of the groyne wall. This causes a considerable draw-down and disturbance near the nose of the groyne wall.

Surface floats from near the berm or parent channel do not even enter the approach chamber. Water approaching the head gets a twist, and there is produced a horizontal roller with axis parallel to the central line of the parent channel. Muddy water at the bottom of the parent channel thus enters the head and surface water escape into the parent channel. The conduction of the head was, therefore, 89·7% in comparison with that in the parent channel. Round eddies are produced just at the entrance of the tunnel, and work rotating with their axis perpendicular to the bed, below the platform. Round eddies cannot carry silt through the tunnel. When the tunnel gets choked to on appreciable extant, the efficiency of the head as a silt excluding device falls. The remedy is to provide shutters as actually done in the case of the head regulator.

(*b*) **Cantilevered skimming platform.**

This type is constructed at the head regulator of the Khunan Distributary taking off at R. D. 207,000 of the Northern Branch, Lower Jhelum Canal. The design of the head is given in Fig. 6. It is simply a flume regulator provided with a centilevered skimming platform.

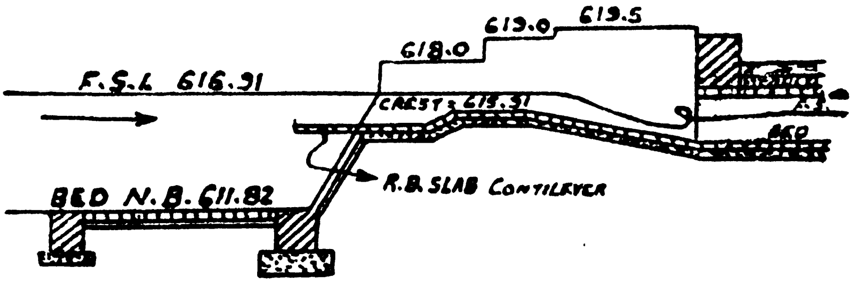


Fig. 6.

The head took silt 72·4% by weight on the average in comparison with that in the parent channel. It was evident from the silt analysis by sieves that the silt taken by the head was distinctly finer than that passing in the parent channel.

7. Author's Silt Selective Distributary Head-Regulator.

The basic principal on which the head is designed, lies in the fact that a flowing stream carrying silt in suspension, the concentration of the silt charge in the lower layers is greater than that in the upper ones Fig. 9 chapter, VI Part II. Consequently, if we can escape the lower supply without interfering with the silt distribution, the water remaining will have less silt in it per unit of volume than the water upstream of the head.

(*i*) The essential features of the design should, therefore, be to provide concentration of silt charge near the bed by some such devices as reduction of friction by pitching or plastering the bed and the sides.

(*ii*) The other and the most important feature of design should be to provide seperation of the bottom water charged with concentrated silt from the top water without any disturbance. This requires that water should enter the approach of the head with same velocity as it is flowing in the canal, approaching the work without disturbance.

(*b*) **Design description.**

A typical design of a "Silt selective Head Regulator" is given in Fig. 7. The structure can be divided into three parts; approach chamber, regulator in the form of *Karries*, and weir flume. The approach chamber selects the silt. The supply is regulated upstream of the weir flume, which meters the supply. The student should refer to paper No. 186. Punjab Engineering Congress. Lahore for a detailed description.

Pitching upstream of the Head in the parent channel on the side accelerates the side velocity, and it is further increased by the reduction of depth provided by raising the bed of the parent channel in front of the head. The difference in the mean velocity in the middle and the side velocity in the parent channel, is reduced. Water on the side simply sweeps straight along the head without any disturbance. Cross waves of silt near bed of the channel from the middle towards the sides are cut off by this quick-moving water in the front of the head regulator. The floor of the approach is kept higher than the bed of the parent channel and excludes bed silt of the parent channel according to requirements. The profile of the side $\frac{1}{2}:1$ is kept the same from bed of the parent channel to the floor of the approach chamber as the vertical wall is likely to cause disturbance. A smooth entry is provided into the chamber both upstream and downstream on sides. The waterway at the entrance to the approach chamber is the chief factor which controls the silt-selective power of the head.

It is convenient to maintain the required supply level in the parent channel by regulation at the control point. A needle regulator is the most suitable device which interferes the least with the silt carriage in the parent channel and is easy and convenient to manipulate. However, if the distributary head regulator is designed according to the everage supply level in the parent channel for want of a control point in it, regulation is provided as shown in Fig. 7 in the form of ·2 foot *Karries* secured and locked in the grooves. In this case a straight length of at least 2H is required from the regulation *Karries* to the gauge hole of the meter flume. This distance will be increased according to disturbance caused by regulation by means of *Karries*.

The regulator is followed by a meter flume. The depth on crest of the flume is kept about $\frac{2}{3}$ the depth in the approach chamber. It has got a length of crest equal to 2H and the distance of the gauge hole from the beginning of the crest is 3H. The opproach curve in the bed is laid with radius 2H. The crest is followed by a convenient glacis. Water to the gauge well is admitted through a single hole, the area of which is about 1/2000 of the area of the gauge well. The hole is provided in an iron plate secured flush with the wall and located an inch or two below crest level at a distance of 3H from the beginning of the crest.

(*c*) **Experimental test of an existing head of this type.**

The head regulator was built at the head of Melay distributary at R. D. 83332 R Southern Branch, Lower Jhelum Canal. The design was similar to the type design given in Fig. 7. The silt observations published in paper No. 189 Punjab Engineering Congress, showed that this type of head regulator took 70·2% silt by weight in comparison with that in the parent channel downstream of it. It took distinctly a finer grade of silt in comparsion with that in the parent channel. This proved as efficient as head with a skimming platform.

(*d*) **Experimental result of models.**

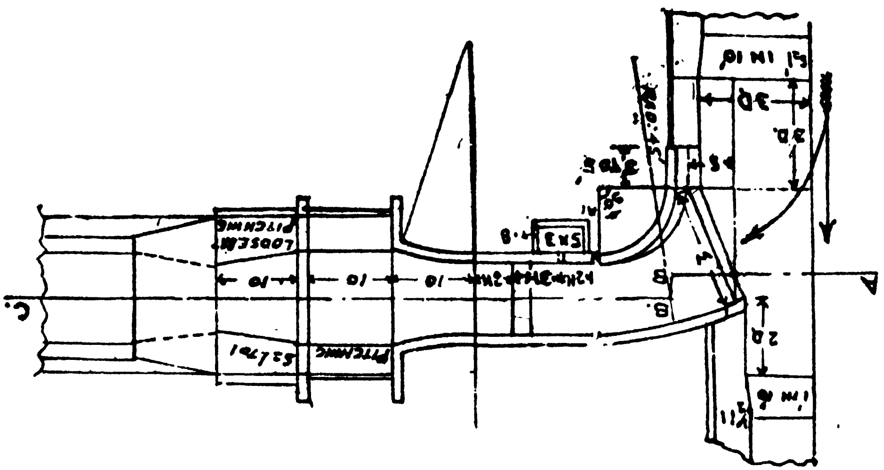
Very detailed experiments were carried out on this type of head on full-size glazed models to trace the silt laden stream lines entering head. The following conclusions were arrived at :—

(*i*) The silt conductive power of a silt-selective head regulator working under ideal conditions does not vary with the discharge of an off-take, so long as the depth in the approach chamber is not changed or, in other words, so long as the ratio of the depth in the approach chamber of a silt-selective head to the depth in front of it in the parent channel is not varied.

Ideal conditions of working mean that the width of the entrance to the approach chamber has been suitably selected, so that the bottom water of the parent channel flows straight along the parent channel along the sloping $\frac{1}{2}$ to 1 crest and has no tendency to rise into the approach chamber. These conditions are available, where no rolling bed floats rise up into the approach chamber and the surface floats indicate neither drawdown at the entrance, nor eddies in front of the bed.

(*ii*) Under ideal conditions of working, the silt conduction varied with depth in the approach chamber. In other words silt conduction varied according to same power of the ratio depth in the approach chamber to the depth in the parent channel in front of the head on the pitched floor. Results of the experiments satisfy the following relation.

Silt Selective Head Regulator.



Section on A, B, C.

Fig. 7

$$p_1 = 100\left(\frac{H_a}{D}\right)^x \qquad \text{(A)}$$

where P_1 = Silt conductive power of the head (expressed as percentage) in ideal conditions.

H_a = Depth in the approach chamber ; D = Depth in the parent channel opposite to the dead.

The value of index x in these observations is nearly $\frac{1}{3}$.

(*e*) **Advantages of this type.**

(*i*) This is a very efficient excluder of silt as compared with other types of head regulators described before.

(*ii*) This can be designed with any required silt conductive power to suit the silt conditions which could be safely allowed in the off-taking with the available slope and C.V.R.

(*iii*) The design is very simple and free from the encumbrances of groyne walls and platform, which often cause disturbance and reduce the efficiency as a silt excluding device.

(8) Example of Design of a Silt Selective Head Regulator.

Notation for Fig. 7. Discharge in canal U.S. of the off-take = Q ; Depth in canal = D_u ; Bed width in canal = B ; Discharge of the off-take 2q ; Bed width of the off-take = B_o ; Depth in the off-take = D_o ; Depth on pitched floor in front of head = D = 0·9 D_u.

Permissible silt conduction of the head = P% } From Nomogram
Depth in approach chamber = H_a } Plate No. XIII, Vol. III.

Projection of the downstream wing, $S = q/Q \times (B + D_u/2)$; Width of approach = W_a = $K\frac{S \times D}{H_a}$ where K = 1·5—2 ; Radius of U/S side wall of approach, $R_1 = 3H_a$; Width of flume = B_1

(not less than $W_a/2$) ; Depth on crest of flume $=H=(q/CB_t)^{2/3}$ (from $\frac{3}{4}$ H_a to H_a). Straight portion upstream beyond Gauge hole $=2{\cdot}5H$; Distance of gauge well hole from beginning of crest $=3{\cdot}5H$. Length of crest $= {\cdot}5H$.

(*b*) Design of silt-selective Distributary head Regulator, for the parent channel conditons :—

Slope = 1 in 5715 ; F.S.L. = 615·2 ; C.V.R. ·98 ; Bed = 610·2 ; $B/D=\rho=8{\cdot}6$; Depth = 5·0.

Discharge = 520 cusecs ; $B=8{\cdot}6\times5=43{\cdot}0$ ft. and $V=2{\cdot}29$ ft. per second.

Off-taking channel.

Discharge = 63 cusecs ; $B/D=\rho=4{\cdot}5$; Slope = 1 in 4000 ; Depth = 2·6′ ;

$B=2{\cdot}7\times4{\cdot}5=12{\cdot}0'$;

$V_o=1{\cdot}0$; $V=1{\cdot}84$; F.S.L. = 611·4 ; Bed = 611·4.

(*c*) To determine the required silt selective power of the head. Silt index of parent channel = C.V.R. = ·98 ; Silt index of off-take = C.V.R. = 1·0

$$R_c=\frac{\text{C.V.R. in off-take}}{\text{C.V.R. in parent channel}}=\frac{1{\cdot}00}{{\cdot}98}==1{\cdot}01$$

$$\lambda=\frac{\text{depth in parent channel}}{\text{depth in off-take}}=\frac{5}{2{\cdot}60}=1{\cdot}925$$

r_1 = silt selective power of head : $R_c=r_1^{1/3}.r_2^{\cdot 3}.\ \lambda^{1/6}$ and assuming $r_1=r_2=r$, $R_c=r^{\cdot 63}.\ \lambda^{1/6}$; $1{\cdot}01=r^{\cdot 63}\times1{\cdot}925^{1/6}$; $\therefore$ $r={\cdot}85$ and thus $r_1=r_2=85\%$.

Note :—It is assumed that the silt grade shall also vary as the silt charge by weight. The observations plotted in plate 7 of the paper No. 189 Punjab Engineering Congress, Lahore, show that the ratio of the average diameter was very nearly the same as the silt conductive power by weight.

(*d*) Depth in approach chamber for 85% silt selection relative to silt in the parent channel.

$$p=100\left(\frac{H_a}{D}\right)^{1/3}\text{ but } D={\cdot}9\times D_u=4{\cdot}5\ ;\ {\cdot}85=\left(\frac{H_a}{D}\right)^{1/3}$$

$$\frac{H_a}{D}={\cdot}6\text{ or }H_a={\cdot}6\times4{\cdot}5=2{\cdot}7\text{ ft. }\therefore\ R_1=3H_a=8{\cdot}1\text{ ft.}$$

(*e*) Set back of the upstream wing.

$$S=\frac{q}{Q}\left(B+\frac{D_u}{2}\right)=\frac{63}{250}(43+5/2)=5{\cdot}5\text{ ft.}$$

(*f*) Width of approach $=W_a$; $W_a=K\dfrac{S\times D}{H_a}=1{\cdot}5\times\dfrac{5{\cdot}5\times5{\cdot}0}{2{\cdot}7}=15$ ft.

(*g*) Radius of upstream approach curve $=3H_a=3\times3=9{\cdot}0$ ft. say 10.

(*h*) The width of flume = 15/2 = 7·5 ft. say 8·0 ; C (from Fig. 27, Chapter X) = 3·03.

$$H=\left(\frac{63}{8\times3{\cdot}03}\right)^{2/3}=1{\cdot}89\text{ ft. ; }V_a=\frac{63}{2{\cdot}9\times8}=2{\cdot}91\text{ ft./sec. and }h_a=\frac{2{\cdot}91^2}{64{\cdot}4}={\cdot}13\text{ ft.}$$

$\therefore$ $G=1{\cdot}89-{\cdot}13=1{\cdot}76$ ft. ; Length of crest = 2·5 ; $H=2{\cdot}5\times1{\cdot}9=4{\cdot}75$ ft.
Gauge hole form the beginning of crest $=3{\cdot}5\ H=6{\cdot}7$ ft. and straight portion upstream $=2{\cdot}5\ H=4{\cdot}75$ ft. say 5·0 ft.

11. Practical Field Test of all Silt-Excluding Devices.

(*a*) It is easy to work out the proportional surface width of the parent channel for the discharge of the off-take. A surface float should be allowed to float in the parent channel from 15 feet upstream of the head of the off-take at a proportional distance from the water edge. If it just enters the off-take, then it is taking its due share of bed and surface water. The silt conductive power of the head will be about cent percent. If the surface float dropped from even beyond the proportional share of the surface width, gets into the off-take head, it is then working as a silt excluder, and silt-laden water at bed is entering the head less than the proportional. This applies to all silt excluding devices discribed in this chapter for distributary head regulators and also to distributors.

Proportional Discharge and silt Distributor.

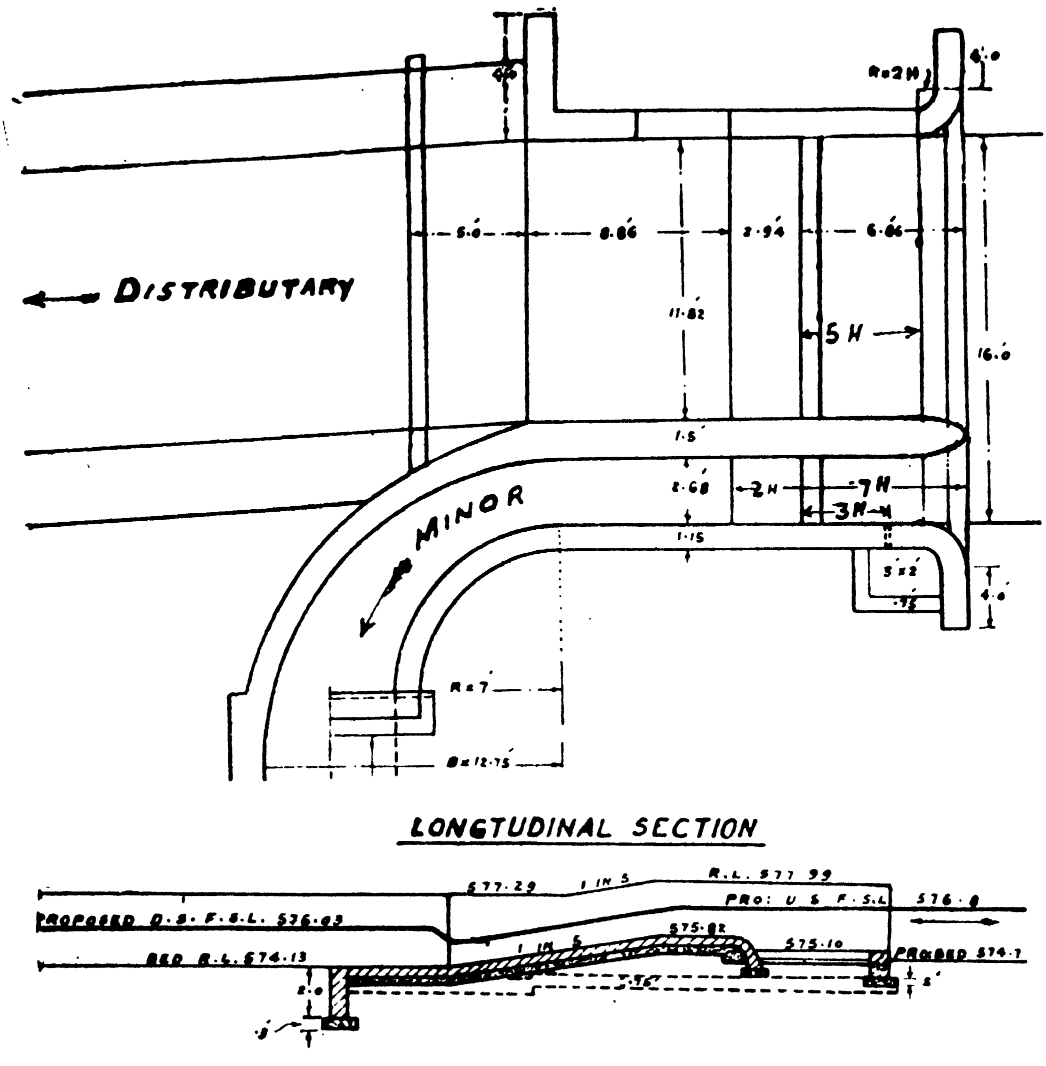


Fig. 8

(*b*) In the case of a silt-selective distributary head regulator, the silt selective power can easily be predicted by simply running the surface floats. Determine the distance from the water edge upto which the floats enter the off-take head. It should be the same as the width of approach allowed in the design for the design for the stipulated silt conductive power for which the head is designed. If it is less, the silt selective power will be correspondingly reduced in direct proportion. The fact was established in the experiments described by the author in Paper No. 189 Punjab Engineering Congress, Lahore.

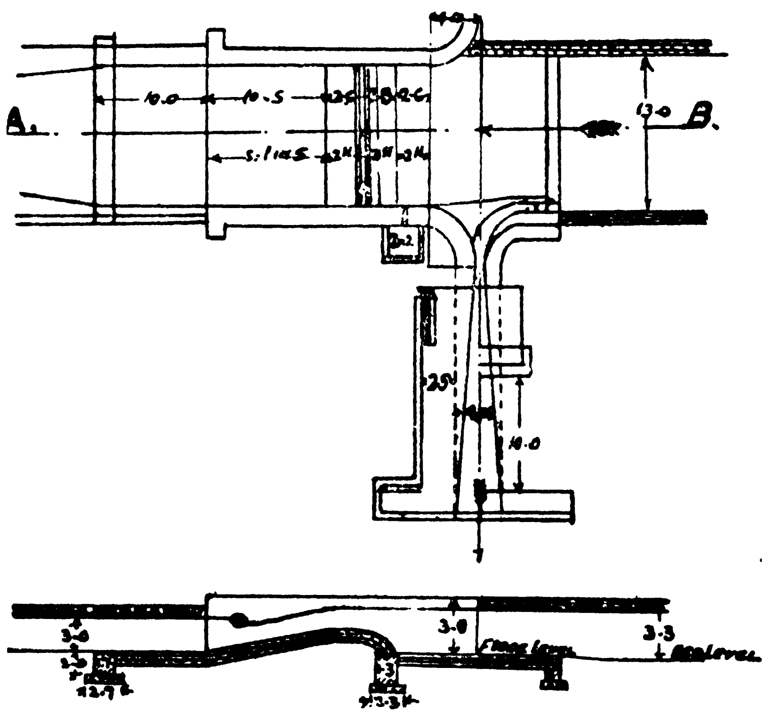
12. Proportional Distributors.

(*a*) In distributing channels, proportional distribution of supplies is desirable and is very much appreciated by the cultivators. It is a good practice to combine the off-taking minors and outlet into one distributor. The proportional distribution of supplies is at once arranged by making all off-takes including the parent channel into open flume weirs with their crests at the same level. There are two arrangements usually adopted as shown in Fig. 8. and another in Fig. 9. When the off-taking minors and outlets are small as compared with the parent channel, the arrangement shown in Fig. 9. is suitable and economical. When the off-taking channel carries a discharge more than 1/4 of the parent channel, the arrangement shown in Fig. 8 is desireable though usually somewhat expensive.

(*b*) The Proportional distribution of the silt charge of the parent channel can easily be attained in the arrangement as shown in Fig. 8 by dividing proportional water-way of the channel approaching each flume by extending the partition walls upstream of the crest. In fact the partition walls can continue as silt varies with height equal to $\frac{1}{3}$ to $\frac{1}{4}$ depth to restrict or increase the entry of bed silt into any off-take according to the requirement.

When the off-taking minor is a small one as compared with the parent channel, the arrangement shown in Fig. 9 is convenient and economical. There is enough experimental work on record (Khan Bahadur Minhaj-ud-Din I.S.E. experiments on silt distribution, Indian Engineering, Calcutta, 1928) to show that the right-angled off-takes as shown in Fig. 9 take relatively high silt charge as compared with the parent channel due to the curved entry of water. This can, however, be set right at little cost by constructing there a silt vane as shown so that only proportional bed silt enters the off-take.

Proportional Distributors with silt vanes. Scale 1/200



F. 9.

The calculations of design of distributors are given in Fig. 9. The structural details are as per standard meter flumes design as given in para 15 Chapter X.

13. Examination Questions.

Give a Sketch of an escape and a regulator for a distributary having a discharge of 150 cusecs, a bed width of 20 feet and F. S. depth of 4·15 ft. The escape has to take a discharge of 100 cusecs and will have a bed width of 14 ft. and water depth of 3·5 ft. R. L. of bed of distributary 560·0., R. L. of bed of escape 557 0.

Explain a Walton gate for a canal head regulator (*P. I. B.* 1937)

2. Work out and sketch the design for a proportional silt and discharge distributor for a trifurcation with the following data :-

	Parent channel	Straight off-take	Right off-take at 60°	Left off-take at right angle
Discharge	520	300	100	120
Depth	5 65	4 6	3·5	3·8
Bed width	34·0	28	14·3	17·0
Slope per thousand	·2	·2	·23	·235
C. V. R.	1·0	1·0	1·0	1·0
Sides	½ to 1	½ to 1	½ to 1	½ to 1

*3. (*a*) Describe the basic principle on which the problem of silt exclusion is based.
(*b*) What are the advantages of silt selective head-regulators over other types ?

4. (*a*) Explain how you would determine by running surface floats in the parent channel whether a head is working as a silt selective and silt excluding device.
(*b*) Explain the basic principle on which the design of King's silt vanes is based

5. (*a*) Why did Wood's design of rising cill gates fail even though it aimed at tak.ng surface water by overshot regulation ?
(*b*) When do the tunnels below the skimming platforms choke and why ? Sketch and explain the conditions when they will not choke.

6. Design a silt-selective distributary head-regulator for the following data:—

	Parent channel	Left off-take at right angle.
Discharge	520	63
Slope	1 in 5715	1 in 4000
C. V. R.	·98	1·0
Bed	43 0	12·0
Bed width depth ratio	8·6	4·5
Velocity	2·29	1·84

7. What are the three functions of a distributary head-regulator ? Explain their development in the modern distributary head-regulator designs ?

8. What are the various devices used for regulation of Irrigation supplies on (*i*) Head regulator (*ii*) under sluices (*iii*) canal falls (*iv*) Minor heads (*v*) out lets. (*P. U. 1952*)

9. How are supplies regulated in a canal system in order to give maximum benefit for the water available with due regard to safety and economy in the water use. (*P. U. 1953*)

10. Enumerate the various structures on an irrigation system from the head works of a canal to a *Zamindari* water course, which are required for regulation and distribution of supplies. The function of each work should be stated breifly. (*P. U. 1953*)

11. What type of silt ejection device would you use for proportional distribution of silt in a distributary. (*P. U. 1955*)

PART II

CANAL IRRIGATION

CHAPTER XV

Outlets And Tail Clusters

1. The masnory structure through which water is admitted from a government distributing channel into a water course (cultivator's channel) is known as an outlet or *mogha*. In America it is called a turn-out. An outlet may be a module or a non-module as defined below :—

Definetion.

The term module, in hydraulics, was originally applied to a contrivance devised to pass a fixed supply of water independent of water surface levels both in the supply and delivery channels. With the envention of the gauge-outlet and similar devices it would, however, seem more convenient to broaden the term "module" to include the latter inventions also, and to define it as meaning "a device arranged to pass a supply of water independent of water surface level in the channel into which the supply is delivered." With this definition, modules fall into two main classes :—

(*a*) **Rigid modules**—Passing a fixed supply.

(*b*) **Flexible moudles**—or (Semi-modules). Passing a supply which varies in some characteristic manner with surface level in the supply channel but which is independent of the variation of the water level in the delivery channel (water course).

(*a*) Among rigid modules are Gibb's Vortex Modules, and the Kent "O" (a pressure float device); Ghafur and Khanna's rigid modules.

(*b*) Flexible modules may be sub-divided into :—

(*i*) **Orifice type**—The K.G.O. (Kennedy Gauge Outlet) and the original Harvey Stoddard Standing wave outlet. Crump's A. P. M. (Adjustable Proportional Module). Sharma's improved A. P. M. (S. S. O. O.)

(*ii*) **Weir (or flume) type.** Various forms of open weirs *e.g.*, that used by Gibb for tail clusters, short-throated flumes as used by Harvey and Lindly for tail clusters ; Crump's open flume outlets , minor-heads, proportional distributors, meters, and long-throated flumes ; Sharma's narrow open flume outlets.

(*iii*) **Combined type**—The Harvey Stoddard Improved (proportional) Outlet, which is essentially a combination of submerged orifice and weir.

2. It follows from the definition of flexible module that a vertical gauge, or scale, fixed relative to the module can be calibrated to show the discharge of the module corresponding to any mark on the gauge at which surface level in the supply channel may stand. The height in feet, measured to this surface level from the zero of the gauge, will be designated by the letter G, it being understood that the zero mark represents zero discharge as calclulated from the formula used to express module discharge in terms of gauge. In flexible modules of orifice type, for instance, G is measured either from the center or the upper surface of the jet, according as the jet is aerated all round or only on the upper surface and discharge varies as $G^{1/2}$. In the weir type, G is measured from the weir crest, and discharge varies $G^{3/2}$. In the combined type, the zero mark is at weir crest level, and the discharge is given by $\left(\frac{q}{k}\right)^2 + \left(\frac{q}{K}\right)^{2/3} = G$; where K and k are constants for any one module.

For rigid modules, the gauge G has no meaning since discharge is independent of gauge.

3. With varying field levels, and with the *zamindar* at liberty to silt clear his water course, whenever and as often as he wishes, the supply drawn by a non-modular outlet is for ever changing independently of surface level in the supply channel, and thereby affecting the general distribution of supply in a manner entirely beyond the control and management of those responsible for distribution. On a moduled channel, on the other hand, distribution is rendered entirely independent of the arbitrary changes in water course conditions, and is dependent only upon conditions in the supply channels under Government control. This great advantage of the module is by now generally recognized ; the old non-modular outlet, except as purely temporary expedient, is doomed to disappear and in what follows, it will be tacitly assumed that we are concerned only with moduled channels.

4. Following the definition of the term module of paragragh 1, all modules—whether rigid or flexible—may be more precisely classified, in relation to the supply channel, in terms of a single characteristic ratio :—

$r = \frac{(dq)/q}{(dQ)/Q}$ representing the ratio between the fractional deviation $\frac{dq}{q}$ in the normal supply q of the moudle, and the corrresponding fractional deviation.

$\frac{dQ}{Q}$ in the normal supply Q of the parent passing below the module. This ratio 'r' will be called the FLEXIBILITY of the module. For a rigid module, the FLEXIBILITY is zero. For orifice types like the K. G. O. it is usually less than unity ; while for weir (or flume) type it tends to be greater than unity.

(*b*) When the "r" of a module is just unity, the module is proportional, *i.e.*, it shares, proportionally with the parent, in any small deviation in the normal supply of the parent ; so that on a distributary fitted throughout—including the heads of its minors—with such modules, diurnal fluctuations of supply would affect all parts of the distributary to the same extent *i.e.*, by the same percentage increase or decrease of supply so that if these fluctuations were the only difficulty to be overcome, the proportional module would offer a simple but perfect solution of the problem of distribution. The proportional setting of the orifice type semi-module is when G = ·3 D and for the weir type outlets is when G = ·9 D, where G is the gauge and D the depth in the channel. The proportional setting of the crest of the outlet, therefore works out to be 6/10 D and 9/10 D in the case of the orifice type and weir type outlets respectively. The mathematical proof of this was worked out by E. S. Crump in his paper No. 26, I. B. Publication, Punjab.

(*c*) The problem is, however, more complicated, as changes in channel regime, coupled with the usual restriction of head supply to within a prescribed maximum limit, introduce a second difficulty, with which the proportional module is poorly adapted to cope. The rigid modules, on the other hand, behave in directly opposite manner ; while completely immune from the effect of regime changes it takes no share whatever in fluctuations of supply. In fact the problem of distribution presents two independent difficulties the requirements of which are in direct opposition ; the first is met by proportional modules, and the second calls for rigid modules. Whether the K. G. O. or the A. P. M. set at bed or below or modules of similar flexiblity, which occupy an intermediate position between these two extremes offer for this reason the best solution of the problem, remains to be considered.

5. Every reach of a channel ends in what may be called a "control point", that is to say in a masonary work usually so designed as to maintain (*a*) a permanent relation between upstream surface and discharge, and (*b*) parallelism of surface and bed lines with varying discharge. Fluctuation in supply may therefore, be regarded as causing a general raising or lowering of the surface line without affecting its slope. Regime changes, on the other hand, result in a steepening or flattening of the surface line, and their effect on distribution is readily appreciated by visualising the surface lines of any reach as swinging slowly about its downstream end *i.e.*, the "control-point" as pivot. All off-takes in the reach are affected in the same way ; but to an

extent increasing with their distance from the 'control point'. The reach as whole benefits or loses as the case may be according to the loss or gain of the reaches below it. The advantage of frequent control points in reducing the effect of regime changes is at once evident ; the effect is reduced invarsely as the number of sub-reaches into which a silting reach is divided.

6. The sensitiveness of a module to regime changes obviously varies directly with its. FLEXIBILITY as defined in paragraph 4 above. It will, however, be useful and conductive to clearness, to destinguish between the two terms and to define the sensitiveness 'S' of a module as "the fractional increase (or decrease) of module supply per *hissa* rise (or fall) in channel surface." With this definition as per Crump's paper No. 26 Class A. P. W. D. Technical Irrigation Branch Publications. $S = \frac{r}{6D}$ applies to all modules, and enables the precise effect of a given regime change to be calculated in terms of the FLEXIBILITY 'r' of the modules concerned and of the normal depth D of the channel.

For example ; applying the above relation to the case of a head reach of 2·8 feet average depth, in permanent regime, and subject to an average surface swing of $\pm 0{\cdot}1$ foot ; the percentage seasonal variation in the total draw-off of the reach would be about ± 6 percent for proportional module, ± 2 percent for K. G. O. or other orifice modules having a setting (G/D) of 0·9 (a fair average for such modules) and nil for rigid modules. In other words, K. G. O.'s were used in the head reach, no resetting of them would be called for ; whereas if proportional modules were substituted, they would have to be readjusted every three months or so, to maintain a well-balanced distribution through the year.

7. The idea of adjustable modules is not new. With non-modular outlets, re-adjustments meant reconstruction. In the case of K. G. O. it takes the form of resetting on a masonry or concrete foundation. By adjustability is meant the provision of some suitable means of altering the size of the orifice say of a module, with a minimum of trouble to those empowered to make re-adjustments, but with a degree of difficulty sufficient to prevent re-adjustment being made in an illegal or unauthorized manner, by the *zamindar* or petty official. In Crump's opinion, the right mean between facility and difficulty is to be attained by means of a key of solid masonry or concrete, substantial enough to defy tampering, but at the same time small enough to be reasonably cheap in renewal. Such a key, would, of course, have to be dismantled and rebuilt at each adjustment of the module. It is assumed and regarded as feasible, that re-adjustments would be carried out under the sanction of higher authority, by the Sub-Divisional Officer concerned, and that he would be held responsible for the correctness of the re-adjustments made and recorded by him. As regards the frequency of the re-adjustments, it must be remembered that all regime changes and it is these alone that call for adjustability take place slowly, so slowly that, even if proportional modules were used in head-reaches, re-adjustment would ordinarily be necessary not more frequently than once a crop. Crump brought forward the idea of adjustability and proportionality and therefore his outlet is called Adjustable Proportional Module (A. P. M.).

8. Modular Limits and Minimum Modular Head (M. M. H.).

All modules, whether rigid or flexible, require a certain minimum head to ensure modularity. In the case of rigid modules, there is also an upper limit beyond which constancy of discharge fails. This limit must obviously be high enough to avoid trouble within the range of running conditions. In all modules the advantage of low M. M. H. is self-evident, since it means less expense in earthwork, both in moduling exisiting channels and in constructing new ones with a view to modular equipment, and in the former case less disturbance of existing regime. In moduling existing channels it is usually found necessary to raise channel surface levels in order to obtain madularity for the worst-commanded outlets. This is due to the fact that with the old non-modular outlet, the importance of insisting upon an ample margin of command was not generally appreciated by irrigation officers. There indifference has commonly resulted in the *Zemindar* being allowed—and even encouraged to bring under irrigation, land which, by reason of its elevation should in the interests of efficient distribution, never have been admitted to irrigation by flow. Needless to say, a continuance of this easy-going attitude is quite incompatible with the modular equipment of channel on scientific lines.

9. Essential Conditions to be Satisfied in Outlet Design.

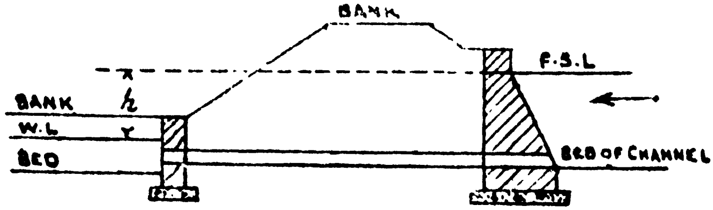


Fig. 1

(*d*) An outlet must be strong and should not have moving parts liable to derangement or requiring periodic attention.

(*b*) Interference by the cultivator must be difficult, and if made should be readily detectable.

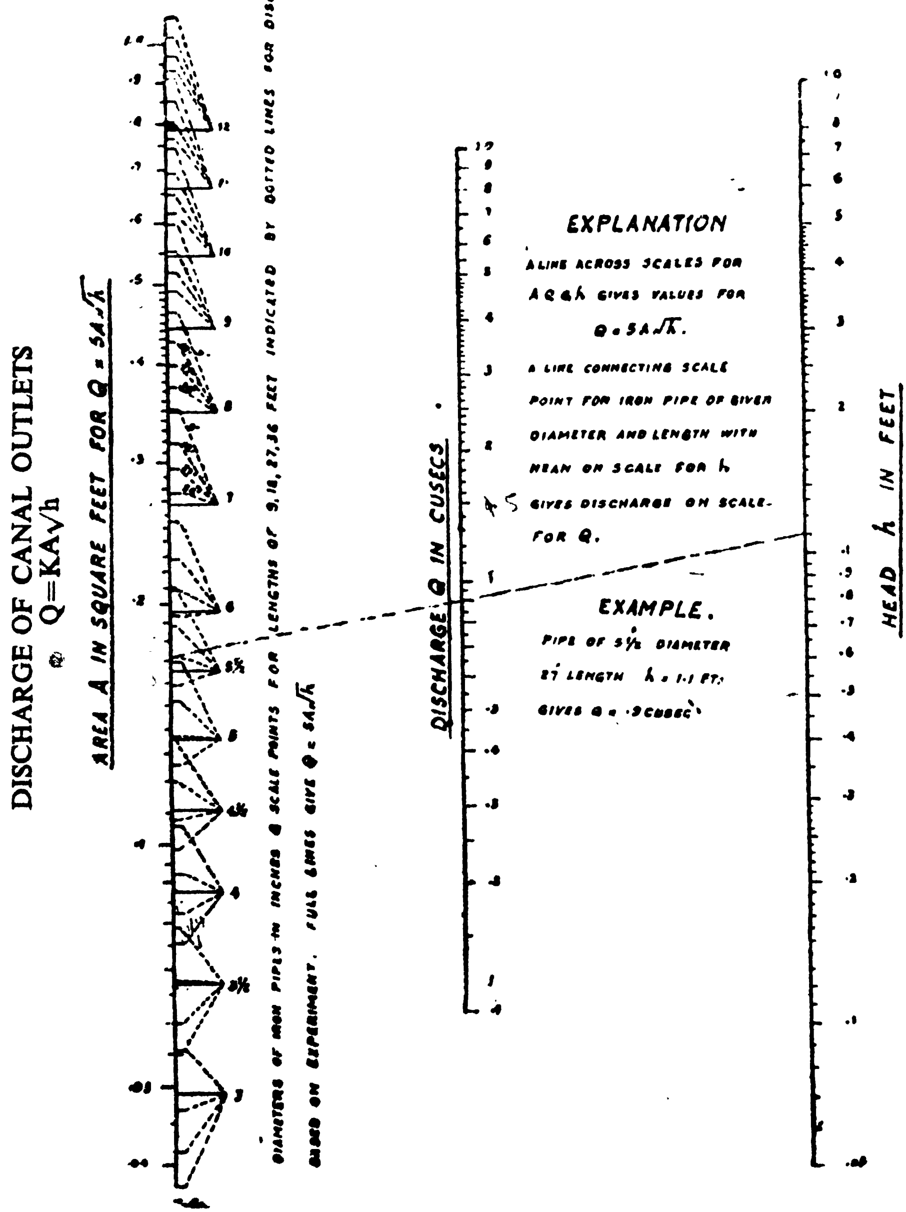


Fig. 2

(*c*) The outlet should draw its fair share of the silt carrid by the parent channel.

(*d*) It should be possible for the outlet to work efficiently with a small working head. The larger the working head the higher the water level required in the parent channel and the higher the cost of the distributary system.

(*e*) The cost should not be high.

10. Non-Module Outlets.

The non-module outlets, are masonry orifices, pipe outlets and wooden shoots.

They are usually put at the bed level of the distributary. They may be circular or rectangular in section. The dischage formula is $Q=K.A.\sqrt{h}$, where Q is the discharge in cusecs, A is the sectional area in square feet and h is the working head in feet (Difference between the water level in the parent channel and the water level in the water course) and K is a constant which is taken as 5 in the case of masonry orifices and 6 in the case of wooden and iorn orifices. There is usually provided a face wall on the distributary side as shown in Fig. 1 and drop wall at the end of the bank in the case of *pacca* outlets.

The wooden shoots are usually put in during the construction of a channel. They are simply put in the bank with puddled earth around them. They are meant to be temporary and *kacha* outlets and are replaced by permanent outlets after running the channel 3 to 4 years when the *chak* boundaies are established by usage and some idea of the actual water levels in the water courses is available.

It is apparent form Fig. 1, that the discharge of the outlet can easily be increased by digging the water course and thereby lowering the wate level in it. This will increase the value of h and, therefore, the discharge. Equitable distribution is not possible with this type of outlet when the cultivators can increase the discharge by simply digging the water course which is a *zamindari* channel. The solution of the formula $Q=5\ A\sqrt{h}$ is given in Fig. 2.

11. Tilted Pipe Outlets. (Free fall pipe outlets)

The masonry of pipe drowned orifices as described above were replaced by tilted pipe outlets as sketched in Fig. 3. The pipes is supposed to discharge free fall above the water level in the water course. The pipe may be tilted as shown in Fig. 3 or horizontal.

The discharge formula of this outlet$=C_d.\ A\sqrt{2gh}$; where $C_d=0{\cdot}6$, co-efficient of the discharge ; A=sectional area of the pipe in sq. ft. ; h=head from F.S.L. in disty ; to the centre of the pipe at the outfall.

This type of outlet is no doubt a semi-module and the discharge cannot be increased by digging the water-courses. This succeeded very well for some time. But the cultivators soon invented the method of constructing a ramp in the water course by heading up water to the top level of the pipe. The discharge increased by 15 to 20%.

The long pipe outlets do not usually run full at the exit and the discharge co-efficient 0·6 is mostly a measure of the co-efficient of the contraction of the free fall jet at the *vena contracta*. By drowning the pipe, it starts running very nearly full bore. The control section shifts from the *vena contracta* of a falling jet to the sectional area of the pipe at the exit. The discharge co-efficient increases to 0·8 and even more in some cases. The discharge is increased even though the head is lost equal to half the diameter of the pipe by drowning.

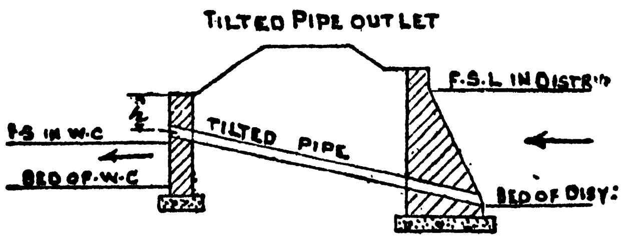


Fig. 3

12. Kennedy's Gauge Outlet.

This was the earliest type of semi-module used in India. It is sketched in Fig. 4.

It was invented by R.G Kennedy, Chief Engineer, Irrigation, Punjab. The outlet is provided with a bell-mouthed approach. The jet at the control section spring clear from the bell-mouthed approach in to an expanding delivery pipe as shown in fig. 4. The jet in the air chamber is aerated all round. The discharge formula $q=C_d.\sqrt{2g}.A.\sqrt{G}$, where G=the head measured from F.S.L. in disty ; to the centre of the orifice ; $C_d={\cdot}97$=Discharge Co-efficient ; A=Sectional area of the orifice at the end of the mouth.

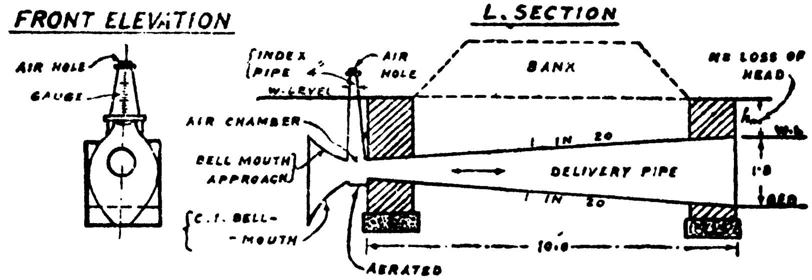


Fig. 4.

This outlet required in practice a Minimum Modular Head of about ·22 to ·25G and its discharge was independent of water level in the water course when the working head exceeded M.M.H.

This module has, however, been superseded because its discharge could easily be increased by closing the air holes feeding the air chamber. After the air holes are closed, the air of the chamber is sucked away by the jet and its pressure in the chamber drops below the atmosphere. The pressure head causing flow increases to $(G+A-A')$ where A is the atmospheric pressure and A' actual air pressure in the chamber below atmospheric pressure.

13. Harvey Stoddard Improved Outlet.

The Harvey Stoddard improved outlet (Fig. 5) was an early design to attain proportionality. It consisted of an orifice outlet combined with a raised standing wave flume in continuation. The loss of head was relatively more than in a standing wave flume. M.M.H. was about 22 to 25%.

L. SECTION

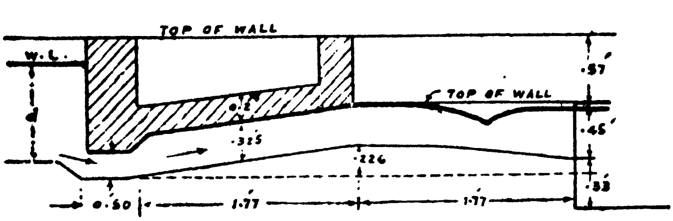


Fig. 5

A standing wave flume gives proportional discharges with a small loss of head, so long as the depth of the parent channel is small. When however, the distributary is a deep one, the flume, which would have to be approximately the same depth (to be on the right side of proportionality) would have to be made so narrow that practical considerations would rule it out.

14. Kirkpatrick Outlets.

Jamrao type open flume is shown in Fig. 6 and the Jamrao orifice semi-module is shown in Fig. 7. The open flume of Fig. 6 has no level crest and has a short throated section about 2" or 3" wide, where an angle-iron frame is fixed. The upstream approach is 2 ft. long and has a splay of 1 in 4. The downstream approach flume is 10 ft. long and the width of the flume at its downstream end is 3 B. Kirkpatrick found that the shapes of the upstream and the downstream flumes were not capable of further improvement and that any further increase in their lengths or rates of divergence could not improve upon this type of design. The values of co-efficient K in the formula $q=KBG^{1\cdot5}$ were not found to be constant for different values of G but if the index 1·5 were changed to 1·6 the formula agreed within 3 percent of the actual observations, which covered a wide range of both B and G. The

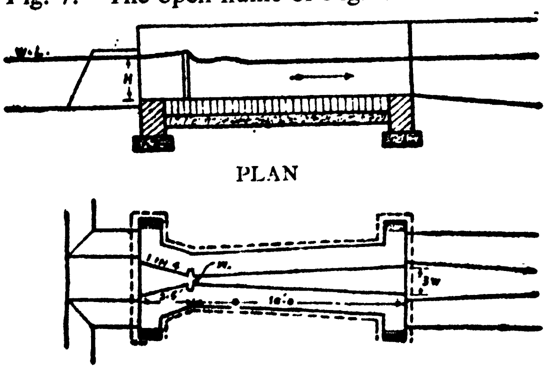


Fig 6.

discharge formula as evolved by Kirkpatrick for this type of open flume outlet is $q = 3{\cdot}2\ BG^{1\cdot5}$.

The M.M.H. found to be necessary is $= \frac{G}{7}$.

It will be seen that the M.M.H. required for this outlet is more than that required for the Crump type of open flume outlet, also the narrow, as the open flume, outlet suffers from two other defects :—

(*a*) The control section does not remain within the angle-iron frame depth as the depth on crest increases.

(*b*) These outlet are more susceptible to getting choked with jungle than the long throated open flumes.

Orifice type outlet in Fig. 7 in an attempt to reproduce a K.G.O. in masonry. The upstream approach is only 2 ft. long and is of the shape of a truncated cone with a convergence of 1 in 4. The control section is a square orifice in an angle iron frame. The downstream flume 10 ft. long has a horizontal floor with the wide walls at a splay of B/10. The must novel feature of the Jamrao semi-module is the introduction of baffles as a means of recovering head. The baffles work as a "roof" sloping gradually upward, in the downstream flume and by their addition "considerable extra head is recovered." At the same time the outlet discharges under free atmospheric conditions. "Even if an aperture is closed, the effect is not considerable as air will enter from the next aperture and so on." "The optimum slope for the roof *i.e.*, lower edge of baffles is 1 in 15. The optimum number of baffles is 9 of which the first 6 are equally spaced and the last 3 somewhat spread out." The discharge formula applicable to this orifice semi-module was determined experimentally, by Kirpatrick and is ; $q = 7{\cdot}2\ B.Y.\sqrt{H_c}$. As the orifice is square and $B = Y$ the formula may be rewritten as $q = 7{\cdot}2\ B^2\ \sqrt{H_c}$ as will be apparent from the drawing. This type is not at all adjustable. The minimum working head required is taken from the table given below.

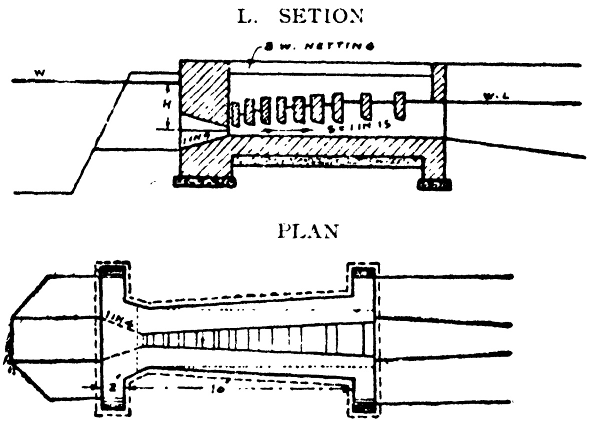


Fig. 7

TABLE No. 1.

Size of Module.	M.M.H. with 6 Baffles.	M.M.H. with 9 Baffles
8″	$\frac{H_c}{4{\cdot}1}$	$\frac{H_c}{4{\cdot}8}$
7″	$\frac{H_c}{4{\cdot}0}$	$\frac{H_c}{4{\cdot}7}$
6″	$\frac{H_c}{3{\cdot}9}$	$\frac{H_c}{4{\cdot}6}$
5″	$\frac{H_c}{3{\cdot}7}$	$\frac{H_c}{4{\cdot}5}$
4″	$\frac{H_c}{3{\cdot}5}$	$\frac{H_c}{4{\cdot}4}$
3″	$\frac{H_c}{3{\cdot}3}$	$\frac{H_c}{4{\cdot}3}$

15. Crump's Adjustable Propotinal Module.

The setting of the crest for proportionate discharge is 6/10 D. The structural details are explained in Drawing in Fig. 8. The immunity from tampering is provided in the form of cast iron base and cheek plate 1·0 foot wide with a cast iron roof block. The adjustability of the outlet is arranged by providing bolts passing through the roof block and the cheek. These bolts can be removed after dismantling the masonry on top of the roof block and it can then be raised and lowered according to requirements. The masonry above the roof block can again be put in. The cost of adjustment of the size of the outlet is very small. It is an orifice type outlet with the additional abvantage of adjustability and proportionality. The discharge formula used is given below :—

$q = c_d \sqrt{2g}\, B.y\sqrt{h}$. Where q = Discharge in cusecs ; $c_d = 0{\cdot}91$ = discharge co-efficient ; B = Width ; y = Height of opening ; h = depression of roof block. $H = h + y$ = depth on crest.

The head "h" is measured upto top of the jet leaving the orifice under the roof block. There is level length H provided in bed after the roof to ensure that the jet of water leaving with y depth should exert pressure on the floor. It is, therefore a submerged orifice. It is semi-module because the discharge is supposed to be independent of the conditions in the water course and the modularity is indicated by the formation of the hydraulic jump downstream of the orifice Fig. 8. A.P.M. is, therefore essentially a submerged semi-module.

Crump in his experiments suggested a co-efficient $C_d = c_d\sqrt{2g} = 7{\cdot}3$. He also gave a table of the minimum modular limits based on his experiments in paper No. 26, P.W.D. Irrigation Branch, Punjab publications class A. His experiments were carried out with width 0·5 ft., and depth on crest 2·0 feet. It was soon found that the co-efficient C and M.M.H. as given by him did not apply to all cases. Crump's A.P.Ms. have been manufactured and used in eight standard widths ·2, ·25, ·32, ·4, ·5, ·63, ·8 and 1·0 ft.

TABLE No. 2

Sharma's table of silt conductive power of outlets.

V/V_o	A.P.M. as per I.B. paper No. 26 crest set at 6/10th (Crump)	A.P.M. modified by Sharma set at 6 10th (S.S.O.O.)	A.P.M. modified by Sharma set at 8/10th (S.S.O.O.)	A.P.M. modified by Sharma set at bed level (S.S.O.O.)	A. P. M. modified by Sharma type B at bed level (S.S.O.O.)	O.F. modified at 9/10th or O.F. at bed level as per I.B. paper No. 26.	Pipe outlet set at bed level.	Bend outlet at 6 10th (Minhaj-ud-Din)	A.P.M. as per I.B. paper No. 20 at 9/10th (Crump)
1·10	90	100	117	126	132	114	114	114	114
1·00	92	100	110	124	130	112	112	112	112
·98	93	100	109	122	127	111	111	111	111
·96	94	100	108	120	125	110	110	110	110
·94	95	100	107	118	123	109	109	109	109
·92	96	100	107	116	121	108	108	108	108
·90	97	100	106	114	119	108	108	108	108
·88	98	100	106	112	117	107	107	107	107
·86	98	100	105	111	115	107	107	107	107
·84	99	100	105	110	114	106	106	106	106
·82	99	100	104	109	113	106	106	106	106
·80	100	100	104	108	112	106	106	106	106
·78	100	100	103	107	111	105	105	105	105
·76	100	100	103	107	110	105	105	105	105

Note :- (*i*) The silt conductive power of the outlets is expressed as percentage relative to the silt charge in the parent channel determined in Paper No. 168 read in the Punjab Engineering Congress 1933. Its variation with respect to the quality of the silt carried in the channel was noticed in the experiments described in Paper No. 168 and was varified by analysing distributaries known to be in regime

Branch; S.S.O.O.=Sub-

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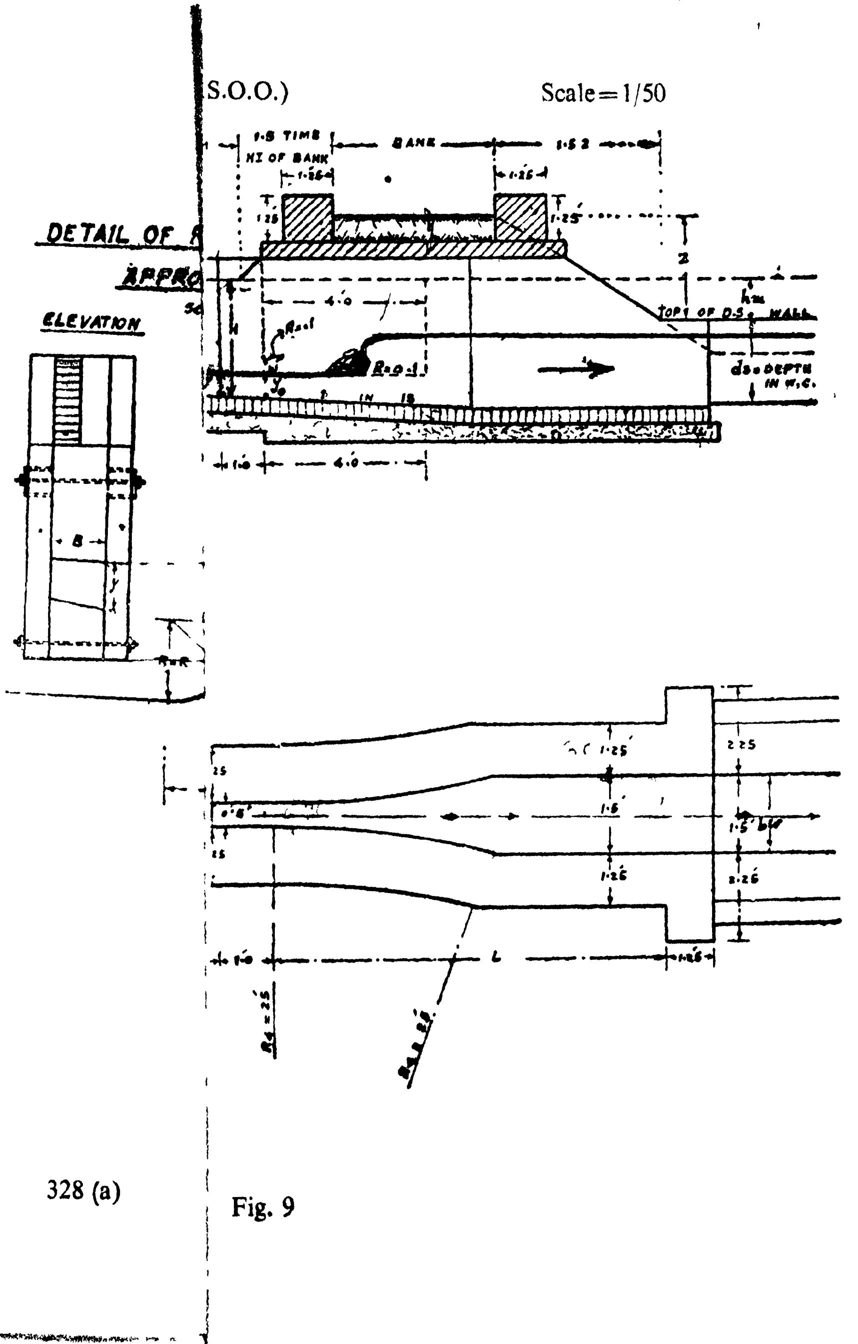

Fig. 8.

328 (a)

Fig. 9

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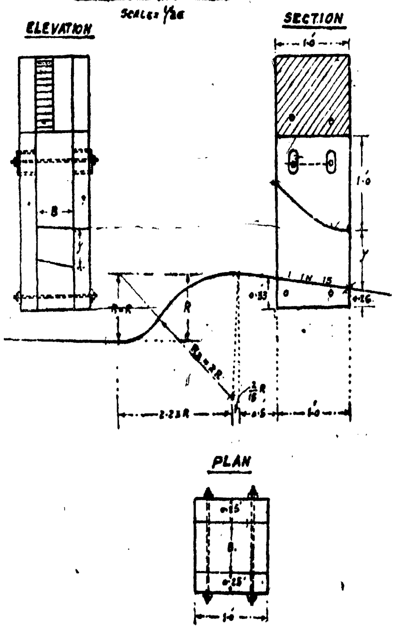
(a) (i) ... with A.P.M. outlets set at 6/10th had a tendency to silt up because A. P. M. outlets as constructed did not take their due share of silt. The proportional silt conduction was not enough because 10 to 12% water was lost in absorption and the due share of silt to be drawn by an outlet was considered to be 110 to 112 percent. The improvements

DETAIL OF ROOF BLOCK AND APPROACH IN BED



Author's Improved A.P.M. (S.S.O.O.) Scale = 1/50

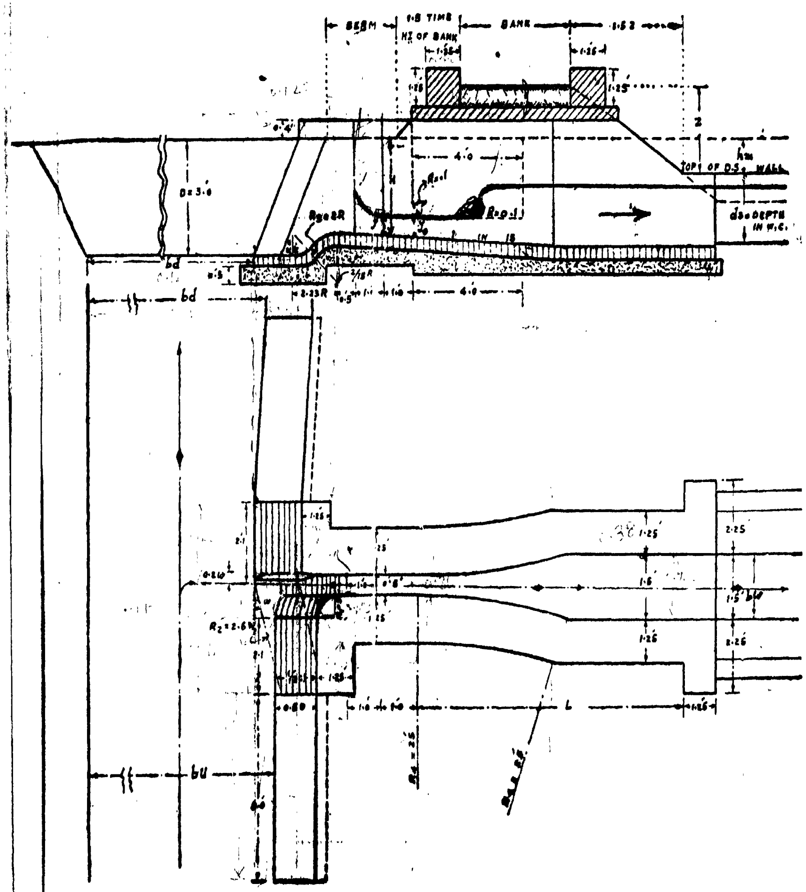


Fig. 9

Formula $q = cBy\sqrt{h}$

Bed width in channel U. S. of outlet	... $= b_u$
Depth in channel U.S. of outlet	... $= d$
Discharge in channel U.S. of outlet	... $= Q$
Bed width in channel D.S. of outlet	... $= b_d$
Discharge of outlet	... $= q$
Setting	... $= s$
Depth on crest $= H$	... $= s_d$
Width of A.P.M.	... $= B$
Depression of roof block	... $= h$
Height of opening	... $= y$
Minimum Modular head (from Diagram)	... $= h_m$
Actual Working Head	... $= h_w$

Set back of U.S. wing
Width of approach $\left.\right\} = w = K\frac{q}{Q}\left(b_u + \frac{d}{2}\right)$

Splay of U.S. Side wall $= S$	... $= w - B$
Radius of splay on side $= R_1$	... $= S$
Length of approach curve on side	... $= S$

Radius of curvature of D.S. Projected wing $R_2 = 2{\cdot}6w$

Distance of the back of A. P. M. Block measured from the centre line of

U. S. channel $= \frac{b_u}{2} + \frac{d}{2} + S + 1{\cdot}5$ or $= \frac{b_u}{2} + 2{\cdot}23R + 1{\cdot}5$

R = Rise of approach curve in bed above distry.

Bed U.S. $= (1 - s)\, d + 0{\cdot}1$

Radius of convex part of approach curve in bed $= R_3 = 2R$

Radius of concave part of approach curve in bed $= R$

Length of outfall	... $= L$
Radius of curvature of outfall	... $= R_4 = 25$

Height of h_m' reducing device $y_o = y + 0{\cdot}05$ where $y > 0{\cdot}5$, $y_o = 1{\cdot}1y$

Width of W. C. $= \begin{cases} 0 \text{ to } 1{\cdot}0 \text{ cusec} = 1{\cdot}5 \text{ ft.} \\ 1 \text{ to } 2{\cdot}0 \text{ cusecs} = 2{\cdot}0 \text{ ft.} \\ \text{Above } 2 \text{ cusecs} = 2{\cdot}5 \text{ ft.} \end{cases}$

$K = \dfrac{\text{Average velocity in channel U.S.}}{\text{Velocity near the side U.S.}}$

$K = 2{\cdot}0$ for head reaches of Major Distys:
$= 1{\cdot}5$ for middle reaches in Major Distys:
$= 1{\cdot}25$ for tail reaches in Major Distys:
$= 1{\cdot}00$ for channels below 10 cusecs.

distributaries known to be in regime.

(*ii*) A.P.M.=Adjustable Proportional Module; I.B.=Irrigation Branch; S.S.O.O.=Submerged Semi-module Orifice Outlet.

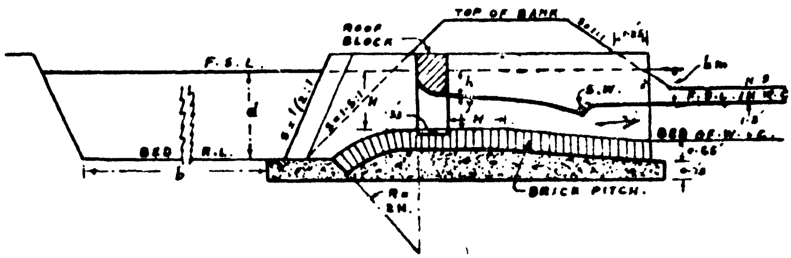
(*iii*) In the case of pipe outlets set at bed level the silt conductive power drops to about 90% with all values of $\frac{V}{V_o}$, if vertex is formed upstream of the outlet in the parent channel.

This outlet is used all over the Punjab. This has proved to be a very successful outlet on account of the immunity to tampering and the adjustability. The cultivators have been attempting to increase the discharge by raising and tilting the roof block. This defect is very easily detected and set right. They sometimes construct a covered drain behind the side wall below water level in the parent channel and then drop it in the out-fall downstream of the roof block by piercing the side wall which is only 1½ brick thick. Though difficult to detect this defect in the supply turns, it is easy to discover it in closures.

16. The Author's Improved A.P.M. (Submerged Semi-module Orifice Outlets.)

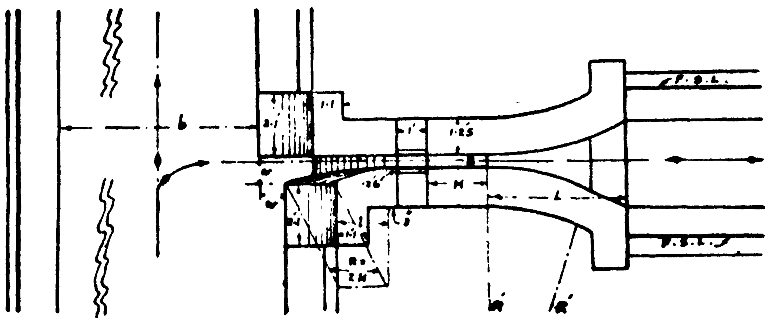
The author carried on very exhaustive research work supplementing the work already done by Crump for all widths and depths on crest from 1·0 to 4·0 feet. The results were published in papers No. 168 and No. 176, Punjab Engineering Congress, Lahore. The first Paper gave the measurements of the silt conductive power of the irrigation outlets as given in table No. 2 and

L. SECTION



PLAN

Fig. 8.



the second one investigated the hydraulics of the A. P. M. outlets. While discussing the paper No. 168 and 176 the following conclusions were accepted unanimously by a committee consisting of E. S. Crump and A. W. M. Jesson, Superintending Engineers and the author, published in Appendix 1 of the author's Paper No. 237 P. E. C. Lahore.

(*a*) (*i*) The channels fitted with A.P.M. outlets set at 6/10th had a tendency to silt up because A. P. M. outlets as constructed did not take their due share of silt. The proportional silt conduction was not enough because 10 to 12% water was lost in absorption and the due share of silt to be drawn by an outlet was considered to be 110 to 112 percent. The improvements made by the author in the approaches of the A. P. M. outlets as shown in Fig. 9 were satisfactory.

(*ii*) The discharge co-efficient C in the formula of an A.P.M. ($q=CBy\sqrt{h}$) was not constant, 7·3, but showed considerable variation from 6·2 to 8·4. In the case of narrow outlets ·2 ft. width with H=4·0 and y=2·0, the co-efficient dropped as low as 6·2. In Crump's outlet the co-efficient decreased with the decrease in width and with the increase in depth on crest.

(*iii*) The Crump's A. P. M. ceased to be a submerged semi-module when the hyraulic jump took place on the glacis beyond the level floor of length H downstream of the roof block. In such cases, the co-efficient as high as 8·4 was recorded. In this case the mere formation of jump did not indicate modularity.

(*vi*) The minimum modular head required for the Crump's A. P. M. was found to be considerably more than that given by him in paper No. 26. When the outlets were narrower than ·5 ft. and also when the depth on crest was more than 2·0 ft. M.M.H. was roughly 33% of H.

(*b*) Further experiments were carried out by the author to attain perfection in the points (*ii*) to (*iv*) enumerated above under the supervision of E.S. Crump, Superintending Engineer, Upper Jhelum Canal, at Shadiwala and the results were published in Paper No. 237, Punjab Engineering Congress, 1940.

In the case of the A. P. M. outlets to ensure modularity by visual examination of the hydraulic jump, the following conditions must be fulfilled at the control section upstream of the jump :—

(*i*) Depth not more than 2/3 H (or allowing for frictonal losses in the aproach not more than 2/3 K) total energy.

(*ii*) Filaments beyond the control section either convergent, adherant or rectilinear.

(*iii*) Jet leaving the orifice adherant, that is, in contact with the bed.

The first one was fulfilled in Crump's design by limiting the valve of y not greater than ·5H and the last two conditions were not attained as indicated by the low pressure pockets beyond the control section under the roof block at its end. The jet leaving the orifice was therefore liable to pressure inflations and capable of attaining a co-efficient of 8·4 even beyond the theoretical value of 8·0. These defects were rectified in the author's improved A.P.M. as shown in Fig. 9 by making the following changes with a view to reduce the co-efficient variation and also to make it work as a perfect submerged semi-module.

(*i*) The curve of the roof block (Bernaulli's Lemniscate) was given a tilt, of 1 in 7½ greater than slope of the bed to ensure an adherant jet and to reduce the variation in C by giving a bell-mouth approch upto the control section.

(*ii*) The level floor was removed and a slope of 1 in 15 was given in bed to ensure rectilinear or convergent adherant filaments in the flow beyond the control section indicating pressure on the bed equal to y or slightly more than y and in no case less than y.

(*iii*) The values of y should be between 0·2H and 0·5H.

The upper limit of the value of y was fixed by Crump to ensure that the outlets should run full bore with no vertical starvation of the control section and the lower limit of y=·3H was introduced by the author on the Lower Jhelum canal in 1930, because when y is less than this, the hypercretical jet becomes non-adherent and the discharge increases. In the author's improved A.P.M. this limit is y=·2H.

The defects as given in (*a*) (*i*) and (*a*) (*ii*) were completely removed. The co-efficient variation was very much reduced as given in plate No. XVI A. Vol. III. The variation of C due to H has now totally disappeared and the co-efficient now varies only with the size of the orifice *i.e.*, ratio (B/y). Whatever be the position of the jump, the discharge does not now vary so long as it is formed.

Co-efficient Variation.

B/y	·2	·4	·6	·8	1	2	3	4	5	6	8	10
C	7·0	7·28	7·43	7·54	7·6	7·76	7·82	7·86	7·87	7·88	7·89	7·90

(*c*) The minimum modular head is comprised of the following losses in the A. P. M. outlets :—

(*i*) Loss in the approach upto the control section indicated by the co-efficient C.

(*ii*) Loss in the hydraulic jump.

(*iii*) Loss in the outfall in expansion.

(*iv*) The frictonal loss from the control section to the water course.

Crump simply stated that the mathematical formula for the losses was very cumbersome. The author worked out the mathematics as published in the Punjab Engineer ; The Mall, Lahore, January, 1940. It takes the form $h_m = h\left(1-\frac{C^2}{2g}\right) + \cdot054\,\frac{[F(k)-2k]^3}{F(k)\times(k)} \times h + 3\cdot6f_1 \times \frac{C^2k^2}{2g\,F(k)^2}\,h + f_2h$, where h_m = M.M.H. ; C = Co-efficient in the formula of discharge ; $k = y/h$ = Height of orifice/Depression of roof block ; f_1 = a factor for outfall losses = ·5 ; f_2 = factor of friction loss in outfall = ·15 to ·25.

$F(k) = \sqrt{k^2 + \cdot288\,C^2k} - (k)$. An approximate formula as sometimes suggested by mere guess work in the form $h_m = \cdot75h$ is used in actual practice.

The author analysed a lot of observations of M.M.H. as published in Paper No. 237 and found that the outfall losses were more than 50% of the total M.M.H. It was thought desirable to have a standard design of the outfall. ~~The bed profile has already~~ been fixed and the side walls were carried straight for 1·0 ft., length beyond the roof block and than expanded with a radius of 25 ft. on both sides to the width of the water course in all cases. M.M.H. observations are not comparable unless the outfall design is the same and standardised. This formula cannot be used in practice but actual observations were plotted in Plate No. 4 of paper No. 237 and are now drawn in Plate No. XVII (A), Vol III, for use of the students, M.M.H. is roughly 20 to 25 percent. of H.

(*d*) Though the M.M.H. results as given in Plate XVII (A) show improvement over the Crump's A.P.M. outlets, they are still very high. In some cases it is necessary to put an A.P.M. in cases of water courses with poor command. It was investigated in paper No. 237 to reduce the M.M.H. further by covering the outfall beyond the straight sides with a flat roof with curved edge in the begining as shown in Fig. 2. The reduction in M.M.H. was recorded by about 30% and the results are given in plate No. XVIII B, Vol. III, for use of the students. This advantage is gained by the addition of the surge chamber in which the jump takes place under controlled conditions. Better results are possible by improving the design of roof covering into a gradual expansion.

(*e*) Design of an A.P.M.—Crump designed a nomogram with C = 7·3 giving the solution of the equation $q = CBy\sqrt{h}$, but C is variable. The author designed a nomogram for the variable C and published it in paper No. 176 Punjab Engineeing Congress 1934, and it is reproduced in Plate XVIII B, Vol. III, for use of the students. The method of its use is explained therein by meens of a key diagram.

With a slide rule the following alternative method for the design of A.P.M. may be of interest. Fix H according to the required setting of the outlet. To determine the size of A.P.M. keep $h = y = H/2$ and calculate $B = \frac{q}{Cy\sqrt{h}}$. A. P. M. size, having width of the flume equal to B or just higher than B (calculated) to suit the standard width will give the required width of the A.P.M. Then calculate, $a = \frac{q}{CB}$

$$y\sqrt{h} = a \qquad \text{(A)}$$

$$h + y = H \qquad \text{(B)}$$

The solution for equations (A) and (B) is given by one setting of the slide rule. This method is not only more accurate but also quicker than the Nomogram method.

Rule. "Set the reversed slide to 'a' on the D scale. The values of h and y are given under the cursor of the A and C scales respectively.

(*f*) **Proportionality versus rigidity.**

Proportionality in an A.P.M. is a great scientific achievement, and is an asset of incalculable value in outlets of regime channels.

It is the considered opinion of Crump, who invented this outlet and the author, who developed it that proportionality should not be sacrificed on regime channels, which could be run nonsilting by suitable grading and silt selection at distributary head regulators ; (The author's paper No. 189, Punjab Engineering Congress, 1936).

If a channel cannot run in regime, proportionality in its outlets becomes a disadvantage. Once it is discarded, there is no limit to the lowering of the setting and to make outlets sub-proportional, *i.e.*, rigid. The author's S.S.O.O. may be set even below bed level, if the available working head permits. The use of an S.S.O.O. approaching a rigid module is for superior to the known devices of rigid modules with moving parts such as Kent's.

(*g*) **Practice of cultivators to foul the control section.**

The co-efficient of discharge C_d in formula of the discharge of a submerged semi-module is the product of C_a and C_v as in the case of *Vena contracta* of a free fall orifice. If the co-efficient of sectional area C_a and the co-efficient of velocity C_v are changed, there shall be a corresponding change in the discharge even if the jump be formed and the control section be convergent adherent. C_a can be changed in two ways.

(*i*) **Vertical starvation.** This means that the outlet does not run full bore. This happens in Crump's A. P. M. when the depth on crest is more than 2·5 ft. The discharge can be increased by having a wider roof block or by adding a lip downstream of the roof block. In such outlets *zamindars* usually put in a wooden strip of metallic flap downstream of the roof block. They just make up their shorage although the discharge cannot be increased beyond the theoretical. The chances of vertical starvation have been cured by a tilt in the lemniscate curve of the roof block in the author's improvements of the A.P.M.

(*ii*) **Horizontal starvation.** It is either due to excessive width of an approach resulting in back flow or due to rough sides. The *zamindars* usually cure the former by putting a horizental wooden plate upstream the roof block and the latter by rubbing the sides approaching the A.P.M. block.

Similarly the value of C_v can be varied by two methods :—

(*i*) **Excessive length of approach.** The losses in friction are increased before the control section resulting in draw down. It has been observed to be as much as three inches in some cases.

(*ii*) **Unsuitable approaches.** The head lost in entry causes serious error in the discharge.

We have so far explained the hydraulics of the Engineers and the cultivators but the subordinates executing the construction on the outlets introduce sometimes changes to benefit the cultivators. The Engineer in-charge should be careful to watch them. These are practised in three ways resulting in 12 to 15% increase in discharge. (*i*) Back tilt in the roof block, resulting in concavity of stream lines with consequent increase in discharge (*ii*) Groove dawnstream of the roof block in bed causing partial aeration below the jet. (*iii*) Upward tilt in downstream level floor causing a low pressure pocket. All these factors have been effectively cured in the author's improvements of A.P.M. It is enough to show that the hydraulics of the submerged semi-modules (A.P.M.) is much more complicated than it is usually supposed to be. It does not mean that the control section of these types of outlets can only be affected but that other types of outlets K.G.O., Harvey Stoddard or Jamrao, Bend outlets are worse still in this respect.

17. Crump's Open Flume Outlets.

The design of the open flume outlet is given in Fig. 10. The length of crest is 2H. It is exactly the same design as originally invented by Crump but the author's approaches and the standard outfall design as accepted by Crump have been added.

Open flume outlet with its proportional setting of 9/10D takes its due share of silt, that is, about 112%.

Type Design of Narrow Open Flume Outlet

Scale = 1/50

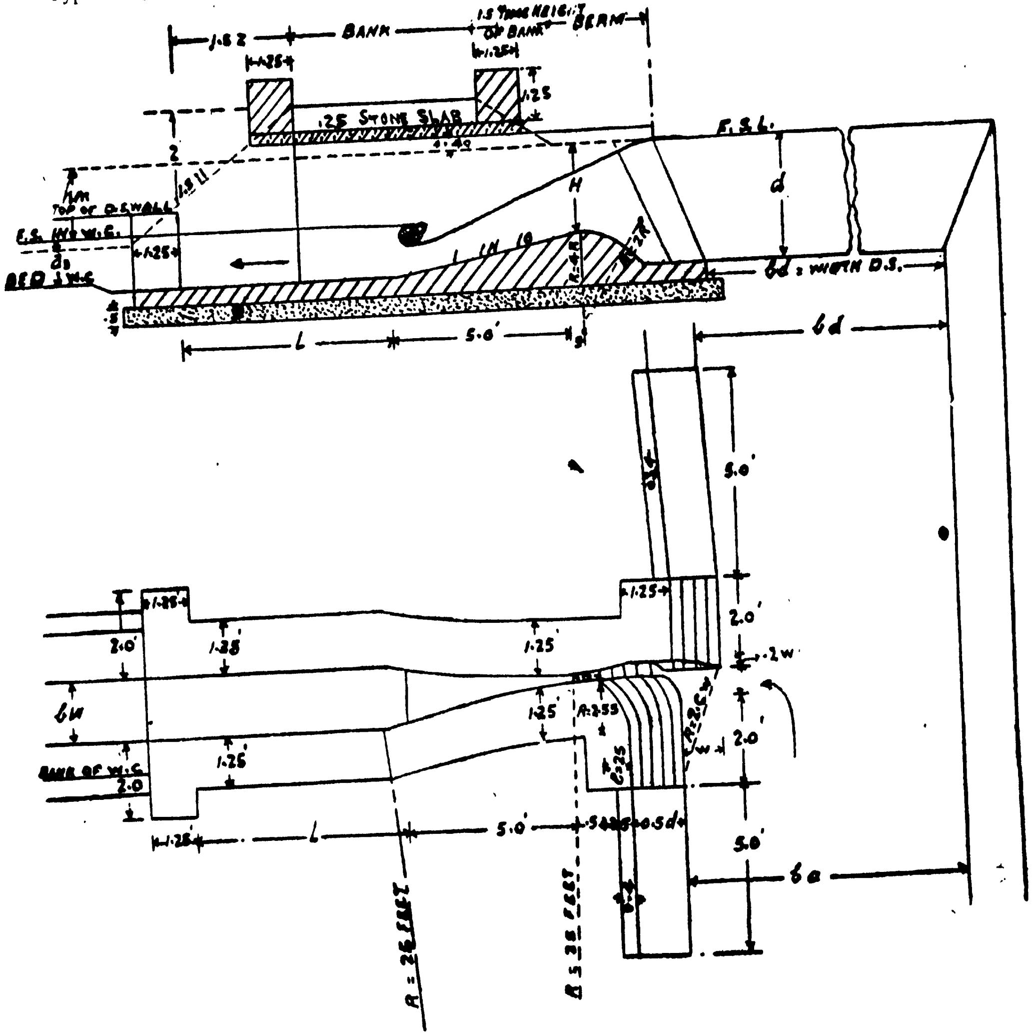


Fig. 11

Bed widih in Channel U.S. of outlet	$=b_u$
Depth ,, ,, ,,	$=d$
Discharge ,, ,, ,,	$=Q$
Bed width in Channel D.S. of outlet	$=b_d$
Discharge of the outlet ...	$=q$
Setting of the outlet ...	$=s$
Depth on crest ...	$=s_d$
Width of outlet ...	$=B=\frac{\quad}{CH^{3/2}}$
Co-efficient of Discharge (from Diagram)	$=C$
Length of crest ...	$=0{\cdot}5$ ft.
Minimum working head (from Diagram) ...	$=h_m$
Actual working head ...	$=h_w$

Set back of U.S. wing
Width of Approach } $w=K\frac{q}{Q}\left(b_u+\frac{d}{2}\right)$

Splay of U.S. side wall ...	$=S=w-B$
Radius of splay on side ...	$R_1=2{\cdot}5\ S$
Length of Approach curve on side ...	$=2S$
Radius of curvature in D.S. Projected wing ...	$=R_2=2{\cdot}6w$
Rise of Crest above Disty: Bed U.S. ...	$=R$

Distance of Beginning of Crest from Centre Line of Disty: W.S. $=\frac{b_u}{2}+\frac{d_u}{2}+2S$

Radius of Curvature of Approach curve in bed $=R_3=4$ R in the convex part and $R_4=2$ R in the concave part Radius of the outfall wall = 25.

Width of water course = b_w $\begin{cases} 0 \text{ to } 1 \text{ cusec} = 1{\cdot}5 \text{ feet} \\ 1 \text{ to } 2 \text{ cusec} = 2{\cdot}0 \text{ feet} \\ \text{Above } 2 \text{ cusec} = 2{\cdot}5 \text{ feet} \end{cases}$

$K = \frac{\text{Average velocity in channel U.S.}}{\text{Velocity near the side}}$

= 2·0 for head Reaches in Major Distributaries.
= 1·0 for Middle Reaches ,, ,,
= 1·25 for tail ,, ,, ,,
= 1·0 for Channels below 10 Cusecs Discharge.

Minimum Modular Head is 10 to 15% of the depth on crest.

The co-efficient of discharge is steady, 3·0 in the formula $q=CBH^{3/2}$ for all values of H and width from 0·3 to 1·0 foot so long as the length of crest is 2H, for narrow open flume.

18. Author's Narrow Open Flume Outlets.

(*a*) Crump did not investigate the behaviour of narrow open flume outlets. In Crump's design the length of approach is very long and the length of crest is 2H. In the case of narrow open flume outlets, the co-efficient drops very much due to very high frictional losses and M.M.H rises. The author carried out detailed investigations on outlets with widths from 0·3 ft. to 0·5 ft. The experiments were divided into three sections.

(*i*) Length of crest 2H.

(*ii*) Length of crest 1·25H.

(*iii*) Length of crest ·5 ft. in all cases.

In the first two cases, the co-efficient variation in the formula $q=CBH^{3/2}$ as found out, is given in Plate XVIII (A) Vol. III., and the Modular Minimum Heads, as observed, were found to satisfy the relation $h_m=0{\cdot}18\left(\frac{3{\cdot}09}{C}\right)^2\times H$, which can be obtained from Plate XVIII (A)

In the third case it was suggested by Crump to reduce the friction losses further when discussing the results of the first two cases and the observations in this case were taken under his supervision and results were published in paper No. 237, Punjab Engineering Congress Lahore. The design of the outlets is given in Fig. 11 and the results of the value of C and h_m are given in Plate No. XVIII (B_2) and XVIII (B_1) respectively. The length of crest being constant and small, the control section shifts beyond the contracted section and under certain conditions the co-efficient of discharge increases even more than 3·09. The visual examination of the jump does not indicate modularity in all cases but the provision of the required h_m as per Plate XVIII is essential to ensure modular conditions.

(*b*) Open flume outlets both narrow and wide are very sensitive outlets because the discharge varies as $H^{3/2}$. The discharge of these simple outlets can easily be increased by adopting undermentioned devices fouling the control section.

(*i*) The discharge is increased by heading up water in the small channels by cattle and men sitting across the section downstream of the outlet.

(*ii*) There have been cases where two men sat the whole night in water with a cloth stretched across near the tail reaches of distributaries. This device passes water below while heading up 3" to 4" and cannot be detected.

(*iii*) The cultivators usually throw bricks and rubbish across the channel downsream the outlet which is usually known as *Daff* (*Bund*).

(*iv*) The discharge of the open flume outlets can be increased by inserting horizontal lamina as shown in Fig. 12.

The upper portion runs as a weir and the lower one as orifice. The sum total of these discharges is more than the weir formula discharge. The greater the number of lamina the larger the increase. By inserting even one plate, increase of 10 to 12% is likely.

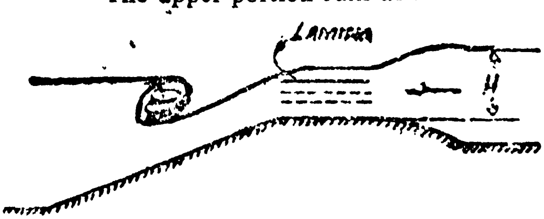


Fig. 12

(*v*) The discharge of the open flume outlet can also be increased by simply putting a pumpkin or a plate in front of it, as shown in Fig. 13. The control section shifts from middle of crest at C to C′ in a wider section and more discharge is passed even though it may act as a drowned weir. The increase is about 20%.

19. Double Module Outlets (Orifice-cum-A.P. M. or Open Flumes).

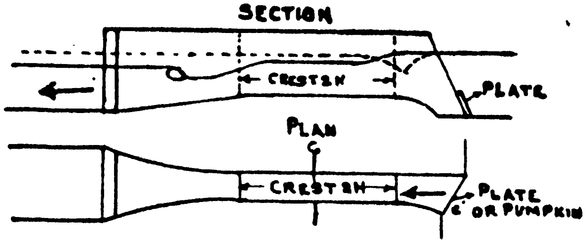


Fig. 13.

The first known outlet of this type was called after Scrachley's name. It used to be a masonary orifice outlet discharging into a well and then followed by an orifice in the well. The modern practice (Paper No. 146 P. E. C. Lahore, 1931 by K. R. Sharma) is to construct a pipe outlet at bed level of the parent channel and then to take out an open flume or the A.P.M. from the well as shown in Fig. 14. These types of outlets are economical and ensure a good silt entry and easy adjustablity where

Pipe-Cum-O. F.

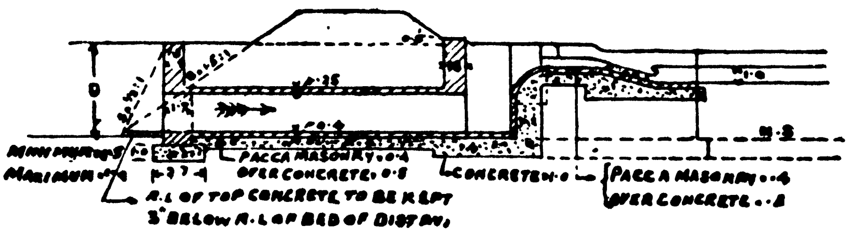


Fig. 14 (a)

Pipe-Cum-O.C.M.

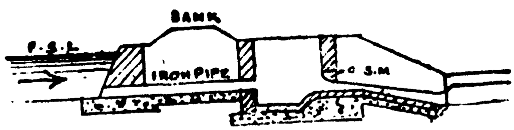


Fig. 14 (b)

they have to be constructed across wide patrol-road banks in the case of branch and main canals. The design presupposes that the available working head is enough and to spare for double loss, first on the drowned pipe outlet and then in the module outlet. The loss of head in the pipe or massonry barrel may be kept 0·1 ft. all cases and the rest of the available working head to be used to set O.F. or O.S.M. (A.P.M) as low as possible.

20. Rigid Modules.

(A) **Kent's module.** Its design was based on a pressure float device as originally used by Italians in the design of their modules. These outlets failed because floats could be the raised by inserting sticks through the air-vents and the discharge could be increased.

(B) **Gibb's module.**

(*i*) This outlet was invented by A.S. Gibb, Executive-Engineer, Punjab Irrigation as published in Paper No. 13-A Class A, P.W.D. Irrigation Branch Publications. The module is named after its inventor and gives an almost constant discharge over a considerable range of water levels upstream and downstream. It is only a rigid module without any moving part. The module consists of a chamber, semicircular in plan, called the eddy chamber, round which the water flows.

Water enters this chamber through an inlet pipe, which delivers water to a 180° rising pipe in which free vortex flow is developed. It then enters the eddy chamber—vide Fig. 15.

The exit at the downstream end of the module discharges through a spout or flume into the outlet channel. In a free vortex flow, velocity × radius = constant. In this condition, the water at the outer circumference of the stream flows at a comparatively high level due to centrifugal force and the surface slopes down towards the inner circumference. A series of baffles are fixed in the eddy chamber with their lower edges sloping at the required height above sill of the module. If the head causing flow is in excess, the water banks up at the outer circumference of the eddy chamber and impinges against the baffles, imparting an upward, rotational, direction of flow to the water, which spins round in the compartment between two successive baffles and finally drops on the oncoming stream of water, thus dissipating excess energy. The action of the baffle is not unifrom and, except at maximum dissipation, some baffles are out of action. In general, one baffle is for more effective than the others.

PLAN

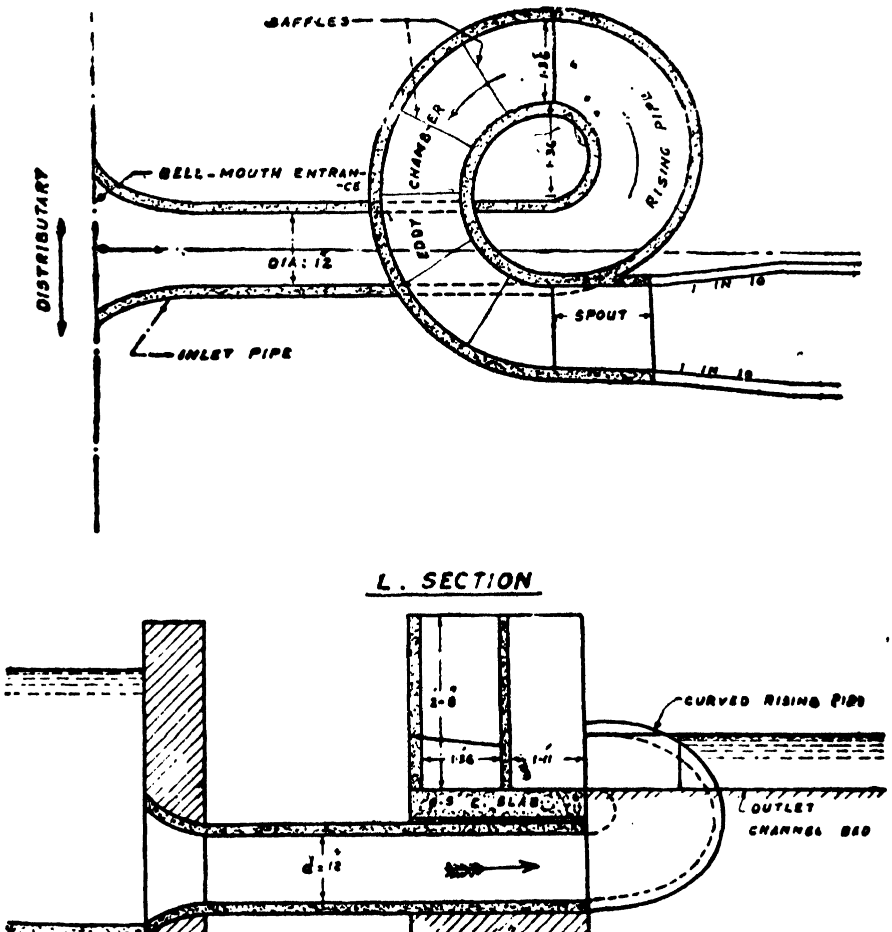


Fig. 15

(*ii*) **Discharge formula.** Gibb's formula for discharge was

$$Q=r_0\sqrt{2g}\ (d_1+h_0)^{1.5}\left\{\frac{m^2-l}{m^3}\log_e m+\frac{l}{m}\log_e m-\frac{m^2-l}{2m^3}\right\}$$

where r_1=Radius of inner semicircle of the eddy chamber ; r_0=Radius of the outer semicircle d_1=Depth of water at inner cirumference.

d_0=Depth of water at outer circumfernce ; h=Head causing flow, varying from h_0 to h_1 ; h_p+D=Total difference of level measured from the minimum water level in the parent channel or distributary to the floor of the eddy chamber (*i.e.*, including head lost in inlet pipe=h_p)

$m = r_0/r_1$ = Ratio of outer radius to inner radius ; Q = Discharge.
B = Width of eddy chamber = $r_0 - r_1$.

The formula is based on free vortex flow, in which the velocity at any point varies inversely as the radius and by Bernouilli's theorem that the total energy head of all filaments is constant :

(Total energy head = head due to velocity plus static head measured from the same datum.)

Gibb's formula held only for his standard design in which

$m = r_0/r_1 = 2$, and $\frac{h_0}{D} = 1/7$

(*iii*) **Poona experiments of Gibb's modules.**

The results of the Poona Experiments on Gibb's module have been Published in Research Publication No. 3 by C.C. Inglis and Rao Sahib D. V. Joglekar of Central Irrigation and Hydrodynamic Research Station Poona, Bombay, 1940, The conclusions arrived therein are summarised :-

(*a*) Of the 6 baffles in the eddy chamber of the standard 3 cusecs module in which

$$\frac{\text{radius of outer circumference of eddy chamber}}{\text{radius of inner circumference of eddy chamber}} = 2{\cdot}0$$

and the rario of $\frac{h_0 = \text{head causing velocity at outer circumference}}{D = \text{depth of water at outer circumference} + h_0} = 1/7.$

The first 4 baffles were more effective in killing the head *viz.*, 0·12 ft. in a 3 cusecs module than baffles 5 and 6 which increased the range by only 0·18 ft.

(*b*) When a bell-mouth was added to the inlet pipe, conisderably less head was reqired to attain modularity and the range was increased.

(*c*) The range was a minimum with $m = r_0/r_1 = 2$ as adopted by Gibb ; but he was wrong in assuming that the range of his module could be increased by increasing the ratio B/D ; and the values of B/D within which a high range is obtained are 0·8 to 1·5.

(*d*) Increasing the number of baffles in a semicircular module had a negligible effect on range, though fluctuations of discharge within the range, were slightly reduced.

(*e*) The range was not increased by increasing the lengh of the arc of curvature through which water flows before entering the eddy chamber.

(*f*) The range of a module can be increased by increasing the number of spirals of the eddy chamber. Gibb held that the range of a module with 3 semicircles would be three times the range of a module of one semi-circule. This was not borne out by experiments, the range of one cs. spiral module with 3 semi-circles was only 1·42 times that of "a single semicircle" module.

(*g*) Maximum permissible downstream depth above the module floor was found to be 0·61 G against 0·56 D according to Gibb. When a 1 in 10 diverging flume was added to the spout, the maximum downstream water level increased to 0·7D. This modification is, therefore, desirable

(*h*) When the length of exit flume or spout is = 2D, the range is maximum. Gibb's idea that the length of spout should be = B, was incorrect.

KHANNA'S RIGID MODULE OUTLET

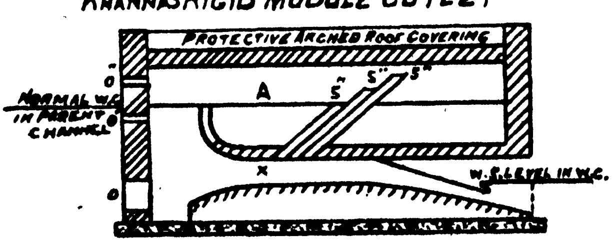


Fig. 16

(C) **Other rigid modules.**

R. K. Khanna, Assistant Engineering, Punjab Irrigation and Ghaffur, Executive Engineer, Punjab Irrigation, have brought out designs of Rigid Modules without moving parts. The limits of their working need to be determined in actual practice in both cases. The designs are shown in Fig. 16 and 17 respectively.

21. Tail Clusters.

The tail masonry work on a channel is

called a tail cluster. When the discharge of a distributary or minor reduces below 5 cusecs it is desirable for equitable distribution to construct all the tail outlets in the form of a cluster. The length of the water course from the tail cluster should in no case be more than 2 miles. The most suitable outlet for use on the tail cluster is an open flume. All outlets at the tail are open flumes with crests at the same level. Usually the depth on crest is kept one foot. The type designs of the tail clusters are given in Fig 18 to Fig 20.

22. Design of Semi-Module Outlets in Channels With Varying Discharges.

(*i*) The supply available in the Sutlej Valley Canals is so short in the critical sowing and maturing periods that it has been found necessary to run these channels with what is called normal supply for certain important periods in the agriculutral rotation. This normal supply is 55 percent of the full supply. The design of outlets on these channels should, therefore, be such that they take their proper share of the discharge both at full supply and at normal supply. In other words, it is necessary to design the outlets on these systems as proportional outlets.

(*ii*) The following are the three types of proportional outlets which are suitable for use on these canals :—

(*a*) An open flume outlet having its crest at 0·9 D and having an available working head of not less than 0·4D will draw a proportional discharge at both full and normal supply conditions in a channel.

(*b*) An O.S. M. with $H_s = 0{\cdot}43D$ and an available working head of not less than 0·4D will draw a proportional discharge at both full supply and normal supply conditions in a channel provided G does not exceed 0·69D.

(*c*) An O. S. M. with $G = 0{\cdot}75D$ and $Y = H_s$, will draw its due share of discharge at full and normal supply levels. In the latter case it works as an open flume. The working head required in this case is also 0·4D.

(*iii*) It will be seen that a working head of 0·4D is required for the design of outlets with the required conditions. An examination of the available working heads of outlets on the Sutlej Valley Canals shows that whereas conditions vary considerably, a substantial number of outlets do not possess a working head of 0·4D. It will thus be recognized that whereas it is possible to design a large number of outlets such that they would work proportionally under both full and normal supply conditions the number of those which cannot be so designed is considerable.

(*iv*) For purposes of design, therefore, the outlets on the non-perennial distributary on the Sutlej Valley Canals may be classified as follows :—

(A) Outlets which can be designed to work porportionally under both full and normal supply conditions—such outlets must have an available working head of not less than 0·4D.

(B) Outlets which cannot be designed as class (A) but have a sufficient working head to be modular at full supply conditions. Such outlets are those which have a working head of 0·2D to 0·4D.

(C) Outlets other than those included in classes A and B. Such outlets have a very poor command *i. e.*, less than 0·2D.

After the outlets of a distributary have been classified as above and before proceeding with the design of outlets, an attempt should be made as explained in paragraphs 10-25 to see if any of the outlets classed B cannot be converted into class A or those classed C to class B or A. For successful distribution it is essential that the number of outlets of classes B and C should be reduced to the minimum.

(*v*) The type of outlets that should be adopted for each class is as follows :—

I. Class (A). For this class of outlets which can be designed to draw a correct discharge at both normal and full supply conditions, the following type should be adopted :—

(*a*) An open flume outlet with crest set at 0·9D provided B_t does not work out to less than 0·2 ft.

(*b*) If in (*a*) above B_t works out to less than 0·2 ft. an O. S. M. set at 0.75D may be designed such that the value of H_s ranges from 0·375 to 0·43D.

(*c*) If an O. S. M. outlet as above cannot be designed for a particular discharge, then O. S. M. set at 0·69D may be designed satisfying the condition $H_s = 0{\cdot}43D$.

II. Class (B). For this class of outlet, it is not possible at present to arrange for proportional working at both full and normal supply conditions. The best type of outlets for this class would be :—

(*a*) An open flume with H equal to five times the working head available.

(*b*) An O. S. M. outlet set at 0·75D with H_s from 0·375D to 0·48D. Whichever of the two would give a lower setting, so as to enable the outlet to draw some discharge at low supplies.

(*c*) In case the width B_t of the open flume outlet works out to less than 0·2′ or it is not possible to design an O. S. M. according to (b) above for the particular discharge then the outlet should be designed as an O. S. M. with crest an 0·69D, provided it could work modularly under full supply conditions.

III. Class (C) For this class of outlet which has a working head of less than 0·2D it is not possible to design an outlet of the semi-moudle type. The only outlet possible is of the pipe or orifice type and the best type is Scratchley outlet.

The working head H_w to be adopted for purpose of design of the outlet should be the average of working heads observed during the time of keen demand (1st to 15th September) for a period of 10 days. A fair proportion of these working heads should be personally checked by the Sub-Divisional Officer (by surprise visits if possible).

(*vi*) When water level in a distributary rises above the designed full supply level on account of changes in regime, the open flume outlet draws a high percentage of exess. To guard against this, all open flume outlets, whether of class A or B, should be fitted with roof blocks.

23. Water Course Discharge Observations.

The discharge of water courses downstream of the outlets are observed by any of the following methods :—

(*i*) Float observations.

(*ii*) Trapezoidal notch cipollette Weir.

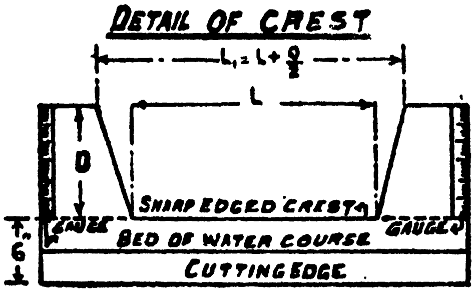


Fig. 21

(*iii*) Wooden flumes or Portable Detachable Tin flumes.

(*iv*) Calculations from the discharge formula of the outlet.

In the first case, the surface width is measured say w feet and depth D in the middle is taken. The surface velocity is taken by running a float or a cow-dung piece for 20 feet length in a straight reach and by taking the time by a stop-watch. The velocity per second is then worked out say V ft. per second The discharge = 0·6 wDV cusecs. This is a fairly approximate method.

In the second case a trapezoidal sharp-crested notch is used as shown in Fig. 21. It is called cipollette weir. Its use is described by A.S. Gibb in paper No. 14 Punjab P.W.D. Irrigation Branch, Publication. It is fixed across a water course without heading up water in it or into a field by diverting the supply from ihe water course. The gauge reading is taken and the discharge can be got from tables published in the said publication. This method gives accurate results only when free fall conditions for the weir are available without heading up water in the water course. The working head is required equal to the depth on crest which is not generally available.

24. The Author's Portable Tin Flume for Water Course Discharge.

Experimets were carried out by the author to design suitable flume for this purpose and the results were published in the Punjab Engineer, Lahore. February, 1940. A brief summary is given below for ready reference :—

(*a*) An accurate determination of water course discharges is a very essentail factor for successful irrigation and equitable distribution. Water is supplied to the cultivators from an outlet which is supposed to discharge the permissible discharge of the water course. The design of the outlet is based on certain co-efficients which are liable to vary with different conditions of the design of the outlet and its imperfect construction. The public opinion is getting enlightened day by day and the cultivators are now keen to get their permissible discharge correct to the second place of decimal.

(*b*) A cultivator is not satisfied to know that the Irrigation figures are more than the permissible.

(*c*) The discharge observations of water courses by means of floats generally result in great dissatisfaction among the cultivators. The overseers and *zilladars* always report different discharges taken by means of floats. The discharge observations of a water course are, thus, very unreliable. The use of the cippollette weir, as described in Irrigation Branch Paper No. 14. Class A by A.S. Gibb, gives fairly accurate results. The free fall conditions for a cippollette weir are not generally available and the observations in the drowned conditions by an average subordinate generally give wrong results. It will, therefore, be a great improvement to observe the discharges by means of an open flume under modular, conditions, when the hydraulic jump is formed downstream of it. In practice, it was observed that it needed special efforts to make the cippollette weir water-tight because water falling down the crest generally scours the downstream bed and the weir is creeped out. Moreover, the velocity of approach under different conditions of the upstream water course section, affects the discharge considerably.

(*d*) The idea of discharge observations of water course by means of a portable flume, was introduced by Mr. Abdul Gafoor P.S.E. (Sub Divisional Officer. Tandalianwala) is his note dated 2-8-1926 circulated by the Chief Engineer. A co-efficient of discharge 3·0 was adopted for the wooden flume for all readings which is not correct. A gauge of the flume was provided upstream in the return wall which is liable to be affected by the loss of head in entry and velocity of approach under different conditions of the water course upstream. Moreover, the design of the flume was not suitable as it required as much as 25 percent of the head for free fall conditions. The experiments were taken in hand by the writer in summer 1931 to improve the design of the flume and to calibrate it for different heads and under different drowning ratios by means of actual discharge observations. The discharges were observed by means of tank measurements.

(*e*) The drawing for the type design of two sizes of portable flume is given in Fig. 22. The flume No. 1 of one foot width is capable of measuring discharge upto 1·00 cusecs. The flume No. 2 of 1·5 feet width is capable of measuring discharge upto 3·00 cusecs. The flumes were made of galvanised iron sheet G.W. No. 22. They are quite light and can easily be carried by a *beldar* or on the carrier of a motor car. One *beldar* can fix them in position in ten minutes. Each of the flumes is in two parts. One part comprises the glacis and the other consists of the upstream approaches and the crests. One part can be folded on to the other.

(*f*) The flumes are made according to the standard design of a meter flume described by the writer in his paper No. 154, Punjab Engineering Congress, 1932 and sketched in Plate No. 2 of the said paper. They have a uniform width throughout their length. The length of the crest is 2H. The upstream approach curve in bed is laid with radius 2H. The gauge is provided on one side in feet reading upto ·01 foot and on the other side in cusecs at a distance of 3H from the beginning of the crest. The flume is straight upto a length of 2H upstream of the gauge and is curved out with a radius 2H from the upstream return wall. The floor is kept level opposite the gauges. The downstream part consists of glacis 1 in 5. This design ensures a standing wave with a working head of 10 percent of the depth on crest. The proper location of the gauges and the silt-free level flow opposite them remove the error generally introduced by the velocity of approach, which would otherwise vary with the shape of the water course upstream.

(*g*) The graph for the discharges of both the flumes is given in Fig. 23 for different heads in modular conditions, so long as the hydraulic jump is formed on the glacis. It will not generally

Design of Portable Flume

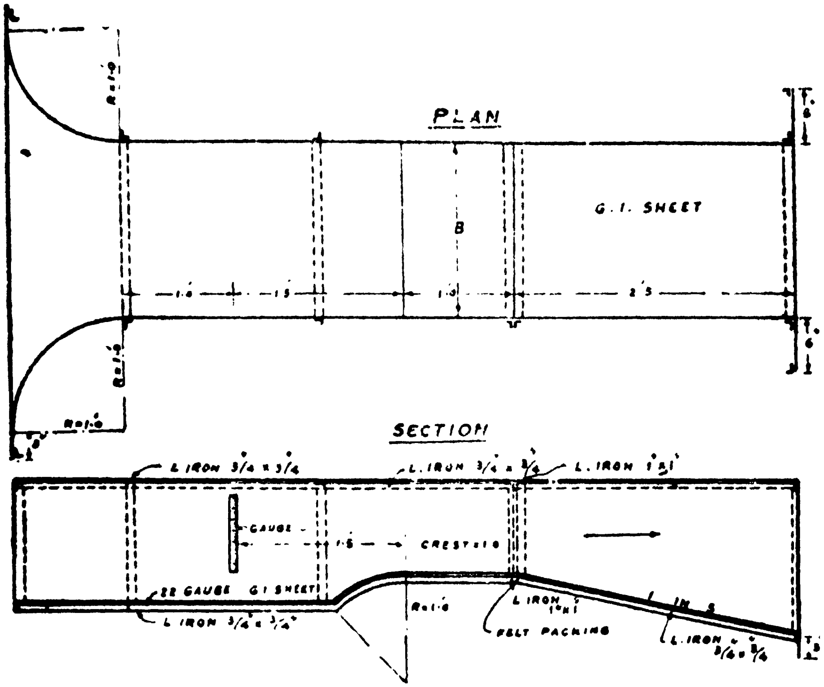


Fig. 22

be necessary to run these flumes in drowned conditions, because in no case the required working head is more than 0·1 foot for modular conditions of the flume. Such a small head is generally available by putting a flume in a branch water course or in an open field. The discharge can be read from the graph or a cusec gauge may be provided in flume itself or by using table No.3. The main and the most important precaution in observation of the discharge by means of a portable flume is that its crest should be absolutly level. This can easily be arranged by means of a mason's spirit level. The spirit level may even be rigidly fixed with the side walls and on the rigid cross bar across the crest. This should in practice be tested invariably by letting in a

TABLE No. 3

Sharma's Table for Portable Flume.

Gauge	Discharge		Gauge	Discharge	
	Flume width 1·0′	Flume width 1·5′		Flume width 1·0′	Flume width 1·5′
0·16	0·22	0·30	0·44	0·99	1·47
0·18	0·25	0·36	0·46	1·06	1·57
0·20	0·28	0·42	0·48	1·13	1·67
0·22	0·32	0·49	0·50	1·20	1·78
0·24	0·37	0·56	0·52	1·28	1·90
0·26	0·42	0·64	0·54	1·36	2·03
0·28	0·47	0·72	0·56	1·44	2·16
0·30	0·53	0·80	0·58	1·53	2·29
0·32	0·59	0·89	0·60	1·62	2·42
0·34	0·65	0·98	0·62		2·56
0·36	0·71	1·07	0·64		2·70
0·38	0·78	1·17	0·66		2·84
0·40	0·85	1·27	0·68		2·98
0·42	0·92	1·37	0·70		3·12

little water before the water course is cut on to the meter flume so that the depth on crest is 0·005 foot as read from the gauge. This should clearly show unevenness in the crest.

The upstream ends should be well puddled and the water course berm and bed should be shaped upstream of the flume so that the entry is smooth. There should be no leakage below the bed or on the sides. This can be very easily arranged on account of the edge provided upstream and the curved return walls. Care should be taken that the floor opposite gauge is clear of any deposit at the time of observation.

(*h*) When the discharge is to be observed for an outlet which is working free fall or with a hydraulic jump, it does not matter if water level is somewhat headed up or depressed by insertion of the flume. The gauge of the meter will become steady after a few minutes in such a case. But when the discharge of an outlet is to be observed when it is not working modularly, it is imperative that the water level in water course should not be changed by insertion of the flume in water course or on the sides. The discharge of the watercourse is generally approximately known from the conditions of the working of the outlet. The flume should be fixed so that the gauge reading on the cusec gauge is in level with the water level in the watercourse. A peg should be fixed in the watercourse with its top flush with the water level in watercourse before water is opened into the flume. In case water level in the watercourse overtops the peg or is depressed below the top of the peg, the observation should be repeated by adjusting the crest according to approximately observed gauge.

The flume should be fixed near the outlets as much as possible so that the loss in observation on the watercourse does not affect the observation. A distance of about 100 feet downstream of the outlet would be sufficent.

(*i*) The writer thinks that the introduction of this accurate method of observation of discharges will lead to great satisfaction among the *Zamindars*. Every subordinate should be provided with these portable flumes and the discharge of the watercourse should always be observed jointly by the *Zilladar* and the Overseer with these flumes on complaints of shortage from the cultivators. It is very difficult to satisfy the *Zamindars* afterwards when a wrong discharge is once intimated to them by a *Zilladar* who generally bases it on inaccurate float observations.

25. Outlets Chaks.

The area irrigated from one individual outlet is called *chak* of the outlet. The discharge of the outlet is worked from the culturable commanded area in the said *chak* based on the permissible intensity of the irrigation and the water allowance. The maximum discharge of an outlet is 2·75 cusecs and the minimum 1·0 cusec. The length of watercourse irrigating a *chak* should not be more than 2·0 miles. The procedure to design the water course and to fix the *chaks* and the method to work out the area, command and draw-off statements is explained in Chapter XVII of this part on irrigation projects. The A.P.M. outlet set below 8/10D or even upto 12/10D or rigid modules are suitable types of outlets in the head reach of channel so that the increase of supply at head is conveyed to the tail reaches where it is most wanted. The proportional outlets such as O. F. @ 9/10th, and A.P.M. @ 6/10th, are suitable for the middle reach.

The open flumes are suitable for the tail reaches of channels and preferable in the form of distributors and clusters. They are sensitive and can absorb the excess which usually floods the tail in rains or when the upper outlets are closed by the cultivators for silt clearness of the watercourses.

26. Examination Questions.

1. What do you know about Kennedy's gauge outlet and explain the use of it. (*T. C. E. 1933*)
2. Give the formula used for calculating the discharge of three types of outlets used in Irrigation Branch. What is the normal setting for each type ?
3. Give the formula for measuring the discharge of water-course with fair accuracy, explaining the same and any co-efficient used. How would you measure the discharge with great accuracy ? (*P. I. B. 1941*)
4. How would you calculate the minimum Water level required at an outlet from a contour plan of the outlet *chak* ?

What is the minimum size of outlet *chak* you would normally permit on a canal in which the water allowance is 3·25 cusecs per 1000 acres ? What are the objections to outlets of smaller size ? (*P. I. B. 1940*)

5. What difficulties, if any, would you expect to find in the irrigation of a *chak* which has an outlet of 3·5 cusecs ? (*P. I. B. 1940*)

6. Make a dimensioned sketch of a distributary tail cluster of two outlets, one 'front' and the other on the left side (at right angles), the particulars for which are as follows :—

Full supply discharge front 2·0 cusecs. Left 1·5 cusecs. Working head front ·5 ft. left ·75 ft. (*P. I. B .1939*)

7. The enclosed blue print plan shows the area which is proposed to be irrigated from a channel marked on it. Propose in it a suitable *chak* without marking *nakas* of watercourses, and design suitable outlets for these. Each rectangle may he taken to contain 25 acres. Permissible annual intensity is 75 pc. *Kharif Rabi* ratio 1 :2 and full supply factor at outlet head is 88 cusecs of *Kharif* permissible. (*P. I. B. 1939*)

8. Give the standard formula for calculating the discharge of each of the following outlets.

(*a*) A.P.M. outlet, (*b*) O.F. outlet, (*c*) Pipe outlet and state the relative advantages and disadvantages of each of these types of outlet. (*P. I. B. 1938*)

9. On the enclosed blue print, mark the alignment of the principal water course from the two tail outlets allowing one *Naka* to each square. (*P.I B. 1938*)

10 A certain minor distributary would be improved if the water level were, lowered as much as parcticable. A tail cluster for this minor has to be constructed for the first time. The R.L 's, of water surface in the water course at the site of the cluster are 512·40, 512·0 and 512·1. These levels are sufficient for command. The reduce level of the water surface in the minor as now measured is 513·2. The discharge of the water courses are to be 1·3, 1·4 and 1·7 cusecs. The positions of the approaching water-courses are shown. Design the tail of cluster. (*P. I. B. 1938*)

11. Describe the two methods of measuring the discharge in a water-course, Point out what precautions are necessary and how the formula are derived. (*P. I. B. 1937*)

12. Design the water-courses for 2L minor on the enclosed blue print. Describe briefly the essential working principal of an A.P.M. How do you tell on inspection when an A.P.M. or an O.F. outlet is working properly ?

What are the respective advantages of an A.P.M. and an O.F. and in what situations would you perfer one to the other ? (*P. I. B. 1937*)

13. Give a dimensioned sketch of a two-way tail cluster to distribute 5 cusecs supply at the tail of a minor. The front water-course is aligned on the same center line as the minor. It is to be given 3 cusecs with a working head of 5 ft. The outlet on the right takes off at right angles and is to be given 2 cusecs with a working head of 1·5 ft. (*P. I. B. 1936*)

14. What governs the size of an outlet *chak* ? What is the optimum size for it ? (*P. I. B. 1941*)

15. What is the difference between a semi-module and a rigid outlet ? What is the main advantage and disadvantage of these types of outlets as compared with those of iron pipe outlets ? (*P. I. B. 1936*)

16. What is the maximum length suitable for a water-course ? (*P. I. B. 1935*)

17. Describe the hydraulic features of the following types of the outlets and state under what condition you would use each type.

(1) A.P.M. and (2) O.F.

18. What are the hydraulic shortcomings in the working of a Crump's Adjustable Proportional Module. How can they be remedied to make it work as a perfect submerged semi-module ? (*P. U. 1943*)

19. (*a*) What measures are adopted by the canal officers to see that the distribution of supplies to the various cultivators are equitable and efficient.

(*b*) Sketch 3 types of outlets current in the Punjab and give the special advantages of each type. (*P.U. 1952*)

20. Design and sketch a tail cluster for a distributary with three outlets having the following data :

	Tail right outlet	Centre outlet	Tail left outlet
Discharge	1·0	2·5	1·5
Working heod	0·5	1·0	0·8

F.S L. =1120·0 in the distributary.

R L. Bed =1118·50 in the distributary. (*P.U. 1954*)

21. (*a*) Why do irrigation channels silt up most in head reaches and suggest remedies.

(*b*) Compare relative merits of three classes of outlets and briefly state which classes of outlets should be adopted in a given case. (*P.U. 1956*)

22. Write short notes on the following :—

Minimum modular head, standing wave through, drawning ration, sensitivity, duty of water, flexibility. (*P.U. Examination*)

23. Distinguish between modules and non-modules. (*P.U. 1957*)

24. Describe with sketches Crump's A. P. Module. At what setting is this module proportional ? What is its main defect and how has it been remedied. (*P.U. 1958*)

PART II
CANAL IRRIGATION
CHAPTER XVI
Irrigation Projects

1. Introduction.

Irrigation Projects usually fall in various categories such as :—

(*i*) Gravity system canals irrigating the tract by flow. The rivers usually have a relatively steeper slope than the adjoining country due to their tortuous length. If the main canal be taken out from a suitable site in a river, it is designed with a relatively flattened slope. The main canal at its tail with full supply above the natural ground surface is split into distributing barnch canals depending upon the configuration of the ground.

(*ii*) Pumping system. In this case the supply is pumped from the sub-soil resevoir or from a natural stream into the distributing channels.

(*iii*) Drainage Scheme. The drains may be surface, seepage-cum-surface drains, seepage drains and underground covered drains to remove storm water or lower the water table.

(*iv*) Storage reservoir schems as described in Part III, Vol. II, of this book.

(*v*) Reclamation Schemes. They are necessary to reclaim the *thur* and water logged lands.

It is not considered necessary to describe all such schemes and projects in this book. In what follows, the gravity system project is described at length. The principal requirements are generally the same ; although the detail and design differ a lot in other cases.

2. Preliminary Investigations.

(*a*) **Reconnaissance Survey.** Before a project is selected for detailed investigation, it is essential that the whole of the country in its neighbourhood should be examined by means of a reconnaissance. The principal objects of this preliminary survey are to be obtained, at comparatively small expense of time and money, general information as to the nature of the tract examined, the facilities offered by it for irrigation, and the relative merits of all project practicable in it. Such a reconnaissnce enables a general plan to be drawn up so as to utilize those facilities in the best and the most comprehensive manner possible so that each individual project proposed will work in, and will not clash with other schemes feasible.

To enable a proper comparison to be made of the relative advantages of competing schemes, approximate general surveys of the works and irrigable areas should be undertaken, and rough plans and estimates of the proposed works should be made. The designs for these should not be considered as final ones, nor the estimates for them as exact, but sufficient care should be taken in their prepation to obviate extensive alterations thereafter, as those may greatly lessen the value of the preliminary work. Care should also be taken to prepare the designs and estimates of competing projects as far as possible on the same general lines, so that a fair comparison as to their costs and advantages may be obtained.

(*b*) **Hydrographic survey.** As soon as it has been ascertained that a favourable scheme is practicable in the area under reconnaissnce, all hydrographic investigations necessary should be started, and they should be continued for as long as possible.

Rainfall data must be studied. High mean annual rainfall does not preclude the necessity of canal irrigation if the rainfall is not available at the sowing time of the principal crops.

(*c*) **Soil-Survey.** Soil survey should be taken to know the alkalinity and acidity of the soil and its suitability for the irrigated crops. Agricultural Department should be consulted about the crops which would be suitable for the tract.

(*d*) **Communications.** The Roads and Railways deparments should be consulted to extend means of communications to cheapen the cost of construction and to develop the rapidity.

(*e*) **Geological survey.** The Geological Department should advise on the general geological conditions, and indicate what special precautions are necessary to ensure the safety and success of the works.

(*f*) **Drainage.** The drainge of lands should be investigated and provided for in the schemes.

3. Detailed Surveys.

After the feasibilty of the project has been established with due consideration of the preliminary investigations, datailed surveys are required to investigate the following aspects :—

(*i*) **The location of headworks.** The detailed river surveys are required. The usual plane-table or Techometers survey methods are not applicable in river beds of large rivers special methods not usually described in survey books are required. The student is advised to study the book written by Rai Bahadur A.N. Khosla "Levelling of Precision Across Rivers 1924 (Disc. Method)" which deals with this subject in detail.

(*ii*) **Alignment of canals and distibutaries.** Contour surveys usually to a scale of 2 inches to a mile are required. These contours may be at 5 feet intervals. They are generally available from the Survey of India Department, and if not, they have to be prepared.

(*iii*) **Alignment of water-courses and Irrigation chaks.** The contoured survey plans showing contours at one foot intervals on a scale of 8 inches to a mile are required. The areas are divided into rectangles and squares and levels every 500 ft. distance observed along long and cross lines. These are not generally avilable with the Survey of India Department and have to be prepared by a special staff of the Irrigation Department.

4. Alignments.

(*a*) Having determind the source of supply and its relation to irrigated lands, the third question is the alignment of the canal. This should be so made that the canal should reach the highest part of the irrigable lands with the least length of line and a minimum expense in constuction. The line of the canal should follow the highest line of the irrigable lands, preferably, skirting the surrounding foot hills, and passing down the summit of the watershed dividing the various streams.

To get at the best alignment, preliminary surveys are necessary. On a large scale contour plan the C.L. (Centre line) of canal, may be marked. The final location may be made on the ground with the aid of a few trials

(*b*) **Obstacles to alignment.** The best method of avoiding the cross drainage works should be considered. Estimates of cost of materials required for construction and subsequent maintenace should be made.

(*c*) **Curvature.** A direct or straight course in the most economical, as it gives the greatest freedom of flow and causes the least erosion of hanks. The insertion of sharp bends inevitably results in the destruction of the canal banks.

MINIMUM CURVES IN IRRIGATION CHANNELS

The minimum radii far curves are given below :—

Capacity of channel	Minimum Radius of the curve
Over 3000 cusecs	5000 ft.
3000 to 1000 ,,	3000 ,,
1000 to 500 ,,	2000 ,,
500 to 100 ,,	1000 ,,
100 to 10 ,,	500 ,,
Less than 10 ,,	300 ,,

As for as possible longer radii should be given.

(*d*) **Bench marks.** Permanent Bench Marks should be left at regular intervals. These should give inscribed reduced levels for cross reference purposes.

5. Slope and Cross Section.

According to Lacey's silt theory, there is only one section of a channel and only one slope at which the canal carrying a given discharge will carry a particular grade of silt (silt factor). The water in the main canal carries silt with a high silt factor. This is gradually reduced in Branch Canals and Minors. The channel section slope is hence fixed by silt analysis. Any excess in the slope of the country has to be destroyed by suitable drops (Falls).

The cross section of a canal may be so designed that the channel may be wholly in excavation, wholly in embankment or partly in excavation and partly in embankment. The typical cross sections are given in chapter VII of part II.

Fall and Drainage Works – L. Section.

As the natural fall of the country through which a canal runs is usually greater than the slope of the canal, falls are provided. The location of these is usually fixed at the site where the canal comes too high above the surface of the ground, while their distance apart is so arranged that they shall not have an excessive height or fall. If a canal can be so located and aligned that it will skirt the slope of the country on a grade contour, it becomes possible to give it the most desirable slope throughout its length without the introduction of falls, but where it runs down the slope of the country, compensation must be made for the difference between the excessive ground slope over that of the canal. If the alignment of the canal cuts the natural drainage lines, either the drainage should be diverted or crossings provided. In no case are the drainage lines to be obstructed.

6. Location af Headworks.

The aim of the canal Engineer is to supply water to the irrigable lands by direct flow from the distributing channels. Hence the headworks have to be so located that this condition is brought about. It may either result in tapping water upstream or raising the pond level of the river by artificial obstruction. A headwork comprises the following :—

(1) Diversion weir or barrage ; (2) Uudersluices ; (3) Head regulator ; (4) Silt Excluder ; (5) Silt Ejector, (6) Escape.

The design of the first these works has already been described in Chapter IV of Part II and items Nos. 4 and 5 have been explained in Chapter XI of Part II.

Canal Escapes. Escapes are intended to escape surplus water from a canal in to the nearest river or some other natural drainages such as river creeks. They serve as safety valves for the canals. Their function is three-fold :—

(*i*) The excesses upto ten percent of the authorized discharge can generally be utilized without endangering the canal or distributary banks. The abnormal excesses which sometimes enter a canal due to sudden changes in river supply levels in freshets have to be escaped at some suitable site below the Head Regulator of the canal.

(*ii*) Escapes are also necessary at tail of Branch canals and upstream of high embankments in a distributary to escape the excesses in such channels due to sudden rainfall.

(*iii*) Escapes are very desirable at convenient places to permit reduction of discharge in the distributaries and Branch Canals to close normal breaches and intentional cuts by the cultivators for unauthorized irrigation.

The effect of reduction from the canal head takes usually a long time. It is expedient to locate reasonable sized escapes at every 50 miles in a long canal.

The capacity of the escapes depends upon the effectiveness of the factors contributing excesses in the channels. As the escapes also serve as safety valves to expediate repairs to the damage done to canal banks by cuts, it is the rough rule to fix the capacity of an escape equal to about one half the discharge of the channels from which the capacity of an escape takes off.

The head regulator design of an escape is just similar to the design of a canal head regulator described in Chapter IV Part II. Usually the escape head regulators have, low crest levels with undershot regulation so that the escape head may also work as scouring sluices when water can be spared for escapage.

The channel section design of an escape is just similar to the design of surface drains as described in Chapter II Part IV. The slope should be as steep as the levels can permit.

The design of other masonry works such as bridges and falls if unavoidable, is similar to one used in the case of canals.

7. Distributries and Minors.

No direct outlets should be given from the Main Canal and Branches, as the regulation of supplies is very difficult. All outlets should take off from Distributaries and Minors.

The water is drawn at proper intervals from the main line into moderate sized branches which are so arranged as to command the greatest area of land to supply the laterals (distributaries and water courses) in the most direct manner.

Location of Distributries. Distribution from a distributing canal is most economically effected when it run along a ridge, so that it can be supply water to its branches and to channels on either side. In the case of main canals this location can be made only in occasional instances, but the distributaries taken from these mains should be made to conform to the dividing lines between water courses. The capacity of the distributries, which traverse the separate drainage divides, are proportional to the duties they have to perform, (the natural bounding streams limiting the area they have to irrigate).

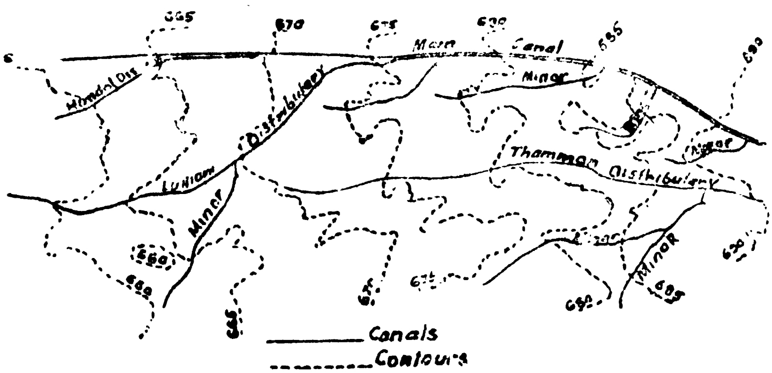


Fig. 1

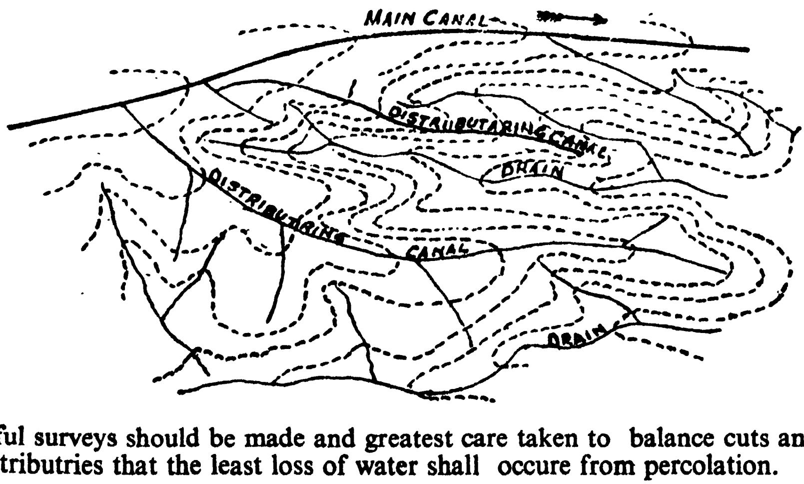
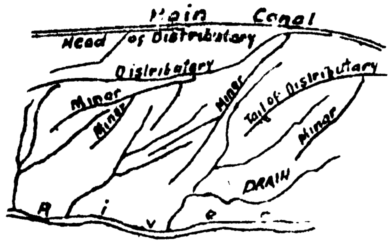


Fig. 2

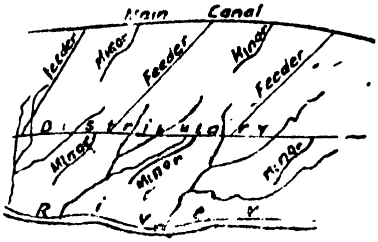
Careful surveys should be made and greatest care taken to balance cuts and fills and to so locate the distributries that the least loss of water shall occure from percolation.

The figures 1, 2 and 3 show an ideal distributary system. Such an arrangement enables the least mileage of channel to command the greatest area of country by furnishing water to both sides of its line. At the same time perfect drainage is obtained by the water flowing in both directions into the natural drains. Fig. 4 shows the faulty alignment of distributaries and drainage channels.



Good

Fig. 3



Bad

Fig. 4

Design of distributaries. The capacity is proportional to the duty performed and water surface kept at level where flow irrigation is possible. The cross sectional area is diminished with steady draw-off from the outlets. The capacity of the different parts of the system must be based on the full supply capacity.

In order to avoid high banks and to ensure the surface of water being above that of the country, the slope of the disty : should be made as nearly parallel as possible to that of the land it traverses. Hence aligment falls must be introduced.

Dimensions of distributries. The greater the amount of water discharged by a distributary, the smaller will be the proportional cost of maintenance. Hence large sized distributaries are better then small ones. The minimum bed width usually kept is 3·0 ft.

8. Intensity of Irrigation and Water Allowance.

The relation of water supply to the land depends on the rainfall and the quality of land, as the depth of each watering is fixed by the composition of soil. The soil survey should give both the chemical composition and the feasible texture. The soil must contain chemical plant food and humous (decayed organic material). It should be free from Alkaline salt. The periodicity of watering is fixed by the climate, the crop period and the number of waterings required for the maturity of a crop. The lossess in transport have also to be considered.

Generally speaking the intensity of irrigation in the Punjab may be kept about 50% in the case of proprietary villages, where means of irrigation such as wells exist. In the case of Crown Waste land of the colony areas, the intensity of irrigation from 75 to 80% will do giving about 20% land for rest during the year. Near the cities where the vegetable crops are raised, the intensity of irrigation should be cent percent.

Generally speaking a water allowance of about 3 cusecs per thousand of the C. C. Area is enough in the Punjab and in the case of the sugarcane, rice and garden area, about double the quantity will do.

9. Capacity of Channels.

The record of areas taken from the settlement records of the villages to be irrigated by a distributary is used to work out the capacity of the channel. The area is abstracted in a statement called the *chak-bandi*, form given on page 348 in form A. Usually *chahi*, *salab*, *abi* and *banjar shamlat* including *ghair mumkin* areas are not allotted any water.

The uncommanded area is excluded. The remaining area is called C. C. Area to which water allowance is applied and *wari* is allotted.

The command statement is worked out as shown in Form B to work out the levels required downstream of the outlet to command the *chak* served be it. It is a bad practice to include the uncommanded area in the C. C. Area, because it results in temptation to the cultivators to head up supply in the parent channel. This upsets the equitable distribution of supply and results in silting of the channel. Then the Capacity Statement of the distributaries and minors is worked out as per statement form C attached. The columns of the Capacity Statement are self-explanatory and need no comments as detailed information in designing the sections has already been given in Chapter V of this part.

FORM A

Chakbandi Missal by outlets with

Zilladari Disty. Side

FORM A

Detail of Fields for village

R.D. of outlet Tehsil District

Serial No.	Name of *Patti*	Name of *Khata*	Name of Owner	Settlement Field No.	Detail of area as per settlement record										Area not to be given *wari* Col : 10 to Col : 14	Area to be given *wari* Col : 5 to Col : 16	*Wari*	Remarks
					Nehri	*Chahi Nehri*	*Barani*	*Banjar Malkiat*	*Banjar Shamlat*	*Chahi*	*Sailaba*	*Abi*	*Ghair Mumkin*	Total				
						Commanded / Un-commanded	Commanded / Un-commanded	Commanded / Un-commanded	Commanded / Un-commanded	Commanded / Un-commanded	Commanded / Un-commanded	Commanded / Un-commanded				(C.C.A.)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

COMPOUND STATEMENT

FORM B

Canal Disty

Serial No.	R.D. of outlet	Right or left.	Critical R. L. of ground to be irrigated.	Command at field	Fall in W.C. at 1/5000	Level required at head of W. C.	R.L. of F.S. required in Disty, Col. 7 plus M. M. H.	R.L. of F.S. proposed in Disty.	Working head proposed	Working head existing.
1	2	3	4	5	6	7	8	9	10	11

CAPACITY STATEMENT

FORM C.

Canal Disty : Minor

Serial No.	R.D. and Side of outlet	Gross area	C. C. area	Area for which water is to be given	Intensity per-cent	A. P. area	F. S. F.	Discharge of outlet	Reach	Discharge by reaches	Absorption by reaches	Total discharge at each reach	F_l or C.V.R	Slope	Bed width	F.S. depth
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

ALTERATION FORM FOR OUTLETS.

FORM D

Distributary Branch Division
Minor

Date of last alteration
Sanctioned with S. E.'s No.

Intensity
Dated

F.S. Factor
Sanctioned S.E.'s No.

Dated

Serial No.	R.D. and Side of outlet.	Village.	C.C.A.	Annual permissible	Average Irrigation last 5 years.	30 percent of column No. 6	Accepted Area.	Discharge Authorised / Existing	F.S. Disch. of Disty.	W.P. of Disty	Section of Disty. b	Section of Disty. d	F.S.L. of Disty. Designed / Existing	F.S.L. in W C.	Working Head	Size of outlet: Type	B.	Y.	H.	R.L. of Crest	h.m	Description and R. L. of referring B.M.	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

EXISITNG

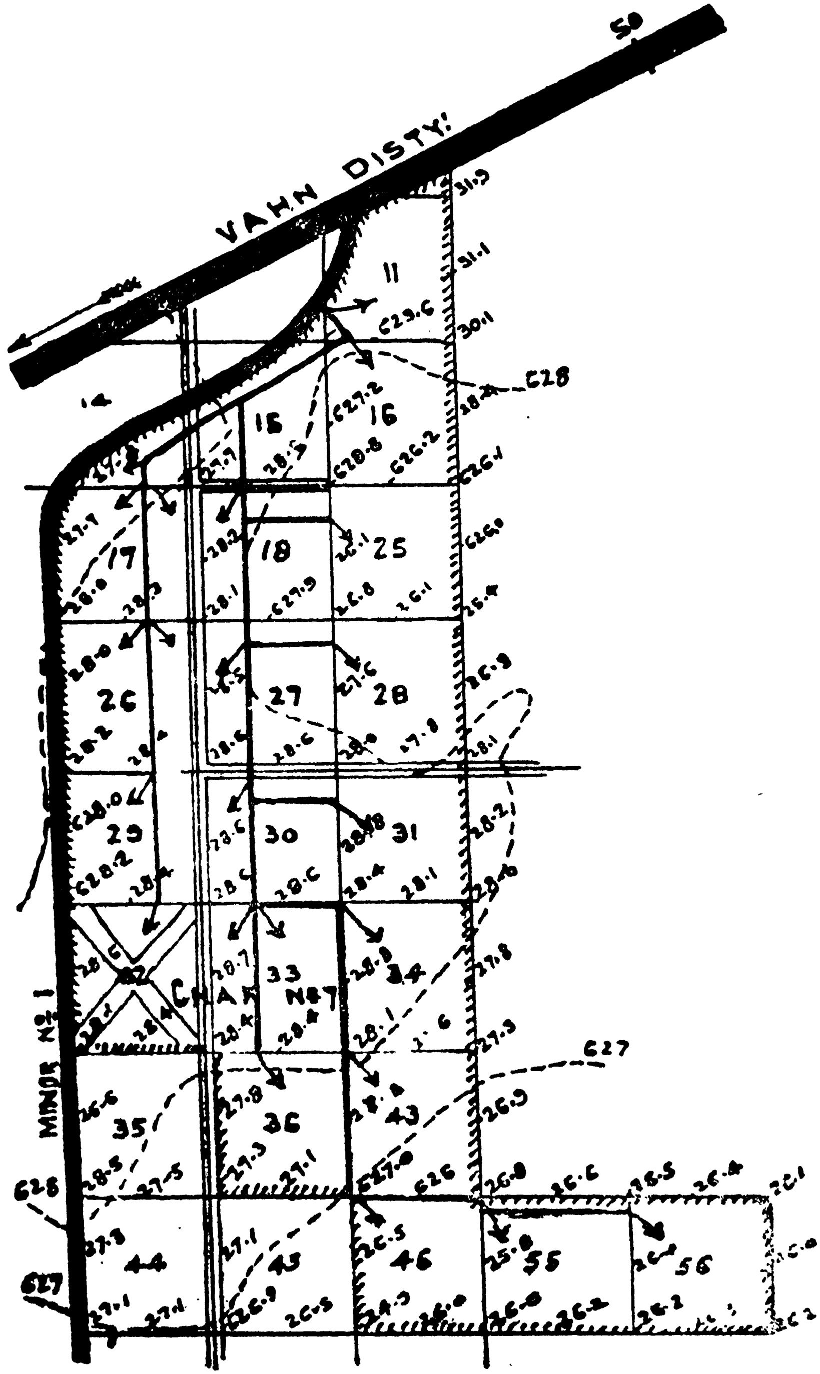
PROPOSED

Form 'D'

Reasons for Alterations	Serial No.	Certificate	Dated Signature	Date of Receipt of form	No. and date of letter with which form despatched
	1	Certified that the changes as per "proposed" data overleaf have been executed at site under my personal supervision on .... and that my outlet note-book has been amended accordingly.	Overseer ......Section		
	2	Certified that the changes have been correctly carried out at site, and have been checked by me on.................. My outlet note-book, irrigation register and the *chak* plan have been corrected accordingly.	*Zilladar* ......Section		
	3	Certified that the alterations have been carried out as sanctioned by Superintending Engineer, and that the sub-divisional irrigation register, outlet register, and *chak* plans have been corrected accordingly. The third copy of the alteration form, duly signed by Superintending Engineer, has been pasted by me into the skeletion album.	Sub-Divional Officer Sub-Division		
	4	Certified that the divisional office record, *chak* plans have been corrected as per "proposed" data overleaf.	Head Draftsman ......Division		
	5	Certified that the *chakbandi* register, *chakbandi missal*, plan, and *shajras*, have been corrected, and that the fifth copy of the outlet alteration from has been filed with the *chakbandi missal*. The Deputy Collector's signature has been obtained where necessary.	Ahlmad ..... Division		
	6	Certified that the divisonal irrigation and outlet registers have been corrected as per "proposed" data overleaf. The fifth copy of the outlet alteration form has been checked and corrected where necessary, certified and despatched to the Ahlmad through the Deputy Collector. The Ahlmad's certificate regarding correction of *chakbandi* records has been obtained on the fourth copy of the outlet alteration form. Both the second and fourth copies of the outlet alteration form have been pasted into the skeletion album adjacent to one another. The head draftsman has signed his cretificate on the fourth copy of the outlet alteration form. Any differences have been brought to the notice of the Executive Engineer whose orders are noted on the fourth copy.	Revenue clerk ......Division		
	7	Orders of the Exective Engineer.			

10. Watercourses and Outlets.

The watercourses are designed on contour plans, scale 8 inches to a mile, showing contours at one foot intervals. The watercourse usually follows the ridge to avoide embankments. The length of watercourse is generally limited to 2 miles. The watercourse is usually branched



to reduce the length and to gain command, lowering there by the highest levels required down-stream of outlets when working as calculated in form (B) given in para 9 above. The usual watercourse slope is 1 in 5000 and the command at the field 6 inches. The working head of the outlet is kept not less than 1·0 foot. A typical layout plan of a watercourse is given in Fig. 5.

The design of the outlet is worked in a statement called the Outlet Statement. A copy of the usual form used is given in form D. The columns are self-explanatory and need no elucidation. The sub-proportional outlets are used in the head reaches of a distributary, such as A.P.M. set below ·6 D down to 1·2 D. Proportional outlets are desirable in the middle reaches set at ·6D up to ·8 D because the setting of the A.P.M. to give due share of supply is ·8D. In the tail reaches of a channel where the command is poor, open flume outlets are desirable, as they need comparatively low working heads and are sensitive to absorb the excess reaching the tail in the case of slack demand. Theory of outlet design has been described in detail in Chapter XV.

11. Rules Governing the Submisson of Estimates for and the Construction of Irrigation Works.

I. General.

1. The following rules define the procedure to be adopted in connection with irrigation projects. Throughout the rules the term "Irrigation projects" or "Irrigation work" should be held to include the Navigation , Embankment Drainage, water-storage proiects or works.

The expenditure on works only should be the criterion for determining the authority, whose sanction is necessary to an estimate (original or revised).

II. Classification of works.

2. Irrigation works for which capital accounts are kept are classified as either "Productive" or "unproductive". The criteria to be adopted in deciding question of classification are given in paragraph 12 of this Chapter.

III. Rules regarding the preparation of irrigation projects.

3. The papers to be submitted with the project for an irrigation work consist of a report, such plans, measurement, quantities, rates and specifications as may be necessary to enable the suitability of the designs and the adequancy of the estimated cost of the project to be adjudicated upon, and an abstract classified under the heads prescribed in rule II, bringing the various component items together. These documents together form what is called the "Estimate".

4. Every project for an irrigation work should contain a full report explanatory of the project. It is impossible to lay down headings which will be suitable in all cases, but the following points should, in general, be dealt with.

5. The report should give a general description of the proposed works, including the sources from which the supply of water is to be drawn, the maximum floods likely to be experienced, the quantity of water available at different periods of the year and the quantity proposed to be utilised. Reference should also be made to any rights in the water already existing. The reasons for the adoption of the particular scheme recommended in preference to any other should be explained, if neeessary. The rainfall and depth of spring level in the tract affected, the sources of existing irrigation, and the means of drainage proposed, if such are required, should be dealt with. The lengths of the main channels and distributaries should be given. These lengths being compared with those of channels of similar capacity actually constructed elsewhere, and a full account of the construction governing the designs of all important works, appended, with a careful analysis of any engineering questions involved. The cost of additional communications (other than the cost of construction communications) and other extraneous works involed in the project should, be regarded as part of its cost, but the report should include an estimate of expenses under this head so far as data for such an estimate are available.

6. Figures should be given showing the area of land commanded, the average area usually cultivated and the area probably irrigable. The opinion of the local revenue officers as to the desirability or necessity of the projected work the fairness of the water rates proposed and the probabitity of the anticipated financial results being realized should invariably be stated. The returns expected from the works, with a note as to the basis on which they are calculated, should be exhibited in form No. III. appended.

7. The report should also deal with the question of labour and the sources, whence it is obtainable, the paobable effects of the operations on the existing rates, the localities whence materials can be obtained and the facilities for manufacture. The manner in which the rates adopted for estimate purposes have been arrived at, should also be commented upon.

8. The method proposed for carrying out the work should be described and in particular such matters as the extent to which it is intended to use mechanical appliances, to employ manual labour to give out work to large manual labour contractors or to resort to the agency of departmental labour or petty contractors should be dealt with. The No. of circles and division into which the work is to be divided and the time likely to be occupied on construction should also be stated. Any permanent increase in the cadres of Engineers which will be necassary to provide for eventual maintenance of the works should be calculated and explained, and sanction should be applied for any posts, temporary and permanent, which the authorities subordinate to the Provincial Government are not themselves competent to create.

9. Special attention should be given to the following matters, which have been the cause of excessess in the past :—

(*a*) The preliminary operations, including the surveys, both of alignments and soil, with trial borings, where necessary, should be completed, under the orders of competent authorities. It should be clearly stated, in each case, to what extent this has been done and what still remains.

The cost of such operations should in the first instance be charged to the Major Head "18—Other Revenue Expenditure financed from Ordinary Revenues (2) Misc : Expenditure."

(*b*) In addition to the usual charge of 5 percent for unforeseen contingencies which percentage may, for reasons recorded be increased if circumstances render it desirable, all incidental expenditure which can be foreseen, such as sheds for workmen and stores, etc., should be seperately provided for in the estimate. The utilization of "over all" rates, intended to include, such expenditure, is to be deprecated.

(*c*) The estimate of the cost of acquisition of land should receive special attention, and a valuation should be obtained in every case from the District Officer concerned.

(*d*) The District Officer should also be consulted as to the number of bridges which will be required, and care should be taken that an adequate number of escapes is provided.

(*e*) Special attention should be given to the probable cost of foundations, the measures to be adopted in connection with them, and the amount of pumping likely to be necessary.

(*f*) The allowance, if any for special tools and plant should be carefully considered. The inclusion of a lumpsum for the purpose is generally to be deprecated and, if this method is adopted, the provision allowed should be tested with reference to the probable cost of the actual plant required. The amount of credit anticipated under this head and the source whence it will be obtained should be fully explained.

(*g*) Where the estimates of several structures of the same kind are derived from a single type design, the most unfavourable conditions likely to occur should not be overlooked.

(*h*) Estimates framed on the basis of analogies from existing works are usually unreliable, and, before this method is adopted, the correctness or otherwise of the analogy should be very carefully tested for selected portions of the work. In particular, analogies drawn from small works should never be relied upon for the preparation of larger projects.

(*i*) All calculations of quantities and costs should be independently checked to obviate clerical errors.

(*j*) When the circumstances of a project are such that there is reason to suspect that expenditure, other than that which can be definitely foreseen at the time of framing the estimate may become necessary during the course of construction provision to meet it, under the head "unforeseen expenditure" should be made in the estimate. When this is done, the circumstances should be fully explained in the Report.

(*k*) In estimating the revenue likely to be obtained from a project, consideration should be given to the fact that, in some cases, the full discharge or storage will not invariably be available and that, in others even if available, it may not be required. The average discharge or storage likely to be utilized, arrived at by analogy from similar works elsewhere, should be the basis of the revenue estimates.

(*l*) In calulating what price will be secured for Government land, the sale proceeds of which form part of the estimated revenue from the project, regard should be paid to the probable relation of supply and demand, specially when a project or different projects, are likely to bring land into the market simultaneously in the Punjab or any other province or an Indian state.

10. The expenditure proposed to be incurred upon a project should in every case be restricted to the minimum amount necessary to ensure the success of the undertaking. Estimates should be framed keeping in view the rule that every public officer should exercise the same vigilance in respect of expenditure incurred from Government revenue as a person of ordinary prudence would exercise in respect of the expenditure of his own money.

11. The complete estimate for a project besides including all anticipated direct charges should further include as indirect charges the amount required to cover the capitalization of abatement of land revenue on the area occupied by the works, cost of audit and accounts establishment and simple interest on Capital Outlay, prior to work being brought into oreration. The direct charges should be classified under the appropriate sub-heads of Account, the main heading being (1) Works, (2) Establishment, (3) Tools and Plant and (4) Leave and Pensionary charges. The items included under the head "Works" should be classified under the prescribed head "Service" and 'Detailed heads' enumerated in Form II. The cost of surveys, including expenditure incurred prior to the submission of the project, should also be included in the estimate. An abstract framed on these lines, in the Form No. II appenned, must accompany every project submitted for sanction. In the case of a large project the sub-works may, if desired, be further sub-divided so as to show individual large works.

12. If it is known that the project will constitute the sole charge of one or more Superintending Engineers, the estimate of the cost of Suprintending Engineers and other establishment should be framed in detail, a suitable amount being added to cover an adequate share of the cost of the Chief Engineer's establishment. Even where a Suprintending Engineer's charge is not in question, the cost of the establishment required for the supervision of the construction of the project should ordinarily be estimated in detail, but Government may at their discretion, calculate the provision to be made for establishment in such a case on a percentage basis, provided that the percentages are justified by comparison with past actuals. In the case of ordinary Tools and Plant, either the actual anticipated cost, or a reasonable peacentage allownce, based on past actuals may be entered in the estimate.

Unless Government directs otherwise the provision to be made for establishment should include 25 percent on the estimate of Works Outlay for departmental establishment and 1 percent for Audit and Accounts, while the provision for Tools and Plant will be 1½ percent on the Works Outlay. Estimates for large surveys for new irrigation projects should, however, provide only for a charge of 5 percent on the cost of special establishment to cover the supervision charges thereon. In the case of irrigation projects, for which neither Capital nor Revenue Accounts are kept, it is unnecessary except in the case of large surveys for new irrigation projects, to enter provision for Establishment and Tools and Plant in the estimate unless, for any reason, it may be deemed desirable to do so in order to forecast the ultimate result of the project.

13. Charges for capitalization of abatement of land revenue should be calculated at twenty times the amount of land revenue remitted, while provision for Leave and Pension allowances should be at the rate of the 21 percent on the gross charges for Establishment.

14. Simple interest on Capital will be calculated at the rate sanctioned by Govrnment from time to time, on the probable annual outlay.

15. No provision should ordinarily be made for the minor head "Suspense" as this head in the accounts represents services of a general character not necessarily pertaining to a particular project. If however, one or more Divisions are expected to be maintained exclusively in connection with stock required for the project, provision for "suspense" may be included, but only to the extent of the balances likely to be outstanding under Suspense on the date of closing the construction.

IV. Storange projects.

16. The Report should, in addition to the information specified in rules 3—15, given the

area of the tank and contents when full, the area to be irrigated per unit ofstroage, the length of the dam, its maximum hight and shape, materials proposed to be used in its construction and the mode in which the water is to be let off for irrigation. The questions, of the available water supply number of times the reservoir will probably fill, rainfall and proportional flowing off the catchment, loss by evaporation and absorption etc.,should be fully dealt with as well as the quantity of floodwater for which provision must be made, the flood absorption capacity of the reservoir, and the waterway of the escape weirs or sluices. The results of any experiments bearing upon the strength of the materials proposed for use in the dam should be dealt with, as also the silt content of the water and the probable effective life of the reservoir.

V. Projects affecting any other province or an Indian State.

17. Where any other province or an India State is also concerned, the report should detail the arrangements mutually agreed upon for financing the works, the terms upon which the water is to be shared, the agency by which the works will be constructed and where an Indian State is concerned, the agency by whom the accounts are to be audited on behalf of the State. When a project or different projects are likely to bring land in the market simultaneously in different provinces or Indian States, and the sale-proceeds of such land form part of the estimated revenue from the project, the report should state what arrangement the Governments concerned have made to meet the contingency. A draft of any formal agreement into which it is proposed to enter to regulate these and any other matters in respect of which agreement is deemed is necessary should accompany the project.

18. In all projects which may affect riparian or other interests in Indian States Government will ascertain the views of the *Durbar* or *Durbars* through the political authorities concerned.

VI Embankments.

19. In the case of new lines of river embankments, the report should show clearly the financial responsibilities of Government in connection therewith, and the manner in which it is proposed that the outlay shall be recovered.

VII Nature of sanction.

20. The sanction accorded by the Provincial Government to a project for an irrigation work shall be regarded as in the nature of an administrative approval to the project, and not as the final technical sanction to the detailed estimates of the works. Such technical sanction will be accorded by those officers of the Public Works Department, Irrigation Branch, to whom powers have been delegated by Government. Detailed working estimates for individual works in excess of Rs. 1 lakh must also further be approved by Government.

VIII Cemmencement of works.

21. It is a fundamental rule that no work, to which these rules apply, shall be commenced upon an Irrigation project, until the following conditions have been fulfilled : —

(*a*) The approval of Government to the project has been obtained.

(*b*) There is a sanctioned design and detailed estimate for the portion to be commenced.

(*c*) Funds have been allotted for the work.

When these conditions have been fulfilled, Public Works Department, Irrigation Branch, are competent to authorise the commencement of construction.

22. Government in the Finance Department, in consultation with their Audit Officers may prescribe rules to regulate expenditure debitable to a sanctioned project upon such survey and preliminary operations as may be necessary in order to enable the detailad estimates to be drawn up.

IX. Rules governing the accord of technical sanction.

23. When a project has been sanctioned by Government, an officer of the Public works Department. Irrigation Branch to whom power has been delegated by Government, may sanction detailed, estimates for component parts of the project against the amount provided for the 'Sevice' or "Detailed" head in the abstract estimate (Form No. II).

24. (*a*) Detailed estimates, subsidiary to a project estimate may be for a single work such as a bridge, a certain number of miles of excavation etc., or for a distributary in which a number of small works are iucluded. In the first case, the cost of the work should be specified

by detailed heads in the abstract, while in the second case the abstract should be prepared to show the component items and liabilities separately in detail under the various service heads of classification. But if a work of exceptional magnitude costing more than Rs. 10,000 becomes necessary, a separate estimate should be prepared for it.

(*b*) A separate estimate should be prepared for each distributary or, where there are minor channels, two estimates one for the main distributary and its important branches, and one for the minor channels, but the whole expenditure under "A-- Preliminary Expenses" should be provided for in the estimate for the main distributary.

(*c*) General charges for establishment, audit and account, and tools and plant should be entered in a project estimate, but will not appear in any estimate subsidiary to it, but after a project estimate has been closed, estimates chargeable to the open capital account should contain provision for these charges except 1 percent on account of audit and accounts establishment, at the rate given in rule 12.

25. When it becomes evident that the amount provided for a "Sub-works" or a "detailed" head in the project estimate will be exceeded the following rules must be observed.

(*a*) Subject to any orders which Government may pass in the matter, the officer-in-charge of the project may transfer provision to meet such exess from another detailed head of the same sub-work on which a saving is anticipated.

(*b*) Should it become evident that an excess over the amount provided in the abstract estimate for any sub-work willl be exceeded, the officer-in-charge of the project must report the fact immediately to the Finance Department. He should, at the same time, intimate what savings, if any, are anticipated upon other sub-works of the project.

(*c*) The Finance Department may transfer provision from one sub-work, on which a saving is anticipated, to meet a probable excess on other, or it may permit an excess over the provision made in the abstract estimate for any sub-work upto an amount to be stated by it.

(*Note.*—The sub-works into which the minor head "Works" is divided are enumerated in Form No. II appended to these Rules.)

X. Modification after accord of sanctions.

26. After the Government's approval of the Project for an irrigation work, the Chief Engineer may, if necessary, and subject to the provisions of rule 25, modify the details of the works; provided that if any such modification is in the opinion of the Chief Engineer substantial, a report of such modification should be made to Government.

Note.—Modification will include abandonment of items included in the original estimate or provision of items not included therein, and an increase or reduction in the area to be irrigated by the Project.

XI. Reports of probable excesses.

27. Whenever it is ascertained that the expenditure upon any project is likely to exceed the amount sanctioned by the Provincial Government by an amount higher than that which the Public Works Department, Irrigation Branch are empowered to pass, Finance Department should be immediately advised of the anticipated exess without waiting for a revised estimate. The revised estimate, if necessary should be prepared in due course, and submitted to Govrnment with a full explanation of the causes and of the probable effect on the financial results of the work. The Finance Department should also be immediatly informed if at any time during the course of construction, it becomes probable that a work, sanctioned as a productive will fail, in operation, to satisfy the criteria which must be satisfied before a work can be regarded as productive.

XII. Supplementary and revised estimates.

28. Any development of project thought necessary while the work is in progress which is not fairly contingent on the proper execution of the work at first sanctioned, must be covered by a supplementary project estimate, accompanied by a full report of the circumstances which render it necessary. The abstract must show the amount of the original estimates and the total of the sanction required including the supplementary amount.

29. A revised estimate must be submitted when a original sanctioned estimate is likely to be exceeded by more than 5 percent and a second revised estimate when any excess is anticipated over a sanctioned revised estimate. The revised estimate should be accompained by a comparative statement in Form No. III appended, comparing the revised estimate with the latest existing sanction of competent authority, and by a report showing the progress made to date.

30. Whan a revised or supplementary estimate is submitted under Rule 28 or 29, it should be accompanied by revised financial forecast statements as required in the case of an original estimate (vide Rule 6).

XIII. Utilization of completion report as revised estimate.

31. When excesses occur at such an advanced period in the construction of a work, as to render the submission of a revised estimate purposeless, the excesses may with the concurrence of the Finance Department be explained in a completion report prepared as prescribed in rule 33. The adoption of this procedure in this way dispenses with the necessity for the immediate report of the excess required under Rule 27.

XIV. Completion reports.

32. The estimate for construction of an irrigation work should be closed as soon as the project is practically in full operation, although there may be certain works provided for in the estimate, either unfinished or which it is not desirable to construct at once.

33. As soon as the construction estimate has been closed, the Public Works Department, Irrigation Branch will prepare, for submission to Government through the Finance Department, a completion report of the project comprising the following documents in Form No. IV appended. Completion report in this form should be prepared only in respect of projects sanctioned by Government. For smaller projects, and other open capital works which are sanctioned by the Public Works Department, Irrigation Branch, completion reports should be prepared in one of the forms prescribed.

Schedule A. A statement showing by works and sub-works, classified under the relavent detailed heads, the actual expenditurs on works completed upto the date of the closing of the construction estimate.

Schedule B. A statement of works which are within the scope of the sanctioned estimate, and of which detailed estimates have been prepared and sanctioned by competent outhority, which were incomplete or had not been begun on the date of the closing of the construction estimate.

Schedule C. A statement of works, whether included in the construction estimate or not, which have been sanctioned by competent authority, between the date of closing the construction estimate and the date of the submission of the completion report.

Schedule D. A statement of works for which no estimates have been sanctioned upto the date of the submission of the completion report but the probable expenditure on which can be foreseen and which are necessary to complete the project.

Schedule E. A statement compiled as a combination of statements A, B, C and D. This statement should also show, for purposes of comparison, the sanctioned estimate by works and sub-works, classified under the relevant detailed heads of account.

A report on the works executed upto the time of the closing of the construction estimate and an index map or maps showing the Project as completed, will accompany these documents. The report will discuss the financial result already attained and expected in the future and will be accompanied by forecast financial statements in Form No. I, based on Schedule E, above *i.e.*, on the total anticipated ultimate expenditure on the Project. Part III of this form will be signed by the Chief Revenue Authority of the province.

34. The schedule A to E accompanying completion reports should intially be signed by the Officer-in-charge of the Project (who is particularly responsible for figures in columns 5 – 9 of Schedule D, and consequency column 10 of Schedule E.) and counter-signed as "varified" by the Accountant General in token of his actuals and classification.

35. The financial statement submitted with completion reports should similarly be signed and countersigned, but in this case the Audit Officer should do so under the words "Actuals and calculations checked".

36. These documents should ordinarily be prepared and submitted to Government within

6 months of the closing of the construction estimate, or 12 months in the case of an exceptionally large work. If this is not found possible within the period specified, the Finance Department should be advised of the reasons for delay and the probable date when the documents may be expected to be ready.

37. Schedule E will be treated as a revised forecast of expenditure against the sanctioned Project. All important works which had not been commenced and which were within the scope of the sanctioned estimate should be included in schedule B, C or D as the case may be.

38. Subject to the restriction that the total expenditure against the project shall not exceed the amount sanctioned for the Project by an amount greater than that which the Departments are empowered to pass, the Public Works Department, Irrigation Branch is competent to pass expenditure between the date of closing the construction estimate, and that of the approval of the completion report by competent authority on—(*a*) Works entered in Schedules B and C. (*b*) Works entered in Schedule D, within the limits and subject to the conditions specefied.

39. On receipt of approval of the Government to the completion report, works included in Schedules B and C, may be carried to completion by the Public Works Department, Irrigation Branch, within their powers of sanctioning excess over estimated amounts, approval of Government being obtained to any higher excess. Public Works Department, Irrigation Branch may also on receipt of such approval, sanction further outlay on other works against the open capital account of the Project subject to the conditions usually laid down.

FINANCIAL STATEMENTS.

FORM NO. 1, PART I.

Summary of the estimated direct charges to capital account.

Years.	Works.	Establishment including leave salary and pension charges.	Tools and Plant.	Suspense	Total.	Less receipts on capital account.	Net total.
1	2	3	4	5	6	7	8

Form No. I, PART II

Summary of the estimated indirect charges to Capital.

Year.	Capitalized abatement of land revenue.	Charges on account of audit and accounts establishment.	Total.
1	2	3	4

FORM NO. I, PART III.

Estimate of growth of Irrigation and revenue receipts and charges.

		Revenue receipts and charges.						
		Gross revenue due to work.				Net revenue due to work.		
Year.	Irrigated area at end of year.	Direct receipts.	Enhanced land revenue or indirect revenue.	Total.	Charges both direct and indirect against revenue account.	Including enhanced land revenue.	Excluding enhanced land revenue.	Remarks.
1	2	3	4	5	6	7	8	9

FORM NO. I, PART IV.

Estimate of net financial results of years after the probable date of completion of the work.

Year.	Direct capital outlay during the year.	Direct capital outlay to end of year.	Simple interest at % on capital outlay to end of previous year plus half outlay during the year.	Net revenue including enhanced land revenue column 7 of Part III.	Simple interest less net revenue.	Net revenue less simple interest.
1	2	3	4	5	6	7

Form II. Abstract Estimate of cost .... Project.

Minor head.	Sub-work.	Detailed head.	Amount.
Works.	Direct charges		
	1. Headworks.	A. Preliminary expenses.	
		B. Land.	
		C. Works.	
		K. Buildings.	
		O. Miscellaneous.	
		P. Maintenance.	
		Contingencies.	
	2. Main Canal.	A. Preliminary expenses.	
		B. Land.	
		D. Regulators.	
		E. Falls.	
		F. River and Hill torrent works.	
		F. (1) Other cross drainge works.	
		G. Bridges.	
		H. Escapes.	
		I. Navigation.	
		J. Mills.	
		K. Buildings.	
		L. Earthwork.	
		M. Plantations.	
		N. Tanks and rivers.	
		O. Miscellaneous.	
		P. Maintenance.	
		Contingencies.	
	2. (*a*) Branch No. 1	As for main canal.	
	2. (*b*) Branch No. 2	do	
	3. (*a*) Distys : group No. 1		
		A. Preliminary expenses.	
		B. Land.	
		C. Works.	
		K. Buildings.	
		O. Miscellaneous.	
		P. Maintenance.	
		Contingencies.	
	3. (*b*) Distys ; group No. 2	As for group No. 1.	
	3. (*c*) Distys ; group No. 3	do	
	4. Drainge and protective works.		
	5. Water-courses.		
	6. Special Tools and plant.		
	Unforeseen expenditure.		
Establishment including leave salary and pension charges.			
T. and P.			
Suspense.			
Deduct receipts on Capital Acconnt			
Total Direct charges.	Indirect charges.		
Capitalized abatement of land revenue.			
Audit and Accounts.			
Total indirect charges			
Grand total.			

FORM III. REVISED ESTIMATE.

Comparison between original and revised estimate.

Minor head	Sub-work	Detailed head	Original estimate	Modifications Sanctioned by competent authority	Total Sanctioned estimate	Revised estimate	Savings	Excess	Remarks

FORM IV. COMPLETION REPORT.

Schedule A.

Schedule of works showing actual expenditure on works completed upto the date of the closure of the construction estimate.

Serial No. of items	Classification			Name of work	Cost of work
	Minor head	Sub-work	Detailed head		

FORM IV. COMPLETION REPORT.

Schedule B.

Schedule of works of which detailed estimates had been sanctioned prior to the date of the closure of the construction estimate but which were in complete or had not been begun on that date.

Serial No. of items.	Classification			Name of work	Outlay to date of the closure of the construction estimate.	Probable further outlay.	Probable further total outlay.	Probable date of completion.
	Minor head.	Sub-work.	Detailed head					

FORM IV. COMPLTION REPORT.

Schedule C.

Estimates sanctioned against the Open Capital account subsiquent to the date of the closure of the construction estimate.

Serial No.	Minor head.	Sub-work.	Detailed head.	Name of work.	Amount Sanctioned				Date on which sanction lapses.
					Works.	Establishment.	Tools and plant.	Total.	

FORM IV. COMPLETION REPORT.

Schedule D.

Schedule of works of which no detailed estimates have yet been sanctioned but the construction of which is necessary for the completion of the Project.

Serial No. of item.	Classification.				Probable cost of work.			
	Minor head.	Sub-work.	Detailed head.	Name of work.	Works.	Establishment.	Tools and plant.	Total.

FORM IV. COMPLETION REPORT.
Schedule E.
Comparison of expenditure by main and sub-heads with the provision in the estimate sanctioned by Government.

Classification.			Provision in estimate sanctioned by the government.	Expenditure up to the date of closing the construction estimate.			Probable further expenditure				Difference between probable total outlay and provision sanctioned		
Minor head.	Sub-work.	Detailed head.		On completed works schedule A.	On uncompleted works schedule B.	Total.	On works uncompleted or not commenced schedule B.	Sanctioued schedule C.	Unsanctioned schedule D.	Total.	Probable total outlay.	Excess.	Saving.
1	2	3	4	5	6	7	8	9	10	11	12	13	14

12. Classification of Irrigation, Navigation, Embankment and Drainage Works into "Productive" and "Unproductive".

1. Projects of Irrigation, Navigation, Embankment and Drainage, are of two classes :— (1) Productive and (2) Unproductive.

2. To admit of a new work being classed as Productive Public Work, the following conditions must be satisfied :—

(*a*) There must be good reason to believe that the revenue derived from it, will within ten years after the probable date of its completion, repay the annual interest on the capital invested calculated at 6 percent, but in preparing a project for sanction no deduction is to be made from the total capital outlay on account of anticipated excess of revenue over simple interest.

Note :—Capital invested includes (1) direct charges, (2) indirect charges and (3) all arrears of simple interest, if any *i.e.*, balance of total interest over total net revenue.

(*b*) It must be susceptible of having clear Capital and Revenue Accounts kept for it.

(*c*) Its classification as a Productive Public Work must be authorized by competent authority.

3. The Rules for determining (1) whether a work which has been classed as productive shall continue to be so classed and, (2) whether an unproductive work may be reclassed as productive are as follows, the percentage rates referred to being those prescribed for the time being and being subject to alteration at the discretion of the competent anthority :—

I. Every irrigation, navigation embankment or drainage work for which capital accounts are kept should, until ten years after the date of the closure of its construction estimate, be classed as Productive, if the net revenue anticipated from it appears likely to repay, on the expiry of that period, the annual interest charges on the capital invested (including direct and indirect charges and arrears of simple interest), calculated at 4% in the case of works sanctioned before the first April, 1919, at 5% in the case of those sanctioned between tne first April 1919 and the 1st August 1921 and at 6% in the case of those sanctioned after 1st August 1921. Conversely, if it is not expected to yield the relevant return, it should be classed as unproductive. If moreover, at any time during the period of construction or within ten years of the date of the closure of its construction estimate, it becomes apparent that a work originally classed as productive will not actually be remunerative according to the criterion prescribed above, it should be transferred from the productive to the unproductive works, and similarly if it becomes obvious, during the same period, that a work sanctioned as unproductive will actually prove remunerative, the transfer of the work from the unproductive to the productive class may be effected.

II. Every work classified in accordance with clause I above will retain its classification unchanged during the eleventh, twelfth and thirteenth years after the closing of its construction estimate.

III. If any irrigation, navigation, embankment or drainage work for which a capital account is kept and which is classed as productive fails, at any time after the expiry of ten years from the date of the closing of its construction estimate, in three successive years, to yield the relevant return prescribed in clause I above, it should be transferred to the unproductive class. A work classed as unproductive which succeeds in yielding, in three successive years, the relevant return prescribed for a productive work may, on the same principle, be transferred to the productive class.

IV. If an exisiting irrigation, navigation, embankment or drainage work be extended or improved, the criterion of productivity prescribed in clauses I to III above or improvement, as whole system, including such extension or improvement had been executed simultaneously with the original work and the date of sanction referred to in those clauses for the purpose of determining the percentage to be returned by the system as a whole, shall be that of the accord of sanction to the original project. As an exception to this rule, if any extension be, owing either to its nature or magnitude, such as may reasonably be considered to be a separate project and if it be susceptible of having clear capital and revenue accounts kept of it, as distinct from those of the project as a whole, it should be treated as a separate project and in that case the conditions relating to original projects and not those relating to extensions and improvements shall be applicable. In all such cases separate capital and revenue accounts should be maintained for the extension in order to enable the productivity test to be periodically applied.

V. Clauses I, III & IV are, however, subject to the provison that the competent authority may postpone the transfer of a work from one class to the other in cases in which it is satisfied that its success or failure is purely due to transitory cause.

4. For the purpose of the determining the productivity of an old work which has been developed by the Government only the capital expenditure expended by that Government should be regarded as the capital at charge on which interest is chargeable.

5. The transfer of a work from the productive to the unproductive category, or *vice versa*, will affect the recording of all future transactions in connection with it. No adjustment will be made in general accounts in respect of past transactions, but the necessary transfers will be effected by the Accountant General in the *Pro Forma* Accounts of the work in question.

13. Plans.

The following list of plans comprises all those generallly required :—

(*a*) **Index Plan** :—[Art. 58] (a) Usual scale 1 mile to one inch. This should be tracing of foolscap size, or folded to it, so that it may be bound with the report : a duplicate copy of the plan should be placed with other plans.

(*b*) **Storage reservoir** :—(*i*) General Plan of the Catchment :— Usual scale 1 mile to 1 inch. If topographical sheets exist, this plan should be prepared from them, the bounding watershed line should be clearly marked and the area in square miles of the catchment printed on the plan.

(*ii*) **Contoured plan of the reservoir** :—Usual scale 660 feet to 1 inch. The plan should show all the main contours and also all the work connected with the reservoir particularly the tail channel of the waste weir and its outfall; a table of the areas and capacities of the main contours should be printed on it.

(*iii*) **Land plan** :—Usual scale is $\frac{1}{2,000}$. The plan should be pre-prepared from the Government land plans, if there are any, and should show all land to be acquired and the offsets from existing fields to where its boundary changes in direction. They should show the roadcrossings and

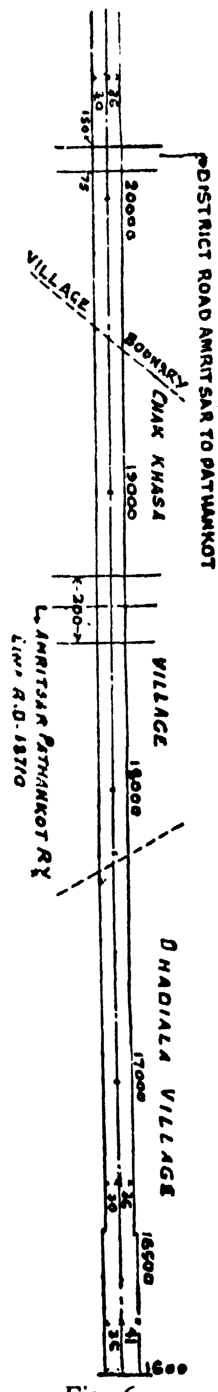


Fig. 6

village boumdaries. A typical land plan for a distributary is shown in Fig. 6.

(*iv*) **Dam** : —Plan longitudinal and cross-sections ; details of foundations and closure arrangements.

(*v*) **Waste Weir :** Plan, longitudinal and cross-sections details of sluices and temporary crest.

(*vi*) **Outlet** :—Plan, longitudinal and cross-sections details of tail and fore-bays approach bridge, valves, lifting rods and capstans.

(*c*) **Weir or headworks :** – (*i*) Contoured plan of the backwater :—As for reservoir.

(*ii*) **Land plan** :—As for reservoir.

(*iii*) **Flood embackments** : – Longitudinal and cross-sections.

(*iv*) **Headworks** : – General plan showing all works longitudinal and cross-sections details of all sluices.

(*d*) **Main canal and branches :** – (*i*) **Longitudinal sections**: —To show canal bed and full supply line, ground line and top of embankment; the location of all proposed works to be indicated. Notes to be given of the bed fall, full supply discharge and the irrigated area under command of different sections of the canal.

(*ii*) **Cross-sections** : To be given of all types and unusual sections.

(*iii*) **Cross-drainge and other works** :—Plan, elevation, and cross section of each main work to be drawn. For minor works type designs to be prepared.

(*iv*) **Plan of irrigation area** :—On this to be shown all villages, roads etc., the main, canal and its branches and sites of the principal works, drainage lines and channels and the boundaries of irrigable and unirrigable land

14. Examination Questions.

1. Mark a suitable alignment for extension of a channel for Irrigation of the area shown on the blue print plan supplied. Divide the area into suitable *cahks* and work out the minimum F. S. L. required in the channel at each outlet head, discharge for each outlet and total discharge required in the channel at the beginning of the extension. Design an A P.M. outlet for one *chak* and a standing wave flume outlet for another Draw free-hand dimensioned sketches for both. Intensity 70%. Full supply factor at outlet head 38 for *Kharif* premissible *Kharif, Rabi* ratio 1 : 2. (*P. I. B. 1941*)

2. On the attached contoured plan, the alignment of a minor is marked. Discuss the advantages and disadvantages of this alignment and propose a better one if you consider this feasible. (*P. I. B. 1941*)

3 On the attached contoured blue print plan mark the alignment of a minor taking off from the most suitable point on the distributary to irrigate the area in side the irrigation boundary of the proposed minor.

Divide the area into suitable *chaks* keeping to the village boundaries as for as possible and mark these *chaks*, in the plan finally showing the position of outlets and water course to irrigate each square in each *chak*. (*P. I. B. 1936*)

4. Describe briefly what information you would collect and the survey you would conduct for the preparation of a complete irrigation project. (*Mysore 1939*)

5. (a) What are the main points to be kept in view, in aligning a canal for irrigation purposes ?

(*b*) What preliminary surveys would you conduct to enable you to fix up final alignment ?

Mention the important data to be collected for properly designing the channel. (*Mysore 1939*)

6. What data would you collect and what are the plans you would prepare for an irrigation project ?

7. Work out the detailed flow irrigation project for the Karol Tube-Well Scheme area from the Lahore Branch. The following alternatives should be adopted :

(*a*) The existing Shalamar Distributary taking off from R.D. 218500 R. Lahore Branch to be enlarged and remodelled to irrigate this area.

(*b*) A new channel to be designed taking off from R.D. 2300 R. crossing the Shalamar Distributary.

(*c*) F.S. Factor=250 *Rabi Kharif* ratio 1 : 1. Intensity of irrigation 100 percent.

Complete draw off, command and area statements and alteration forms of outlets to be worked out. The alignment should be marked on the attached contoured plan and Longitudinal Sections to be prepared with cross sections at every thousand feet. The land plan should be prepared showing the detail of the land to be acquired. The typical design of the following works should be given :—

(*i*) Distributary Head Regulator. (*ii*) Meter Flumes. (*iii*) Falls. (*iv*) Crossing of the Shalamar Distributary. (*v*) Outlets. (*vi*) Syphon over the drainage and tail clusters.

The cost of the scheme will be valuated under the following sub heads :—

(*i*) Preliminary surveys. (*ii*) Land (*iii*) Earthwork. (*iv*) Service roads. (*v*) Masonry works. (*vi*) Outlets etc. (O. Miscilaneous.) (*P.U. 1942*)

8. Describe brief what information would you collect and surveys you would conduct for preparation of a complete irrigation project. (*P.U. 1952*)

PART II
CANAL IRRIGATION
CHAPTER XVIII
Remodelling Channels

1. Introduction.

In the last chapter, the student was acquainted with the design of new channels and the preparation of their projects. This chapter deals with the troubles and the working of the existing channels. It is intended to describe here briefly how to diagnose the trouble and to apply the remedies. The object of all remodelling of the existing channels is to aim at equitable distribution of supplies with a reduced cost of maintenance. As the holdings of the cultivators in the Punjab are generally small, the question of equitable distribution of supplies is very important. In the colony areas, the educated cultivators clamour to have the discharge of their outlets correct even to the second place of decimal.

The work of remodelling channels requires a great experience of this type of work, because a good deal of complaints and dissatisfaction starts when once the outlets are changed. Really honest work alone can make a remodelling successful. A brief summary of the author's experience of this type of work is given here and the student should study the following paper, available in the proceedings of the Punjab Engineering Congress, Lahore for a detailed study. (a) The Author's paper No. 154, 1931. (b) S. L. Malhotra's Paper No. 172, 1934 describing remodelling by R. A. Routh, (c) A. W. M. Jesson's Paper No. 230, 1940.

2. Necessity of Remodelling Channels.

The channels having the following troubles need to be remodelled :—

1. Chronic shortage at the tails.
2. Widespread complaints of individual outlets.
3. Drowned bridges.
4. Excessive silting up and frequent silt clearances.
5. Excessive supply over the authorised full supply discharge at head necessary to feed the tail.
6. Poor command at head of a distributary or at heads of its minors.

3. Bench Marks and Hydraulic Survey.

When the necessity of remodelling has been decided upon, then just put reliable bench marks along the channel to be remodelled, by double levelling, along closed circuits. The masonry works are liable to be demolished, the bench marks should be fixed on the R. D. (Reduced Distance) marks or by constructing masonry blocks $1{\cdot}25' \times 1{\cdot}25' \times 1{\cdot}5'$ near the land boundary limits at every mile with top one inch above ground level. An arrow head marked in cement plaster should indicate the point actually observed in levelling.

A detailed hydraulic survey of the channel should then be carried out and the longitudinal section and the cross-sections plotted. Typical remodelling L. section and Cross sections are given in Plate XXII D. II, and Fig. 1 to 4 respectively. The hydraulic survey should comprise all the information shown therein.

When preparing a longitudinal section, the points to observe are :—

(*a*) The horizontal scale should be $2''=1$ mile and vertical scale 1/50.

(*b*) Last designed and existing full supply and bed levels should be shown in different colours on the paper section. (When a tracing is made, do not use blue colour, as it produces very faint lines on the print especialy on glazed prints).

(*c*) Crest levels and working heads both existing and proposed, for all outlets and also the water levels in the water courses should be shown. The water level in a water course should be that prevailing when a high field is being irrigated.

Record plans of all masonry works should be checked at site, while carrying out hydraulic survey to see that they are correct.

Cross Sections of Mithalak Distributary

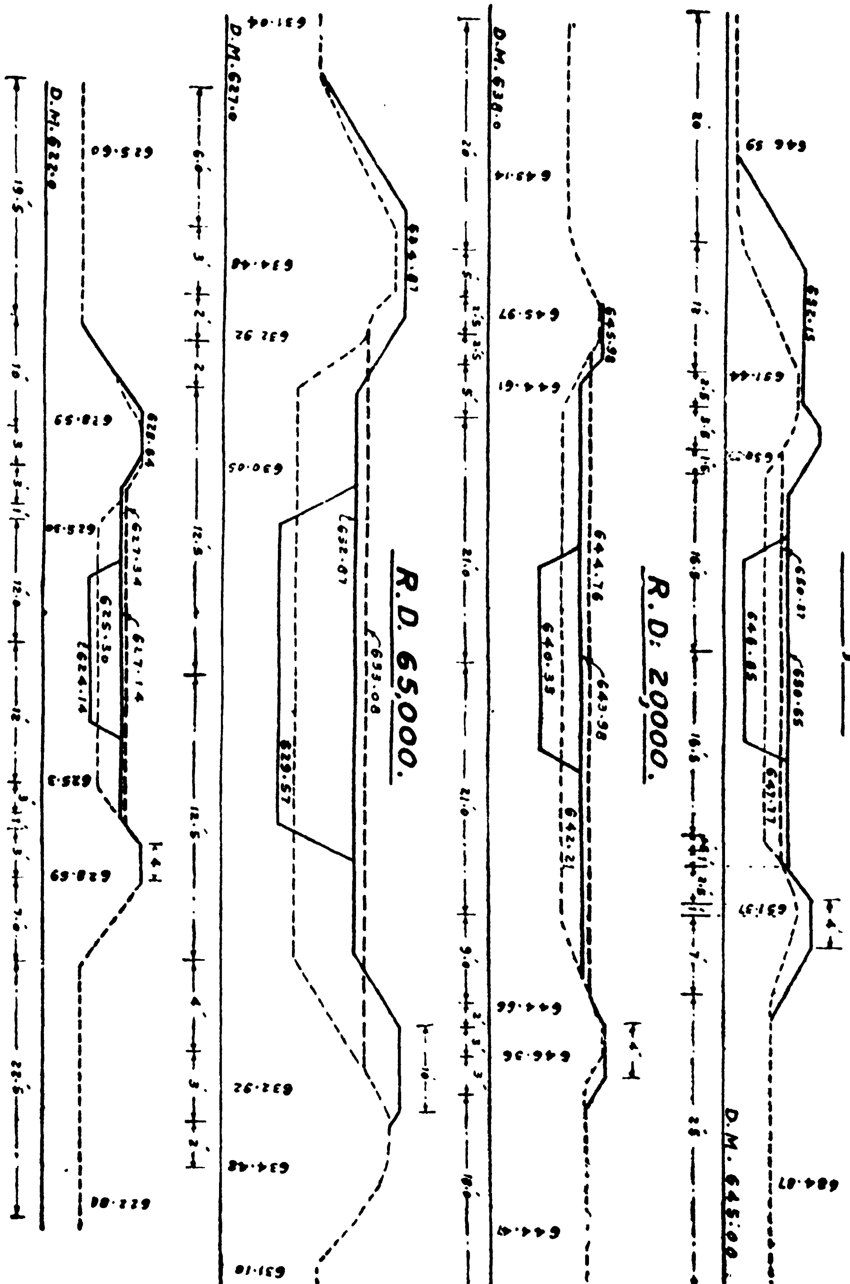


Fig. 1 to 4

4. History of a Channel.

The history of a channel should be read if one is available. If not available, one should be prepared after reading the old files to give the following information. The reasons for all changes in the original channel design with respect to slopes, sections, outlets, falls, distribution minor head and distributary head regulator should be recorded as given out by the then engineers. The record of all silt clearances should also be traced if available.

5. Diagnosis of the Trouble.

In the existing channels, the supply levels of the parent channel cannot be changed appreciably and the supply levels of tails are also fixed from considerations of the command of the area to be irrigated Moreover there should be provided control points in the form of falls or meters of at least 1·0 ft. Drop say at every 5 mile to divide the distributary in reaches to minimise the regime changes by tilting of supply levels as explained in para 5 Chapter XVI, Part II. There is usually little scope of steepening the slopes. The channel has to be run with the available slope. It can generally be run non-silting with the available slope by controlling the silt conditions and the bed width depth ratio (Para 6 Chapter VI Part II). A reference to appendix I of Author's Paper No. 154 P. E. C. 1932, will show that the channels do often suffer from excess slopes steeper than the regime slopes. The excess slope cause silting up of the channels by eroding sides and widening, by meandering and shoaling and above all by sweeping velocities killing the silt carrying vertical and rolling eddies. The causes of trouble should be investigated under the following sub-heads :–

(*a*) Defective slopes (*b*) Wide Channel sections (*c*) Defective outlets (*d*) Defective head regulator (*e*) Drowned bridges (*f*) Lack of control points and distributors.

6. Channel Sections.

The channel sections should be within 20% of those obtained from Lacey's formula $P=2{\cdot}67\ Q^{1/2}$ or Wood's bed-width-depth ratio. If channel is too wide, as is usually the case, the remodelling should provide its contraction to the designed sections. Very wide channel sections are generally attained in course of time by cattle tresspass, and collapsing of berms and pedastrian crossings. A relatively wide section silts up more than a narrower section on account of the decrease of the vertical silt lifting and rolling eddies (para 15 C, Chapter VI). The contraction should be attained by hanging spurs if scouring is also to be attained along with the berm formation which is a slow process. If it is desired to form berms quickly and also not to disturb the equitable distribution of the supplies to the outlets, the channel should be silt-cleard to the designed bed levels with bed width equal to designed bed width plus depth and then to construct longitudinal bushing. Both processes are explained in detail in the author's paper No. 154 and are given in paragraph No. 9 Chapter VII of Part II, Grass should be sown on new berm formation. If bushing be not available the contraction is attained by silt filling on sides protected by *gachi* pitching (berm clods) or by compacted earth protected by dry brick pitching.

7. Outlets.

The outlets should be designed to take their due share of silt, to give the correct discharges based on correct co-efficients and to work modularly under full supply conditions in the parent channel. In the head reach of a channel, A.P.M. outlets should be set as low as possible even (·2D lower than the bed) according to available working head, as permitted by the required M.M.H. In the middle reaches they should at least take there due share of silt which is about 112 percent. The author's A.P.M. with ·8D, setting permits this much silt conductive power and in the tail reaches there should be open flumes set at ·9D or bed level. The author's modifications in approaches increase the silt conductive power of these by about 6 to 8 percent.

8. Head Regulator.

The probable defects in head regulators of different types have already been given in Chapter XIV Part II. Blind silt exclusion by constructing skimming platforms should be avoided, as it is likely to give trouble in the parent channel. With the known C.V.R. in the parent channel and that which can be permitted in the off-take with the available slope, the required silt condutive power of the head can be calculated and a silt selective distributary head regulator can be designed with silt conductive power (Paragraph 8 Chapter XIV. Part II).

9. Drowned Bridges.

The drowned bridges cause unnecessary heading up and upset the regime. The heading up results in silting up of the reach upstream of this. They should be raised with 1·5 ft. clearance by R.C. Slab or T. Beam bridges in the distributaries and with 2 ft. clearance in the case of Branch and Main Canals.

10. *Chakbandi*, **Command and Draw-Off.**

Having decided upon the causes of trouble the next thing should be the preparation of correct *chakbandis* and the area statements. The classification has been given and the preparation of the area statements is explained in Chapter No. XVII Paragraph No. 9. Similarly Command and Draw-off statements should be prepared as explained in pargraph No. 9. Chapter No. XVII. Part II.

11. Lowering of Channels.

No outlet should have a working head less than 1·0 ft. If the working heads are high or if there is no irrigation from a reach, it should be lowered as much as economically possible. From consideration of seepage losses, it is desirable to keep the water levels in the channels as low as pessible.

12. Control Points and Meters.

(*a*) There should be control points at about every 5 mile with a minimum working head of 9″. The suitable design of control point is shown in Fig. 29. The outlets should be grouped to take off from upstream of the control points and combined with in the form of distributors. A meter flume at about R.D. 1,000 of the channel should be provided so that the supply passing the distributary is corrcetly gauged, if correct metering is not possible in the head design.

Two control points in a major distributary are absolutely essential, one about 2 miles upstream of the tail, say where the discharge is 15 to 20 cusecs, so that the silting and berming up of tail reach does not spoil the regime of the whole channel, and the other about 2 miles downstream of the head regulator to localise the effect of any mistake in the design of the head regulator or in selection of silt factor or C.V.R. in the beginning. These should be provided even at a risk of slightly flattening the slope. The middle reaches shall then remain immune from silt trouble even with relctively flat slopes.

(*b*) **Distributors.** There should be distributors at the heads of all minors in the form as given in Fig. 8. and Fig. 9.

13. Raising and Strenthening Banks.

When raising and stregthening banks do not allow borrow-pit measurements, adopt bank measurements. You will get much better value for your money. Below are given suggestions or calculating bank widths etc. Bank width $= D + F + \sqrt{\frac{B}{D}}$ (take the result to the nearest foot) where F = height of designed bed above N. S. ; D = designed full supply Depth : B = designed bed width.

Height of bank above full supply level up to 10 cusecs discharge = 1·0′ ; 11 to 25 cusecs discharge = 1·25′ ; 26 to 100 cusecs discharge = 1·5′ : 101 to 250 cusecs discharge = 1·75′ ; above 250 cusecs discharge = 2·0′. Berm width at full supply level up to 10 cusecs discharge = 1′ + d/2. 11 to 25 cusecs discharge = 1·5′ + d/2 ; 26 to 50 cusecs discharge = 2·0′ + d/2 ; 51 to 100 cusecs discharge = 2·5′ + d/2 ; 101 to 250 cusecs dischaage = 3·0′ + d/2 ; above 250 cusecs discharge = 3·5′ + d/2.

The formulae given above provide strong banks and liberal berms. If funds are scarce a lighter section may be adopted.

14. Remodelling Operations.

In the case of small channels all works required, such as silt clearances, remodelling outlets, raising bridges, constructing meter flumes and control point and contraction of channel sections should be done in one and the same closure. The stregthening and raising banks should follow as soon as possible.

In the case of major distributaries the reaches 3 to 4 miles, starting from the tail reach side should be tackled. All works required as mentioned above, should be finshed in the reach in one and the same closure. It is preferable to take reaches as defined by control points at the ends.

Some engineers are in favour of taking up remodelling reaches in major distributaries, starting from the head. This may be possible only when the distributaries are not taking excess at head, and only outlets and masonry works are to be remodelled to the modern designs. In the case of channels drawing excess at the head, the remodelling reaches must be selected beginning from the tail.

All remodelling of outlets must necssarily be carried out in the beginning of a crop. *kharif* or *rabi*, say in April or October so that the changes in outlets do not adversely affect the crops when they have been sown to avoid discontent among the cultivators and excessive remissions.

15. Silt Clearance and Berm Cutting of a Remodelled Channel.

After a channel has been contracted to the designed sections, the protruding branches of bushing should be cut off and channel should be finally cleared to the designed bed levels.

The tail reaches need annual berm cutting in the month of September generally, due to berm growth throughout summer caused by very fine silt in the water near tail reach which fertilises the bream growth especially in the rainy season. The berm growth heads up supply with the consequent rise in the bed levels.

It may be that in the remodelling, the channels cannot be designed with ragime sections due to the existing flat slopes in certain reaches. Such reaches should be given sections as limited by the wetted perimeter discharge relation or by the bed width depth ratio as far as possible and should be declared that they shall need silt clearances say every 2 or 3 years as the experience will show.

The remodelled channels with correct values of X, S and ρ in Kennedy's designs and with regime slopes, correct values of P_w and silt factor in Lacey's theory, with discharges over about 20 cusecs, do run nonsilting and do not need silt clearances. However, this does not mean that they could be neglected for all times to come. The damage caused to them by cattle tresspass by way of widening should be repaired by bushing, the beriming up should be straightened by cutting and the bed interference by the cultivators by way of *duffs*, bed clods and branches sticking in bed should be removed by bed clearance whenever the necessity arises preferably before the months of keen demand in the Punjab (June and October).

16. Watching Channels after Remodelling.

The remodelled channels need to be specially watched for at least a few years. The following observations should be taken :—

(*a*) **H. Registers (Modularity Register).**

The H. Registers are now maintained as normal routine giving observations by the subordinates once a months of depth on crest of all outlets and the actual working head. The columns of the register may be as per specimen below :—

No.	R.D. of outlet and Side	Type		Designed Data				April		May		June		July		Similarly for August to March
			B	y	H	g	h_m	H	h_m	H	h_m	H	h_m	H	h_m	
1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	

Column 9 shall show the fluctuations of the water above the designed water levels and column No. 10 shall give the indication of non-modular outlets where actual working head is less than h_m required for the modular working of the outlet as got from plate XVIII (A. & B.) column 8.

(*i*) **Special gauge slips.**

Special gauge slips should be introduced for at least 3 years after remodelling a channel giving upstream meter gauge and downstream channel gauges at all control points including the head of the distributary and the heads of the minors daily. All tail gauges should be read daily. After the channel has stabilized, it will be necessary to have just the head gauges of distributaries and minors and the tail gauges. The overseers, the *zilladars* and the S. D. O., inspecting a tail porsonally should report the actual gauge reading. This serves a useful check on the work of gauge readers.

(*c*) **"Characteristic curves," of distribuiaries.**

(*i*) There has been a progressive develpment in recent years in the methods of distribution of supplies form the remodelled Distributaries. With the universal application of modules for outlets and Minor heads, the distribution of supplies has now become automatic and interference by the Gauge Readers has almost ceased, The system has yet by no means attained perfection and frequent silting of channels introduce conditions, which are inmical to the proper working of the newly evolved automatic distribution of remodelled channels. Moreover, distributaries are being required to run with different supplies in Summer and Winter to meet the requirements of land reclamation, and of additional land given out for temporary cultivation. All this seriously upsets the distribution to various outlets and minors. Although the minor heads could be adjusted every crop to make them take their authorised supply and no more, it is neither practicable nor desirable to adjust outlets in the same way. It would, therefore, happen that when excess supplies are run at the head, the intervening outlets would also take a certain share of this excess. It is of utmost importance to be able to calculate readily, even though approximately, the excess discharge that would be needed at the head of any channel to deliver specific supplies at different points of a channel.

(*ii*) To meet this requirement characteristic curves have been evolved by R. B. Hakim Rai, Superintending Engineer. These curves afford a easy means of finding an answer to various questions, which otherwise would need lengthy calculations each time it is desired to carry out such investigations. Some of the problems for which a ready solution could be obtained by the application of these curves are:—

(1) Excess disharge required at head to give say 1·0 cusec at different points of a Distributary.

(2) The effect on tail discharge of lowering the Full Supply Level by silt clearance in a certain reach of the Distributary.

(3) The effect of *tatiling* of outlets in a particular reach on tail supplies.

(4) If any minor is over drawing the effect of reducing the supply in the Minor without reducing the supply downstream of it on the head discharge of the distributary.

There would be many other problems for which these curves would prove useful.

(*iii*) For the present the follwing characteristic curves have been drawn :—

(1) α—Curve. This gives the excess discharge required at each point of the channel to give one cusec extra discharge at the tail. This is the basic curve, as other curves are derived from it.

(2) β—Curve. This gives the excess discharges at head required to give one cs. extra discharge at any point of channel.

(3) γ—Curve. This gives the percentage of the extra discharge at any point that would reach the tail.

(4) λ—Curve. This gives the increase in depth in the various reaches when excess supply, to give one cusec extra at tail, is running. This also gives the effect of silt clearance of certain reaches on the tail supplies.

(5) Σ-Curve. Below the λ Curve is given the excess withdrawal by all the outlets in a reach when extra supply to give one cusec at tail is running in the channel.

Note :—For discharge other than that for which the curves are drawn, values can be obtained by direct proportion (except in the case of γ curve which remains unchanged).

(*iv*) Method of plotting the characteristic curves:—

α Curve is called the basic curve, as calculations are made to determine values for

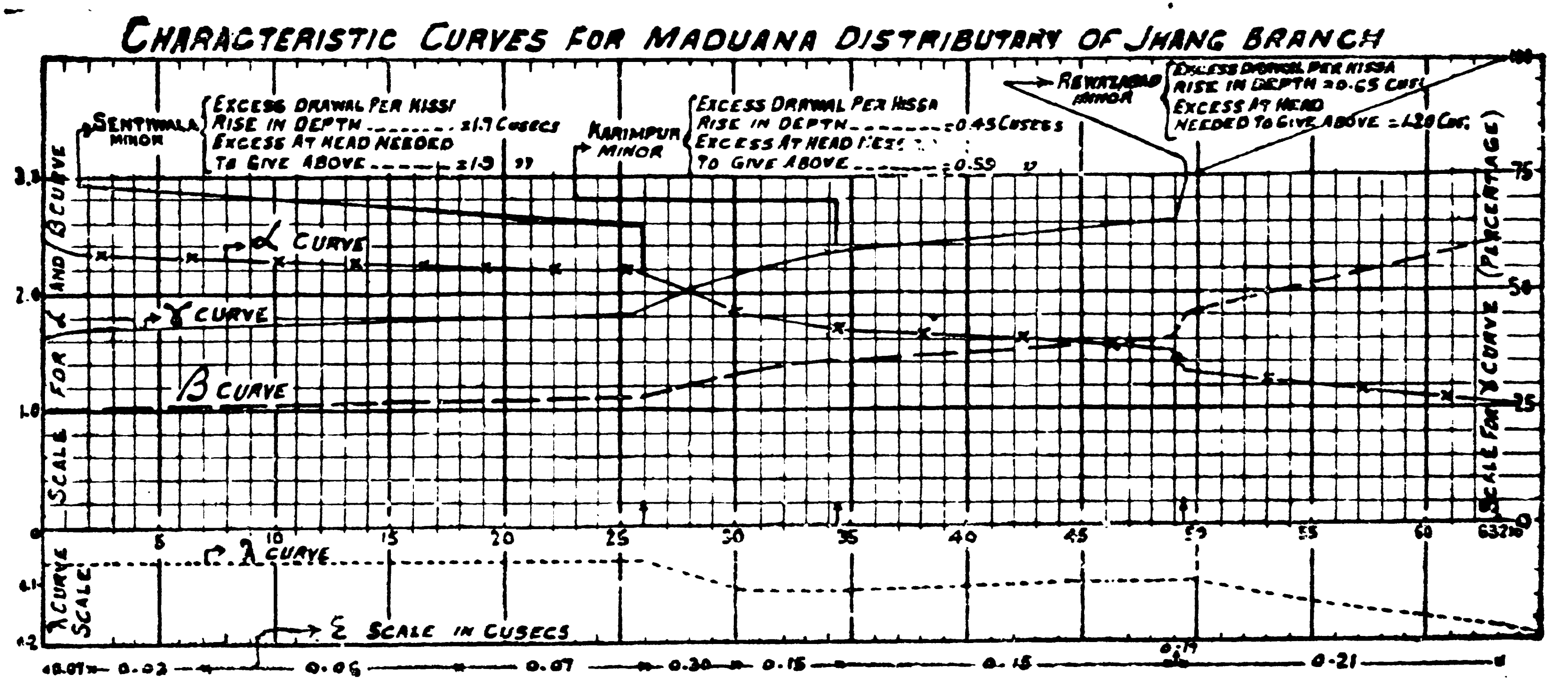


Fig. 5.

Note :—

1. α Basic curve gives exces discharge required at any point to give 1·0 cusec extra discharge at the tail.
2. β Curve gives excess discharge at head required to give 1·0 cusec at any point ; β at point $P = \frac{\alpha \text{ at head}}{\alpha \text{ at point P}}$.
3. γ Curve gives %age of increased discharge at any point that would reach the tail ; γ at point $P = \frac{100}{\alpha \text{ at point P}}$.
4. λ Curve gives increase in depth at any point to give 1·0 cusec extra discharge at the tail ; λ as per col : 6
5. Σ=Total extra withdrawal by outlets in the reach when there is 1·0 cs. excess at the tail; Σ as per col : 14

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plotting this and other curves, are then derived from this. The tabular form used for this, purpose is shown in table 1. The channel is divided into small reaches beginning from the tail upwards. The discharge at the tail is to be one cusec in excess of that sanctioned. This results in increase in the depth of the channel which is found by using the farmula $Q = KD^{5/3}$; percentage increase depth can easily be determined as it is approximately 60% of the percentage increase in discharge. After having determined the increase in depth in the reach the excess discharge drawn by each outlet is worked out and the total withdrawal plus the extra supply reaching the tail plus the absorption losses give the proposed discharge at the tail end of the next reach. The same process is repeated till the discharge originally required at the head, and that now determined is worked out. For purposes of these calculations, it is assumed that all the Minor heads would not draw any share of excess as it would be possible to adjust them for the increased depth. Figures thus obtained are then plotted as ordinates and reduced distances of the channel as abcissa to get α curve. Calculations for each Minor are also made separately from which and with the help of these curves figures can be easily worked out, if the minor heads are not adjusted. The excess discharge drawn by each Minor per *hissa* rise in depth is worked out and the excess discharge required at the head to give this much excess in the Minor is also determined as would be clear from the examples worked out.

The figures for the β curves are obtained by dividing the ordinate at the head as per α curve by the ordinate at the head of the reach. The points thus obtained when plotted give the β curve.

The figures of the γ curves are obtained by dividing 100 by the ordinate at the head of the reach as per α curve.

The figures for λ curves are obtained from column 6 of table No. 1, of this Chapter. The Σ figures are obtained from column 14. These curves are shown for Maduana Distributary of Lower Chenab Canal in figure 5.

(*v*) Examples of the use of curves.

(i) What extra discharge is needed at the head of Maduana Distributary to give 1·5 cs. at tail and 0·75 cusecs at 35000 if :—

(*a*) Outlets are not adjusted and minors not allowed to take excess.

(*b*) Both outlets and minors are not adjusted.

(ii) If one cusec is saved as a result of silt clearance in the reach head to 35,000, how much of it would reach the tail ?

(i) (*a*) From α curve excess required at head to give 1·0 at tail = 2·4 cs.
do do do at R. D. 35000 = 1·4 cs.
∴ Excess required at head to give 1·5 cs. at tail = $1·5 \times 2·4 = 3·6$ cs.
do do do 0·75 at R.D. 35000 = $0·75 \times 1·4 = 1·0$. Total 4·5 cs.

(*b*) If Minors also draw excess, addition for this to be made as follows :—

Rewazabad minor : - For 1·5 at tail, rise in depth at head of minor = $1·5 \times 0·1 = 0·15$ (see curve) ; From the curve for every *hissa* rise in depth at head of Rewazabad minor, the excess discharge needed at Maduana head = 1·2 cs. ∴ Excess discharge needed at head if Rewazabad minor is allowed to take excess = $(·15/·1) \times 1·2 = 1·8$ cs. (A)

Similarly for Karimpur minor. Rise in depth = ·09 ; Excess discharge needed at Maduana head = $(·09/0·1) \times 1·5 \times ·59 = ·8$ cs. (B)

And in the same way for Sehtewala minor excess discharge required at head would be = $(·05/·1)$ $1·5 \times 1·9 = 1·4$ cs.

For supplying 0·75 cs. at R. D. 35000 excess required at head = $0·75 \times 1·19$. (From α curve) = 1·4 cs. (C)

This is equivalent to sending = 1·4/2·4 = 0·6 cs. at tail.

The excess discharge needed at head in consequence of the two minors above 35,000 *i.e.*, Karimpur and Sehtewala, drawing excess is worked out below :—

For Karimpur = $(·09/·1) \times 0·6 \times ·59 = 0·32$. (D)
For Sehtewala = $(·05/·1) \times 0·6 \times 1·9 = 0·57$. (E)

Total excess required at the head on account of minor heads not being adjusted
$=(A)+(B)+(C)+(D)+(E)=1{\cdot}8+{\cdot}8+1{\cdot}4+{\cdot}32+{\cdot}57=4{\cdot}89$.
Total excess at head in the case of (i) (b)$=4{\cdot}6+4{\cdot}89=9{\cdot}49$ cs.

(ii) If one cusec is saved in the reach, head to R.D. 35,000, 60% as seen from γ curve. will reach the tail. Thus if Rewazabad Minor in the way does not draw any excess discharge reaching the tail$=0{\cdot}60\times 1=0{\cdot}6$.

If the minor also draws excess, deduct for this as follows, 0·6 cusecs at tail would raise supply at Minor head by $\cdot 1\times\cdot 6=\cdot 06$ and the minor will get the discharge of $\cdot 6\times\cdot 65=\cdot 39$ cs. and thus tail would get $\cdot 06-\cdot 39=\cdot 21$ cs.

TABLE 1.

Calculations for increase in discharge at head of Maduana Distributary for an increase of one cusecs at tail.

Reach.	Discharge at the tail of reach.	Discharge now proposed.	Designed depth at the end of reach.	Modified depth now worked out.	Increase in depth.	R.D. and side of outlet.	Size of outlet.				Q as sanctioned.	Q with increased H.	Excess.	New Q at the end of reach.	Calculations.
							H	B	Y	h					
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
63210T	4·77					Excess					1·0			5·77	5·77
63210	4 77	5·77	1·4	1·57	·17	61043L	1·6	·25	·45	1·15	·89	·94	·05		3·29
to						57548R	1·6	·25	·33	1·27	·68	·73	·05		·33
49500						55936L	1·6	·4	·49	1·11	1·51	1·62	·11		·58
										Total	3·08	3·29	·21	9·97	
49500	8·64	9·97	FALL	1·10	·1	49465L	1·4	·32	·58	·82	1·23	1·30	·07		9·97
to			1·0			49425L	Rewazabad minor.				4 32	4·32			7·08
49000						49100R	1 4	0·32	0·68	0 72	1·36	1 46	·10		·71
										Total	6·91	7·08	·17	17·76	
49000	16·24	17·76	1·90	2·0	·1	46982L	2·0	·9	·55	1·45	1·94	2·00	·06		17·76
to						45542R	2·4	·4	·43	1·97	1·76	1·81	·05		6·46
34625						40077L	2·28	·32	·51	1·77	1·59	1·62	·03		·65
						38994R	2·71	·25	·36	2·35	1·02	1·03	·01		
										Total	6·31	6·46	·15	24·87	

(SO ON AND SO FORTH)

17. Examination Questions.

1. An existing distributary of 100 cusecs capacity designed for an annual intensity of 80% and a crop ratio of 1 : 1 is to be remodelled to the same intensity but to a smaller *Kharif Rabi* ratio 1 : 2. Work out the old and new outlet allowance and state fully what data you would collect as basis for remodelling the channel and how you would determine the new slope to be given to it ? (*P I.B. 1936*)

2. How will you determine the necessity of remodelling a channel and how will you diagnose the trouble ?

3. In the remodelling of channels, the slopes are fixed according to the existing conditions and cannot be changed, how will you determine whether a particular reach shall need normal silt clearance or not ? Why do the tail reaches need annual berm cutting and silt clearance in October ?

4. (*a*) In remodelling channels, how will you fix the setting of the outlets ?
   (*b*) What is the effect of drowned bridges on the regime of a channel ?

5. What observations and care are necessary after a channel has been remolleded ?

PART II

CANAL IRRIGATION

CHAPTER XVIII

Inundation Canals

1. Introduction.

The canals which usually flow only in summer, when the rivers are high, are called 'Inundation Canals'. They have no permanent headworks and no weirs across the rivers. Owing to the changes in the river course the off-takes of the canals have often to be changed and fresh channels dug or creeks cleared out. The supplies of these canals are not always as desired.

In most of the districts in which inundation canals exist, the rainfall is very scanty, and the crops chiefly depend on canals, river inundation and wells. In order that the water of a canal may be made to flow on to the surface of the country, the canal must generally have a direction making an acute angle with that of the river and a general slope flatter than that of the river, but when the country falls away from the river, a canal may off-take at right angles to the river. Sometimes a canal, after crossing the river valley, runs nearly parallel to the river along the slope of the watershed and irrigates land many feet higher than the flood level at a point immediately opposite. The bed of a canal is below the surface of the ground, but the water level is generally above it, and the irrigation is by "flow". The land near the head of a canal is generally too high to receive flow irrigation except in floods. It may be irrigated by lift either from the canal or from the river. Owing to the smaller lift this is much easier than irrigating form wells, and the water contains, fertilizing silt. Land irrigated by canals yields two crops, the *Kharif* or summer crop and the *Rabi* or winter crop. The land for the latter is soaked with water in August or September and afterwards ploughed. The *Rabi* crop is sown in November and December, and is generally matured by water raised from wells. The principal summer crops are rice, indigo, cotton and mellet, but the largest of all crops is the winter crop of wheat and barley. The inundation canals of the Punjab used to irrigate about 1½ millions of acres yearly. They were more than fifty in number. Nearly half of them have been converted into perennial canals by the construction of the Sutlej Valley and the Haveli Canals. The Dera Ghazi Khan and Muzaffargarh Divisions have inundation canals even now. There are private and Government inundation canals in the Shahpur district.

2. General Description of Canals.

The bed level at the head is generally about the same as the winter sub-soil water level, and this is about the same as the low water level of the river. The depth of water in the canal at any time is thus about the same as the height of the river above its low water level. In most canals it averages about 5 feet, but it may sometimes be 10 to 12 feet. A large canal usually gives off branches or distributaries. These again give off water-courses which are maintained by the people and not by Government. The bed slope of a canal is seldom steeper than 1 in 4,000 or flatter than 1 in 10,000. The side-slopes generally become, by silting, about half to one. The canals were in most cases dug originally by the cultivator without any engineering knowledge and are sinuous. The widths are also somewhat irregular.

The banks of the canals are often somewhat weak and liable to breach. In such cases the banks are patrolled and watched. The strengthening of canal banks, where liable to breaches, is a work which should constatly be seen to.

A canal generalley has a masonry flood regulator, a few miles from the off-take, to enable excessive supplies to be shut off. The ragulator cannot be placed near the off-take because this is often changed and there would also be the fear of its destruction by the river erosion. At the flood regulator there is generalley the off-take of an escape leading back to the river and there is often also the off-take of a branch or distributary.

If the canal crosses a flood embankment the flood regulator is at the point of the crossing. Further down the canal there are other regulators generally at the the off-take of branches or distributaries, but sometimes at other places in order to head up the water during low supplies.

The Head Reach (often called, for shortness, the "Head", while the actual point where the canal begins is called the "off-take) is the uppermost five miles or so of the canal, and this often corresponds roughly with the reach upstrem of the flood regulator.

The banks of the canals generally have trees and jungle growing on them. Fresh trees are also planted in lines on the banks and slopes, but not generally near the channel because they are somewhat apt to fall into it.

A canal generally has a bridle road, and often a proper road fit for wheeled traffic, along one or both banks. The road on one bank is generally called the Inspection Road and is reserved for the use of the Canal Officers. The other bank is open to the public. Where watercourses take-off, rough bridges made of branches of trees, grass and earth are constructed across them.

For the purpose of fixing the dates of opening the canal, they are divided into three classes. An "ordinary" canal is opened about the 6th May. An "early" canal is one which serves that tract of a country where certain crops which require early watering, are grown in sufficient quantities to render an early opening necessary. Such a canal is opened about the 26th April. A "Late" canal is one where there are no considerable areas requiring water till about 15th May or (sometimes) 1st June. All the above dates are averages and approximations. The actual dates depend upon demand for water, the height to which the river has risen, the tendency of silt to deposit in the canal and the probability of damage occurring from breaches. The canals generally go dry about 30th septmber and thus the period of flow is generally about 5 months.

3. Silting of Canals.

The slope in the canal is usually flatter than that in the river. The velocity in a canal being less than that in the river a deposit of silt generally takes place in the canal. The silt consists of sand clay. The sand, with a little clay, is deposited in the head reach of the canal. At the close of irrigating season, the deposit at the off-take is generally from 1 to 5 feet deep and it extends downstream, gradually decreasing in depth, for a distance which may be less than a mile or may be 6 miles. The deposit is generally the greatest when the canal takes-off from the main stream, especially if erosion of the river edge upstream of the off-take of the canal is going on, and least when it takes-off from a creek. There may be some deposits, due to berming up, near the tail of the canal.

There is generally no deposit in the middle portion. There is nearly always a silt deposit at the inside of a bend, but there is a corresponding hollow at the outside, so that the general level of bed is about the same as it is just above or below the bend. Large sums of money have been wasted by removing the silt deposits which are known as 'side-silt'. Their removal makes the cross section greater than elsewhere and they quickly form again on bends.

A great amount of silt is deposited in the water-courses. The silt when removed from the canal is generally placed equally on both sides behind the banks and levelled down to the height of the banks.

The deposit of silt in the head reach of a canal is the greatest evil with which the Canal Engineer has to deal. Deposit in reaches further down the canal, if it occurs ; is not of so much consequence. In many cases a Subsidiary Head is provided and kept closed. Deposit in the head reach not only reduces the supply below the demand, except in high floods, but is also cuts off the supply when the river finally falls and causes damage to, or failure of, standing crops. The subsidiary head is then opened.

Erosion of banks may occur, especially in large canals and at bends. It is likely to be very serious in sandy soil, and this is most common in the head reaches. The best and most common method of stopping it is "Bushing'. The various methods of bushing have already been described in the case of perennial canals. Sometimes fascines and mattresses have also been tried where river action is high.

4. Canal Head Reach.

(*a*) (*i*) The general layout of the head reach of a canal is as shown in Fig. 1.

The main problems of inundation canals is to secure adequate supplies for these canals in the middle of April or very early in May, so that the cultivators can sow valuable crops such as Indigo, sugarcane and cotton instead of cheap ones as rice, fodder crops, *bajra* and *chari*.

If flood water breaks into a canal, it frequently breaks out again on the opposite side and it may from a deep hole in the bed of the canal. This hole may enlarge in all directions, forming a pond whose diameter may be much greater than the width of the canal. When the banks are made up they are carried round the pond forming 'ring banks'. In course of time the pond silts up and the banks can then be brought into proper line.

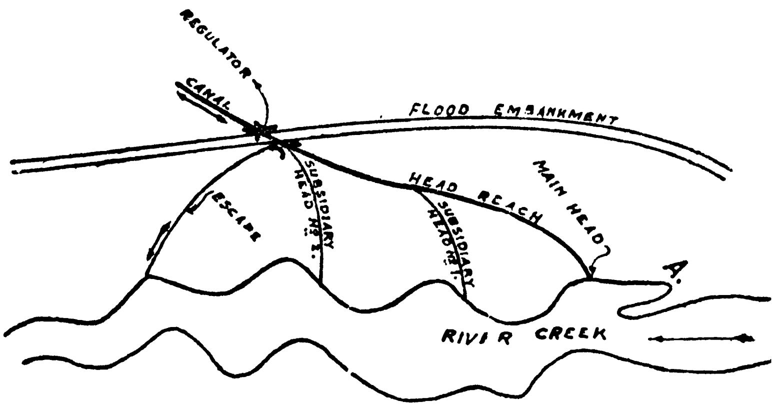


Fig. 1

If the damaged place is in high ground, little harm may result, except the expense of repairs, but if the ground is low, the canel supply will continue to escape and be wasted even after the subsidence of the flood and the closure of the breach may be difficult. The rush and disturbance may also cause falling in of the banks for a considerable distance upstream and a short distance downstream of the breach.

If the water merely breaks into the canel head, and does not breaks out again the heading up, at the point of afflux, is likely to be considerable. It may be desirable to cut the other bank if the ground is not low. It is nearly always difficult, and generally almost impossible to close bad breaches (*i.e.*, breaches not in high ground) in head reaches until the river finally falls and money should never be spent in attempts to close them unless there is a real hope of success and also something appreciable to be gained by success.

(*ii*) Anything which reduces the silt deposit in the head reach of a canal is of the utmost value, and a small change may make all the difference. Any sharp bends should be removed and loops cut off so as to shorten the channel and increase the gradient. The width, suposing the gradient and discharge to be the same, should not be greater than that of the next reach downstream of regulator. It may with advantage be slightly less so as to give a "draw". Anything which causes heading up of the water, such as the continued use of the flood regulator or an influx of water into the head reach at a point downstream of the actual off-take is a source of danger and may cause serious deposit of silt. The inflowing water may be simply flood water or escapage from another canal or flood water impounded in a "pocket". For the reasons just given a flood regulator should never be built witnin, say three miles of the off-take of a canal if the gradient is steep and six miles if it is flat. The actual site is generally so fixed that the regulator can be combined with a bridge for a main road or so that it may suit the off-take of a branch or distributary. It may be many miles from the canal off-take. Unless it is so, it should not be used more than is absolutely necessary to reduce supply and cause the escape to work. If there is a flood embankment which has to cross the canal it should be altered, if necessary, so as to bring the regulator site lower down.

(*b*) **Escape in a head reach.**

At a flood regulator there should always be an escape, and it should be large enough to carry all the surplus water. Otherwise there will be heading up in the head reach of the canal. The size of the escape can be calculated by taking the difference of the discharge of the reaches a-b and b-c (Fig. 2). Escapes usually lead back to the river, but occasionally an escape leads into the canal next below. In this case the junction with lower canal is probably not far from off-take of the latter and the heading up caused by the inflow is likely to be detrimental. Such an arrangement should not be permitted unless the escapagage is likely to be slight or unless the water can again be let out of the lower canals close to where it enters.

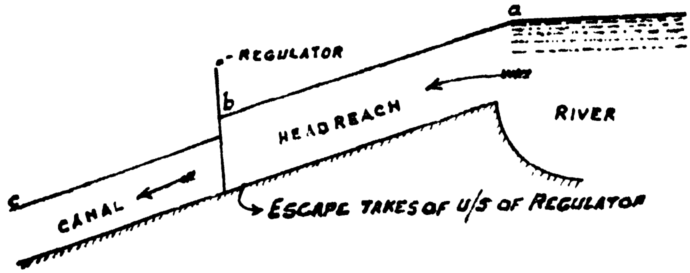


Fig. 2.

An escape, of course extends only as far as the nearest creek of the river. It should be inclined downstream at, say, 45° to the general direction of the river. Sometimes, in order to get a short line, it is run out at right angles. Such a channel is likely to have a poor gradient. It is almost peculiarly liable to have its banks breached in floods. This may not matter much while the floods last, but the breaches have to be repaired afterwards. In the absence of an escape, cuts are sometimes made in that bank of the head reach which is nearest the river, at points where the ground is high and the escaping water find its way to the river. Such cuts close of themselves when the floods subside.

(*c*) **Changes in head reaches.**

Owing to the changes which take place in the rivers, it is frequently necessary to dig new heads. If the heads could be dug rapidly, and at any time of the year, matters would be greatly simplified, but the excavation requires time and it can only be carried on in the winter and the spring when sub-soil water level is low. Moreover, labour can only be obtained is sufficient quantities in the winter and the early spring. As soon as one irrigating season is over, the heads for the next season must be arranged for. The old head may be retained or a new one may be necessary. In arranging for heads regard must be had chiefly to the state of the river as it is at the time. Some little idea may be formed of the changes which will take place in the immediate future, but beyond that all is mere conjecture. A canal head, whether new or old, may begin to work badly soon after being opened. When serious erosion of the river edge takes place just upstream of a canal head the latter may become heavily silted in a few weeks, or the creek supplying the head may silt up, or the stream may move away and the water level be in consequence lowered. Subsidiary heads are also dug as shown in Fig. 1 which are opened if the main head has silted up.

(*d*) **Selection of off-take.**

The following are general rules for off-takes : –

(*i*) A site where erosion is occuring, or is likely to occur, is a bad one. Sites in the main stream, or close to it are thus objectionable.

(*ii*) A site in a small or silted creek is open to the risk of silting up of the creek.

(*iii*) A site in a large creek is generally goods. The site should not be at a bend which seems liable to erosion.

(*iv*) A site near the tail of a creek, but not so low down as to be close to the main streem, is generally an excellent one. If the head of the creek silts up, the supply will be drawn in from the tail.

(*v*) A back water, such as that marked A in fig. 1 is also a good site.

The principle of placing the off-take as far upstream as passible is followed when other consideration do not interfere with it. This ensures a long feeding channel which is desirable. A good site for an off-take can generally be found by going far enough, but both funds and labour are limited and practically the length of a new head seldom exceeds two or three miles. A head

(Head reach) should not run for a long distance near the river edge. Such a head offers facilities for new off-takes in case they are needed, but it is open to the danger of being cut in half if erosion occurs. If an off-take is carried too far upstream, it may result in the supply becoming so great in floods as to be unmanageable.

(*e*) **Subsidiary heads.**

(*i*) When a canal has two heads, the one first opened is called the Main Head and the other the Subsidiary Head. The following points may be in view in deciding which is to be the Main Head :–

(*a*) The main head should be the one which seems likely to give the better immediate supply. It not much use having a good supply for maturing crops if there is not a good supply for sowing them.

(*b*) The main head should be the longer of the two, because there is then less chance of the silt extending down below the junction and so obstructing the flow of the subsidiary head when opened.

(*c*) The subsidiary head should have good banks so that, being closed by dams at both ends, it may be completely "boxed up", and no flood water allowed to break into it. A channel to which flood water has an access is quite unsuitable as a subsidiary head.

Generally, the longest head, *i.e.*, the head whose off-take is farthest upstream, will give the best supply unless erosion seems likely to occur at the off-take.

The main head usually gives sufficient supply for the greater part of the season. At first the deposit of silt in the canal or in the feeding creek is small. By the time it has become considerable, the river has risen high and the silt has little effect.

It is when the high floods are over and the river is falling, that the silt begins to have a serious effect, and this is the time for opening the subsidiary head. A suitable date generally for the Indus is, about the 10th or 12th of August, and for the other rivers about the 20th of Angust. Premature opening of a subsidiary head may be disastrous, causing excessive supplies or silting one of the heads if it has a flat surface slope.

(*ii*) When the subsidiary head is opened it is sometimes desirable to close the main head. It may happen, however, that both heads bring in appreciable supplies (this is likely to occur if both heads are long ones or even if the upper one alone is a long one, and is the more silted of the two) and both should remain open. No doubt some additional silting is caused. In one head or the other owing to both being open, but there is also an additional supply. The question whether the additional supply is worth the additional silting (the cost and difficulty of closing a flowing stream being also considered) is one of judgement. The qustion whether it is desirable to close a head, is also affected by the information received from station higher up the river. Possibly the river may be falling so fast that the head will soon close of itself.

(*iii*) Special cases may now and then arise and give rise to special measures. For instance, if erosion occurs near the off-take of the main head and it begins to silt up, and the erosion seems likely to go on, it may be desirable to close the Main Head, even quite early in the season and to open the subsidiary head. It may even happen that what was intended to be that main head is not opened and the subsidiary head is opened instead. Somstimes if there is spill water sufficient, cuts may be made in the banks to feed the canal thereby.

In a large nubmer of cases the subsidiary head is not used. Either the main head continues to give a good supply all the season or erosion sets in at the subsidiary head or its feeding creek fails or the river recedes from it.

5. *Bunds* (Dams) for Canal Heads in Winter.

The canals having gone dry in September or October, earthen dams are constructed at the off-takes in Novermber. The object is to prevent the rainy river water from accidentally entering the canals, before they are ready for opening. Any such accident may cause immense trouble. Contractor's earthwork may be unfinished or unmeasured or materials for masonry work may be lying about. The danger may occure during winter freshets or in April or May when the river rises permanently.

In order to allow for a slight falling in of the river edge, the dam in the main head must be set back, say 20 to 50 feet, from the actual off-take. To pervent this space from silting up when the river rises, a minor dam is necessary at the actual off-take or as near to it as possible.

Setting bank the main dam several chains is most abjectionable. The whole length may silt up if the minor dam is carried away.

The dams must be paralled tn the river edge. If made square to the canal when the off-take is skew, a space is lest for the collection of fine tenacious silt. Such silt has at times absolutely prevented a subsidiary head being opened and has interfered with the opening of a main head.

The dimenosions of the dam may by :—

		Top width	Height above water level on 15th Nomber
Main Dam	......	10 ft.	8 ft.
Minor Dam	......	8 ft.	5 ft.

If the off-take is from a dry creek, the height can be measured from its bed. The material should be got, if possible, from the bed of the canal. Sand does very well. The side slopes of the dams should be protected by fascining. Long twigs are made into bundless and tied up so as to form fascines, and these are laid on the slopes and secured by pegs driven into the slopes, at short intervals, between the fascines.

If a creek is to be cleared, the dam will be at the off-take of the creek (from the river or from another creek, which is not to be cleared) and not at that of the canal from the creek, unless the dam at the creek off-takes is considered to be insufficient.

If there is any creek or channel or depression which is not to be cleared but by which water can enter the canal or can enter a creek which is to be cleared it must be closed in a similar manner, but the material need not be taken from its bed and there need not by a minor dam. On the top of the small dam a grass hut should be made and a watch-man should live in it. For a subsidiary head the dam should, unless erosion seem likely, be close to the off-take with 15 feet top width, height 3 feet above H.F.L. and sides protected by fascining. If erosion seems likely, the dam may be silghtly set back, and it erosion actually sets in, a second dam (No. 2) should be constructed say 200 feet downstream.

A dam can be built on top of sandy deposit, but it should never be built on the top of tenacious silt or on grass or *dibh* growth. The silt should be dug out to bed level and than the dam made.

In opening a canal it is impossible to remove the whole dam. Some of it must necessarily be left and be allowed to be swept away by the water. If time and labour are short, the whole dam may be left to be swept away a mere cut being made in it. It is better to do this than to delay the opening of the canal. The material in the dam when spread over a great length of canal, causes no appreciable raising of the bed.

6. Miseellanous Works.

(*a*) **Bank erosion.**

Erosion of the banks may occur especially in large canals and at bends. It is likely to be very serious in sandy soil, and this is most common in the head reaches. The best and the most common method of stopping it is "Bushing." The various methods of bushing have already been described in the case of perennial canals. Sometimes fascines and mattresses have also been tried where river action is high.

(*b*) **Bridges and water course crossings.**

In the tracts served by Inundation Canals, roads fit for wheeled traffic are somewhat scarce. Masonry bridges are somewhat few. Many bridges suitable for camels, horses and foot passengers are made of rough tree trunks, placed on end to act as piers, with rough branches laid across them and covered with coarse grass and earth. Such bridges require repair, or reconstruction yearly and not infrequently portions of them fall into the channels. They are gradually being replaced by masonry structures. Bridges of sawn timber are cheap, but do not last. There are very numerous *ghats* or places where the banks are sloped off so that animals or men can get water from the stream or wade across it.

In winter the people put up numerous temporary acqueducts, rough wooden troughs supported on stakes to carry water raised from wells on one side of a canal to fields on the other side. The canal banks, being generally above the leve of the fields, have to be cut through at such places. Before the canal are opened in the spring, the people remove the acqueducts and repair the banks. This procedure is troublesome and objectionable, and is only permitted at places fixed by usage. If a crossing for water becomes necessary at new place, a masonry syphon has to be constructed or a new well sunk, the cost being borne by Government, if a new Government channel has severed the connection between well and field, otherwise by the people concerned.

(*c*) **Drainge crossings.**

In the Dera Ghazi Khan district there are numerous streams which issue from the Suleman mountains (these lie twenty or thirty miles to the west of the Indus and flow towards the Indus). The channels are generally dry, but after rain, streams come down and break into the canals at numerous points, generally towards the tails or into the western branches and break out again on the opposite bank. Much damage is done to the banks and some silt is brought into the channels, but the water brought in is, on the whole, is useful in supplementing the regular supplies which are not generally very good towards the tail of the canals or in the western branches, as these have necessarily some what flat gradients owing to the land rising towards the mountains.

(*d*) **Duty of inundation canals.**

The "Duty" of water on inundation canal is low, being generally obout 70 acres per cubic foot of discharge per second, (measured at the canal head) as against 200 or 300 acres on perennial canals. The duty is low because the canals flow only for about five months of the year, because the soil is sandy porous and because the rainfall is very light. Long water courses also, to some extent, cause low duty.

In matters such as the observations of discharges, the keeping up of Irrigation Registers, the distribution of the supply, the fixing of the sizes and sites of outlets for water courses, the remodelling of water courses and the maintenance and extension of plantations, the principles and practice are the same as for perennial canals, except that trees are not planted where they are likely to be cut away by the river.

The chainage of an Inundation Canal usually starts from the flood regulator and runs in both directions, The mile posts etc., in the upstream reach should be of a kind which can be shifted. The same remark applies for the whole canal until all necessary straightenings have been effected.

7. (*a*) **River behaviour.**

P. Claxton produced on this subject paper No. 119 the Punjab Engineering Congress 1938. His practice is shown in figures 3 and 4. In this paper the writer deals with the Inundation Canal practice in all its bearings. He takes as an example the whole of the Dera Ghazi Khan Division in which he had served for many years, from 1910, off and on, till 1927. He had been able to watch problems as they had arisen and developed.

"It is well known that alluvail rivers, bearing large loads of silt, carry them on, not in one sustained effort, but by a series of jumps, or by saltation. The loads have not travelled down continuously from the hills, but are removed along by the processes of scour and erosion. Of these two, erosion, the action at banks, in contradiction to scour, that is over the bed, is the more important. It gives the heavy excess loads at various points and causes the river to meander. In doing so it presents the problems with which the inundation canal practice has to deal".

One of the first principles lays down that silt eroded from a bank very soon deposits as a shoal on that side. Many engineers argue that it may be, and is, carried over to the other side. The argument is based on experiments on very narrow streams in which the whole surface slope is affected. This brings about a cross current which has the power of laterally deflecting particles. This is due to the surface slope and in rivers is confined to the bank. The silt for the remaining width is entirely controlled by the strong forward current, across which nothing may pass. The writer does not appeal to laboratory experiments for his proofs, but has shown by a few examples in other papers that the principal is supported by large river movements, sometimes involving square miles of shoal. These shoals appear and melt away in step with erosions above them, and have the power of deflecting even the main stream. They all obey the same law which the writer has enunciated. He has, therefore, considered it fully established. The theory is illustrated in Fig. 3 which is believed to be drawn strictly to principle.

In this diagram there is erosion from A to B on the right side. At A the stream comes into the bank, having deposited its load of silt on that side in the shoal above. The water A is,

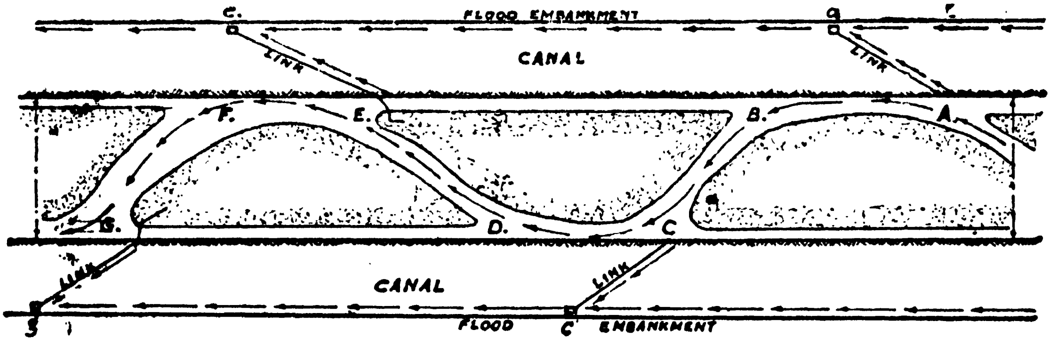


Fig. 3

therefore, clear. From A to B erosion is active, and the load of silt is consequently increasing. At B it has grown so great that the river is no longer able to carry it forward, and a shoal begins to from, diverting the stream across to C. From C the process. as at A, is repeated, only this time on the left side. The erosion from A to B is responsible for the shoal BCDE and the erosion from C to D for the next shoal on the left.

It will now be shown how, on this theory the favourable points for inundation canal heads are determined. The favourable non-silting points are these just above A, C and E. The canal off-takes may be placed a little higher up in the creek which forms the out fall above one of these points. Creeks are favoured since they are usually more constant than the main stream, being subject to milder attacks. They also allow of the building of groynes or dams and thereby the off-take may be carried higher up towards the head of the creek itself. Inundation canals invariably work upstream for better command and such canalizing of creeks is very usefull.

(*b*) **River control.**

In controlling rivers for inundation canals, there being no stone available, use of the still-water pocket principle must be adopted to the greatest extent. The principle is illustrated at every Bell's Bund built for protection and training at important works of the province. It need not, therefore, be described here. Water to be held by earth embankments must be brought to a complete stand-still, and this is effected by the formation of the still-water-pocket. Such a pocket is formed when a creek is completely closed by a dam. This, however, is not always, passible, and a compromise is got by means of a groyne. The groyne is a dam which has the canal bank or the high main land for one flank while the other flank is in the open river bed. Whenever possible, the nose of the floating flank is placed on an island, or is carried up to a high shoal, across the main creek, working upstream at the same time so as to create a pocket. Sometimes it cannot be carried across the stream but is finished off to from a pocket on one side of it, The nose is protected by stakes and brush-wood, and spurs may be added to throw off longitudinal flow along the groyne. These groynes are not only helpful in securing the early supplies but are strong factors in keeping the canal heads clear of silt. With experience they may often be maintained even through the floods. Whether or not throughout the season, they form undoubtedly a feature of inundation canals which should be recognized. A head with some sort of groyne may nearly always be found and by it much silt clearance may be avoided. Another very important advantage is the control of the river which groynes afford. Developmets of down side creeks are prevented, and the main stream is kept away. These are some of the leading features of control.

(*c*) **Policy combining river behaviour and control.**

The usual form of river is shown by the meandering course A, B, C, D, E, G, within the normal flood embankments, which of course, will not be so regular, but will be more or less constant, marking the limits within which the river may wander. Beyond these limits the flood embankment and main lines of canal should be built as at a-e and c-g. One of the features of policy was the combination of the canal and flood embankment by aligning the canal along the borrowpits of the embankment on the river side. Thus the cost would be that of the embankment only. The canal would have no river bank but will spill over the whole river front, and itself, would remain clear, since it would be the deepest channel which would form naturally along the embankment, even without the help of borrowpits. On the river edge it would have a high margin of spill which would keep it from returning to the river. The cross section at the time of construction would be as shown by the diagram Fig. 4 and in time with successive deposits of silt this would tend to grow into that shown by the dotted line.

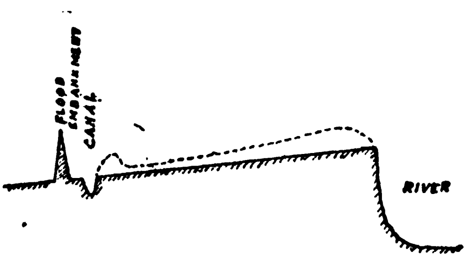


Fig. 4

This trunk canal would run the whole length of the district, more or less constant in capacity. It would feed other branches off-taking at the points A, E, C, G, etc. The trunk canal itself would be fed by links, run in form each of the favourable points A, C, E, as described above. They would be at angle so as to form pockets with the trunk canal, at the junction with which regulators and escapes would be built. These points would also be the heads of the branches. Each link would be under entire control at the junctions A, E, C & G, a regulator into the next compartment, and a head of a branch canal, being built at each point. By means of any of these regulators, the flow in the compartment above a link would be brought under entire control, even to the extent of causing the link to work back to the river by heading up. This control would regulate silt deposits in the upper compartment to some extent. These compartments by arresting silt, would grow valuable for *Rabi* Irrigation, while behind the embankment, *Kharif* crops would be sown. The system would be unique in making all the heads on the river available for service. The trunk canal also would be impregnable, *i. e.*, it would be impossible for river erosion to sever it. Such action frequently ruins an inundation canal. Hear, however, should erosion threaten to eat into the canal, it would also threaten the embankment ; and as this must at all times be maintained for the preservation of the district, it would have to be built further back as a loop, and the canal would automatically follow it. Not only would existing favourable points of the river be made available, but short cuts to others, might at any time be made along with other links. The canal, as a trunk canal could make up shortage at one off-take with excess at another, thus making the supply more even. The large compartments would also act as dampers and would neutralize temporary shortages. Thus, this plan provides for every difficulty with which an inundation canal is faced. It aims at affording a steady, even and assured summer supply.

It may be noted that one of the objects of a trunk canal, such as that described above, is to obtain, if possible, not only a summer supply, but also a small winter one. This is often possible even with the present system of canals which are not in a position to tap all the available heads. By having a continuous trunk canal, all the available heads will be greatly enhanced.

8. Examination Questions.

1. Define Inundation Canals. Describe the function, location and the working of subsidiary heads in Inundation Canals.

2 Describe briefly the Layout and the points you will keep in view for determining the suitable main head for the head reach of an inundation canal.

3. Why are escapes necessary at the end of a head reach of a canal ? Why cannot they be done away with by heading up supply at the regulator to top excess supply ?

4. Describe Claxton's system of aligning head reach of an Inundation Canal ?

5. What are Inundation Canals ? What are the requirements of a good site for head works of such canal ? What are the advantages and disadvantages of such a canal ? (P. U. 1954)

PART II

CANAL IRRIGATION

CHAPTER XIX

Discharge Observations and Regulations

1. Introduction.

Discharge denotes the volume of water passing a section of a stream in a unit of time. It is measured in cubic feet per second, commonly called cusecs in India.

For successful irrigation operations the correct knowledge of the discharge in the rivers feeding the canals is very essential. The supplies available will be distributed into the canals taking off from the river. The discharge sites along the great rivers of the Punjab are fixed, where daily discharges are taken.

Similarly correct discharge observations are necessary along the canals and branches to ensure equitable distribution of supplies in the distributing channels called distributaries and minors.

The methods commonly adopted for the river and the canal discharge observations are described in this chapter.

2. Objects and Applications of Discharge Observations.

The objects of discharge observations and uses to which the data so called are put, are as below :—

(*i*) **Statistical.** By collecting daily discharge data for a sufficient number of years and scrutinizing it, it is possible to forecast the normal supplies for basing the irrigation projects on and also the highest supplies for fixing the magnitude of works and storage projects. Accuracy in observation of discharges at all stages is therefore needed. The Sind-Punjab dispute shows the importance of collecting discharge data for inter-provincial distribution of river supplies. The importance of collecting, in the first instance, correct data for highest discharges is illustrated by the subsequent additions at a comparatively higher cost required at the Panjnad Headworks and Kalabagh Railway Bridge.

(*ii*) **Regulation and distribution.** Accurate measurement of river supplies for distribution among partners of an irrigation system and of canals, branches distributaries and outlets for equitable distribution is an essential feature of the successful running of canal system.

(*iii*) **Scientific and hydraulic investigations.** For this, accurate observations of discharges both in the field and laboratory are necessary. Of the numerous investigations in connections with the advance of hydraulic and irrigation engineering some are estimation of absorption losses in channels, determination of co-efficients of roughness or various types of channels both lined and unlined, sluice gate co-efficient studies of the conditions of flow in streams and investigations of hydraulic laws.

3. (*a*) River Gauging Site in Plains.

The site chosen must so far as possible comply with the following specifieations :—

(*i*) It must not be located where the river is too wide and shallow, nor too tight and deep. In the former case it results in shoals and slack water, while in the latter during high floods it may result in excessive and dangerous velocities.

(*ii*) Widths may vary from a few hundred feet or even less during the winter season, to several thousand feet in the summer months, but the site is to be such that all times the section is reasonably uniform. A site where the section is very deep on one side and very shallow on the other, or shows some such departure from normality at different times of the year must be avoided.

(*iii*) The discharge section line as for as possible is to be located in a straight reach of the river.

(*iv*) It is preferable to locate the site for obvious reasons where the river is flowing in one channel. But if this cannot be avoided, it means that the multiple section line will require observations to be taken on each arm of the river in flow. On the other hand, a very large river may more conveniently be treated at more than one arm of flow when in all probability each arm would be treated as an independent site with its own seperate observational staff and equipment.

(*v*) The rection line must be so orientated as to be as nearly as possible at a right angle to the direction of flow.

(*vi*) A site must never be located upstream of a confluence and sufficiently near to it, as to be affected by heading up at the discharge site due to increased flow in the channel into which the flow through the discharge site falls resulting in higher and false levels. Similarly, a discharge site should never be located above a Weir or Barrage, within the effect of the Pond formed there by the heading up of the river.

(*b*) **River gauging site in hills.**

In this case the same arguments apply as for plains gauging sites, but generally speaking sites in the hills are more difficult to select and all the desirable features cannot be found ; but it essential nevertheless, to select a site which is :—

(*i*) Free from projections from either the bed or the sides.

(*ii*) The site should not be too close to any bridge or fall. Any incoming torrent upstream or downstream of the discharge section should be as for away as possible and not nearer than 500 feet.

(*iii*) Straight for at least 500 feet, above the section line.

(*iv*) Accessible in all seasons.

(*c*) **Artificial channels (canals).**

Such sites generally are easy of selection, but nevertheless must be :—

(*i*) In a straight reach of length net less than 10 times the mean width of the channel.

(*ii*) Not in close proximity to any fall, bridge or work of any kind likely to cause obstruction to smooth flow and consequent eddy action.

(*iii*) If the channel is not pitched, the existing channel must be kept to a clean section free from fallen berms. For a permanent discharge site, the side slopes and preferably the bed also should be pitched over a length equal to at least 200 feet.

4. Permanent and Temporary Guages.

(*a*) Permanent gauges may take the form of :—

(*i*) Graduations directly engraved into rock or masonry.

(*ii*) F.I. Gauges fixed to an angle iron, old rail, or masonry-pillar and given secure foundations.

(*iii*) E.I. Gauges fixed to masonry structure such as Railway Bridge, Piers etc.

It is most important that the zero R.L. of a permanent gauge, when once fixed is on no account changed.

(*b*) A temporary gauge does not require such secure foundations and may be used as on extention gauge to a permanent gauge which has been temporarily left high and dry or in connection with a discharge site the section line of which is not in itself a permanent fixture. When temporary gauges are required to be fixed not less than 500 feet upstream and 500 feet downstream of a discharge site section line, for the purpose of recording water surface slope at that discharge section line, they, must be graduated to read to 0·01 foot in order that surface slope and ultimately Kutter's constants may be sufficiently accurately determined.

(*c*) All gauges, whether permanent or temporary, must have their zeros fixed relative to a permanent bench mark which, if not existing near enough to the gauge or gauges to be set from it, must separately be located and provided.

5. Methods of Stream Gauging.

The measurement of discharge is made by the following methods :—

(*i*) Velocity area method, (*ii*) Gauge discharge curve method (discharge table), (*iii*) Weir method (Meters, Chapter X.)

The above three methods are the most commonly employed for discharge observations and shall be described in detail hereafter. In additon, the following methods are also used, through they will, interest engineers only.

(*i*) **Pilot tubes.** This method is based on the principle that the dynamic or impact pressure of a current moving with velocity V, and striking a glass tube with a nozzle, pointing upstream, makes the water rise inside the tube by an amount 'h' above the surface and that the velocity is represented by the equation $V=G\sqrt{2gh}$. The value of the co-efficient G can be determined by calibration of the tube.

(*ii*) **By chemical means.** This method is based on the principle that if a weight w ℔ of a chemical is added each second to a stream discharging Q cusecs and after a thorough mixture if a ℔ of the chemical is contained in n ℔s of water then $\frac{w}{62\cdot5\,Q}=\frac{1}{n}$ or $Q=\frac{nw}{62\cdot5}$. This method requires special apparatus and arrangements and is not easily applicable under conditions prevailing in the field.

(*iii*) **By venturimeter.** This is applicable only to discharges capable of being passed through a pipe and its theory is described in all text books on hydraulics.

(*iv*) **By a travelling screen.** In this method a light varnished canvas screen fixed in a rigid frame is hung from a wheeled carriage and is made to move with the water in a specially prepared rectangular channel. The velocity with which it moves after being corrected for leakages from sides is taken as the mean velocity of the stream.

6. Velocity Area Method.

This method involves measurements of area and velocity. It being impossible to determine the mean velocity for the entire section of a channel in one single measurement, the area is divided into vertical strips of suitable size and mean velocity for each component strip is observed, the total of the discharges for all strips being the discharge of the channel. In other words the cross section of a river is observed by soundings and the mean local velocity perpendicular to the cross section is observed at as many points as possible. The discharge at these points are denoted by the product of the mean velocity and depth. If now a curve is plotted with the water surface width as the base and the discharge at each point represented by product of depth and velocity as the ordinate. The total discharge is represented by the area of the enclosing polygon. The area of the polygon may be determined either by taking straight avarages or by Weddles and Simpson's Rules.

(A) **Measurement of area.** The area of cross section of a stream involves two elements *viz.*, the horlzontal width and vertical depth which are both measured as below :—

(*i*) **Width.** If the discharge site is located at a bridge or other masonry work. the measurement of width is a simple affair. In the case of an open channel of width upto about 1,000′ a wire is stretched across the channel and segments are marked on it by means of pendants. At Rupar on the Sutlej a weir has been stretched at a width of 1700 feet of the river. The necessary precaution in such cases is to make a suitable allowance for the sag. Alternatively, the pendants should be fixed by a theodolite. In bigger channels across which wires cannot be stretched, the segments are marked by pivot-point method which is based on the principle of similar triangles to be described hereafter.

(*ii*) **Depth.** The points where depths have to be measured are located as above and in deep and fast stream in which wading is not possible, boats have to be employed for reaching such sounding points. Depth is measured either by direct reading on a sounding rod or log line or by indirect methods.

(*a*) **Direct method.** (*i*) A sounding rod consists of an oval wooden rod with E.I. gauge on it, flat iron of 2″×½″ size or bamboos 2″ to 3″ diameter all graduated in tenths of feet and with iron discs of 4″ to 6″ diameter at bottom to prevent sinking in bed. Bamboo rods have been used even up to 30 feet depths in low velocities. The depths are measured at downstream ends thus omitting effects of afflux due to velocity.

(*ii*) **Lead line.** In higher velocities and depths, observations are made with a weighted log line. Log lines consist generally of copper cores covered over with hemp. Such a line does not shrink when wet nor stretches under weight and remains free from knots. Alternatively, when such metallic lines are not available, weighted manilla rope can be used, but with due precautions regarding stretching and wetting it before use. Constant checking during use is essential.

The lead or sounding weight is generally of the shape of frustum of a cone. The weight varies from 10 lbs to 56 lbs, depending upon depth and velocity encountered. At Kalabagh even a weight of 56 lbs has to be used with care and experience to measure depths accurately.

It requires an experienced hand to observe depths properly with a log line. The depth of water surface below a reference point is first measured and marked on the line. The weight is then swung and released a little upstream of the observer. After touching the bed it trails down and the rope or line is pulled, till the weight is vertically under the observer when the lenght of the rope is marked against the reference point. The distânce between the two points gives the

Haigh's Depth Meter. Scale 3/8″=1″

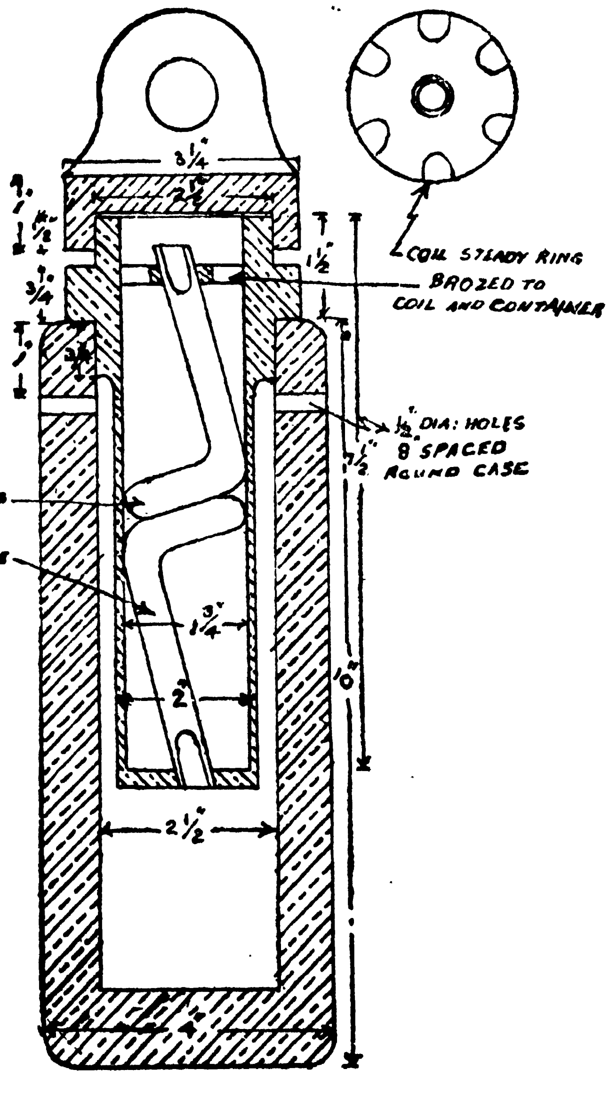


Fig. 1.

depth at the point. A difference of 6″ between two consecutive readings is ignored. For large differences observations have to be repeated.

(*b*) **Indirect method.** In deep and rapid rivers the observations of depth with log line requires skill, experience and preseverance and the accuracy of results can to some extent be questioned. The problem of measuring depths with ease, rapidity and accuracy has not yet been satisfactorily solved and attempts are being made to apply some indirect type of sounders for the purpose of which the following are worth mentioning.

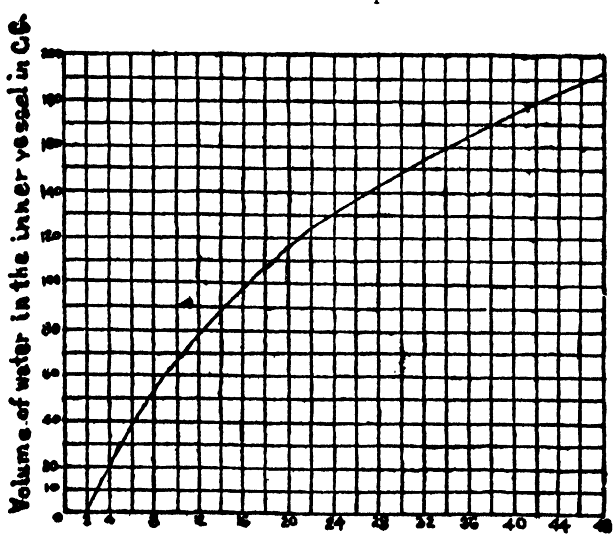
(*i*) **Haigh depth meter.** This is an ingenious divice recently invented by F. F. Haigh C.I.E., Chief Engineer, Punjab Irrigation and is based on Boyle's law *viz.*, PV is constant and the observations so far made give remarkably consistent results. It consists of a tube coil in a cylinder container in which water under pressure is entrapped by compressing the volume of air in it. The volume of water entrapped is proportional to the depth upto which the meter is immersed and the depth is read from a calibration curve Fig. 2 prepared by actual observations before it is brought into use. The details of the meter are as shown in Fig. 1. The instrument besides being used in discharge observations will be of immense value in observing scour depths at salient points of Headworks, training works and railway bridges.

(*ii*) **Kevin tubes.** These have been tried in Sind and have not proved to give consistent and accurate results.

(*iii*) **Echoe Sounder.** Suggestion has been made by Montagu, to explore the possibility of using such sounders as are used on ships for observing depths. This needs further investigation.

(*B*) **Measurements of velocity.** The flow being turbulent in opon channels, three to four observations of velocity should give the correct average velocity at the points of section of a channel.

Calibration Curve for Depth Meter



Depth in feet Fig. 2

Even though the flow in open channels is sinuous (turbulent) and subject to continuous pulsations, the distribution of velocities in a cross-section has some general characteristics. Attempts at determining location and magnitude of one single value for mean velocity for the entire cross-section of a channel have so far not been successful and therefore for discharge measurement mean velocities for component strips of area of cross-sections are observed. The vertical velocity curve showing velocities at various depths is accepted to be a parabola with a horizontal axis. Actual observations by various investigators in America and India have shown that the position of mean velocity in a vertical occurs under normal conditions of flow, at 6/10D. It has been proved mathematically also by Lindell from the parabolic shape of the vertical velocity curve that the mean velocity occurs at 0·6D and the maximum velocity at 0·15D. Further, it can be proved that the mean velocity equals the average of the velocities at 2·21D and 0·79D generally accepted as velocities at 2/10th and 8/10 depth which is the basis of the "two and eight tenths method". When measurements of velocities at known depth are not possible, surface in segments or even central surface velocities in a cross-section are measured and corrected into mean velocities by application of a "reduction co-efficient". Velocities are measured by the following methods :—

(*i*) **Surface floats.** These consist generally of wooden discs from 3″ to 6″ in diameter. Observations with such floats are liable to be influenced by wind, ripples and eddies on the surface of water and bends in the stream. The shape and weight should, therefore be such as to minimise the effect of the disturbing forces. To be safe against effects of wind, globular floats are better than flat discs. Simpler floats consisting of corked bottles, oranges, blocks of wood, or even floating debris are used when improved types are not available.

The floats register the surface velocity and a reduction co-efficient has to be applied to get the mean velocity. The exact value of this co-efficient is still a controversial matter. The value is stated to be greater for sandy bed rivers and minimum in beds of gravel. Experiments on different rivers have shown the value to be between 0·76 to 0·91. In Holland the value has been found to be 0·87 in sandy beds. In the Punjab Goodman collected a large number of observations and found the value to be 0·89, which is already in use. The method is not of an unqustioned accuracy and is applied in high floods only when observations by more definite and accurate methods are physically impossible.

Sometimes sub-surface or double floats, consisting of metallic hollow, cylindrical weighted submerged floats attached by means of a thin cord to surface floats were used. They have, however, become obsolete with the development of better methods.

(*ii*) **Rod or tube floats.** A rod float consits of a wooden rod, square or round, in section of width or diameter from 1″ to 2″ proportional to length for strength against rough field use. It is weighted at bottom by means of a lead weight for immersion to required depth. Hollow tubes of tin or galvanised iron were also used but have been discharged being unsuitable due to cracking and developing leaks.

That a rod float extending from surface to bottom of a channel shall represent very closely the mean velocity in the vertical, is an obvious inference. Cunningham in his Roorkee enperiments has, by assuming that the force acting on the rod at any point is proporional to the square of the difference between the velocities of the rod and water in contact with it and that the vertical velocity curve is a parabola, mathematically shown that V_r the velocity of the rod is equal to V_m the mean velocity when the immersed length of the rod is from 0·95 to 0·97 the depth of water, the exact value depending upon position of maximum velocity in the vertical. He takes mean value when length of immersion is 0·94D. Parker found that V_r equals V_m if length of immersion is 0·952D with maximum and minimum values of ·97D and 0·91D. In the Punjab Irrigation Department, rods with length of immersion of 0·94D are used and they ere made of full length and weighted so as to have ·06 of length out of water.

Parker is of opinion that rod floats give too high a result as compared with weir or current meter.

The method of rod-floats is applied only for canal obsevations and is rarely, if ever, applied to rivers. The rods form a simple inexpensive and direct method of measuring velocity easily followed by untrained men and are not affected by the nearness of boundaries of channel section. They have, however, defects like requiring a larger party for observation, difficulty to control their course, motion along a pulsation without neutralising effects of changes in velocity observation of more than one cross section and possibility of fouling bed by ·94D rods and a consequent tendency for use of smaller rods leading to over-measurement of discharge. This over-estimation of observed discharge by the rod-floats is an admitted fact and in order to solve this difficulty the rods described below have been introduced as alternatives.

(*iii*) **Lacey tabular rods.** This type of rod consists of a hollow closed tube working in another hollow tube, closed and weighted at the bottom so as to float in water with only the top sticking out obout two inches in length. The length can be adjusted by pulling out the inner tube and the depth of immersion remains upto the same point from top, the increased length of the outer hollow tube getting filled up with water. These rods are available in sets which can serve for depths from 1·0′ to 10·0′.

Lacey has mathematically proved the relation $V_m=2V_2-V_1$ where V_m=the mean velocity or velocity at 0·6D and V_2 for a rod of 0·8D immersion and V_1=the velocity for a rod of ·4D immersion. He has, therefore, recmmended the simultaneous use of two iods immersed to depth ·4D and ·8D and observations and comparison with current meter velocities at ·6D has proved this method to be quite accurate. It, however, involves too elaborate a system and excessive time and suggestions to substitute a single ·8D rod have been made as described hereafter.

(*iv*) **Current meters.** Current meters are of two kinds *viz.*, Cup meters, consisting of a wheel with conical cups revolving on a vertical axis and screw or propeller meters, consisting of vanes revolving on a horizontal axis. To the first kind belong the Price (Fig. 3). Ellis and Watt meters and to the second, Fteley and Stearns, Haskell, Amsler (Fig. 4) Stobroni, Richard and Ott. Price meters known as Gurley meters after the manufacturers, W and L.E. Gurley are now almost universally employed in America, Egypt and India.

7. Price or Gurley Meter.

The equipment consists of the following five principal parts Fig. 3.

1. **The head.** As shown in the sketch the head is yokeshaped, carrying a wheel of 5″ diameter with six conical cups, '2' attached to a horizontal frame. This wheel revolves in a counter-clock-wise direction on a shaft, '4' resting on a pivot point, '5, 6' at the lower end, and passing into cummutator box '11' at the upper end in which arrangements to indicate electrically the number of revolutions by single or penta strokes are provided. In Gurley No. 622 single and penta arrangements are provided in the same box and in meter No. 623 separate boxes are provided. Meter No. 623 is in use in the Punjab generally.

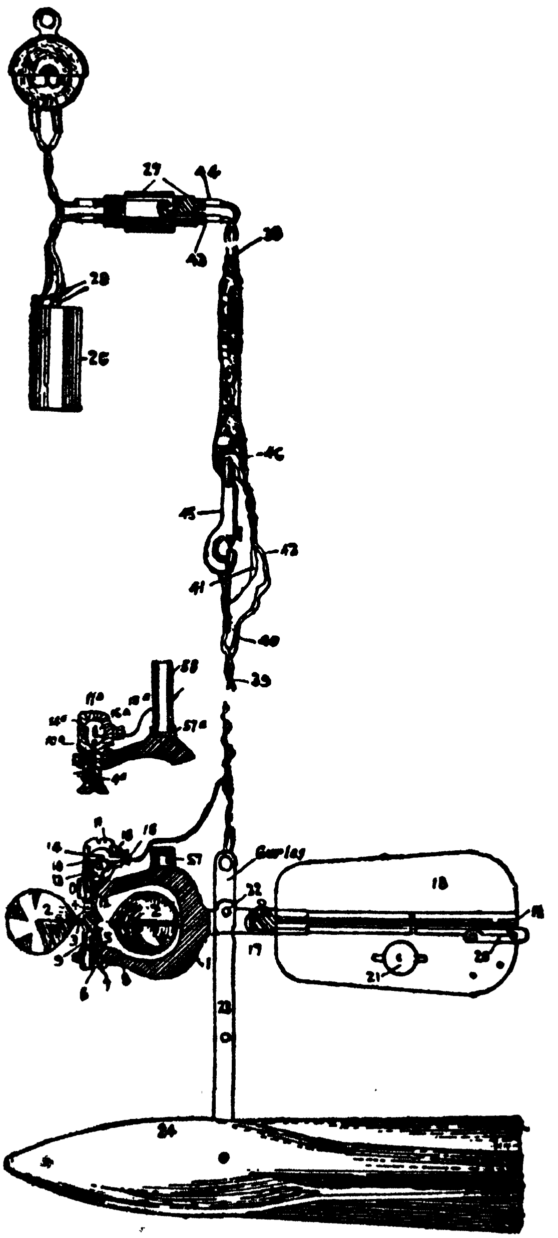


Fig. 3

2. **The tail.** The tail '18' consists of a stem on which there are two vanes, one being rigidly fixed and the other capable of being pulled out by sliding in a groove. An adjustable weight in a slot is provided on one of the vanes. The functions of the tail are to balance the head and keep the meter parallel to the current.

3. **The hanger.** The hanger '23' consist of a thin steel bar, passing through the frame of the meter, with torpedo-shaped weight at the bottom, the function of which is to keep the meter in plumb when hung by a cable.

4. **The recording or indicating devise.** This consists of a telephone reciever '27' with necessary battery '26' equipment and ticks are conveyed to the observer, on closing of circuit after the single or penta number of revolutions according to the commutator box in use.

5. **Suspending device.** This consists of a rod, cable or rack-and-pinion and provides for lowering the meter and weight (when used) in water. The rod is used in shallow streams of wadeable depth. The cable is used from boats and bridges, but it has the defect of bowing and being deflected from the point of observation in high velocities. In order to ensure the location of the meter being truly at the point and depth of observation, a rack-and-pinion device is used in practice. This consists of a rigid rack rod with teeth which works by means of a toothed gear and can be made to stand at any depth in the water. Details of such an appliance are shown in Fig. 5. The rack rod is graduated by reducing each foot to 6/10ths as the velocities are

measured at 0·6D. At present velocities upto depths of 20′ can be easily measured by rack-and-pinion arrangement and designs for measuring velocities at greater depths are in hand.

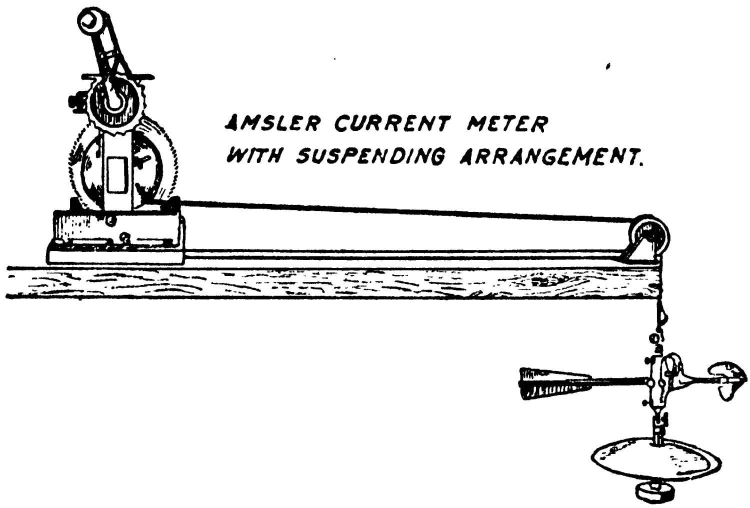


Fig. 4

RACK-AND-PINION
Syspension arragement for current meter.

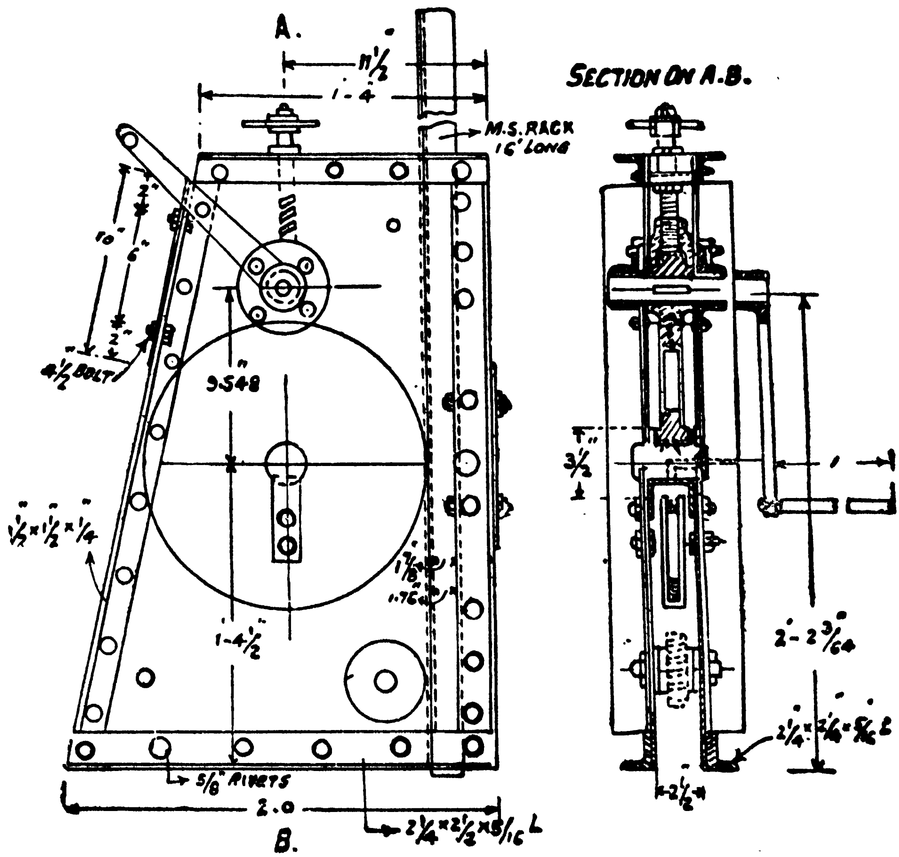


Fig. 5

The Gurley meters possess reliability in service under most trying conditions in the field, simplicity in design, rigidity in construction, adaptability for use in all sizes of streams, compact size and are now used to the exclusion of all other meters in the Punjab.

In using a meter revolutions are recorded and the relation between the revolutions and the velocity is determined by rating the meter. The processs of rating consists in towing a meter through still water at known uniform speeds and noting time, number of revolutions and distance. Revolutions per second and velocity in feet per second are calculated. For each meter under rating eight such observations are taken at different speeds, and the relation between velocity and revolutions per second if determined by either plotting the curve or by the method of least squares. The relation has a square line equation of the form $V=\alpha N+\beta$, where V is velocity feet per second, N is revolutions per second and β is the intercept of the straight line on velocity axis representing minimum velocity at which the wheel starts revolving. A rating table for each meter in use is prepared from this. For Gurley meters in use in the Punjab, α lies between 2·18 to 2·38 and β between 0·0 to 0·05. An up-to-date arrangement for rating meters consisting of a masonry tank 400 ft. long 8 ft. width and 7 ft. deep with an electric trolly fitted with distance, time and revolutions recording devices, exists at Lahore.

Rating is carried out in still water and at the temprature of the season and it will not be out of place to mention briefly the effect of turbulent flow and changes in temperature on the performance of meters. Turbulence postulates variations in both magnitude and direction of velocity in pulsations, durations of which depend upon degree of turbulance. Investigations in America have shown that cup meters over-register and screw-meters under-register velocities and some authorities have recommended the mean of observations by both kinds of meters to be taken as the correct discharge. Regarding effect of temperature, Blench found by analysing the discharge data at the head of the L.J. Canal that a Price meter over-registers by one percent for every 6° C fall in temperature. This may be attributed to changes in velocity observations.

The mean velocity in a vertical is observed by a current meter in one of the following ways :—

(*i*) **Six tenths method.** The meter is held at 6/10th depth from surface and the observed velocity is taken as the mean velocity. This method is invariably employed in the Punjab, except in conditions of turbulent flow.

(*ii*) **Vertical velocity curve method.** This consists in measuring velocities at a point 0·5 ft. below surface every tenth of depth and a point nearest possible to bed and plotting a curve with velocities as abscissae and corresponding depths as ordinates. The area of this curve divided by the depth gives the mean velocity. This method is employed where, due to turbulence, the position of mean velocity does not occur at 0·6 D as can be the case in some hill sits only.

(*iii*) **Surface velocity method.** The velocity is measured at 0·5 ft. or 1·0 ft. below surface depending upon velocity and turbulence at the surface. Mean velocity is obtained by applying a reduction factor. This method is employed only when (*i*) and, (*ii*) are physically impossible to apply, say in floods in hill torrents.

(*iv*) **Two point or two-and-eight tenth method.** Velocities are obtained at two tenth and eight-tenth depth and average of the two gives the mean velocity of the vertical.

(*v*) **Summation integration method.** In this method the meter is lowered to the bed and than raised to surface at a uniform rate. The reading of the meter, which represents the mechanical average of the rates at which it turns during its journeys up and down is assumed to correspond to the mean velocity. Cup type meters are not suited for this method which is never used in the Punjab.

8. Precautions for Upkeep of Current Meters.

(*a*) Although the current meter will stand considerably hard usage, it needs carefull handling and attention to ensure its proper working. In this connection the following instructions should be carefully observed :—

1. Be sure that the set screws are all tightened up before putting the meter in the water otherwise some of the parts may be lost.

2. Before beginning a measurement loosen the raising nut and see that the meter runs freely. Spin the meter cups occasionally during a measurement to see that they are running freely

only by blowing and not by hand.

3. See that the meter is swivelling fully and freely on the supension rod or rack. If cable suspension is used see that the weights are free to following the direction of the current.

4. If any apparent inconsistency in the results of an observation throws doubt on its accuracy, investigate the cause at once. Grass may be wound round the cup shaft ; the cups may be tilted by tension on the contact wire the channel may be obstructed immediately above the meter ; the meter may be in a hole ; or the cups may be bent so as to come in contact with the yoke.

5. After a measurement, it is absolutely necessary to pour out any water that may have collected in the commutator box, to clean and oil the bearing (in order to prevent rust) and to inspect the pivot point. The overseer must do this personally, using only the oil supplied with the meter by the Discharge Division.

6. When the meter is not in use, the cups must never be permitted to ride on the pivot-point.

7. Always see that the Lock Nut, on the Pivot-point is screwed firmly against the Frame Nut, so that it will stay in place and carry the cups properly.

8. In measuring low velocites, be sure that the meter is in a horizontal position. If it has a tendency to tip, the Balance Weight on the Tail is to be moved forward or backward as necessary to give a horizontal balance.

9. Avoid taking measurements in velocities of less than 0·5 foot per second, because the accuracy of the meter diminishes as zero velocity is approached.

10. Should it be necessary to take measurements in high velocities by cable suspension instead of rack-and-pinion suspension, it is essential to use sufficient weight and also a stay-line in very high velocities, so that the meter is vertically suspended below the water surface as nearly as possible.

11. When the meter is not in use, disconnect the meter line from the battery, so that it will not become exhausted.

12. Do not strike the telephone receiver, as a heavy jar will injure the receiver.

13. Care must be taken not to short-circuit the dry battery when the meter is not in use. To avoid this, the poles may be wound with insulating tape.

(*b*) **Current Meter Outfit.**

1. Meter itself, with its rating table. 2. Telephone connected up with insulated wire in circuit, with dry cell and connecting plugs. 3. Oil-can filled with spindle oil. 4. Small screw-driver. 5. Cable for supporting the meter. 6. Hanger. 7. Hanger screw. 8. Small tin-box containing 2 spare set-screws. 9. Lead weight pin-screw. 10. Commutator box screw-driver. 11. Penta Commutator box (Model 623 only). 12. Cotton square for cleaning.

When sending meters for re-rating all the equipment listed above must be sent along with it.

9. River Discharges.

The following procedure is adopted for observation of river discharges :—

(*a*) **Selection of discharge site.**

The essential feature for the accurate measurement of a discharge at a site is that the flow should be streamline and regular, devoid of errors caused by irregularities in the motion of water. This postulates the following *deciderata* in selecting the site :—

(*i*) It should be in a straight reach of the river with regular streamline flow, undisturbed by bends leading to unequal distribution of velocities, cross flow and eddies, which affect accuracy of measurement.

(*ii*) The section should be regular and deep with least tendency for wide spills even at high discharges.

(*iii*) The section should have a reasonably stable bed and sides and be amenable to a regular discharge relationship as far as possible.

(*iv*) It should be far removed from confluences and structures causing vitiation by back-water heading up.

(*v*) The site should be easily accessible and free from cracks.

(*vi*) In case of hill sites, the stream should be straight at least for 500′, site should be free from projections in bed and from sides and away from disturbing influences of rapids, falls, and confluence of torrent.

(*b*) **Segmentation or spacing of sounding points.**

The distance between sounding points depends upon the width of the stream profile of the bed and the accuracy desired. In short, the greater the number of sounding points the more accurate the area measured and discharge observed, as there are greater chances of fluctuations in velocities being evened out. The following rules are used in practice for segmentation of river discharge section lines :—

(*i*) The number of segments should not be less than fifteen.

(*ii*) For river width in excess of 2500 ft., segmentation is to be fixed by the Executive Engineer, keeping in view the prevailing cross section, distribution of discharge across the cross section, equipment available and time involved in completing an observation.

(*iii*) For river width from 1500′ to 2500′ segments should be 100′ apart over the portion of section passing 75 percent of total discharge and 200′ apart for the remainder.

(*iv*) For river widths 750′ to 1500′ segments are to be 50′ apart.

(*v*) For smaller widths segments to be 40′, 30′, 20′ or less subject to condition (*i*).

The principle to be followed is that in the portion of the section where the discharge is concentrated, the segments must be nearer and farther apart in the slack portion so as to have segments of more or less equal discharges.

(*c*) **Procedure and equipment.**

At each discharge site the following equipment is required :—

(*i*) Boat—This is required for reaching the points for observation of depths and velocities. A 35′ long Sukkar type boat is in use in the Punjab.

(*ii*) Suitable anchor with about 250′ long 1″ diamether manilla rope.

(*iii*) Sounding rods of line and weight according to conditions of site.

(*iv*) Current meter with accessories complete in all respects including a swivel. At each site a check meter is also kept and the local meter in use is checked with this once a fortnight to make sure that it works satisfactorily under conditions in the field. A bracket for handing check meter is also provided.

(*v*) Rack-and-pinion suspension arrangemet, described above.

(*vi*) Stop-watch for use with meter. An extra-check stop-watch is also kept at each site.

(*vii*) A torpedo float with cotton cord for indicating direction of current.

(*viii*) Pocket sextant, with which the angle made by the current with the section line as indicated by the float is read. In order to work out the component of the velocity, normal to the section line, the observed velocities are multiplied by the sine of this angle and a table for such corrections is provided at each site.

(*ix*) A levelling instrument for checking reduced levels of gauges and a theodolite for reading angles.

(*x*) The cross section line is marked by three flags, 200′ apart on each bank. In case of swift currents, the boat drifts down and drift flages apart are fixed for a requisite distance downstream of the X section line.

(*xi*) In case of narrow streams, say upto a width of 1000′ as the case with all the Punjab rivers in winter, the section line is marked across the stream by a wire, on which pendants are hung at sounding points Fig. 6. The boat is held by another wire, stretched a little upstream so as to have the rack-and-pinion under the pendants.

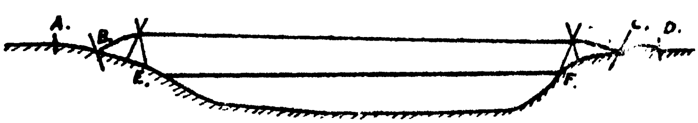


Fig. 6

(*xii*) When streams are too wide for stretching a wire rope, the sounding points are located by pivot method which is illustrated in Fig. 7.

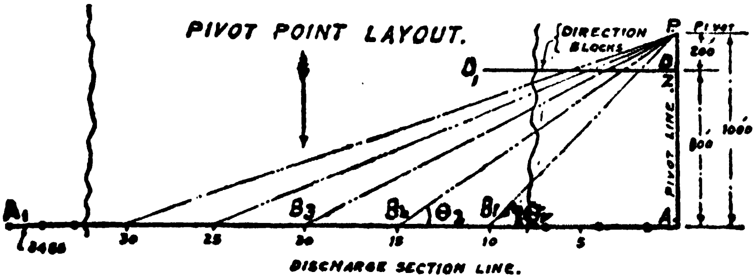


Fig. 7

From point A on the cross section line on the bank, a line AP at right angles is drawn and from a point D on it, a line parallel to the section line is drawn. The ratio PD:PA is generally kept 1:5 and the length of AP it 1000′ or upto half the width of the river. On line DD, point say 20′ apart are marked and rays from P passing through these points intersect the section line at B_1, B_2, B_3 etc., each 100′ apart as is evident from the principle of similar triangles. The point P is called the pivot point. For accurate daily work the position of the flags along the section line, pivot point and direction lines are marked by cement concrete blocks with holes for flags and the arrangement is thus marked semi-permanently. The layout can be checked by the theodolite, as angles subtended by B_1P, B_2P etc., with AP should have tangents increasing in ratio of segments *e. g.* with 100′ segment and 500′ length of AP ; tangent $\theta = \frac{100}{500}$; tangent $\theta_2 = \frac{200}{500}$, tangent $\theta_3 = \frac{300}{500}$ etc. For rivers wider than 2,000′, pivot point layouts are made on both banks, for work by two different parties from the two banks. Methods of pivot point layout are shown in Fig. 8.

In case of physical difficulties like uneven ground, the length and angle of the pivot-point line can be varied and the observer can easily locate the sounding points with theodolite by his knowledge of trigonometry. The section line is marked by R. Ds. and the water edges go on changing with season.

Various Pivot Point Methods of Layout.

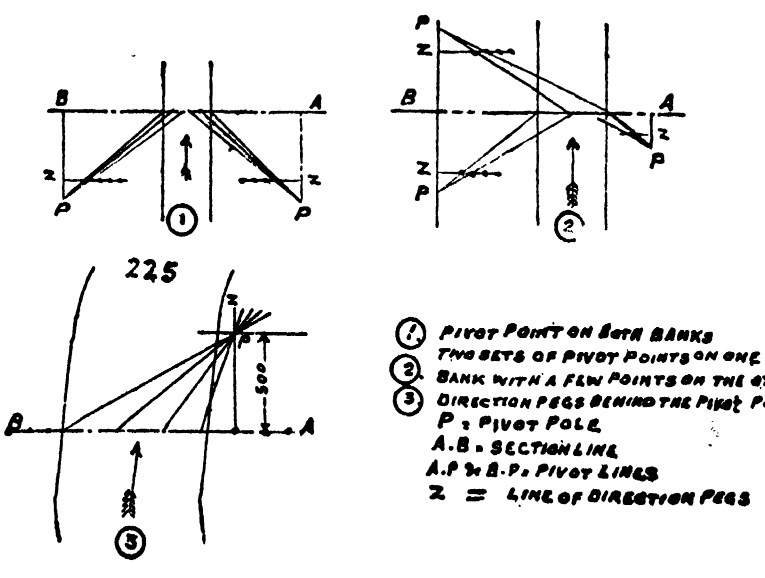


Fig 8.

In slack velocities and low depths the boat is kept in position by a boatman standing in water and in high velocites or greater depths the boat is kept in position by an anchor. Upto about 5′ per second velecities, the boat can be rowed up, from station to station. In higher velocities while pulling out the anchor it loses headway and has to be brought to bank or in slacker water for being rowed up. This detouring involves a lot of time. To reduce the number of detours the anchor is thrown about 300′ upstream of the section line by means of a subsidiary flag line or pivot-point arrangement, and at every station a little headway is lost, enabling about 3 or 4 stations to be done at a time without detour in velocities as 8 to 10 ft. per second. The difficulty and risk involved in pulling out the anchor under such conditions are appreciable, and the latest method of securing the anchor shown in Fig. 9 is the safest, though it involves introduction of some drift. Trained crew can, however, cope with even the most difficult ccnditions met with at some sites during floods. Under extreme conditions met with at some sites provision of power launches may be necessary which shall facilitate the process of observaion.

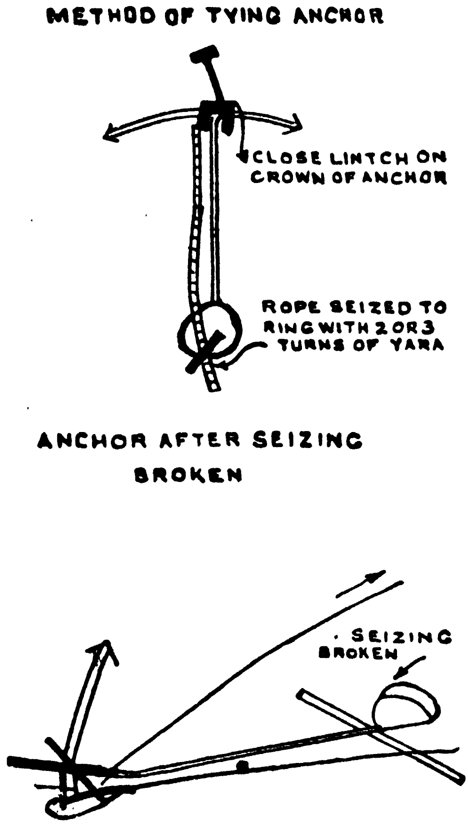


Fig. 9

(*xiii*) Having divided the section into segments, the first procedure of discharge observations consists in taking the boat to a station by wire rope or pivot-point method as is in use, at site and observing the depth. The current meter is then lowered, by rack-and-pinion to 0·6 depth and velocity is recorded. The angle of the current with the section line is then read by means of the torpedo float and pocket sextent. These observations are noted in a discharge note-book, a specimen copy of which is shown in Appendix 1 which shows full procedure for calculating the discharge. The velocity at each station is modified for its angle and the product $d \times V_m$ gives the ordinate of the discharge curve at each station. The total discharge is found by straight averaging. Further, the wetted perimeter is also calculated and the slope observed for working out the value of Kutter's N. This is taken as the mean of the slopes on the two banks. Details of meters and other equipment used, along with calculations for discharge and Kutter's N as prepared by observers at each site daily are shown in the form, a specimen of which is given in Appendix 1. The method of recording is clear, self evident and foolproof and the data collected can be used to check the accuracy of fieldwork from day to day.

In the absence of boats, improvised arrangements consisting of bull skius and *charpoys*, called *khatnaus* are prepared for discharge observation in hills.

Another approximate method of estimating a flood discharge in the absence of suitable equipment, is to observe the slope and flood level at the time. A cross section is observed at lower supplies and from it the Cross Section at the highest flood deduced. Having determined the slopes and hydraulic mean depth R, the mean velocity is determined by one of the following emperical formulae :—

(*a*) Chezy's formula $V = G\sqrt{RS}$; where the value of G is determined either by Kutter' or Bazin's formula given in all books on hydraulics.

(*b*) Manning's formula $V = \frac{1 \cdot 4858}{N} R^{2/3} S^{1/2}$

(*c*) Lacey's formula $V = \frac{1 \cdot 3458}{N_a} R^{3/4} S^{1/2}$

10. River Discharge with Current Meter from a Bridge.

When there is a bridge near or at a discharge site, that bridge may be made use of if necessity accordingly dictates, either by holding a current meter directly from the bridge, or, and preferably, by holding a discharge boat at a sufficient distance downstream of the bridge by means of a rope traversed along the bridge. The distance marks for segments should be painted on the upstream and downstream railing or parapet. One flag should be fixed on each railing and the boat brought in line with the section line and these flags, the section line being marked by three large poles on each bank.

The section line must be far enough from the bridge so that the effect of the piers on the flow of water is reduced to a minimum. A distance of 500 feet should, however, be sufficient. From a bridge were there are railings, the handling of a boat by means of a rope is fairly easy, but from large 'N' Truss Railway bridges the handling of a boat is difficult. The rope used is in three pieces jointed in the form of the letter "Y", the tail piece should be 400 feet in length and the two arms about 160 feet each, and all three jonted to a steel ring of about 2" diameter. In the Fig. 10 when the boat is below point A, and an observation is in progress, the position of the three ropes will be as shown by the firm lines. When the observation is over, the Flagman on the bridge is given a signal from the boat by means of a green flag, to move to the next station. The two sets of boatman holding the two ropes at A and D, so that the final position of the ropes becomes as is shown by the dotted lines. When the boat has finally reached the new position and the rope DE has fully taken the strain, the boatman holding the rope at A which is now entirely slack and free of any load, now travel to the third station G, crossing the rope over the rope DE at E, so that is takes up the position GE. By this method which is repeated for each succeeding station, the boat is securely held all the time and autumatically comes to each succeeding station.

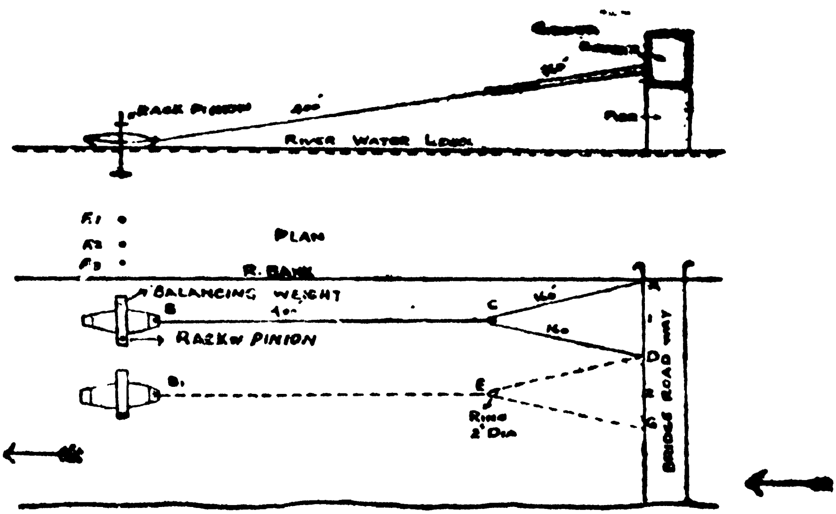


Fig. 10

In cases where a discharge is observed by directly lowering a current meter from a bridge ; each bay should be divided into a suitable number of segments, so that no observation point comes too near a pier or within the zone of disturbed water. The meter is to be held just below water surface so that the surface velocities only are recorded ; if attempts to hold the meter at 0·6D are made there will be no guarantee that the meter is really at 0·6D owing to excessive deflection of the supporting cable. Great care must be taken in measuring depths which in this case will be observed by means of a log line held from the bridge, and since tne point of suspension of the log line will be some distance from the water surface, excessive deflection from the vertical will result due to the perssure of the water in flow, and there is, therefore, the possibility of a considerable error being introduced in the observation of depths in this manner.

11. Canal Discharge Observation.

(*a*) The observations are divided in three classes of accuracy.

Class I. Depths measured at the upper and lower ends of the float run and at the centre section line and meaned.

Class II. Depths measured at the upper and lower ends of the float run only and meaned.

Class III. Depths measured at the central section only.

For all the three classes of observations, three satisfactory observations through each velocity station over the whole float run are to be taken, divergence not being permitted to exceed-half a segment width. The boat holding wire for velocity rod observation is to be shifted immediately above the upper float run wire, so that velocity rods may be let into water exactly oppsite the desired pendants. Placing a velocity rod in water is a job requiring previous practice as it is essential that it is released in such a way that it does not bob and spin. The best way to do this is to hold it tightly by the upper end between the fore-finger and thumb and lower it gently so that it is in its floating position tending to trail downstream ; then the rod should be released with a slight forward push so that it immediartely gets into the vertical and floats along steadily before it passes under the upper float run wire. The time taken for the run is then to be timed by an observer on the bank. He times the rod over the whole run, using a stop watch, watching it carefully all the while because if it suddenly fouls the bed or proceeds in jerks the observation has to be rejected even if the rod passes perfectly below all the desired pendants. If after recording three results one of them differs by more than 5% from the mean of the other two, it is to be rejected and the run repeated.

(*b*) The standard velocity rod is weighted to float at 0·94 of the depth *i.e.*, if the depth of water is 3·0 feet the rod to be used will be 3·0 feet long and it will float with $3{\cdot}0 \times 0{\cdot}94 = 2{\cdot}82$ feet submerged and the balance of 0·18 ft. above water surface. As, however, a velocity rod can only record the mean velocity between water surface and the bottom of the rod, it follows that no velocity rod can possibly read the mean velocity between water surface and the bed of the channel, and it is that mean velocity which is required. It may also be found that even if for example a 3·0 feet rod is used submerged by 2·82 feet, it will foul the bed at some point or other when the use of a shorter rod cannot be avoided. The application of a correction factor, therefore, becomes essential and correction may be made from the Francis relationship :—

$V_{mean} = V_{rod} \left(1{\cdot}012 - {\cdot}116 \sqrt{\frac{D-l}{D}}\right)$; where l = length of the submerged portion of the rod used.

For different ratios of velocity rod length to the mean depth of water along the path of a velocity rod, the correction factors to be applied in order to correct a rod velocity to mean velocity, by the above foumula are as follows :—

$$\frac{l}{D} = 0{\cdot}75,\ 0{\cdot}80,\ 0{\cdot}85,\ 0{\cdot}90,\ 0{\cdot}93,\ 0{\cdot}95,\ 0{\cdot}96,\ 0{\cdot}97,\ 0{\cdot}98,\ \&\ 0{\cdot}99$$

$$\frac{V_{mean}}{V_{rod}} = 0{\cdot}954,\ 0{\cdot}961,\ 3{\cdot}968,\ 0{\cdot}975,\ 0{\cdot}981,\ 0{\cdot}986,\ 3{\cdot}989,\ 0{\cdot}992,\ 0{\cdot}996\ \&\ 1{\cdot}00$$

In the example given above if it was necessary to use a 2·5 feet rod submerged to $2{\cdot}5 \times 0{\cdot}94 = 2{\cdot}35$ feet while D remaind as 3·0 feet then $\frac{l}{D} = {\cdot}783$ and the correction would be $= 0{\cdot}957$, meaning, the error by using the smaller rod and not correcting for it would be 4·5%. This error would be on the tight side, meaning that there was really 4·5% *less* water flowing in that particular segment than the observation showed.

The fundamental difference between a current meter observations at 0·6D and a velocity rod observations is that the current meter measures mean velocity between water surface and the bed over a period of time at a fixed point, while a velocity rod measures the mean velocity between water surface and the bottom of the rod only, not over a period of time but over a distance, and as the latter introduces two errors, a current meter is preferably to be used when the highest degree of accuracy is required, and necessarily to be used with the greatest degree of care wnen observations for the calibration of a meter flume or for the preparation of a discharge table are to be made.

(*c*) **Segmentation.**

The cross-section will be divided into 5 main segments, as follows :—

1 Central segment, w_3 ; 2 Side segments w_2 & w_4 ; 2 Slope segments, w_1 & w_5.

The width of the central segments $6w_3$ should be taken approximately at the full surface width, less 3 or 4 times the general depth, and should, if possible, be made a multiple of 6, so that w_3 may be a whole number. If this cannot be arranged, it should be multiple of 3.

The widths of the side segments w_2 and w_4 should be equal to each other, extending as near as possible to the foot of the slopes. The balance of full surface width to be divided equally between the slope segments w_1 and w_5.

The typical section given below will show the general arrangement, the notation to be adopted, and the points at which velocities and soundings are to be taken :—

Widths, of segments	w_1 w_2..............$6w_3$........... . ...w_4 w_5
Distances between velocity station.	w_1 w_2 w_3 w_3 w_3 w_3 w_3 w_3 w_4 w_5
Paincipal soundings	d_1 d_2 d_3 d_4 d_5 d_6 d_7 d_8 d_9 d_{10} d_{11}
Intermediate soundings	d_3 d_4 "d_4" "d_5 d_5" "d_6 d_6" "d_7 d_7" "d_8 d_3" d_9
Mean depths.	D_1 D_2 D_3 D_4 D_5 D_6 D_7 D_8 D_9 D_{10} D_{11}
Mean velocities.	V_1 V_2 V_3 V_4 V_5 V_6 V_7 V_8 V_9 V_{10} V_{11}

Soundings (d_1 and d_{11}) should be taken in the centre of the slope segments and velocities (V_1 and V_{11}) observed when practicable. When velocities V_1 and V_{11} cannot be taken they may be assumed at $\frac{V_2}{2}$ and $\frac{V_{10}}{2}$ respectively.

Streamers will be attached to the section ropes at all the points of the principal soundings which are also velocity stations. Intermediate soundings will be taken only in the central segment at intervals of $\frac{w_3}{3}$. Streamers an not necessary at these points.

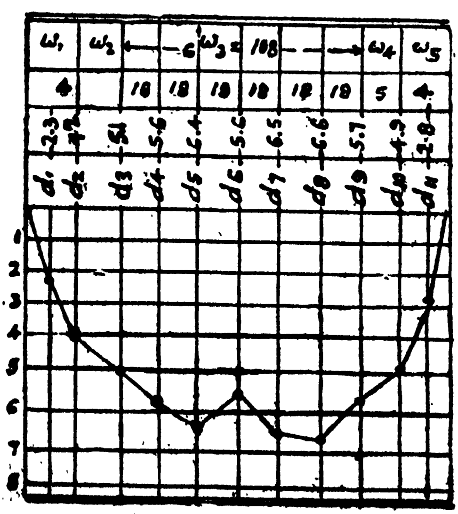


Fig. 11

The method of recording observations and working out discharge is shown in Fig. 11 and appendix II.

12. Preparation of Discharge Tables. (Gauge Discharge Method).

(*a*) A series of discharges are to be observed at round about steady full supply, there quarters, half and one quarter full supply, against a permanently fixed gauge. For each series of observations one meaned value of 'Q' the Gauge reading 'G', and the area of waterway 'A' is to be taken out by rejecting any obviously erroneous observation. 'D' or mean water depth as area divided by Mean Width is then evaluated, and from the four values of 'Q' and 'D' so arrived at, the connection between 'Q' and 'D' is determined by evaluating for 'K' and 'n' in the equation $Q=K.D^n$ using the method of least squares. The final stage consists of preparing the discharge table for values of 'D' corresponding to gauge reading as read from a plotting of 'D' against 'G' in order to determine intermeniate values of 'D' corresponding to intermediate values of 'G'.

(*b*) For a channel section.

When framing a discharge table using the relation for a channel section $Q=K.D^n$ it is insufficient to assume, $n=5/3$ as actually it must vary for each individual site; on the other hand, where the highest degree of accuracy is not required, it is sufficient to evaluate 'K' and 'n' graphically by plotting logQ against logD as shown in Fig. 12.

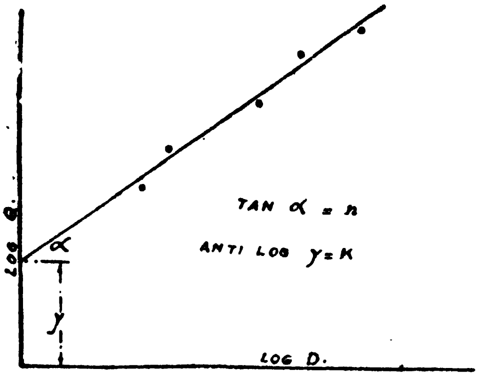


Fig. 12

As $Q=K.D^n$ expressed in the form of :—

$\text{Log}Q = n\log D + \log K$ is the equation of a straight line, the four points representing the series of observations, at round about steady full supply, three quarters, half and one quarter supply, will fall in such a away that :—

1. The tangent of the angle, the line through the points makes with the logD axis $=n$.

2. The ordinate, the line through the point makes with the log Q axis $=\log K$.

The solution to the values of 'K' and 'n' is thus completed but this graphical method depends on the correct visual appreciation of the divergence of the points from the straight line and the fitting in of the appropriately fitting straight line; the mathematical method appreciates the minutest divergence and provides a solution which mathematically fits the entire range of supply over which the observations have been taken.

13. Special Precautions for Calibration Discharges.

The following precautions are to be taken before and during a calibration class of observation :—

1. The gauge; whether in a gauge-well or outside, against which the discharges are to be correlated must be absolutely firm and secure. To frame a discharge table against a gauge attached to a loose and shaking post is a complete waste of time, and in any case no such gauge should ever be used in connection with any recording of discharge.

2. The gauge well, if any, must be functioning properly and it should be seen that it is not choked with silt, and that the holes or slits are perfectly clear.

3. Each observation of discharge should be in duplicate, using two newly re-rated check meters on the same suspension rod or rack-and-pinion, and operated by two indepednent observers. This serves as a check on the work during its progress as, if one obsever gets at a station an entirenly different velocity from an other observer's, the observation can immediately be repeated after checking over both the meters. Finally the two results, if differing by not more than 1% may be accepted and meaned.

4. A complete observation must take the form of a double traverse, that is, one set of observations will be from left to right and immediately followed by another set from right to left; with two Observers there will in this way be four sets of results and it is essential that they are all worked out finally at site and cross checked.

5. A regular programme for taking the whole series of observations of suitable supplies is to be prepared in consultation with the local ragulating officers so that the series may not drag on indefinitely and yet cover as much as possible of the whole range of supply from full supply down to not less than 40% of full supply.

6. Special efforts must be made by the Regulating Staff to keep the supply steady and if the variation in gauge during a calibration discharge observation is greater than 0·01 ft. the observation is to be rejected.

7. The discharge site must be as near the gauge of the flume under calculation as possible in order to avoid the effect of time-lag. If the distance, however, is unavoidably appreciable then time-lag between the discharge site and the gauge or flume, has to be allowed for. If an observation, for example, is from 10-0 A. M. to 11-30 A. M. and the distance is 5000 ft. and

the mean velocity is 2·5 feet per second the lag will be $= \frac{5000}{2\cdot5 \times 60} = 33$ minutes, when the comparable gauge, if it is on upstream of the site, will be that prevailing from 9-27 to 10-57 A. M.

8. When observing velocities, times must be recorded to the nearest 1/5th second and the products of "D × V" must be taken out to three places of decimal, and when finally preparing the equation not less than seven accepted series of observation are to be taken into mathematical analysis.

14. Regulation.

(a) Use of Discharge Table.

The discharge tables are calculated to give the discharges for every *hissa*, that is, up to first decimal place of a foot. The discharges for the intermediate Gauges for the second decimal place are worked out by interpolation. Usually enamelled Gauges with divisions upto second decimal of a foot are fixed. The discharge tables are maintained for all distributary heads and for the indenting site at the beginning and at the end of a canal sub-division on the branch or main canals. Usually there is a fall available which is designed to work a meter at the indenting site. If no fall is available, a discharge site is constructed in the open channel by pitching the bed and sides for a length upto 150 feet, so that, a straight run of 100 ft. length is available for the discharge observation. The Gauge is located in a gauge well constructed about the middle of the pitched reach. The discharges for such sites are worked out as explained in para 12 (*b*) above.

(b) Indents.

Every Sub-Divisional Officer works out the requirements of supply for the sub-division by adding authorised. Full supply discharge of all off-taking distributaries and the seepage losses in the main or branch canal in the sub-division which are usually sanctioned by the Superintending Engineer in the Capacity Statement of the canal. In the case of rain-fall or slack demand in certain portions of the sub-divisions, he forestalls the requirements of the distributaries and then adds up the required discharges to work out the indents. The Tail S. D. O. wires to the Upper S. D. O. and so on to S. D. O. Head works of the Canal. The indents should specify both the Gauge and the discharge at the indenting gauge. Twenty four hours notice is usually required to meet the indent. The sub-divisional officers on the tail reaches of a canal should calculate the time when the effect will reach considering veloclties in Main Canals 3 miles an hour and in Branch Canals 2 miles an hour. Regulation is a very important part of the duties of an Irrigation Engineer. There are usually printed instruction hung up at the important regulators on the main and branch, canals specifying the duties of the regulation staff. The regulation staff is on duty for 24 hour on the important regulators. They at once attend to the orders issued by the officer-in-charge of regulation ordering changes in indents.

(c) Immergent indents.

In case of accidents to canals such as breaches or cuts, immergent indents are issued for reduction. The foremost duty of the officers and the regulation staff is to attend to them. The regulation staff and the signallers are supposed to attend to the Telephone and Telegraphy instruments even at night.

Bibliography.

1. Paper No. 272, Punjab Engineering Congress, 1945 "Methods of Discharge Observations" by S. L. Malhotra I. S. E. Executive Engineer, Punjab Irrigation; 2. Discharge Observations by Hayat and Grover. 3. Lewett's Hydraulics.

15. Examination Questions.

1. (*a*) Describe different methods of measuring the discharges of rivers and canals.
(*b*) Describe a "discharge rod." What is the best ratio of the length of the discharge rod to the depth of water? *(T. C. E. 1933)*

2. Describe in detail how you would proceed to measure the discharge of a river about 400 yards wide Max: depth about 20 ft. max: surface velocity about 2 ft. per second. *(T. C. E. 1935)*

3. The observed discharge of a canal with 7·5ft. gauge was 600 cusecs and with 6·5ft. gauge 450 cusecs. Frame a discharge table showing discharges with 6·0ft. gauge and 8·0ft. gauge. *(P. I. B. 1941)*

4. Describe the method for taking a first class discharge of a canal with current meter. A branch caual has a discharge of 1500 cusecs with gauge 9·5ft. at site and 1000 cusecs with gauge 8·5ft at site.

How will you proceed to frame a discharge table for this site?

What will be the discharge with a gauge of 6 5ft.? *(P. I. B. 1939)*

5. From your experience describe an accruate method of measuring the actual discharge of a channel having 10 to 15 cusecs capacity, you have a stop watch, tape, measuring rod. Give specifications of the measuring rod.

6. Describe the method of observing the discharge:—
 (*a*) Of a river where it is not possible to stretch a rope across it;
 (*b*) Of a canal with 1st class observotions;

7. (*a*) Describe any one type of Current Meters used for discharge observations;
 (*b*) Give briefly the percautions you will take for proper upkeep of Current Meters. *(P. U. 1942*

8. (*a*) How will you work out the indent of supplies in the canal for your Sub-Division?
 (*b*) Explain the process of segmentation in a first class canal discharge.

9, (*a*) How are Weddle's and Simpson's rules used for working out canal discharges?
 (*b*) How will you make correction in the discharge observed by velocity rods, when a strong wind is blowing

10. (*a*) How are the velocity observations corrected by velocity rods to get correct mean velocity?
 (*b*) What points will you keep in view while selecting discharge site in an artificial channel?

11. What are the different methods adopted for measuring to velocity of flow of water in a stream? Mention the conditions to be satisfied in selecting the site for accurate gauging of flow in a stream. *(Mysore 1941)*

12. How would you observe the discharge of a channel using;
 (*a*) Velocity rods.
 (*b*) Current meter? Discribe any type of current meter with which you are best acquainted. *(F. Sc. 1941)*

13. How are the supplies in a river gauged?.
 (*a*) Describe any one type of current meter known to you.
 (*b*) What are usual causes of observational error in a discharge measurement and how far can we avoid them. *(P. U. 1952)*

14. How is the discharge gauged in the case of
 (*a*) A river. (*b*) A canal and (*c*) A distributory. Describe the appratus used. *(P. U. 1953)*

APPENDIX 1 FORM D 1.

DAILY DISCHARGE DATA.

River Sutlej Site Harike Date 10-9-1944

Meter No. and make Gurley No. 171-22,304-S Time from 7-30 hrs. To 12-0 hrs.

Gauges.	Permanent	Temporary R. B.	Temporary L. B.
Zero R. L.	670·56	670·56	670·56
Beginning.	9·10	8·95	8·97
End	9·08	8·93	8·95
Mean	9·09	8·94	8·96

Equation of Meter $V=2{\cdot}35\,N+{\cdot}03$

Date of last rating 6-4-1944

Description of surface floats ... ...

Length of float run ... ...

Float run marked with ... ...

Section line marked with Pivot Points

Sounding taken with Sounding Rods

Method of suspending meter Rack and Pinion

Timepiece used Stop Watch

Weight used lbs.

Weather Clear

Wind direction and strength

very slight
slight
strong
very strong

Condition of water: Fairly Clear / Ordinarily Silty — Yes / Intensely „

Current

SURFACE SLOPE OBSERVED.

	Right Bank Back	Right Bank Fore	Right Bank Difference	Left Bank Back	Left Bank Fore	Left Bank Difference
500 D/S	7·25			6·76		
0						
500 U/S		7·06			6·58	
Fall 1,000 ft.			0·19			0·18

Mean=0·185

$\therefore S=0{\cdot}00019$

$V=\text{Mean Velocity}=\frac{Q}{A}=3{\cdot}05$

$R=\text{H.M.D.}=\frac{A}{P}=8{\cdot}62\quad\sqrt{R}=2{\cdot}94$

$\sqrt{RS}=0{\cdot}040$

$C=\frac{V}{\sqrt{RS}}=76{\cdot}25$

$X=41{\cdot}6+\frac{00284}{5}=56{\cdot}40$

$Y=\frac{CX}{\sqrt{B}}=1{\cdot}463$

$Z=C-X=19{\cdot}90$

$N=\frac{\sqrt{(Z^2+7{\cdot}244Y)}-Z}{2Y}=0{\cdot}028$

Character of river bed Sandy

Class of roughness under which it falls II

Passed for check by Initials S. D. O. II dated...........

Checked by Collator dated...........

Rough-volumed by Collator dated...........

Compared & recorded by Head Collator dated...........

ON HIS MAJESTY'S SERVICE

THE SUB-DIVISIONAL OFFICER,
DISCHARGE SUB-DIVISION.

....

Stamp

Observed by O. S.

Date

DAILY DISCHARGE DATA 10-9-44

R. D. on Section	Water Depth	Difference of Depth	Increase of Bed.	Time	Revolutions of meter.	Surface Velocity	Mean Velocity	Angle of current with the section line=α	Correction for Angle,	Modified Velocity Col. 8× Col. 10	Drift in feet.	Velocity Correction for Drift.	Final Modified Velocity.	Col. 2× Col. 14	Correction + or — for unequal Segments.	Kind of Boat used.	Whether anchored or not.	Remarks.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
L. B. 0—2480	Dry.	..	...	m.s.	...		...	...	...	...			...	...				
2500	0·4	0·4	...	2-24	25		0·44	92	...	0·44			0·44	0·18				
2600	1·7	1·3	·01	2-16	40		0·72	92	...	0·72			0·72	1·22				
2700	3·5	1·8	·01	2-22	65		1·11	97	·01	1·10			1·10	3·85				
2800	6·4	2·9	·04	2-02	80		1·57	99	·02	1·55			1·55	9·92				
2900	11·4	5·0	·12	2-02	125		2·44	98	·02	2·42			2·42	27·59				
3000	14·5	3·1	·04	2-12	160		2·88	100	·04	2·84			2·84	41·18				
3100	11·8	2·7	·03	2-10	250		4·55	92	...	4·55			4·55	53·69				
3200	9·8	2·0	·02	2-14	250		4·41	89	...	4·41			4·41	43·22				
3300	13·0	3·2	·05	2-27	250	Velocity observed at ·6 D	4·03	88	..	4·03	Nil	Nil	4·03	52·39	Deducted (i) 80/2×0·18=7·20 (ii) 60/2×1·32=39·60 = 46·80	S. T. Boat.	Anchored.	
3400	10·2	2·8	·03	2-14	200		3·54	89	...	3·54			3·54	36·11				
3500	11·0	0·8	...	2-01	160		3·14	93	...	3·14			3·14	34·54				
3600	11·5	0·5	...	2-08	160		2·97	95	·01	2·96			2·96	34·04				
3700	10·2	1·3	·01	2-06	160		3·01	98	·03	2·98			2·98	30·40				
3800	9·6	0·6	...	2-22	160		2·68	100	·04	2·64			2·64	25·34				
3900	7·8	1·8	·02	2-20	125		2·13	104	·06	2·07			2·07	16·15				
4000	2·7	5·1	·13	2-04	25		2·50	104	·01	2·49			0·49	1·32				
4040	...	2·7	·09	...	...		...	...	...	...			...	...				

Total	135·5	Total	·60
Multiply by common width of segments	100	Add surface width	1560·00
Product	13550·0	P= Wetted perimeter	1560·60
Deduct ...	97·0		
A=Area ...	13453·0	Say	1561 ft.

Say :—13453 sq. ft.

Total	411·14
Multiply by common width of segments ...	100
Product	+11140
Deduct total Col. 16	46·8
Q=Discharge ...	41067·2

Say—41067 C. S.

Detail of deduction :—(i) 80/2×0·4=16·0
(ii) 60/2×2·7=81·0
97·0

Note :—1. Col. 16=(common width of segments—½ the sum of segments on either side of the R. D.)×Col. 15

2. When floats are used, or more than one meter observations are taken at the same section, each observation of time and revolutions must be recorded in a separate line in Cols. 5 and 6 respectively. In Cols. 1 and 2 all the lines relating to one station will be bracketed and R. D. on section and water depth will be recorded once.

3. Co-efficient employed converting surface velocity into mean velocity should be noted in the remarks column. unless specially directed it is to be taken as 0·89.

4. If no drift occurs, the same is to be shown as nil the column is never to be left blank.

Appendix II

First Class Observations

Discharge taken on..........,.........at R. D. 10,000 Main Line.

Between 10 A.M. and 1-0 P.M. *By* Executive Engineer.....................................

Assisted by Assistant Engineer. *Reading of gauge* at R.D. 1,000 below regulator

Before observation	... 6·0	Mean 6·05
After ,,	... 6·1	

Surface slope—R. L. 500 feet above centre of run ... 834·70
R. L. 500 feet below centre of run ... 834·52
Difference or fall in 1,000 feet ... 0·18
Fall in 1,000 feet 0·18 or S= ... 0·00018

Slight breeze———→ Strong breeze———→

Much Wind ———→ Direction—————→

Length of run—100 feet *Timing done*—with chronograph.

Floats used—loaded tin tubes *Surface width*=126. *Central Segment*=108

$W_1=W_5=4·0'$. $W_2=W_4=5·0'$. $W_5=18'$.

TABLE OF SOUNDINGS.

$<W_5 \times W_4 \times W_3 \times W_3 \times W_3 \times W_3 \times W_3 \times W_2 \times W_1>$

Distance from right bank.	Notation	Upper.	Middle.	Lower.	Total.	Mean.	Average.	Notation.
0								
2	d_1	2·0	2·3	2·1	6·4	2·13	2·13	D_1
4	d_2	3·5	4·2	4·0	11·7	3·90	3·90	D_2
39	d_3	4·7	5·1	4·8	14·6	4·87		
15	d_3''	5·1	5·3	5·9	16·3	5·43	5·24	D_3
21	d_4'	5·0	5·4	6·0	16·4	5·47		
27	d_4	5·4	5·6	6·1	17·1	5·70	5·70	D_4
33	d_4''	5·5	5·9	6·4	17·8	5·93		
39	d_5'	5·6	6·0	6·5	18·1	6·03		
45	d_5	6·0	6·4	6·4	18·8	6·27	·17	D_5
51	d_5''	6·1	6·3	6·2	18·6	6·20		
57	d_6'	6·0	6·4	6·0	18·0	6·00		
63	d_6	6·0	5·6	5·7	17·3	5·77	5·85	D_6
69	d_6''	6·0	5·8	5·5	17·3	5·77		
75	d_7'	6·6	6·0	5·3	17·9	5·97		
81	d_7	6·2	6·5	5·2	17·9	5·97	5·91	D_7
87	d_7''	6·0	6·2	5·2	17·4	5·80		
93	d_8'	5·9	6·3	5·4	17·6	5·87		
99	d_8	5·7	6·6	5·6	17·9	5·97	5·95	D_8
105	d_8''	5·7	6·5	5·8	18·0	6·00		
111	d_9'	5·6	6·0	5·9	17·5	5·83		
117	d_9	5·5	5·7	5·4	16·6	5·53	5·73	D_9
122	d_{10}	4·0	4·9	4·3	13·2	4·40	4·40	D_{10}
124	d_{11}	2·2	2·8	2·5	7·5	2·50	2·50	D_{11}
126								

NOTE.—$D_3=\frac{1}{3}(d_3+2d_3'')$ and $D_9=\frac{1}{3}(2d_9'+d_9)$; D_4 to $D_8=\frac{1}{3}(d'+d+d'')$.

VELOCITY OBSERVATIONS.

Notation.	Distance from right bank.	Length of rod used.	Time of passing first rope.	Time of ending run.	Seconds.	Total.	Mean.	Velocity per second.
V_1	2	2·0	1·03 2·31 4·02	2·15 3·40 5·09	72 69 67	208	69·8	1·44
V_2	4	3·75	7·00 3·15 9·30	8·04 9·20 10·34	64 65 64	193	64·3	1·56
V_3	9	4·5	12·00 13·17 14·40	12·59 14·15 15·42	59 58 62	179	59·7	1·67
V_4	27	5·25	20·05 21·10 23·00 25·02 28·10	20·50 21·52 23·47 25·48 28·55	45 42 47 46 45	255	45·0	2·22
V_5	45	5·75	0·14 1·20 2·31	0·54 1·56 3·07	40 36 36	112	37·3	2·68
V_6	63	5·5	5·12 6·40 8·00 9·10 10·50	5 54 7·21 8·44 9·53 11·30	42 41 44 43 40	210	42·0	2·38
V_7	81	5·0	13·01 14·00 14·59	13·45 14·41 15·38	44 41 39	124	41·3	2·42
V_8	99	5·5	0·00 1·12 2·15 3·02 4·12	0·37 1·52 2·55 3·45 4·56	37 40 40 43 44	204	40·8	2·45
V_9	117	5·25	5·00 7·00 8·16	6·49 7·52 9·08	49 52 52	153	51·0	1·96
V_{10}	122	3·75	10·40 12·08 15·00	11·42 13·11 16·04	62 63 64	189	63·0	1·59
V_{11}	124	2·0	18·00 19·30 21·12	19·10 20·42 22·25	70 72 73	215	71·7	1·39

CALCULATION OF DISCHARGE

Central segment (Weddle's rule).	$D_3 V_3 = \quad 5{\cdot}24 \times 1{\cdot}67 =$	8·75	
	$5D_4 V_4 = 5 \times 5{\cdot}70 \times 2{\cdot}22 =$	63·27	
	$D_5 V_5 = \quad 6{\cdot}17 \times 2{\cdot}68 =$	16·54	
	$6D_6 V_6 = 6 \times 5{\cdot}85 \times 2{\cdot}38 =$	83·54	
	$D_7 V_7 = \quad 5{\cdot}91 \times 2{\cdot}42 =$	14·30	
	$5D_8 V_8 = 5 \times 5{\cdot}95 \times 2{\cdot}45 =$	72·89	
	$D_9 V_9 = \quad 5{\cdot}73 \times 1{\cdot}96 =$	11·23	
	Discharge $= \frac{3W_3}{10}$ or $5{\cdot}4 \times$	270·52	1,460·81

CALCULATION OF DISCHARGE (continued)

Side segment.	$D_2V_2 = 3{\cdot}9 \times 1{\cdot}56$ = ...	6·08	
	D_3V_3 = as above = ...	8·75	
	D_9V_9 = as above = ...	11·23	
	$D_{10}V_{10} = 4{\cdot}0 \times 1{\cdot}50$ = ...	7·00	
	Discharge $= \frac{w_2}{2}$ or $2{\cdot}5 \times$	33·06	82·65
Slope segments (Simpson's rule).	$4D_1V_1 = 4 \times 4{\cdot}13 \times 1{\cdot}44$ = ...	12·27	
	D_2V_2 = as above = ...	6·08	
	$D_{10}V_{10}$ = as above = ...	7·00	
	$4D_{11}V_{11} = 4 \times 2{\cdot}5 \times 1{\cdot}39$ = ...	13·90	
	Discharge $= \frac{w_1}{6}$ or $0{\cdot}67 \times$	39·35	26·30
	Total Discharge ...		D=1,569·76

OTHER CALCULATIONS.

AREAS.					Wetted Perimeter.	
Central Segment.		Slope and Side Segments.				
D_3 ...	5·24	D_2 ...	3·90		$\sqrt{(w_1)^2+(D_2)^2}$ ...	5·6
$5D_4$ ...	28·50	D_3 ...	5·24		w_2+w_4 ...	10·0
D_5 ...	6·17	D_9 ...	5·73		$6\ w_3$ ...	108·0
$6D_6$ ...	35·1	D_{10} ...	4·40		$\sqrt{(w_5)^2+(D_{10})^2}$ ...	6·0
D_7 ...	5·91	$\frac{1}{2}\ W_2\times$ ...	19·27	=48·17		
$5D_8$ ...	29·75	$\frac{1}{2}\ D_2$ ...	1·95			
D_9 ...	5·73	D_1 ...	2·13			
		$\frac{1}{2}\ D_{10}$ ...	2·20			
Total ...	116·40	D_{11} ...	2·50		Total ...	129·6
$\times \frac{3w_3}{10} =$	5·4					
Area= ...	628·56	$\frac{1}{3}\ w_1\times$ ...	8·78	=17·56 64·78		

Area of central segment	...	...	...	... 628·56
,, other segments	...	...	...	... 65·73
A=Total area	...	...	...	... 694·29
P=Wetted perimeter	...	...	...	... 129·60
Hydraulic mean depth $= \frac{A}{P} = R$	=	...	...	... 5·36
$A\sqrt{RS} = 694{\cdot}29\sqrt{5{\cdot}31 \times 0{\cdot}00018}$	=	...	...	... 21·558
Co-efficient $= \frac{D}{A\sqrt{SR}} = \frac{1569{\cdot}76}{21{\cdot}588}$	=	...	...	... 73·8

PART II
CANAL IRRIGATION
CHAPTER XX
Water Power From Canal Falls

1. Introduction.

The necessity of creating canal falls (drops) has been explained in Chapter X of this part. When the slope of the country is steeper than the permissible (regime) slopes allowed for the irrigation channel, the additional head available in a canal is given the form of a canal fall. Such falls can be utilized for the development of motive power and for other useful purposes as under :—

(1) Generation of Hydro-Electric power which can be utilized for lighting, heating and various industrial purposes.

(2) Direct drives, such as flour miles and lift wheels. The latter can be used for lifting water to command the high adjoining areas.

(3) Hydromats (Suction or compression type). (Already discribed in chapter XIII of this part).

(4) Autosuction weirs. (Described in Chapter IX of this part).

2. Power Available From Canal Falls.

An approximate idea of the energy available from a canal fall can be obtained from the following calculations.

Let Q=The quantity of water in cubic feet per second flowing through the canal.

H=Head in feet through which the water drops at the canal fall.

W=Weight of water in lbs per cubic foot=62·4 lb. The theoretical water power available from the canal fall is$=\dfrac{Q.H.W.}{550}=\dfrac{62{\cdot}4\ Q.H.}{550}$ horse power. The actual useful energy available depends upon the efficiency of the various parts of the installation. The usual efficiencies may be taken as under :—

(*a*) Pipe lines, intake channel etc.$=e_1=94$ to 98%

(*b*) Hydraulic turbine$=e_2=80$ to 90%

(*c*) Electric generator$=e_3=90\%$

Thus the overall efficiency from water to electrical side will be $e=e_1\times e_2\times e_3$. This usually about 75 to 80%

Note :—In modren pratice the turbines are directly coupled to the generators on a common shaft. The old practice was to use a gear arrangment but as this involves a loss of power of about 5% direct coupling is preferable.

Taking the overall efficiency at 75% the electrical power available from a fall $=\dfrac{62{\cdot}4QH\times 75}{550\times 100}=\dfrac{QH}{12}$ horse power approximately. The common unit of electrical power is a kilowatt which is equal to 1·341 horse power.

Expressed in kilowatts the available electrical energy$=\dfrac{QH}{12}\times\dfrac{1}{1{\cdot}341}=\dfrac{QH}{16}$ Killowatts (approximately). Taking an actual example, suppose the quantity of water flowing through a canal is 500 cusecs and the hight of the canal fall at a given point is 10 ft. Then the electrical energy available from this fall would be :—

$$\frac{500\times 10}{16}=312{\cdot}5 \text{ killowatts}$$

To have an approximate idea as to what this energy is capable of doing, the normal consumption of various electricle appliances are given below :—

(i) Ordinary electric lamp = 40 watts.
(ii) Electric table fan. = 40 to 50 watts.
(iii) Electric ciling fan = 60 to 80 watts.
(iv) Electric heater for room = 500 watts.
(v) Flour mill $2\frac{3}{4}$ ft. dia: stone = 10 Kilowatts.
(vi) Tube-well with a discharge of 1·5 cusecs and lift of 20 ft. = 12 Kilowatts.

One Kilowatt = 1000 Watts.

3. Hydro-Electric Installations on Canal Falls.

A Hydro-electric installation on a canal fall will consist mainly of the following parts :—

(a) Headworks for diverting and controlling the flow of water from the main canal to the headrace or intake channel leading to the power house.
(b) Headrace or intake channel.
(c) Water turbines together with the control apparatus.
(d) Electric generators together with switch gear and control apparatus.
(e) Housing for the hydraulic and electrical apparatus, items (c) and (d) above.
(f) Tail race leading the water back into the main canal.

4. Design of Hydro-Electric Installation.

In designing the various parts of a hydro-electric installation the engineer has to consider very carefully the most suitable velocities of water at each part so as to determine the cross section and design. High velocities mean the sacrificing of a large percentage of the total head available for developing power at the turbines. On the other hand, low velocities require a lrage section of intake and out-flow channels thereby involving a higher capital cost. The engineer has to determine the economical mean between the two. In low head plants it is most important that the head lost in friction and eddies be as small as possible and, therefore, velocities should be as low as possible. Suitable velocities in various parts of a hydro-electric installation are given below :—

(i) Headrace or intake channel.

The maximum velocity in open flumes should not exceed adout three feet per second as a higher velocity is likely to cause whirlpools and eddies which will enter the runner and cause disturbances in the smooth flow of water. In steel pipes used on high head installations higher velocities can be used. Usual velocities for heads up to 200 ft. are 8 to 10 ft. per second and for higher head the economical limits for velocities are still higher. To maintain a constant head and constant supply of water to the intake channel an overflow weir is necessary which in modern practice is designed as spillway syphon. (See chapter VIII of this part). The intake channel should be designed non-silting. (Chapter VI, Part II).

(ii) Turbine pit.

In open flume plants, the intake channel leads direct into the turbine pit. The velocity of water on entrance to the runner on this type of installation should be about 2 ft. per second to avoid eddies in the runner. When turbines are provided with spiral casings so as to guide the water in smooth passages, higher velocities can be used. When spiral casings are of concrete, velocities of 5 to 6 ft. per sec. are recommended. For plate steel spiral casings velocities as high as about 20 ft. per sec. can be used.

(iii) Draft tube or suction pipe.

The velocity at the exit end of the draft tube should be about, the same as in the tail race. In low head plants this should be about 3 ft. per sec.

(iv) Tail-race.

In low head plants the velocity of water in tail-race should not exceed about 3 ft. per sec. So that the out-going water leaves the turbines with as little kinetic energy left in it as possible. Besides, higher velocities cause eddies and destroy the suction action in the draft tube which leads to lower efficiencies. Slightly higher velocities upto 5 or 6 ft. per sec. may be allowed as the head increases. In the case of Pelton wheel installations the tail water has no effect on the turbine and its velocity is determined by such conditions as the available area of channel with respect to station foundations etc.

(*v*) **Inlet bend to Pelton wheel nozzles.**

The velocity in the inlet pipe of a Pelton wheel is generally kept at 10% of the spouting velocity of water *i. e.*, 10% of $\sqrt{2gH}$ where H is the effective head of water under which the turbines operate. For heads exceeding about 600 ft. it is preferable to reduce it to $7\frac{1}{2}$% of the spouting velocity and for very high heads of the order of 2000 ft., and above, this percentage may be reduced still further to about 5%. The velocity should in no case exceed about 30 ft. per second.

5. Selection of Equipment.

Hydraulic turbines may be classified under two main headings :—

(*i*) Reaction turbines which work by means of the potential and pressure energies of the water. Francis turbine runner is the chief example of this type in the present day use. It is a mixed flow turbine being partly inward flow and partly axial flow.

(*ii*) Impulse turbine work under the kinetic energy of water, and the power is abstracted from the water by allowing the jet of water to act on a number of buckets fixed to the rim of a disc or wheel. The principal example of this type is a Pelton wheel.

The characteristics of these two types of turbine runners are described in detail in a later part of this Chapter. Hydro-electric plants using reaction turbines are further subdivided into the following principal categories :—

(*a*) Open flume turbines.
(*b*) Concrete spiral cased turbines.
(*c*) Plate steel spiral cased turbines.
(*d*) Cast iron or cast steel spiral cased turbines.

(*a*) **Open flume turbine.**

In this type of construction the intake water channel leads direct into the turbine runner. For low heads not exceeding about 30 ft. this is the simplest and most economical construction as it eliminates altogether the necessity of pipe lines, valves, turbine casings etc. Single runner vertical shaft type constrution is usually adopted as this gives better efficiency. The water after doing its work in the runner discharge through a suction tube into a tail water canal just beneath the foot of the turbine pit. The turbines should be so arranged that the distance from the surface of the incoming water to the highest point of the turbine runner, or guide vanes when these are provided, is at least equal to the diameter of the runner, as otherwise air is likely to enter through eddies, thereby reducing the suction action in the draft tube and diminishing the output of the turbine.

When rocky foundations are available, this type of construction may be economical even for heads upto 50 ft.

(*b*) **Concrete spiral cased turbines.**

These are used principally for capacities above 400 H.P. at 10 feet head and above 5000 H.P. at 100 ft. head. They are seldom used for heads above 100 ft. due to the necessity of reinforcement in the concrete which makes them more expensive and less reliable. They are almost invariably constructed in the single runner vertical shaft type.

(*c*) **Plate steel spiral cased turbines.**

These turbines are used for heads exceeding 40 ft. upto 375 ft. Above this head a cast type casing is used. These turbines are usually constructed in vertical shaft single runner type as this has a greater efficiency.

(*d*) **Cast iron and cast steel sprial cased turbines.**

Cast iron casings are used only for moderate size units working under medium heads as cast iron is not reliable material for high stresses. For large units and high heads upto 1000 ft. cast steel casings are used. When the head exceeds 1000 ft. the Francis type of runner is unsuitable as it gives too high a speed, and impulse turbines are used.

(*e*) **Impulse turbines.**

These are generally used for heads above 850 ft. They are also used in smaller units down to 200 H.P. at 100 ft. head. These turbines are usually of the horizontal shaft type either one or two sets of buckets being used to drive each generator. Turbines of this type, arc in use for heads upto 5000 ft, in Switzerland and for heads varying from 2000 to 3000 ft. in other parts of the world.

6. Suction Pipe or Draft Tube.

The suction pipe or draft tube is simply an air-tight tube fitted to all reaction type turbines on the discharge side. It extends from the discharge end of the turbine runner to about 18 inches below the surface of the tail water level. The suction action of the water in this tube has the same effect on the runner as an equivalent head so that the turbine developes the same power as if it were placed at the surface of the tail water. The action is similar to that of a syphon, the water exerting a suction proportional to the height of the column. Theoretically the centre of the turbine shaft can be placed about 34 ft. (height of the water barometer) above the tail water at sea level, but full advantage of this cannot be taken owing to the dissolved air in water and due to small air leaks. In practice the maximum head that should be used in suction is about 25 ft. at sea level, and less according to the altitude of the site.

Straight draft tubes have generally a flare of from 4 to 6 degrees, depending on the length, so as to reduce gradually the velocity of water, which the discharges quietly into the tail water and with as little energy left in it as possible due to residual velocity.

The draft tube should be supported securely otherwise severe vibrations are set up which disturb the whole station. Large draft tubes are usually moulded in concrete as it is easier to obtain gradual curves in concrete than in steel plate. At sites subject to high floods the electrical equipment has to be placed above the high flood level and in such cases the maximum length of the draft tube has to be used so as to avail of the total head upto tail water level. Such contingencies should be carefully studied from the data of rainfall and flood levels extending over a period of 40 years or over before designing the installation. If data available is for a lesser period, a safe margin should be added.

7. Specific Speed.

The specific speed of a turbine may be defined as the r. P.M. at which the runner would run if it were so reduced in size, without in any way changing the design that it would develop one horse power under one meter head. Meter has been selected as the unit of head as the performance hydraulic turbines is more often expressed in metric units. Runners having the same specific speed have similar charecteristics of performance, so that specific speed is a very convenient term indicating the type of a turbine runner. High specific speed runners are suitable for low heads and low specific speed runners are suitable for high heads. The specific speed is a complete measure of the possible performance of a given runner under any head, both as regards power and speed and thus it gives an indication of the suitability of a given design of runner for any given set of conditions of head, speed and power. When considering a turbine which has more than one runner on its shaft, the specific speed is based on the capacity or out put of one runner ; hence the total capacity of the turbine must be divided by the number of runners on the shaft for emboying in the specific speed formula given hereafter.

Let D=Dia : of runner in meters. Q=quanity of water passing through the runner in cubic meters per minute ; N=B.H.P. developed ; n=Speed in revolutions per minute ; and H=net or effective head on the runner in meters.

Then :—(*i*) $n \propto \sqrt{H}$ (*ii*) $Q \propto \sqrt{H}$; (*iii*) $N \propto H^{3/2}$ for a given design of runner ; (*iv*) $N \propto D^2$; (*v*) $Q \propto D^2$; (*vi*) $n \propto 1/D$; (*vii*) $D \propto \sqrt{N}$. The formula for specific speed is

$$n_s = \frac{n\sqrt{N}}{H^{5/4}}$$

When investigating any given scheme the head, quantity of water, and the total power output are readily available and the first problem is to find a suitable type of turbine and its speed.

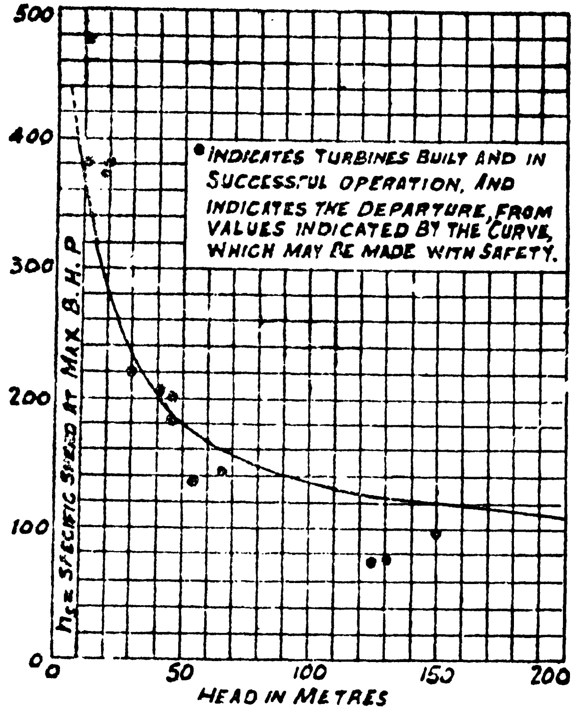


Fig. 1

The curve in Fig. 1 shows the maximum specific speed suitable for various heads, and enables the maximum specific speed being determined at a glance. This curve has been obtained by collecting the data of a number of existing plants, and is not based on any theoretical calculations. The application of the specific speed will now be demonstrated by the following example.

Example I. Suppose it is desired to develop 6400 H.P. at 10 meter head by a single machine. The electric generator is to generate alternating current at frequency of 50 cycles. To determine the speed and the type of turbine suitable for this development It will be noticed that the head is suitable for an open flume turbine and this should be adopted. From the curve in Fig. 1 it will be served that the maximum specific speed for 10 meters head is 360. Assuming one runner, the actual speed of the turbine may be determined from the specific speed formula as below :—

$$n_s = \frac{n\sqrt{N}}{H^{5/4}} \; ; \therefore \; n = \frac{n_s \times H^{5/4}}{\sqrt{N}} = \frac{360 \times 10^{5/4}}{\sqrt{6400}}$$

$$= 79{\cdot}2 \text{ r. P.M.}$$

This in not a suitable speed for the alternator, hence we will try two runners on one shaft. The output per runner will then be 3200 B.H.P. and substituting this in the above equation, we get :—

$$n = \frac{360 \times 10^{5/4}}{\sqrt{3200}} = 112 \text{ r. P.M.}$$

The nearest speed suitable for an alternating current at 50 cycles is 125 r.P.M. and this will be adopted. The specific speed corresponding to this speed will be $n_s = \frac{125 \times \sqrt{3200}}{10^{5/4}} = 401$

This is higher than the maximum spacific speed derived from the curve, but as the head is very low, some deviation from the curve is possible. This speed will, therefore, be adopted.

Example II. A turbine is required to develop 3000 B.H.P. at 80 meters head. The generator is to generate alternating current at 50 cycles frequency. To determine the suitable type and speed of turbine.

From the notes given under selection of equipment it will be noticed that plate-steel spiral cased turbine will be suitable for this head. From the curve in Fig. 1 the maximum specific speed corresponding to 80 meters head is 150. Hence the speed of the turbine will be :—

$$n = \frac{n_s \times H^{5/4}}{\sqrt{N}} = \frac{150 \times 80^{5/4}}{\sqrt{3000}} = 654 \text{ r.P.M.}$$

The nearest synchronous speeds of the generator corresponding to 50 cycle frequency are either 600 r.P.M. or 750 r.P.M. If we adopt 750 r.P.M. the generator no doubt will be less costly, as the higher the speed, the less bulky will be the machine, and hence the cost will be comparatively less. The specific speed corresponding to 750 r.P.M is :—

$n_s = \frac{750 \times \sqrt{3000}}{80^{5/4}} = 172$, **which is 15% higher than the maximum specific speed** obtained from the curve. This deviation is not desirable for a head of this magnitude, hence the next lower speed of 600 r.P.M. will be adopted. The specific speed corresponding to this speed is 138 and turbine runner of this specific speed will be selected.

8. General Principles of Hydraulic Turbines.

It has been stated under para 5 that hydraulic turbines are of two paincipal types (a) Reaction turbines of the Francis type and (b) Impulse turbines of the Pelton wheel type. Below are given a general description and the principles of working each type.

(*a*) **A Francis tubines.** The runner of Francis turbine consists of a number of vanes spaced round the circumference of a wheel which revolves on a shaft. The water is guided into the wheel by means of guide vanes, and the reaction of the water on the vanes of the runner produces a torque causing motion of the turbine. The guide vanes also regulate the flow of water into the runner, being mounted on a shaft about which they can turn through the governor action, thereby reducing or increasing the water input according to load. In Fig. 2 two circles represent the inner and outer periphery of the runner and one vane is shown with tips B and C; Let C_1 represent the velocity of the jet before entering the runner. U_1=peripheral speed of outer casing. U_2=peripheral speed of inner casing. U_1 and U_2 are tangential at B and C respectively; $\frac{U_1}{U_2}=\frac{r_1}{r_2}$ where r_1 and r_2 are the radii of the outer and inner casings respectively.

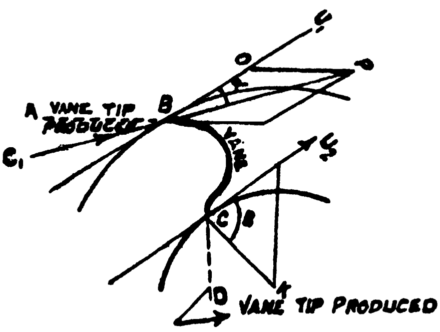


Fig. 2

The triangle BOP is the entrance velocity triangle, in which BP is the velocity of jet at entrance and BO is the peripheral speed, and OP represents the relative velocity between the jet and the casing.

The water should enter the runner smoothly without shock and for this end OP should be parallel to AB, *i.e.* the direction of the vane tip, at entrance. A similar triangle can be constructed for the discharge end of the vane tip, such as CMK, in which CM represents the peripheral velocity, CK is the discharge velocity and MK is the relative velocity between the jet and the vane at discharge. The magnitude of CK, it will be noticed, will depend on the vane tip angle and this velocity should in practice be as small as possible.

When the water enters the wheel, it sets up a pressure tangential to the periphery equal to $\frac{wQ}{g}\times C_1 \cos\alpha$ in which "w" is the weight of water of per unit volume and "Q" is the quanity of water entring the vane in unit time. Similarly, if the dischrge velocity CK be denoted by C_2 the pressure set up at discharge end is $\frac{wQ}{g}\times C_2 \cos\beta$. These two pressures set at leverage of r_1 and r_2 respectively, where r_1 and r_2 represent the outer and inner radii of the runner respectively. Thus, the turning moment at the outer vane end is $r_1 \times \frac{wQ}{g}\times C_1 \cos\alpha$ and that at the discharge end is $r_2 \times \frac{wQ}{g}\times C_2 \cos\beta$. The effective turning moment on the shaft is the difference between the above two moments, and if we multiply each moment by the linear velocity of the wheel at the respective points, the difference will give the work done or the horse power of the runner.

A typical efficiency curve of a Francis Turbine is shown in Fig. 3. It will be noticed that the curve drops rapidly on both sides of the maximum efficiency point and it is therefore necessary to run it at or near the load of maximum efficiency.

(*b*) **Pelton wheel.** As previously stated, a Pelton wheel consists of a number of buckets fixed to the rim of a wheel and a water jet issuing from a nozzle under the pressure head of water impinges on these buckets, thereby setting the wheel in motion. The energy is imparted to the wheel solely by the kinetic energy of the jet of water. The speed of the Pelton wheel has a difinite relation to the theoretical velocity of water issuing from the jet and these relations are given below:—

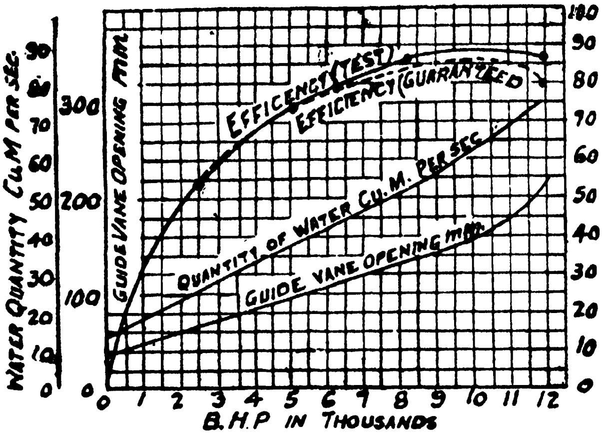


Fig. 3

Let H = the effective head of water in meters, then the theoretical velocity of water, usually called the spouting velocity, will be $\sqrt{2gH}$

(*i*) The velocity of jet $V = 0{\cdot}97\sqrt{2gH}$ meters per second.

(*ii*) Wheel velocity in meters per second is usually taken between $0{\cdot}43\sqrt{2gH}$ and $0{\cdot}47 \times \sqrt{2gH}$. This gives the best efficiency. The lower figure should be used for high specific speed and the higher figure for low specific speeds.

The term wheel velocity for a Pelton wheel means the velocity of a point on an imaginary circle, to which the centre line of the jet forms a tangent. The diameter of a Pelton wheel is the diameter of this imaginary circle.

(*iii*) The diameter of the Pelton wheel is determined as follows:—

Let wheel velocity $= V = 0{\cdot}45\sqrt{2gH}$ meter per sec. Knowing the value of H this can readily be worked out. Let n = r.P.M. of the wheel; then $V = \pi D\, n/60$; or $D = \dfrac{V \times 60}{\pi \times n}$

(*iv*) **Jet diameter.** To work out the jet diameter the total quantity of water required by the Pelton wheel must first be determined. The efficiency of a Pelton wheel is usually about 90 percent and the friction losses in pipes, etc., must be taken another 5 to 10 percent. Knowing the overall efficiency, the quantity of water required for a given capacity can easily be worked out. Let this be "Q" cubic, meters per second' thus:—

$$d = \text{dia of jet} = \sqrt{\frac{Q}{V \times 0{\cdot}7854}} \text{ meters.}$$

where velocity V = velocity of jet in meters per sec. The ratio D/d varies from 8·5 to about 15 or higher efficiencies.

(*v*) **Number of buckets**. The number of buckets has a pronounced effect on the efficiency of a Pelton wheel. The efficiency increases as the number of buckets increases up to a certain point and it then drops if the number of buckets is further increased. The graph shown in Fig. 4 is based on the data collected from a number of existing Pelton wheels giving good efficiency. The number of buckets is plotted against the ratio D/d of wheel to jet diameter. This graph will give an approximate idea of the suitable number of buckets for the design of a Pelton wheel.

(*vi*) **Size of buckets** The width of buckets is usually from 3·7 to 4 times the jet diameter; height of buckets from 0·7 to 0·8 time the bucket width, and depth of bucket is from 0·3 to 0·4 time the bucket width.

It is quite evident from the above relations that if the number of jets is increased, the bucket dimension will become smaller, and as buckets form a very expensive part of the turbine, the cost of the turbine will be less. On the other hand, higher stresses in the revolving parts due to higher speed of the turbine produced thereby will involve higher construction cost and these two factors have to be balanced against each ather to arrive at a suitable arrangement.

The curve in Fig. 5 is a typical efficiency curve of a Pelton wheel. It will be noticed that the Pelton wheel gives practically the same efficiency at part loads as at full load, and in this respect it has a considerable advantage over a Francis Turbine.

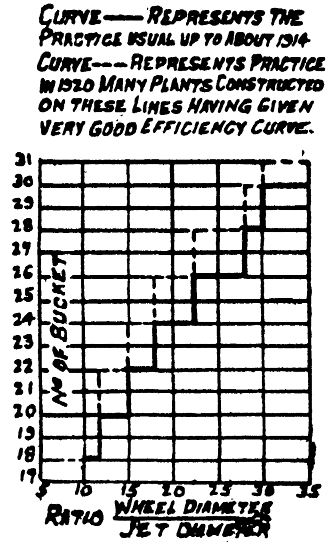


Fig. 4

9. Number of Units.

The greater the number of units in a Hydro-electric plant the more easily can they be run at their maximum efficiency and this has thus an advantage from the operation point of view. On the other hand, the cost of foundations and installation increases as the number of units increases. Usually in an independent installation, four to six units should be installed depending on the capacity of the installation, six units is an ideal installation, four units being designed to take the maximum load, the fifth being kept as a standbye for emergencies, and the sixth unit can always be opened for overhauls by turn. As each unit can take about 25 percent overload, there are in fact two units available as standbyes for emergencies and the factor of security of supply is, therefore, very great. Five units may be installed for slightly less security, and in installations of less importance, four units may be installed. Less than four units are not desirable unless the installation is inter-connected with another so as to depend for inter-change of load in emergencies.

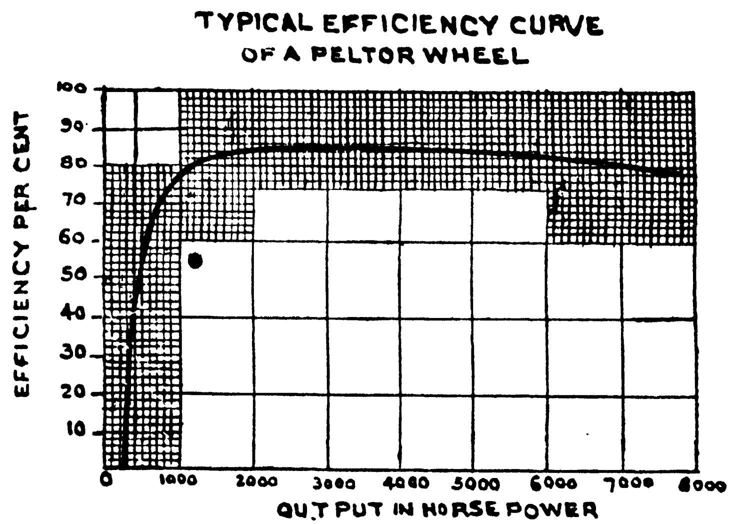


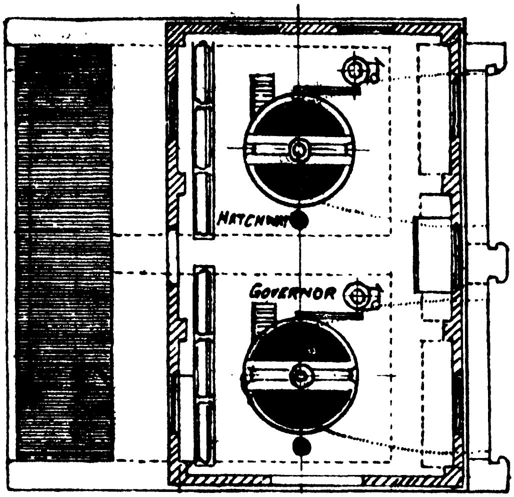
Fig. 5

10. Inter-linked Canal Hydro-electric Developments.

The main disadvantage of Hydro-electric developments on canal falls is the discontinuity of supply as the canals have to be closed periodically:—(*a*) For repairs (*b*) When water is not needed for crops, and (*c*) For rotational running of inter-linked canals due to insfficient supplies in the rivers to feed all canals simultaneously. It is necessary to have steam or oil standbye plants to run during canal closures to have continuous supply of electricity. Such standbye plant involves large additional capital cost, besides high running expenses. Fortunately most of the adjoining canals in the Punjab are interlinked and if one is closed an adjacent one will be running. It is, therefore possible to have interlinked hydro-electric developments on adjoining canals conected with the same net-work of electric supply systems, and this will eliminate the necessity of providing and maintaining the standbye fuel plants. In the present canal systems

Single vertical Hydro-Electric unit in simple open flume for low heads up to 20 ft. head.

PLAN



SECTION

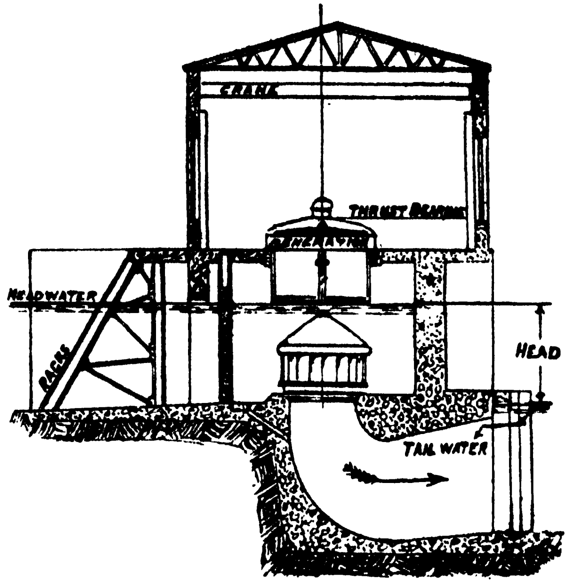
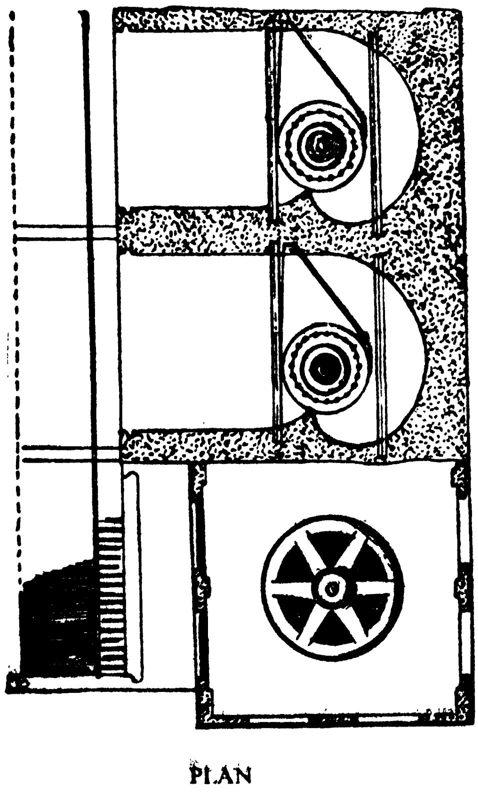


Fig. 6



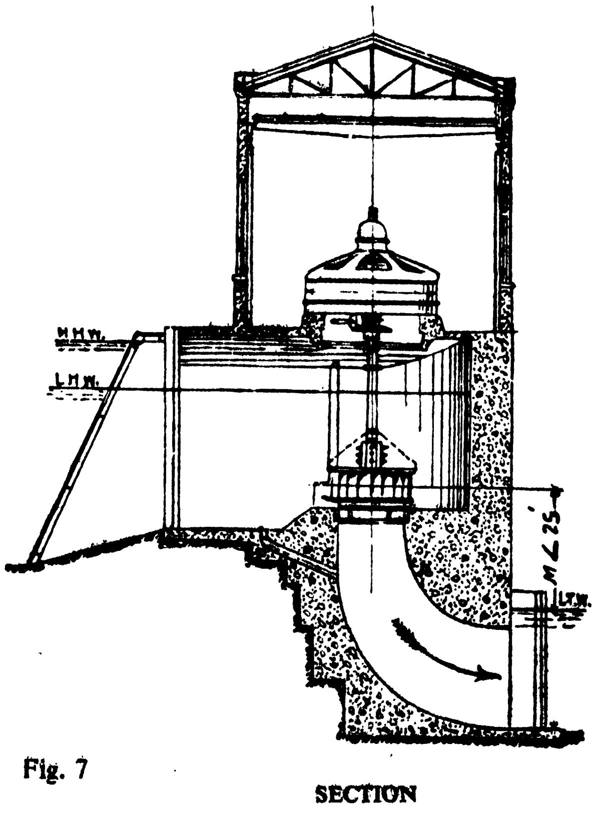


Fig. 7

PLAN

SECTION

in the Punjab there are at some places several small falls within a few miles length of the canal. From the hydro-electric development point of view, falls with bigger drops are preferable, and it is an important point to be kept in view in the future design of canal systems. Typical layouts of low-head plants with open flume and spiral cased turbines are shown in Fig. 6 and 7.

11. The foregoing notes give a general idea of hydro-electric installations and their equipment. The design and selection of hydro-electric turbine and their equipment is a specialized subject on which the advice of hydraulic firms manufacturing such equipment must always be obtained. Names of a few important firms are given below :—

1. Boving & Co. Hydraulic Engineers, Kingsway W.C.2.
2. Allis Chalmers Manufacturing Co. Milwankee, Wis, U.S.A.
3. Escher Wyss and Co., Zurich (Switzerland). The electrical parts of a hydro-electric installation falls within the province of an Electrical Engineer and is not dealt with in this book.

12. Flour Mills Run be Turbines.

In the Punjab, the water power at the canal falls has been extensively used to run flour mills. They have been commonly run by the use of country made wooden gird wheel carrying a pair of stones described fully in the succeeding paragraphs. In a few casses the low head Francis turbines (suitable for falls from 4 to 10 feet) have also been used. Two instances of this type are on the Lower Jhelum Canal at R.D. 25,500 and 64,000 of Northern Branch. Turbine shaft is coupled to a cam-shaft rotating two to four pairs of stones. Generally speaking turbine flour mills are expensive in the initial outlay and expensive to maintain as compared with the gird wheel flour mills.

13. Gird Wheels of Flour Mills of the Old Indian Pattern.

The Floor Mills of the Punjab Canals bring in a considerable revenue to the Government. The income from Mills in the Upper Bari Doab Circle for the year 1944 and 45 is given below :—

Name of the Mill	Number of stones.	Daily Income Rupees.	Amount
1 Aliwal 1 & 2	9	64/-	1033/2×250 (running days) = Rs. 2,58,281/4
2 Aliwal No. 3	5	31/-	
3 Aliwal 4 & 5	12	75/-	
4 Parowal.	11	71/-	
5 Ranewali.	6	54/-	
6 Kohali.	8	36/-	
7 Kotla.	5	30/-	
8 Pakhoki.	5	41/-	
9 Jaura.	5	37/-	
10 Athwal.	5	35/-	
11 Raya.	12	66/6	
12 Nagoki.	5	35/-	
13 Alladinpur.	5	47/8	
14 Sujanpur.	5	33/-	
15 Sarna.	6	41/-	
16 Tugial.	6	38/-	
17 Nanunangal.	5	21/4	
18 Dhariwal.	12	93/-	
19 Bhuchar.	6	36/8	
20 Bedian.	6	36/8	
21 Lulliani.	8	61/6	
22 Bhanba.	6	33/-	
23 Bhanba.	3	16/10	
Total.........	156	1033/2	

The above figures give the income for one canal system. There are fifteen such systems in the Punjab, and very nearly similar figures are obtained on other canals as well.

The quantity of the flour produced per pair varies from 20 to 30 maunds per diem and approximately the number of pairs of stones working in the Punjab is about 2000.

The mill wheel or "gird" varies very little in form or dimennions on the Bari Doab and Sirhind Canals, being universally made about 2 feet in diameter, on the Western Jumna Canal 2 feet 3 inches is the size adopted, while on the Swat Canal wheels 3 feet 6 inches in diameter are used.

The mill stones vary in diameter from $2\frac{1}{2}$ to 3 feet and are about 6 inches thick, when new, and $2\frac{1}{2}$ inches thick when rejected as worn out. They are made of Agra sandstone, except on the Swat Canal, where a very hard conglomerate stone obtained from near Michni Fort is employed.

The "head" under which the mills work varies from $2\frac{1}{2}$ feet to 6 feet, being generally between 3 feet and 5 feet, but in few cases upto 9·0 ft.

14. Design of Gird Wheel.

The water is applied to the vanes of the "gird" through an inclined shoot of a rectangular section, the sides of which converge towards its lower end or mouth. The angle at which the shoots are inclined to the horizontal varies from 30 to 60, an inclination of 35 being a common practice as shown in Fig. 8. The dimensions of the mouth of the shoot vary considerably, according to no apparent rule with reference to the height of fall or size of stones (a common size is 80 to 90 square inches, that is, 7 to 9 inches wide, and 9 to 12 inches deep).

On the Sirhind Canal it has been found that, with working falls of about 5 feet, the discharge necessary for a pair of 3 feet stones (the size in common use) is given by the formula $Q=\frac{45}{h}$, where Q=the discharge in cubic feet per second and h=the working fall in feet, so that a 5 feet fall would require 9 cubic feet per second per pair of stones, and a 4 feet fall about 11 cubic feet per second. It has been pointed out by Higham, that Qh is not necessarily constant for all falls, and he has quoted experiments on the Sirhind Canal showing that a discharge of $\frac{36}{h}$ cubic feet suffices to give good results on a 4 feet fall in the case of some new mills at Akhara, while in another case at Khanpur the discharge was $\frac{76}{h}$ the working fall varying from 5 to 8 feet, but in the latter case more water was probably being admitted than the mills could utilize. It will suffice, however, for purposes of roughly investigating the subject, to assume that Qh has a constant value=45 for all heights of fall considered. It may also be assumed that the co-efficient of discharge at the mouth of the shoot is 0·81 and that the axis of the shoot is inclined to the horizontal at an angle of 30°

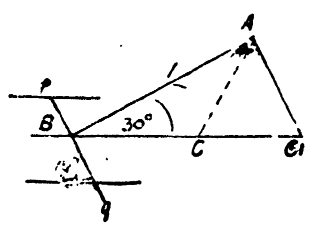


Fig. 8

Let AB represent the velocity and direction of motion of a particle of water issuing from the shoot and striking the vane pq, and let the velocity and direction of motion of the vane at the time of impact be represented by the line CB then the velocity and direction of impact of the praticle relatively to the vane is represented by AC. The particle of water will do work in driving the vane, provided that the velocity of the vane is less than C_1B; where AC_1 is parallel to the vane pq. If the velocity of the vane$=C_1B$ the particle of water will merely slide down the vane with a velocity relative to it$=AC_1$ and do not work, and if the velocity of the vane exceeds C_1B the particle of water will strike the back of the vane, and will tend to retard the wheel.

Let AB=V and $C_1B=U$. We may assume the Plane of the face of the vane pq to be inclined at angle of 60° with the horizontal, that being in accordance with parctice.

Then $C_1B=AB \sec 30^\circ$ $\therefore U=V=1{\cdot}15V$

But $V=0{\cdot}81\sqrt{2gh}$;$\therefore$ $U=0{\cdot}81\times1{\cdot}11\sqrt{2gh}=7{\cdot}48\sqrt{h}$

The following table shows the values of U for varying heads from 3 feet to 10 feet :—

Head in feet	3	4	5	6	7	8	9	10
U in ft. per sec.	12·9	15·0	16·8	18·3	19·8	21·1	22·5	23·7

and evidently no portion of the wheel should have a greater velocity than that assigned to U in the above table.

The usual number of revolutions made by the native mills when working efficiently is 150 revolutions per minute.

The following table shows the circumferential velocity in feet per second of a point at a distance R inches from the axis of a wheel making 150 revolutions per minute :—

R	Velocity in ft. per. sec.	R	Velocity in ft. per sec.
4	5·2	15	19·7
5	6·5	16	21·0
6	7·8	17	22·3
7	9·1	18	23·6
8	10·5	19	24·9
9	11·8	20	26·2
10	13·1	21	27·5
11	14·3	22	28·8
12	15·7	23	30·1
13	17·1	24	31·4
14	18·4		

It will be seen from these tables that with a 5 feet fall (a comman working fall in the canals) the limiting distance from the axis at which a particle of water striking a vane does any work in driving the wheel at 150 revolutions per minute is about 12½ inches so that any addition made to the size of the wheel beyond a diameter of 2′-1″ will not only be of no use but any water striking near the ends of the vanes would in such a case actually retard the wheel. It is a noticeable fact that 2′-0″ to 2′-1″ is exactly the size which the native millers have universally adopted on the Bari Doab and Sirhind Canals.

Similarly it follows that the following diameters should not be exceeded for the falls named below :

Height of fall in feet	Dia : of wheel in inches	height of fall	Dia : of wheel in inches
3	19	7	30
4	22	8	32
5	25	9	34
6	28	10	36

The work is done in these principally by the impact of water, and it is therefore probable that they work most efficiently when the vanes are moving at about half the velocity with which the water over-takes them. This being true of other water motors which are propelled by impact.

The component of the velocity of effluent water resolved in the direction of motion of the vane is $V \cos 30° = \sqrt{\frac{3}{4}} \times V$ and the best velocity for the vane would therefore, be $V\sqrt{\frac{3}{4}}$ and the shoot should be so placed that its center is situated as nearly as possible in the position where this would occur.

The following table shows the distance in inches from the axis of a wheel making 150 revolutions per minute at which the velocity is $V\sqrt{\frac{3}{4}}$ and, therfore, the distance from the axis at which the centre of the shoot should, if possible, be placed.

Head in feet.	$V\sqrt{3/4}$ in feet per second.	Distance from axis in inches.
3	6·0	4·5
4	7.0	5·3
5	7·8	6·0
6	8·5	6·5
7	9·2	7·0
8	9·8	7·5
9	10·4	8·0
10	11·0	8·5

Dimensions of the shoot :—

The shoot is to give a discharge $Q = \frac{45}{h}$ cubic feet per second. The values of Q and V and the necessary areas of cross section of the shoot in square inches are given below :—

Working fall in feet	3	4	5	6	7	8	9	19
Q=Dis : required in cubic ft. per second......	15 0	11·2	9·0	7·5	6·4	5·6	5·0	4·5
$V=0{\cdot}81\sqrt{2gh}$	11·2	13·0	14·6	15·9	17 2	18 4	19·5	20·5
Area of shoot in square inches $\frac{Q}{V}\times 144$.........	191	124	88	68	53	44	37	31

The results obtained are summarized in a tabular form below :—

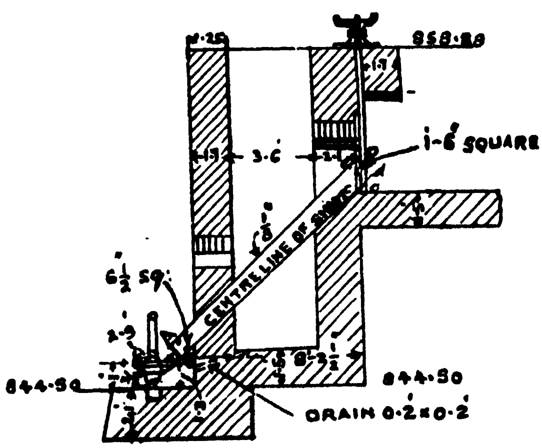
Working head it feet	3	4	5	6	7	8	9	10
Dia : of wheel in inches which should not be exceeded	19	22	25	28	30	32	34	36
Distance in inches from the axis at which the water does most work.	4·5	5·3	6·0	6 5	7 0	7·5	8·0	8·5
Area of orifice of shoot necessary in spuare inches.	191	124	88	68	53	44	37	31
Data assumed :—								

Number of revolutions made by the wheel=150 per minute. Inclination of shoot to horizontal—30°. Inclination of vanes to the horizontal—60°. Velocity of vanes when the water impinging on them a maximum amount of work in driving the wheel=half the horizontal component of the velocity of effluent water. Co-efficient of discharge of the shoot=0·81.

Now if we attempt to design a suitable arrangement for pairs of stones for various falls from 3 to 10 feet in accordance with the above results, we shall notice the following. First with 3 feet of fall, the wheel should be only 19 inches in diameter, and the best, place for the water to impinge is $4\frac{1}{2}''$ from the axis, but the boss is 8 inches diameter, and the shoot ought to measure 191 square inches and we find it quite impossible to bring anything like enough water on to the shoot vanes which are only $5\frac{1}{2}$ inches in length, and scarcely any of what reaches them and strikes them near enough to the axis to work in the most economical manner. With a 4 feet fall we meet with the same difficulties, and the best point to apply the water is $5\frac{1}{4}$ inches from the axis or $1\frac{1}{4}$ inches from the root of the vanes, which are 7 inches long, and as the orifice need only measure 124 square inches, we can apply a large proportion of the discharge from a shoot say 7 inches wide and 14 inches high.

Shoots for Flour Mills, Dhariwal

PLAN



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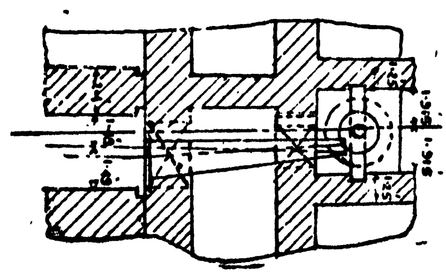
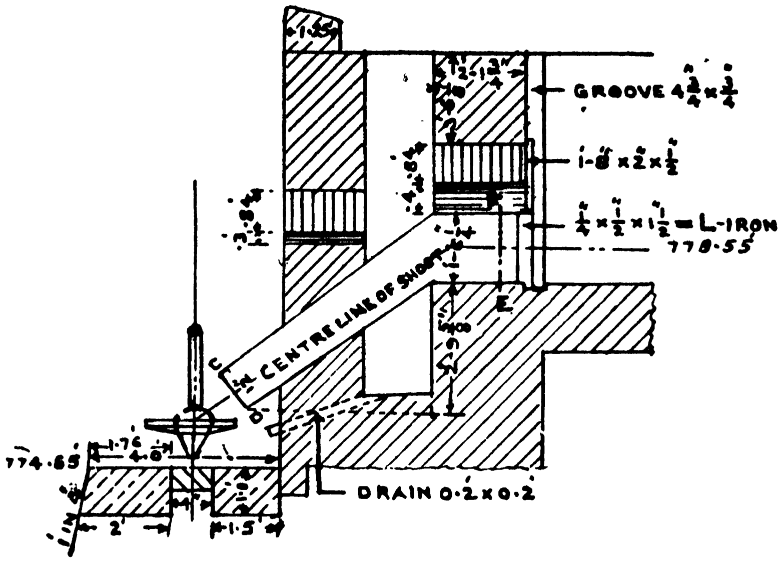


Fig. 9

SECTION



PLAN

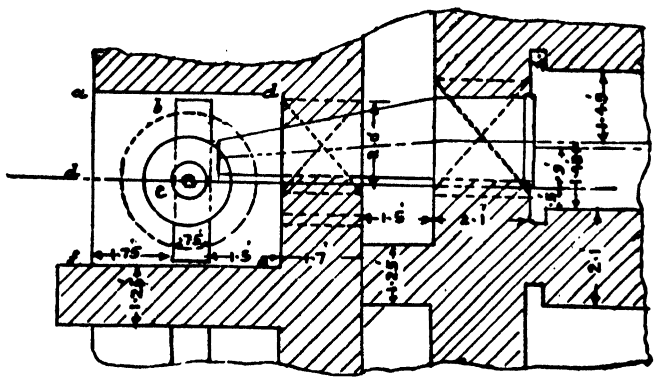


Fig. 10

With 5 feet or more of fall we have no difficulty but when we have to deal with a high fall of say 10 feet, we find that the shoot need only measure about 5½ inches square and that the best point of application is 8½ inches from the axis. There is no objection to the wheel being as much as 3 feet in diameter, but with such a small shoot having its centre only 8½ inches from the axis nothing is gained by making the dia; of the wheel greater than (say) 2½ feet diameter, as we can apply all the water necessary to a wheel of that size in the most advantageous way. The tails of shoots are shown in Figs. 9 and 10.

15. Efficiency of Girdwheels.

Light stone do not give such good results as full sized one, the quality as well as the quantity of the outturn falls off when the stones are driven too slow. With the ordinary pattern of mill now in use both these causes evidently operate in reducing the efficiency of mills working on falls of less than (say) 4 feet, and it would appear that at sites where the fall is as low as this, and the supply of water abundant, better results would be obtained by desingning a shoot which should deliver the water on to a larger two segment of the wheel, or substituting two shoot for one. Also the substitution of a wheel with a thinner spindle, perhaps a metal wheel, would apparently allow of more water being applied at a part of the wheel where it would work economically.

On the other hand, at sites where the fall is considerable and the supply limited, it would be worth while to make experiments with a view to find out whether the shoots in use are not extravageantly large for the stones.

It may also be noticed that where there is a high fall, it would be quite feasible to use much heavier stones. In England stones 4 feet in diameter are commonly used and these are run at 140 revolutions per minute, and grind 5 bushels (about 3 maunds) of wheat per hour. It does not seem customary in the Punjab to attempt to run such large stones, but one reason may be that the mill houses are so built that they could not be set up. In the Pashawar District, however, large stones are prefered, and are said to produce better floor than smaller ones.

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1. "Hydro-Electric Handbook" by William P. Creager and Joel D. Justin.
2. "Water Power Engineering" by Daniel W. Mead.
3. "Hydro-Electric power Developments" by Rushmore and Lof.
4. "Fundamental Principles of water Power Engineering" by F. F. Ferquesson.
5. "Note on Gird Wheel Flour Mills" by A. Reid, Under Secrelary, Irrigation, Punjab, (1895).

ERATA

1. Please substitute H_L for H_l where it has been used for the Head or Energy destroyed in a Hydraulic jump.

2. Page 364 read Chapter XVII, Remodelling Channeles for Chapter XVIII, Remodelling Channels.