THE BOOK WAS DRENCHED

THE BOOK WAS DRENCHED

UNIVERSAL
LIBRARY
OU_170490
UNIVERSAL
LIBRARY

AGENTS FOR THE SALE OF MADRAS GOVERNMENT PUBLICATIONS.

IN INDIA.

The Burma Book Club (Ltd.), 240-A, Merchant Street, Rangoon, Burma

Butterworth & Co. (Ltd.), 6, Hastings Street, Calcutta

R. Cambray & Co., Calcutta

The Superintendent, Diocesan Press, Vepery, Madras

E. M. Gopalakrishna Konr, Pudumantapam, Madura.

Higginbothams (Ltd.), Mount Road, Madras

V. Kalyanarama Iyer & Co., Vellala Street, Vepery, Madras

M. C. Kothari, Bookseller, Publisher and Newspaper Agent, Raopur Road, Baroda

The Law Book Depot (Ltd.), 15 and 16, Francis Joseph Street, Madras

G. C. Loganadham Brothers, Madras

Madras Y.M.C.A. Bookstall, Y.M.C.A., Esplanade, Madras, E.

The Modern Stores, Salem

S. Murthy & Co., Madras

G. A. Natesan & Co., Madras

The Superintendent, Nazair Kanun Hind Press, Allahabad

Nivasarkar, Manager, "Hitawada," Nagpur

P. R. Rama Iyer & Co., Madras

Ramakrishna & Sons, Lahore

The Srivilliputtur Co-operative Trading Union (Ltd.), Srivilliputtur.

R. Sundrr Pandurang, Kalbadevi Road, Bombay

D. B. Taraporevala Sons & Co., Bombay

Thacker & Co. (Ltd.), Bombay

Thacker, Spink & Co., 3, Esplanade East, Calcutta

S. Vas & Co., Madras.

IN STRAITS SETTLEMENTS.

The Federal Rubber Stamp Co., Penang

NOTICE.

Official publications hitherto obtained from agents in the United Kingdom and on the Continent will in future be obtainable either direct from the office of the High Commissioner for India, 42, Grosvenor Gardens, London, S.W. 1, or through any bookseller.

COLLEGE OF ENGINEERING MANUAL

IRRIGATION

BY

COL. W. M. ELLIS, C.I.E., R.E.

Late Chief Engineer for Irrigation, Madras

MADRAS

REPRINTED BY THE SUPERINTENDENT, GOVERNMENT PRESS

1926

PREFACE.

THIS work is compiled for the use of students of the Madras College of Engineering and its scope is confined to flow irrigation as practised in South India.

It is assumed that students have a competent knowledge of the Madras College of Engineering Manuals on Hydraulics, and on Elementary Applied Mechanics by Col. H. D. Love. In compiling this manual assistance has been sought from existing technical books and papers; the following works especially have been freely utilized:—

	Authors.
Practical Design of Irrigation Works.	W. G. Bligh.
Irrigation, Principles and Practice	R. H. Brown.
Irrigation Pocket Book	R. B. Buckley.
Irrigation Works of India ...	Do.
Roorkee Treatise, Irrigation Works in India.	J. Clibborn.
Graphic Hydraulic Diagrams ...	R. G. Kennedy.
Madras Irrigation Manual ...	J. Mullins.
Control of Water	P. M. Parker.
Irrigation Works	B. O. Reynolds.
Indian Storage Reservoirs with Earthen Dams.	W. L. Strange.
Reservoirs for Irrigation, Water Power and Domestic Water Supply.	J. D. Schuyler.
Principles of Water Works Engineering.	Tudsbery and Brightmore.
Design and Construction of Dams.	E. Wegmann.
River Discharges	J. C. Hoyt and N. C. Grover.

Advanced students are advised to study the above works in original; those by Bligh, Kennedy Parker and Hoyt and Grover are especially recommended.

The methods of design of masonry structures described by Bligh in the "Practical Design of Irrigation Works" have been closely adhered to in this manual and the empirical formulæ proposed by him for the dimensions of different parts of irrigation works have generally been adopted.

My best thanks are due to Mr. R. Mahadeva Ayyar, B.A., B.E., of the Madras Public Works Department, for his assistance in noting on the manuscript, checking the formulæ, and correcting the proofs.

W. M. ELLIS.

SYMBOLS ARRANGED ALPHABETICALLY AND REFERENCE TO THE FORMULÆ AND PARAGRAPHS IN WHICH EACH IS USED.

	Formulæ	Paragraphs
From Greek alphabet.		
Δ (delta) depth of water on surface of the land in inches	1	21
λ (lambda) limit of safe stress on masonry	20, 25, 26	213, 220, 238
μ (mu) co-efficient of fluid friction in pipe		488
ρ (rho) specific gravity of masonry		205 and throughout the work.
θ (thita) angle of resultant pressure with the vertical	11 to 13	207
From English alphabet.		
A angle of axis of arched dam with vertical	28	251
A width of impervious masonry apron of drop	56	466
A area of sluice shutter	71	582
a crest width of masonry dam	21	215
	27	240
a crest width of masonry weir	32	259, 262,
	35 to 37	268
B base of duty of water	1	21
B height of shutter	70	578
B top width of canal bank	51	427
b bed width of canal or channel	51	427
b width of arch ring in arched dam	23 to 26	238
b width of horizontal joint of dam or weir	9	197
	10	204, 205
	11 to 13	207
		220
	35 to 37	268
b width of joint at right angles to resultant	11, 12	207
C classification constant of river bed sand	38 to 40	282, 283
	45 to 48	331, 336
C co-efficient of discharge of notch	53 to 55	461

	Formulæ	Paragraphs
C co-efficient of Ryves' and of Dickens' formula	3, 3-a	91
	50	411
C constant in formula for width of berms	52	428
c co-efficient of entry to contracted aqueduct	68	526
c co-efficient in Kennedy's formula for critical velocities . .	8	160, 162
c co-efficient used in formula for run-off from combined catchment area ...	50	411
c distance of centre of pressure from centre of joint in masonry .	9	197
D balancing depth of excavation .	51	427
D canal or river discharge . .	68	526
	2	79
D depth of tail water above base of weir	33	262
		263
D discharge of pipe outlet .. .	64	488
D maximum flood discharge ...	3, 3a	91
	50	411
d depth of canal		133
	56	466
d depth of cutting of canal ..		426
	52	428
d depth of flow over weir crest .	29 to 30	257
	32	259
	33	262
	49	396
d depth of stream flow ..	7, 8	160
		162 to 165
d depth of water on crest of weir when tail water is at depth D above the base		263
d depth of water in canal above an aqueduct . .	68	526
d depth of water passing over shutter.	70	578
d depth of water passing over sill of notch	53 to 55	461
d diameter of pipe in inches	64	488
d diameter of screw spear ..	71	582
d duty of water per cusec .	1	21
d effective size of grains of sand	4, 5	115
d_0 depth of water above sill or crest of drop .. . .57 to 61, 63		467 to 469
d_1 depth of water in canal below a drop.	58	467
E submergence, that is, depth of tail water over sill of notch ...	53, 55	461
		462
F 'fetch' in miles	22	216
F proportion of mean to surface velocity	2	79

	Formulæ	Paragraphs
f overturning moment of falling shutter in foot lbs. per foot run ..	70	578
H height of weir wall above base ...		256
	29 to 31	257
	32, 33	259, 262
	35 to 37	268
H „ of water surface above base of dam	10	204, 205
	14 to 19	209
		220
	21	215
	23 to 27	238, 240
H height (vertical) from crest of tank weir to apron .	49	396
H „ (vertical) from crest or sill of body wall of drop to apron . .		465
H maximum head of water on a shutter in feet	71	582
H_a difference of level between crest of shutter and apron of weir or regulator . . .		283
	45, 47	331, 336
H_b difference of level between crest of masonry body wall of weir and L.W.L. below work . .	39, 40	283
H_s difference of level between crest of shutter and L.W.L. below work ...	38, 46, 48	283
		331
		336
h difference of level of head and tail water at weir or drop . .		262
	57, 58	467
	59 to 63	468, 469
h head producing percolation .	4, 5	115
	6	119
h „ required to force water through culvert or pipe	64	488
	66	519
h heading-up above a contracted aqueduct		526
h height of bank above canal bed .	51	427
h „ wave at dam	22	216
h_o depression head on Kennedy gauge outlet, that is, height of water level above centre of orifice		491
h_a head due to velocity of approach ...		258
h_e head of entry to pipe or contracted aqueduct		488
	68, 69	526
h_r head expended in overcoming frictional resistance in pipe		488
h_t head recovered at tail of contracted aqueduct	67	526

	Formulæ	Paragraphs
K co-efficient of afflux of weir ...		263
L length of body wall of drop	56	466
L length of culvert barrel or pipe in feet	64	488
	66	519
L length (unsupported) of screw spear in feet	71	582
L total width of down-stream apron of weir	40	283
L_1 total width of down-stream apron of river regulator	46	331
L_2 total width of down-stream apron of scouring sluice	48	336
l width of sill of trapezoidal notch	53 to 55	461
l width of trough of contracted aqueduct	68	526
l length of path of percolation ..	4, 5	115
	6	119
		123
	38	283
l_1 length of path of percolation for computation of uplift on aprons .		123, 283
M area of catchment in square miles ...	3, 3 (a)	91
	50	411
M moment of horizontal pressure about a point	30, 31	257
M ratio of bed width to depth of canal		433
M_o overturning moment of weir	31 (a), 31 (b)	262
	34	263
		267
M_r moment of resistance of weir ...	35 to 37 (a)	268
m area in square miles of portion of combined catchment	50	411
m power factor in equation for critical velocity	8	160, 162
N Kutter's co-efficient of rugosity ...		165 (g), 436, 439 525
n 2 tan *a* where *a* is the angle with the vertical of the side of a trapezoidal notch	53 to 55	461
n up-stream division of joint in dam 'reservoir empty'		225
n' up-stream division of joint in dam 'reservoir full'		223
P percolation per million square feet of area		133
P pressure of water on the face of a dam or weir		203, 209 257
P_h horizontal component of do. .	29	257
P_v vertical component of do. ...	29 (a)	257
p back pressure on a weir or regulator		322

	Formulæ	Paragraphs
p proportion of silt to water ...		162
Q discharge of canal or channel in cusecs		426
Q discharge of canal per foot width of stream		162
q maximum discharge of weir per foot run of crest	46	331
	48	336
R hydraulic mean depth		433
	66	519
R radius of intrados of bridge or regulator	42, 43	321
R radius of up-stream face or arched dam		237
	23 to 26	238
R′ radius of downstream face of arched dam . ..		238
R_c radius of centre of arch ring of arched dam	28	251
r slope of a face, viz, horizontal equivalent of each vertical foot .	29 (a)	257
	35 to 37	268
r_1 slope of channel cutting	52	428
r_2 slope of channel bank .	52	428
S clear span of regulator or bridge opening	41	321
S maximum or minimum compressive stress	9	197
		220
	25	238
S_1 mean compressive stress .	9	197
		220
	23, 24	238
S_a strees in arched dam due to weight of arch	28	251
s hydraulic or percolation gradient	6	119
s height of weir crest shutter	31 (b)	262
	35 to 37	268
t depth of water passing over weir when tail water is at base level	34	263
t temperature (fahrenheit) of water	4	115
u down-stream division of horizontal joint of dam from centre of pressure to nearest extremity of base		220, 223
V velocity of discharge	66	519
V velocity of percolation .	4, 5	115
V_1 velocity in canal above and below a contracted aqueduct . .	67 to 69	526
V_2 velocity in aqueduct	67 to 69	526
V_0 critical velocity (Kennedy's)	7, 8	160, 162
V_a velocity of approach ...	66 (a)	519
v middle division of horizontal joint of a dam		223

	Formulæ	Paragraphs
v velocity of flow		488
W weight of masonry of dam, weir, etc.	14 to 19	209 220 and elsewhere
W′ weight of masonry of dam, plus vertical component of water pressure on the face	13	207
W width of down-stream impervious apron of river weir	39	283
W_1 width of down-stream impervious apron of river regulator	45	331
W_2 width of down-stream impervious apron of river scouring sluice ..	47	336
w weight of water per cubic foot ...		206
	14 to 19	209 220
	23 to 26	238
X depth of water cushion cistern below the bed of the tail channel ...	57, 58	467
X height of pivot of falling shutter ..	70	578

ABBREVIATIONS.

F.S.L.	Full supply level.
F.T.L.	Full tank level.
M.W.L.	Maximum water level
L.W.L.	Low water level

TABLE OF CONTENTS.

CHAPTER I.

General Remarks and Definitions.

Para		Page
1, 2	Definition of irrigation	1
3.	Irrigated 'dry crops'	1
4.	'Irrigated wet crops'	1
5.		2
6.	'Flow' and 'lift' irrigation	2
7.	'Perennial' and 'inundation' irrigation	2
8.	Signification of term 'perennial irrigation' as generally used in Madras	2
9	'Direct' and 'tank' irrigation	3
10.	Rice, the irrigated crop generally referred to	3
11.	Considerations which determine the methods of irrigation to be adopted.	3

CHAPTER II.

Water-supplies required for irrigation.

12.	'Duty of water'	4
13.	Duty expressed in 'acres per cusec'	4
14.	Duty in terms of quantity of water impounded	4
15.	American notation for 'duty of water'	4
16.	Duty in terms of 'depth over the area'	4
17.	Relations between different expressions for duty of water	5
18.	'Base' of duty and 'crop period'	5
19.	Place of measurement to which duty has reference	5
20	Requirements for precise statement of duty of water	6
21.	Relation between duty in 'acres per cusec' and 'depth over area'	6
22.	Variations in the duty of water	6
23.	Watering 'dry crops'	6
24	'Dry crops' in Madras	7
25.	'Dry crops' in Northern India	8
26.	Duty for rice	8
27.	Transplantation of rice	8
28.	Rice after transplantation	9
29.	Duties in the Kistna and Gōdāvari deltas	9
30.	Effect of rainfall on speed of transplantation	9
31.	Water required for rice after transplantation	10
32.	Great variation of duty of water under different conditions	10
33	Duty of water for 'tank irrigation'	10
34.	Fixing duty specially for each tract irrigated	10
35	Examples of duties proposed for projects in Madras	10
36	Water for seed-beds	1

CHAPTER III.

Water-supplies available for irrigation.

Rainfall.

37	Rainfall	13
38.	Description of a rain gauge	13
39.	Setting up a rain gauge	14
40.	Observing and registering rainfall	14
41.	Rain recording stations	14
42.	Variation of rainfall over an area	15

PARA.		PAGE
43.	Estimate of average rainfall over any area	15
44.	Variation in annual, or seasonal, rainfall at the same station ..	16
45.	Percentage of errors in estimating mean annual rainfall ..	16
46.	Correction of estimate of mean rainfall in reference to records of an adjacent station	16
47	Fluctuations of annual rainfall and their bearing on irrigation projects	17
48	Relation of maximum and minimum to mean annual rainfall ..	17
49	Statements of mean, maximum, and minimum annual rainfall for the Madras Presidency	17
50	Average rainfall of a bad year	20
51.	Definition of 'catchment basin'	20

RUN-OFF

52.	Definition of 'run off'	20
53	Irrigation supply wholly derived from 'run-off'	20
54.	Measurements of run-off much to be preferred to estimates based on rainfall . .	20
55.	Disposal of rain falling on any catchment	20
56.	Factors which affect run-off . .	21
57	Estimating the run-off from seasonal rainfall	21
58.	Strange's table of run off due to monsoon rainfall	21
59	Estimating run-off from total monsoon rainfall	23
60	Objections to estimating run-off from the total monsoon rainfall	23
61	Strange's table of run-off from daily rainfall	23
62.	Classifying condition of catchment as regards humidity ..	24
63.	Run-off by measuring stream flow	25
64.	Importance of gaugings of stream flow even for one or two seasons	25
65.	Example of above	25
66.	Percentages of run off from rainfall corrected from stream flow measurements	27
67.	Method of recording stages of stream flow	27
68.	Inaccuracies due to recording flow from intermittent gauge readings	27
69.	Selection of suitable positions for erection of gauges	27
70	Considerations affecting the position of gauges in rivers and streams .	28
71.	Some details of gauge erection	28
72.	Zero of a gauge to be connected with bench mark	29
73	Gauge wells . . .	29
74	Automatic continuous water level recorders	31
75.	Methods of computing flow from gauge readings	31
76	Selection of site for measuring flow of a stream ...	32
77	Methods of measuring velocity of flow ..	33
78	Measurement of velocity by surface floats .	33
79.	Method of computing discharge from surface velocities	34
80.	Float observations with velocity rods .	34
81.	Measurements by current meter	35
82.	Methods of observing mean velocities by current meter . .	35
83	Mean velocity by one and two point methods	35
84	Surface velocity by current meter	36
85	Methods of operating the current meter .	36
86	Weirs used for measuring flow-off .	36
87	Gauge calibration curves	37
88.	Run off computed from change of water level of an intercepting reservoir	37

FLOOD DISCHARGE.

89.	Maximum flood discharge from a large catchment .	37
90.	Floods from rainfall in moderate sized catchments .	38
91.	Ryves' and Dickens' formulæ	38
92.	Table of flood discharges by Ryves' formula	39
93.	Rainfall—Formulæ for flood discharge only applicable to moderate and small sized catchments	39
94	Standard area of maximum precipitation of rain ..	40
95.	Selection of co-efficient in Ryves' formula	40

PARA		PAGE
96.	Consideration of suitability of co-efficient with reference to data available other than the maximum rainfall in one day	41
97.	Effect of nature and shape of catchment on the co-efficient	42
98.	Co-efficients frequently applicable in Madras	42
99	Other flood formulæ	42
100.	Marking and recording flood levels and flood discharges	42

CHAPTER IV.

PERCOLATION AND EVAPORATION

101.	Percolation	44
102.	Percolation and absorption	44
103.	Seepage	44
104	Mean annual percolation of a catchment	44
105.	Effect of percolation on irrigation supplies	45
106	Percolation in soils of different kinds	46
107	Percolation through rock	46
108.	Grit and plastic soils	46
109	Permeability of soils	46
110.	Percolation gradient	46
111	Water table and ground water gradient	47
112.	'Saturation gradient'	47
113	Saturation gradient in a bank shown by pipes	48
114	Uplift pressure exerted by ground water	48
115	Velocity of percolation through sand	49
116	Mechanical analysis of sand	49
117.	Sand analysis useful for comparing designs with existing works	49
118.	Percolation past irrigation works	50
119	Stability of works affected by percolation	50
120. 121	Limiting velocity of percolation under foundations	50 51
122	Percolation through embankments	51
123. 124	Upward pressure on works from sub soil percolation	52 53
125.	Best position of pipes or wells for checking percolation	53
126	Rate of evaporation	53
127. 128.	Measurement of rate of evaporation	54
129	Evaporation and absorption	55
130.	Loss from tanks	55
131	Losses from tanks in South India from evaporation and absorption	55
132	Losses in canals and distributary channels	55
133	Losses from evaporation and absorpt on in canals and distributaries	56
134. 135.	Percentage losses by evaporation and percolation	56 57

CHAPTER V.

SILT AND SCOUR.

136	Classes of silt	58
137.	Factors influencing the nature and quantity of river silt	58
138	Nature of silt in Madras rivers	58
139.	Measurement of quantity of silt in suspension	58
140.	Taking samples for estimating quantity of silt	59
141	Mechanical analysis of nature of silt in suspension	59
142.	Punjab system of sand analysis	59
143.	Proportions of silt carried in various rivers	60
144.	Silt quantities in the Kistna and Cauvery rivers	60
145	Relation between measurement by weight and by volume	60
146.	Quantity of silt yearly transported to the sea by large rivers	61
147.	Exclusion of bed silt from silt measurements	61
148.	Alluvium and alluvial formations	61
149.	Rivers running through alluvial plains	61
150.	Silt deposit in lakes	61

PARA.		PAGE
151	Formations of deltas	62
152.	Formation of bars	62
153	Distinctive features of deltaic tracts	63
154	Gradual flattening of surface slope of delta river	65
155.	Liability to constant change of delta river channels	65
156	Characteristics of delta rivers	65
157.	Irrigation facilities in deltas	65
158.	Drainage difficulties in deltas	66
159	Silt transporting power of water	66
160.	Kennedy's theory of critical velocities	66
161	Kennedy's theory of silt transportation	67
162. 163.	Kennedy's law considered theoretically	68
164.	Reduced values of Kennedy's Vo	69
165	Variation of velocity to suit different kinds of silt	69
166.	Deposit of heavy silt near canal heads	71
167.	Silt deposit in tanks	72
168.	Regulation of impounding to avoid silt deposition	73
169	Scour	73
170.	Irrigation works and scour effects	74
171.	Nature of damage done to irrigation works by scour	74
172	Providing protection against scour	74
173.	Eroding power of clear water	75

CHAPTER VI.

HEAD WORKS.

174.	Irrigation head works	76
175.	Diversion works	76
176.	Level of weir crest	76
177	Disadvantages of a solid weir	77
178.	Regulator substituted for a weir	77
179.	Head sluice	77
180.	Scouring sluices	78
181.	Divide groyne	78
182	Plans of typical head works	78
183	System of regulating head and scouring sluices	79
184	Flood banks	79
185.	Sites for diversion works	80
186.	Favourable conditions for sites of head works	80
187. 188	Direction of a weir with reference to river axis	81

STORAGE WORKS.

189.	Storage works as head works	81
190.	Component works of a reservoir	81
191.	Reservoir dams	81
192.	Reservoir supply sluices	82
193.	Reservoir surplus works	82
194. 195.	Favourable conditions of site for reservoirs	82 83

CHAPTER VII.

HEAD WORKS—MASONRY DAMS.

196.	Definition of masonry dam and weir	84

GRAVITY DAMS.

197 198.	Distribution of pressures in a masonry structure	84
199.	Diagram of distribution of pressure in a joint	85

PARA.		PAGE
200.	Deductions in respect to distribution of pressure	87
201.	Stability conditions of a dam	87
202. 203	The middle third rule	87
204.	Security against sliding	88
205.	'The elementary profile'	89
206.	Weight and specific gravity of masonry	89
207. 208.	Maximum stress due to resultant pressure	90 91
209.	Equations giving relations between pressures and stresses	91
210.	Unit for computation of pressures in force diagrams	91
211.	Stresses worked out in tons per square foot	92
212.	Pressures in an elementary profile of specific gravity 2½	93
213	Limit of depth of elementary profile	93
214.	'High' and 'low' dams	93
215.	Crest width of a masonry dam	94
216	Free board of a dam	94
217. 218	Profile for a 'low' dam	94 96

HIGH MASONRY DAMS.

219	Design of a 'high' dam	96
220	Formulæ applicable to design of 'high' dams	96
221	Example of design of a 'high' dam	97
222.	Practical profile based on the theoretical profile	99
223	Essential divisions of a joint of a dam	100
224.	Formula for stress in the joint of a dam	100
225.	Method of moments for testing stability	101
226	Example of test by the method of moments	102
227.	Graphical construction for pressure on up stream sloping face	104
228.	Stability diagram of a dam	105
229	Analysis of results of the above example	108
230. 231.	Another graphic method	109 110
232	Examples of existing high dams	110
233	The Assuan dam	112
234. 235	'Uplift' and 'flotation'	113 114

ARCHED MASONRY DAMS.

236.	Arched masonry dams	115
237.	Stress in arches under water pressure	116
238.	Formulæ giving stress in arched dams	116
239	Theoretic profile, a triangle	117
240.	Crest width of arched dams	117
241.	Maximum stresses allowed in arched dams	117
242.	The Bear Valley dam	117
243.	Australian arched dams	118
244.	The Path-finder dam	119
245.	Arched dams only applicable to narrow gorges	120

BUTTRESS DAMS.

246	Buttress dams	121
247.	Stability of Buttresses	121
248	Panelling between buttresses	121
249.	Arched buttress dams	121
250.	Spacing of buttress for arched dams	122
251.	Stresses due to the weight of masonry of an inclined arch	122
252	Example of a design of an arched buttress dam	122
253.	Quantity of masonry as compared with gravity dam	125

PARA.		PAGE
355.	Qualities of plastic and grit soil	188
356.	Face slopes of earthen bunds	189
357.	Failure by overflow or topping of the bund	189
358.	Failure by erosion	189
359.	Type of bunds of tanks in South India	189
360.	Common dimensions of tank bunds for restoration work	190
361	Earthen bunds, Type B	191
362.	Earthen bunds with core walls, type C	192
363	Saturation gradient in banks	193
364.	Drains under the outer slope of a tank bund	194
365.	Top width and free board of tank bunds	195
366	Breaching sections	195
367.	Revetment to tank bunds	195
	DESIGN OF EARTHEN BUNDS.	
368.	Cross section of bund to suit the earth available	196
369	Foundation of the bund	197
370.	Profile of the bund	198
371	Important points in regard to actual execution of embankments	198
372	Junctions in embankments	200
373.	Draining or filling of scour holes	200
374.	Testing soil for embankments	201
375.	Testing the permeability of a tank bed	201
	RESERVOIR SUPPLY SLUICES	
376.	Tank sluices	202
377	The culvert or barrel of the sluice	205
378.	Foundations of sluice culverts	205
379	The superstructure of a sluice culvert	205
380.	Stop walls across the barrel of a sluice	206
381.	Head wall and wings of tank sluices	206
382.	Sluice with a tower head	206
383.	Method of regulation of tank sluices	207
384	Details of plugs for tank sluices	208
385.	Cistern or wings at tail end of sluice barrel	209
386	Selection of site and sill level of a sluice	210
387	Information which should be recorded on the design of a sluice	211
	TANK SURPLUS WORKS.	
388	Surplus works of tanks	211
389	Types of tank weirs	212
390.	Principles applicable to anicuts also applicable to tank weirs	212
391	Working conditions of anicuts and tank weirs contrasted	212
392	Positions suitable for weirs of types B and C	213
393.	Positions suitable for type A weirs	213
394.	Weirs (type D) with stepped aprons	216
395.	Material for aprons of tank weirs	216
396.	Width of floors of tank weirs	216
397.	Bank connexions of weirs	217
398	Flush escapes	217
399.	Dam stones on tank surplus escapes	218
400.	Sluices as surplus works of tanks	219
401.	Surplus vents or notches in weir walls	219
402.	Approach to surplus works	220
403.	Surplus channel	220
404	Disposal of surplus water from tanks	221
405.	Detail plans of sites of surplus escapes	221
406	Example of site plan and connected weir design	222
407.	Choice of sites for surplus works	224
408	Groups of tanks	224
409.	Computation of capacity of surplus works of tanks	225
410.	Flood absorbing capacity of a tank	225
411	Computation of flood discharge from combined catchments	226

PARA. PAGE

412. Computation of the capacity of a tank 226
413. Co-efficients of discharge of tank weirs and flush escapes 227
414. Capacity of supply channels for tanks 227
415. Working tables 228

CHAPTER XI.

DISTRIBUTION WORKS—CANALS AND CHANNELS.

416. Distribution works 229
417. Classification of distribution channels 229
418. Different methods of alignments of irrigation channels 229
419. Points to be aimed at in the lay-out of a distribution system 230
420. Main canals as contour channels 230
421. Offtakes from contour canals 230
422 Typical distribution system 232
423. Plan showing part of typical distribution system 232
424. Alternative alignments of contour canals 232
425. Lay-out of field channels 233
426. Standards of canals 234
427. 'Balancing depth' of cutting 235
428. Rule for width of berms 235
429. Side slopes of canals 237
430. Curves in canals 237
431. Location of a contour canal 237
432 Velocities of flow and limit of depth of canals 237
433. Proportion of bed-width to depth in the best discharging channel .. 238
434. Cross section of least absorption 238
435. Proportionate bed width to depth for canal of fixed discharge 239
436. Tables of dimensions of canals and distributaries 239
437. Characteristics of channels of non-silting velocities with varying proportions of bed width to depth 242
438. Balancing depth too shallow for large channels and too deep for very small ones 242
439. Kutter's co-efficients for channels of different sizes 242
440. The proportion of bed width to depth to be fixed having in view the slope of the country traversed 243
441. Other considerations for fixing the proportion of bed-width to depth . 243
442 Depth of cutting of a channel 243
443 Depth of a channel bed above a 'drop' 244
444. Capacity of canals 246
445. Command 246
446. Considerations influencing the proportions of cross sections of channels. 247
447. Change of carrying capacity of a canal 248
448. Changes of depth of a channel 248
449. Mile and bench-mark stones 248
450. Contour channels in single bank 249
451. Tanks formed at drainage crossings used as storage works 249
452. Masonry works of distribution systems 250

CHAPTER XII.

DISTRIBUTION SYSTEM—MASONRY WORKS.

CANAL 'DROPS' OR 'FALLS.'

453. Canal 'drops' 251
454. Positions of 'drops' 251
455. Essential features of a drop 251
456. Control of the velocity and of the water level in the approach to a drop. 251
457. Control by 'trapezoidal notches' 252
458. Theoretical and practical form of trapezoidal notch 252
459. Water levels for which notches should be computed 252
460. Details of plan of a trapezoidal notch 252

PARA.		PAGE
461.	Formulæ for discharge of a trapezoidal notch	253
462.	Nethersole's discharge tables of trapezoidal notches	254
463.	Example of calculation for a trapezoidal notch	254
464.	General features of design of drops	255
465.	Profile and length of drop wall and number of notches	255
466.	Width of the main apron of a drop	256
467.	Depth and limits on use of water cushions	256
468.	Lengths of main aprons of drops	257
469.	Length of revetment and bed pitching	257
470.	Bank connexions	258
471.	Example of design of a drop	258
472.	Testing a design for percolation and uplift	261
473.	Splayed wings	261
474.	Example of drop with solid apron and core wall bank connexions	262
475.	Modifications necessary to get a percolation gradient of 1 in 5	264
476.	Types of bank connexions contrasted	265
477.	Designs of existing drops	266
478.	Syphon well drops	268

CANAL REGULATORS AND DIVIDING DAMS.

479.	Canal regulators	271
480.	Regulating notches	271
481.	Designs of regulators	271
482.	Dividing dams	273

IRRIGATION SLUICES

483	Irrigation sluices	275
484.	Shutters of canal sluices	277
485.	Types of irrigation sluices	278
486.	Pipe outlets	280
487.	Fixing size of pipes in outlets	282
488.	Calculation of discharge of pipe outlets	282
489	Table of discharge of pipe outlets	283
490.	Defects in distribution by pipe outlets	284
491.	The 'Kennedy gauge outlet'	284
492.	Limitations on usefulness of Kennedy gauge outlets	285

COMMUNICATION WORKS.

493.	Bridges	285
494.	Rood syphons	286
495.	Cart and cattle crossings	287
496.	Cattle crossings in deep canals	288

RETAINING WALLS FOR IRRIGATION WORKS.

497. 498.	Remarks on retaining walls	289 290
499.	Retaining walls with sloping tops	291
500.	Building a winding slope	292

CHAPTER XIII.

CROSS DRAINAGE WORKS AND DRAINAGE OF IRRIGATED TRACTS.

501.	Methods of disposing of cross drainage	293
502.	Aqueducts and syphon aqueducts	293
503.	Types of aqueducts and syphon aqueducts	293
504.	Selection of a suitable type of aqueduct	294
505.	Headway required at the point of crossing of drainage by an aqueduct	294
506.	Lowering bed of drainage below a syphon aqueduct	295
507.	Considerations to determine the site of aqueduct or syphon aqueduct, and which type of work is most suitable	296

PARA.		PAGE
508.	Crossing a drainage by an irrigation syphon	296
509.	Example of type III syphon aqueduct	296
510.	Profile of abutments of aqueduct	299
511.	Example of type III syphon aqueduct	299
512.	Bank connexions	299
513.	Example of type III syphon aqueduct	300
514.	Uplift pressures on syphon flooring	300
515	Example of type III syphon aqueduct	301
516.	Uplift pressures on overhead covering of syphon culverts	302
517.	Example of type III syphon aqueduct	303
518.	General remarks on uplift on the coverings of syphons	303
519.	Formula for the discharge of an inverted syphon	304
520.	Head on a syphon	304
521.	Example of type III syphon aqueduct	304
522.	General remarks on the design of a syphon aqueduct	305
523.	A type III aqueduct suitable for a surplus work	305
524.	Contraction of waterway through an aqueduct	305
525.	Loss of head or heading up in a canal at a contracted aqueduct	306
526.	Formula and calculations for head required	306
527.	Example of type III syphon aqueduct	307
528.	Normal limit of heading up due to contraction of an aqueduct	308
529	Example of aqueduct of type II	308
530.	Drainage culvert or syphon aqueduct of type I	310
531.	Examples of existing aqueducts and syphon aqueducts	310
532.	Superpassages and syphons	312

LEVEL CROSSINGS, INLETS, AND OUTLETS.

533.	Level crossings	315
534.	Inlets and outlets	316
535.	Inlets	316
536.	Canal outlets or escapes	317
537.	Surface outlets	317
538.	Outlet or surplus sluices	318
539.	Outlets built in connexion with inlets	319
540	Objection to disposal of cross drainage by inlets and outlets	319
541.	Outlets not connected with inlets	319
542.	Outlets for canals carried in single bank	320
543.	Canal scouring sluices	320
544.	Scouring sluice in the first reach of main canal	320
545.	Diversion of natural surface drainage by contour irrigation canals	320
546.	Catch drains	321
547.	Tail escapes	322

DRAINAGE WORKS.

548	Drainage of the irrigated area	322
549.	Grading of drains	322
550.	Banks of drains	323
551.	Drainage outfall sluices	324
552.	Capacity of drains	327
553.	Combined irrigation and drainage channels	327

CHAPTER XIV.

NAVIGATION COMBINED WITH IRRIGATION WORKS.

554.	Navigation canals	329
555.	Limits of velocity of navigation canal	329
556.	Depths and widths of navigation canals	329
557.	Extra water required for navigation	329
558.	Special works required for navigation purposes	329
559.	Canal locks	330
560.	Lock floors and side walls	330

Para.		Page
561.	Dimensions of lock chamber	330
562.	Chopella lock, Gödâvari Eastern delta	331
563.	Lock sluices	333
564.	Lock gates	333
565.	Tidal locks	333
566.	Locks with sloping sides	334
567.	Double locks	335
568.	Mooring posts in locks	335
569.	Site of locks and lock weirs	335
570.	Lock weirs	337
571.	Fender piles	337
572.	The tow-path	337
573.	Bridges on navigation canals	337
574.	Desirability of combining navigation with irrigation	338

CHAPTER XV.

Regulating Shutters and Gear.

576.	Weir crest shutters	340
577.	Fouracres' falling shutters	341
578.	Computation of position of pivot of Fouracres' falling shutters	341
579.	Ashford's patent drop shutter	343
580.	Large span lift shutters	344
581.	Screw geared sluice gates moving on plane bearing surfaces . .	347
582.	Dimensions of spears and screw threads	352
583.	Non-rising screw spears	353
584.	Needle shutters	353
585.	Regulation by horizontal planks	353

IRRIGATION.

CHAPTER I.

GENERAL REMARKS AND DEFINITIONS.

Definition of Irrigation.—Irrigation may be defined as the supply of water by artificial means for the cultivation of crops, and the works required to give effect to such supply are 'irrigation works.' The function of irrigation is to supplement the supplies of water falling in the form of rain on the area to be cultivated at the season and to the extent required for the successful cultivation of the crop.

2. When the natural rainfall on the area cultivated, as regards both quantity and seasonal incidence of fall, is suitable for the cultivation, irrigation is not required, and in countries (such as England) in which the normal rainfall is sufficient for the watering of the crops grown, irrigation is not practised. In Southern India the normal quantity and incidence of rainfall is generally suitable and sufficient for the cultivation of most of the kinds of crops raised, such as millets, cotton, etc., and these crops are in consequence in the Madras Presidency generally referred to as 'dry crops' and are not irrigated. It must, however, be realized that in other parts of India, in Egypt and in other countries where the rainfall during the crop season is smaller these same crops are normally irrigated.

3. Irrigated 'dry crops.'—'Dry crops' are occasionally irrigated in Madras with a view either of raising the crop during the season of the year when normal rainfall is insufficient or of getting a better outturn of crop by securing means of watering the crop at will and thus insuring the crop from injury owing to temporary failure of rain. Considerable areas of 'dry crops,' especially *ragi*, are irrigated in Madras under private sources, generally wells, but the areas normally irrigated by Government irrigation works are small

In years of deficient rainfall, however, considerable areas of 'dry crops' which are not normally irrigated are watered from Government irrigation works.

This is specially the case under the Kurnool-Cuddapah Canal, the area of dry crops irrigated under which expands greatly in years of deficient rainfall and which in consequence is a most valuable famine protective work.

4. Irrigated 'wet crops.'—While crops which are ordinarily grown without irrigation in Madras are termed 'dry crops' conversely those which cannot normally be grown without irrigation are 'wet crops.' Generally speaking 'wet cultivation' in Madras is confined to rice, of which many varieties are grown with

crop periods varying from three months to seven months. The terms 'wet' and 'dry' crop in connexion with irrigated areas may possibly be derived from the fact that in the case of the former the irrigated area is usually kept covered by several inches of standing water, whereas this is not so in the case of irrigated 'dry crops.'

5. In addition to the above there are specially valuable crops subjected to intensive cultivation and requiring water for at least nine months and frequently throughout the year These are termed 'garden' crops or sometimes 'dufassil' crops. These include sugarcane, plantains, etc.

6. **'Flow' and 'lift' irrigation.**—Where the supply of irrigation water available is at such a level that it is conveyed on to the land by the force of gravity, that is by 'flow' it is known as 'flow' irrigation as contrasted with 'lift' irrigation, which must be practised when the water supply is at too low a level to run by gravitation on to the land. In such case the water level must be raised by pumps or other water lifting devices The most common application of 'lift' irrigation is irrigation from wells.

It is not proposed in this text book to deal either with 'lift' or 'well' irrigation and only 'flow irrigation' will be dealt with.

7. **'Perennial' and 'inundation' irrigation.**—There are two main classes of 'flow' irrigation systems—

(*a*) Perennial systems.

(*b*) Inundation systems

Most of the systems in India are of the perennial type. Inundation irrigation is carried on by deep flooding and thorough saturation of the land to be cultivated which is then drained off prior to the planting of the crop, the moisture stored in the soil by inundation serving when supplemented by natural rainfall and in some cases by occasional minor waterings to bring the crop to maturity. This system has been in operation in Egypt, Sind and elsewhere for many centuries It is also termed 'flood' irrigation.

The term 'perennial' irrigation *in contrast to* '*inundation*' *irrigation* implies that the water required for irrigation is supplied in accordance with crop requirements throughout the 'crop period' and that floods are not necessary for passing water by flow on to the irrigated area

The term *used in the above connexion* does not imply that water is supplied throughout the year. It must however be explained that in Southern India (Mysore and Madras) this term is generally used in the latter sense and has reference to crops such as sugarcane, plantains, etc., of a kind requiring watering throughout the year.

There is no distinct line of cleavage between 'flood' and 'perennial' irrigation and many canals which at one time were merely flood channels have by improvement acquired such facilities for irrigating from low river flow as to merge into the perennial class of work Similarly it occasionally happens when irrigation works have been neglected that canals, once perennial sources have become only inundation sources of supply.

8. **Signification of term 'perennial irrigation' as generally used in Madras.**—As there is little inundation irrigation in Southern

India, it is customary there to use the term 'perennial irrigation' as referring to cultivation requiring water throughout the year.

9. 'Direct' and 'tank' irrigation.—'Flow' irrigation is further classified in accordance with the source from which the water is drawn as:—

(1) 'Direct' irrigation (termed 'canal' irrigation in North India).

(2) 'Reservoir' or 'tank' irrigation.

'Direct' irrigation implies that the water is drawn directly from the flow of a river or stream without the intervention of any storage work, while the water for 'tank' irrigation is drawn from tanks. Many systems are a combination of both methods.

10. Rice, the irrigated crop generally referred to.—Flow irrigation in Madras is almost exclusively devoted to the cultivation of rice and the duties and systems of irrigation and distribution described in this text book when not otherwise stated have reference to rice cultivation. In 1916-17 (fasli 1326) out of a total area of 3,818,072 acres irrigated by Government works in Madras, for which capital and revenue accounts are kept, no less than 3,574,393 acres (over 93 per cent) was rice.

11. Considerations which determine the methods of irrigation to be adopted.—An irrigation reservoir does not increase the water-supply of a catchment but merely furnishes a means of regulating the flow-off of that supply in a way suitable for the requirements of the crops and thus by impounding water when natural flow-off is in excess of immediate requirements holds it available for issue when natural flow falls below this limit.

If the river or stream constituting the source of supply of an irrigation system has a normal flow-off throughout the irrigation season never less at any time than the requirement of the irrigated area the 'direct flow' method of irrigation is clearly indicated. Where the whole yearly or seasonal flow-off although of fully sufficient total quantity is normally insufficient during certain parts of the crop season for irrigational requirements, the conditions call for artificial means of regulating the flow-off by storage works and 'tank irrigation' must be resorted to.

CHAPTER II.

WATER SUPPLIES REQUIRED FOR IRRIGATION.

12. 'Duty of water'—The relation between the area of crop irrigated and the quantity of irrigation water required to supply it is technically termed the 'duty of water'

13. Duty expressed in 'acres per cusec'—Duty may be stated in various ways by employing different notations but in India it is most frequently stated in terms of the number of acres of cultivation of any kind, which can be, (or has been), irrigated by a flow averaging one cubic foot per second (one cusec) for the whole crop season, or for any specified period

'Duty' is referred to as being 'high' or 'low' according as the number of acres per cusec irrigated is large or small

In America one cubic foot per second is called a 'second foot' but in India a cusec.

14 Duty in terms of quantity of water impounded—The notation of paragraph 13 above is a convenient one for expressing duty when considering direct flow irrigation, in which the discharges of the distribution canal and channel are expressed in cusecs, but there are occasions in which it is convenient to use other notations to express the 'duty of water'

This is specially so with reference to tank irrigation as in such cases the areas which can be irrigated are dependent on the amounts of water stored in the tanks supplying them

In India reservoir capacity is usually stated in millions of cubic feet, and it is therefore frequently convenient to express the duty of water in connexion with storage systems in terms of the number of acres of irrigation which can be supplied per million cubic feet of impounded water Under such notation duty is stated as '. . . . acres per million cubic feet.'

15 American notation for 'duty of water'—In America the capacity of storage works is usually stated in *acre-feet*, one *acre-foot* being the quantity of water required to cover one acre to a depth of one foot (viz., 43,560 cubic feet) Under such a notation the duty of water is the number of acre-feet required to supply an acre of crop

It is clear that this notation is a convenient one, as, if the depth of irrigation water required for a crop is known, the area irrigable from a reservoir of stated seasonal storage in acre-feet is at once apparent The notation is occasionally used in India.

It may here be noted that one 'acre-foot' is almost equal to one cusec flowing for half a day (12 hours)

16. Duty in terms of 'depth over the area.'—The quantity of water required for irrigation purposes depends greatly on the normal quantity of rain falling on the area to be irrigated and as this is measured in terms of the depth falling on this area, it is clear that it may be convenient to express the 'duty' of water in the same notation; duty is therefore frequently stated in terms of

depth of water over the irrigated area required to bring the crops on it to maturity.

The duty in this form, as also in other cases, is generally stated so as only to include irrigation water supplied, but some times it is stated so as to include the total depth of water, that is, rain *plus* irrigation supply. Where rainfall is included this must always be specifically stated, failing which the duty refers to irrigation water only.

For example, the duty of water for a crop requiring a total depth inclusive of rain of 65 inches on the field, where 25 inches useful rain can be counted on during the crop season, might be stated either as --

(*a*) A duty of 40 inches delivered on the field, or

(*b*) A duty on the field, inclusive of rain, of 65 inches.

17. Relations between different expressions for duty of water --The relations between different notations and units for expressing duty are stated below :—

1 cusec flowing one day	= 0864 million cubic feet.
	= 2 acre-feet (1 98 exactly).
	= 24 acre-inches nearly (23'76 exactly)
1 million cubic feet	= 11'574 cusecs flowing for one day
	= 23 acre-feet nearly (22'96 exactly).
1 acre-foot or 12 acre-inches	= ½ cusec flowing 24 hours (nearly).

One acre per million cubic feet is equivalent (approximately) to a duty of 12 acres per cusec for a crop period (base) of 140 days

18. 'Base' of duty and 'crop period' --Unless the contrary is specified duty is generally taken to refer to the whole period of cultivation from the time when irrigation water is first issued for preparation of the ground for the planting of any crop, to its last watering before harvesting This will be referred to as the 'crop period.'

The quantity of irrigation water required for a crop varies greatly at different times, and the 'duty' for the whole 'crop period' is an average duty and does not give information of the actual rate of supply on any day, or series of days

The term 'duty' is however frequently used with reference to period of time less than the whole 'crop period' and the period to which the stated duty has reference is termed its 'base' *To be precise it is necessary when stating a duty, also to state the base.*

When the base is not stated and when there is nothing in the context repugnant to this assumption, it is generally understood that the duty refers to the whole crop period

19. Place of measurement to which duty has reference – To clearly define the significance of any specified duty, in addition to specifying the base, it is also necessary to state the place of measurement of the water

Losses of water in transmission between the head of a canal system and the fields, by evaporation, percolation, and other causes, are in many systems very considerable, and the quantity of water

delivered on the fields is different from that passing into the system at the head, and therefore the duty 'on the field' is different from the 'duty at the head.'

For instance, if losses in transmission amount to 20 per cent of the water entering at the head, then if the duty on the field is 40 inches (equivalent to 96 acres per cusec to a base of 160 days), then the 'head duty' would be 50 inches (equivalent to 77 acres per cusec to a base of 160 days).

Again in the case of a reservoir if the duty has reference to the amount of water required to be stored in the reservoir, the losses by evaporation and percolation from the reservoir must be allowed for in addition to losses in transmission and added to the quantity required on the fields

20. Requirements for precise statement of duty of water.—In order to clearly define duty it is therefore necessary to state at the same time :—

(*a*) the base,
(*b*) the place of measurement.

The duty stated may be in terms of :—

(1) acres per cusec, or
(2) inches in depth on the irrigated area, or
(3) acres per million cubic feet of water.

21. Relation between duty in 'acres per cusec' and 'depth over area.'—The following is a convenient equation connecting the different notations used for expressing duty :—

If d = duty in acres per cusec,
Δ (delta) = equivalent depth in inches,
and B = base in days,

$$\Delta = \frac{24\ B}{d}, \quad d = \frac{24\ B}{\Delta} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Thus a duty of 90 acres per cusec is equivalent to 0·26 inch per day or say 8 inches per month.

NOTE.—The above equation is not exactly correct, but the error is small, and the form so simple that it is in general use.

22. Variations in the duty of water.—Duty of water varies with the nature of the soil, the nature of the rainfall, and the nature of the crops irrigated. In South India irrigation is almost altogether devoted to the cultivation of rice, but the duties for other crops will be briefly referred to.

23. Watering 'dry crops.'—Although irrigation of dry crops is but little practised in South India it is proposed to give some information on the subject of the methods of applying water to dry crops and the amount of the water required.

Under normal conditions the surface soil in the areas growing dry crops is at the time of watering either quite dry, or only very slightly moist, and the conditions under which water is applied to such areas differ greatly from those under which water is given to rice fields which are normally kept submerged under three or four inches of water.

Under the latter conditions it is not a matter of any great importance how the water required to replenish the supply to a field,

is given, or if each watering is completed in two hours or is spread over two days or is given as a continuous flow.

In the case of water applied to a dry field this is however a matter of great importance, for it will be apparent that when water is supplied from one side of a level field surrounded by a small bund it will spread over the field and reach the other end far more quickly if the flow is large than if it is small; the time taken for water to reach every part of a field varies generally with the area (and shape) of the field and the rate of flow of the water. This is a matter of importance because the soil over which the water is flowing is absorbing water all the time and a great part of this absorbed water is not required for the sustenance of the crop.

The most economical conditions of watering a dry field is when, as happens under the supply by rainfall, the water is applied simultaneously to an equal depth all over the field. This condition cannot be realized by irrigation, but in order as far as possible to approximate to it the size of the embanked area should be small, and the rate of flow into each area sufficiently large to prevent great waste in applying water to dry crops.

In irrigation from wells it is usually impossible to supply a considerable flow of water and the universal practice is to divide up the area into separate embanked plots sometimes as small as 500 square feet, and generally not greater than 1,000 square feet; in Northern India these compartments are known as 'kiaris.'

Under flow irrigation it is not necessary or practicable to divide up the whole area into such small divisions as the flow which can be given to each compartment is much larger than that from wells. Generally it should be arranged that the direct flow supply to each compartment should be of not less than half a cusec and the compartment should be one-quarter of an acre or less in area, if practicable as small as one-eighth acre.

It is found in practice that the crop at the boundary of the plot remote from the source of supply receives a sufficient watering if the supply is shut off as soon as surface flow reaches that boundary, and the quicker this can be effected the less the waste by useless percolation into the soil in the parts of the plot receiving earlier watering.

Apart from the size of the plot and the magnitude of the flow the quantity of water required to water a plot fully is greater for the first watering than for the subsequent ones. Generally it may be taken that for a first watering in loam soil a quantity of water of not less than four inches depth is required, while subsequent waterings require from 2½ to 3 inches

24. 'Dry crops' in Madras.—Flow irrigation to dry crops in Madras is practised only over a very restricted area. The kind of dry crops chiefly irrigated are *cholam*, *jowari* and *ragi*.

A supply, inclusive of rain of 6 inches a month, is sufficient for such crops on loam soils and when grown on very retentive soils, such as black cotton, even less water suffices.

Under the Kurnool-Cuddapah canal in years of deficient rainfall a large area of *cholam* is watered and the duty allowed for is 120 acres per cusec.

The first watering requires about 6 inches depth and each subsequent watering which in the absence of rain should be given at

ıbout twenty days' interval requires about 4 inches depth of water ıver the area.

As it is desirable to water the crop under each distributary as ·apidly as possible, distributary channels for dry crops should be nade to supply the area at a duty of about 60 acres per cusec; his would suffice to give a 6-inch watering to the whole area under :ach distributary in fifteen days and a 4-inch watering in ten lays.

Distributary channels intended for irrigating dry crops should ɔe run full whenever open and between waterings should be kept ɛntirely closed. This results in diminution of percolation losses ın channels and water courses which, in the case of irrigated dry ɔrops, are likely to be proportionately much greater than ın the ɔase of wet crops. This system also keeps every outlet pipe dis-ɔharging full whenever a channel is open and greatly facilitates ɛquitable distribution.

25. 'Dry crops' in Northern India.—In the summer (kharif) season in Northern India irrigated crops are similar to those grown in South India, and the canal head duties realized vary from 60 to 120 acres per cusec according to the varying condition of soil and rainfall.

The winter (rabi) crops are generally of a nature which would not grow in the warm climate of South India. The chief rabi crops are wheat and barley and these in the Punjab require 4 to 5 waterings. The duties realized for the rabi crops are much higher than for the 'kharif,' and vary from 110 to 220 acres per cusec and are greatly dependent on the actual incidence of rainfall during the crop season.

The base of the above duties is five to six months.

26. Duty for rice.—In Bengal the duty for rice on the field *inclusive of rainfall* is stated by Buckley as varying from 56 to 75 inches and this agrees fairly with what is found to be the duty in Madras for a single crop of rice

The above are duties for a 'crop period' of about 140 to 160 days. During this period the amount of water required for the rice crop varies considerably and may be separated into two distinct parts:—

(*a*) Water required for the preparation of the land to receive the crop.

(*b*) Water supply required for the crop after it is planted.

27. Transplantation of rice.—The preparation of land for rice cultivation usually consists of flooding the land till water stands several inches in depth over it and ploughing it when in this state, so as thoroughly to puddle the surface soil, reducing it to a soft slush The water is generally kept standing in the fields for some days after submerging the land so as to kill the weeds.

The rice seed is generally germinated in separate seed-beds, and the young plants from the seed-beds are transplanted being thrust by hand into the soft slush from which surface water is previously drained off; after transplantation a light flooding is given. The operation of preparing a plot and transplanting it takes from first to last about 8 to 10 days, and a very large quantity of water, varying in different soils and localities from about 12 to 24 inches on the field, is required, for the whole operation.

In some places the seed is sown broadcast on the fields and there is no transplantation of seedlings, but the ploughing and puddling as described precedes sowing. In some parts dry ploughing is resorted to and rice is sown broadcast after a fall of rain and is not irrigated till after the crop has come up.

The above, however, do not constitute the ordinary agricultural practice and the great majority of the rice crop in South India is transplanted by hand in fields prepared as above described.

28. Rice after transplantation —After transplantation the crop requires water sufficient to keep the land submerged. This may be given either continuously, or at varying intervals, up to about ten days. Usually the supply to individual fields is given at intervals varying from four to eight days.

29. Duties in the Kistna and Gōdāvari deltas —The 'transplantation water' required varies as already stated, but in these deltas the depth required is believed to average about 16 inches, the operation taking about ten days.

After transplantation about ¼ inch per day is required to supply the transplanted crop.

The supply of 16 inches in ten days works out to a duty of $\frac{24 \times 10}{16} = 15$ acres per cusec, if however the period of transplantation for the whole area is spread over seven weeks the duty for this area works out to 73 acres per cusec.

In this computation no allowance has been made either for rainfall during the period of transplantation, or for the water required to sustain the part of the crop transplanted in the early part of the period to the end of the seven weeks. In the Kistna and Gōdāvari deltas, during this period, 6 inches of useful rainfall may be counted, on even in seasons of rain considerably below the average

The water, for the sustenance of the already transplanted crop, would require about average ¼ inch per day over half the area, equivalent to ⅛ inch per day over the whole area, say 6 inches for the transplantation period. Thus the total irrigation water required over the whole area for transplantation in seven weeks is $(16 + 6 - 6) = 16$ inches

At a duty of 73 acres per cusec discharged on to the fields the above quantities give the equation $B = \frac{\Delta d}{24} = \frac{73 \times 16}{24} = 49$ days.

In these systems the maximum capacity duty provided at the canal head is 66 to 70 acres per cusec, giving about 73 acres on the field; the supply is thus adequate for transplantation in seven weeks, except in times of abnormal failure of rain in the transplantation period.

30. Effect of rainfall on speed of transplantation.—It will be seen that so far as water-supply is concerned with canals delivering irrigation water at the above duty transplantation will be expedited or retarded in accordance as there is more or less useful rainfall in the transplantation period.

The water-supply is however only one of the factors which limit the rate of transplantation, and labour or plough cattle or seedlings available limit the rate where water-supply is abundant

31. Water required for rice after transplantation.—Subsequent to transplantation water is required for the crops from 6 to 4 inches per half month (inclusive of rain) the quantity of water being less during the last two months of the crop period; one watering only is generally required three weeks to a month before reaping, after which the field is allowed to dry.

32. Great variation of duty of water under different conditions.—It will be obvious that the duty of water for rice cultivation will vary greatly with the seasonal rainfall and also with the rate of percolation from the irrigated soil.

Over an area on which water is kept standing for months together loss by percolation even in fairly retentive soils will be considerable, while from free draining permeable lands the percolation must be very great, and the supplies required to maintain water standing on such fields must be greatly augmented. In practice this is found to be the case.

For instance the normal supply duties in the deltas are more than twice as high than those realized in such channels as those taking off from the Cauvery above the Upper Anicut. The ground irrigated by these channels are narrow strips of previous soil running along the margins of the river resulting in free percolation through the subsoil and into the river

The *mean* duty of these channels falls as low as 40 to 50 acres per cusec whereas the *mean* duties in the deltas are as high as 80 to 110 acres per cusec

33. Duty of water for 'tank irrigation'—In loamy soils of fairly retentive character the 'duty' of water for tanks in Madras is about 5 acres of rice per million cubic feet which is equivalent (approximately) to a duty of 60 acres per cusec to a base of 140 days.

This duty includes ordinary losses due to evaporation and absorption in the tank for the period of seasonal supply, but would be insufficient to include these losses in the exceptional cases where the supplies impounded one rainy season are stored for use through the following dry season.

34. Fixing duty specially for each tract irrigated.—It will be clear from the above that the duty of water necessary for rice cultivation must be made the subject of consideration in regard to each separate tract to be cultivated and that the requirements vary with the quantity and incidence of rainfall, and the nature of the soil and subsoil in respect to permeability and subsoil drainage.

35. Examples of duties proposed for projects in Madras.—The following statement gives the duties allowed in the Kistna Reservoir project for irrigation under the upland main canal for a crop period of five months (exclusive of water for seed beds).

For comparison a statement is given of the actual water supplies passed through the head sluice, Kistna Western delta, *plus* actually recorded rainfall being the average of three consecutive years.

The base in each case is five months, and the conditions similar in regard to the soils of the irrigated area.

Period.	Kistna Reservoir Project Upland Canal proposed duties.				Kistna Western Delta observed actuals canal head supply *plus* rainfall.	Remarks
	¾ of mean rainfall	Proposed main canal supply.	Total, columns (2) and (3).	Duty proposed canal supply acres.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	INCHES.	INCHES.	INCHES	PER CUSEC.	INCHES.	
1st month ... 1st half ...	1·71	5·17	6 88	69	5·70	
2nd ,, .	1 91	5 17	7·08	69	7 86	
2nd ,, 1st ,, ..	1·91	5 17	7·08	69	8·92	
2nd ,, ..	2·19	5·17	7·36	69	6·64	
3rd ,, .. 1st ,, ..	2·19	4·75	6 94	75	7·40	
2nd ,, ...	2·37	3·75	6·12	95	5·81	
4th ,, ... 1st ,, ...	2·37	3·75	6·12	95	4·11	
2nd ,, ...	1·38	4·20	5·58	85	5·12	
5th ,, ... 1st ,, ...	1 38	4·20	5·58	85	3·44	
2nd ,, ...	0 13	4·20	4·33	85	3 39	
For whole season ...	17·54	45·53	63·07	80	58·39	

In comparing the proposals with actuals it is apparent that during the first two months, that is during transplantation season, the desire to economise in cost of cutting the canals is the reason for the smaller supply of these months under the proposals as compared with the actuals. The project is based on a two months' transplantation period and from what has already been stated the allowance made will be found to suffice for this. A further criticism on the proposals is that an unnecessarily low duty has been allowed in the project for the fifth month which might be increased to 95 acres per cusec.

As an example of the enormous effect of local rain in the estimation of duties, a similar table is given of the duties proposed for the Madras Cauvery Reservoir project for the area under the Grand Anicut Canal.

Period.	⅔ of mean rainfall.	Proposed canal supply	Total, columns (2) and (3)	Duty of proposed canal acres.
(1)	(2)	(3)	(4)	(5)
	INCHES.	INCHES.	INCHES.	PER CUSEC.
1st month .. 1st half ..	0 6	5·1	5 7	70
2nd ,, ..	1·2	5·1	6·3	70
2nd ,, .. 1st ,, ...	1·4	5·1	6·5	70
2nd ,, ...	1·6	5·1	6·7	70
3rd ,, . 1st ,, ...	1·6	4·0	5·6	89
2nd ,, ..	1·8	4·0	5·8	89
4th ,, .. 1st ,, ...	3·6	2·5	6·1	143
2nd ,, ...	4·0	2·0	6 0	178
5th ,, ... 1st ,, .	3·4	2 0	5·4	178
2nd ,, ..	2·8	1·2	4 0	297
Total ..	22·0	36 1	58·1	102

In this case also the transplantation supply is limited by a canal capacity to a maximum head duty of 70 and the transplantation period is taken as two months. There is however reason to believe that the puddling of the soil in this area requires less water than in the northern deltas. The irrigated tract receives its principal supply of rain during the north-east monsoon, and as a consequence the duty for the two last months of the crop season are immensely higher than that for the Kistna.

The rainfall being so great a factor in total supply ⅔ the mean rainfall has been taken in this case instead of ¾ as in the Kistna project.

36. Water for seed-beds.—The above cases exclude supply of water for any seed-beds raised before the preparation of land for the transplantation commences. The area occupied by seed-beds is from 3 to 5 per cent of the area transplanted, but as seedlings take only from three to four weeks to raise, only a small area of seed-bed has to be planted before water for the transplantation operation is available.

The source of supply of water for raising early seedlings must be considered in each case, and the matter is important, but it does not come under the consideration of duty of water for purposes of design of a canal system. In both the projects mentioned the passing of early water for raising seedlings is separately provided for.

*

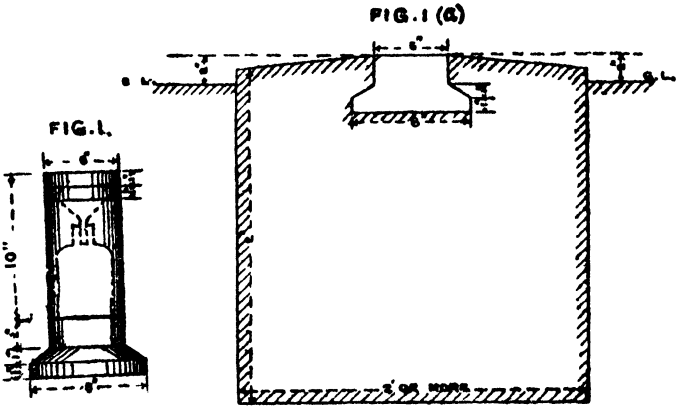
CHAPTER III.

WATER SUPPLIES AVAILABLE FOR IRRIGATION.

RAINFALL.

37. Rainfall.—"Rainfall is the source of all water used for irrigation purposes and therefore a knowledge of its amount, character, seasons or periods and the effects produced by it is of primary importance to all whose duty it is to design, carry out, improve, or maintain irrigation works" (Mullins' Irrigation Manual). Rainfall is measured, at a number of stations distributed throughout the country, by means of rain gauges and the daily fall at each such station is recorded in inches and decimals of an inch.

38. Description of a rain gauge.—A rain gauge consists essentially of a funnel with a circular mouth of known area, which discharges into a receiving vessel, the water impounded in which is measured in a suitably graduated measuring glass. The instrument should be set up in an open place with the circular mouth of the funnel exactly level. 'Symons' Rain gauge' (figure 1) is the



instrument prescribed for use at all Government rain gauge stations throughout India. This is a cylindrical vessel 5 inches diameter with a base enlarged to 8 inches diameter. The top section is a funnel provided with a circular brass rim exactly 5 inches internal diameter. Details of the instrument are shown in figure 1. The funnel shank is inserted in the neck of a bottle. A cylindrical graduated measuring glass is furnished with each

instrument which reads to tenths and one-hundredths of an inch of rainfall, and when filled up to the top graduation, holds water equivalent to half an inch of rain. It is desirable that spare glasses be kept in each district to enable breakages to be replaced without delay, or better still, to have a spare glass at each station.

39. Setting up a rain gauge.—The rain gauge should be set up in a masonry foundation with the rim forming the top exactly level and so that, when in position, the rim is 12 inches above the surface of the ground. Figure 1 (*a*) shows details of a specially built masonry foundation which should not be of less size than 2 feet cube. The site where a rain gauge is set up should be an open place as far as can conveniently be arranged from trees, houses, walls or other obstructions; in no case should it be nearer to an obstruction than 30 yards, or twice the height of the obstruction. The reasons for the above are, that the rainfall registered close to trees, or buildings, frequently varies considerably from that recorded in the open, owing to drip from trees or roofs being carried into the gauge, or the site sheltered from wind, or exposed to eddies or gusts caused by such obstructions. As rain gauges are occasionally blown over unless fixed, the base of the instrument should be set 4 inches deep in masonry. Where there is no masonry foundation, the instrument may be temporarily stood on a flat stone bedded 2 inches in the ground, and gravel or earth packed in round the base to hold it firmly. In every case it must be fixed so that the rim of the funnel is dead level.

40. Observing and registering rainfall.—To secure uniformity the rainfall at all Government stations is measured every day at 8 a.m and the result recorded *as the rainfall of that day.* Thus the rainfall of each day has actually fallen in a period comprising 8 hours of the same day, and 16 hours of the previous day. The receiving bottle as a rule does not hold more than 3 to 4 inches of rainfall, and as during a heavy fall this quantity is frequently exceeded, the rain should be measured 3 or 4 times in a day on days of heavy rainfall lest the receiver fill and overflow; in all cases the last measurement should be taken at 8 a m and the sum of all the measurements during the previous 24 hours added together should then be entered in the register. As the intensity of rainfall in a few hours in a heavy fall is information which may be of much value, it is desirable to enter in the remarks column of the register the hour and rain water gauged at each separate measurement.

If owing to neglect of the above, the receiving bottle should overflow, the cylindrical casing must be removed from the base and the water in it measured and added to that measured from the bottle.

Rain gauges should be examined daily even in dry weather and kept clean and free from dirt. Failure to do this sometimes results in failure of recording owing to the funnel becoming choked.

Inspecting officers should direct their attention to the above points and specially verify the dimensions and the accuracy of the shape of the rim of the funnel, and its being set up exactly level.

41. Rain Recording Stations.—In Southern India rain gauges in charge of the Revenue Department are kept up at the

headquarters of every taluk and a number of recording stations are maintained at other places by the Meteorological, Revenue, Public Works, Forest, and other departments. In addition private rainfall records are maintained on most tea, coffee, and rubber estates. The records of the daily rainfalls recorded at Revenue Department stations are published monthly in the *Fort St. George Gazette* and in the gazettes of the districts concerned. These give the daily, half monthly, and monthly rainfalls in inches and hundredths of an inch, and the number of 'rainy days' in each month signifying days on which a fall of 0·1 inch or upwards has been registered The previous average of daily and monthly rainfall, and of the number of rainy days are also stated for purposes of comparison.

42. Variation of rainfall over an area —It is very noticeable that the rainfall at places close to each other frequently differs greatly, and this is specially so where there are intervening between the stations, marked physical features, especially hills, or where the levels of the two places are very different.

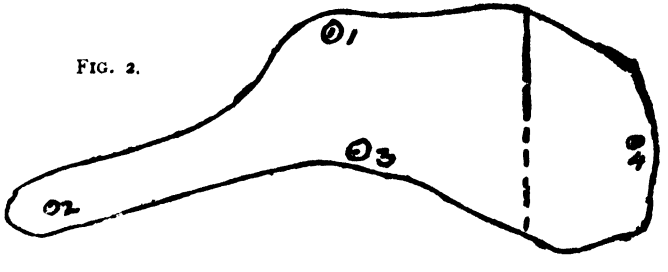
Rainfall generally is greater on higher levels, also the character of rainfall between two stations in the plains separated by a range of hills may be quite different.

For instance the distance between Palghat and Coimbatore, two places separated by a range of hills, is less than 30 miles, but their mean annual rainfalls are 78·38 inches and 22·01 inches respectively.

43. Estimate of average rainfall over any area.—In estimating the average annual, or seasonal, rainfall in any area, from the average annual, or seasonal, rainfalls recorded at various stations within or near the boundaries of the area, it is necessary carefully to consider the relative positions of the stations with respect to the whole area, so that a proper value may be assigned to the records of each station. This is specially important if the differences between the rainfalls of the various stations are great

For instance if the average rainfall in the area in figure 2 has to be estimated from the records of the four stations and the fourth

FIG. 2.



station record differs materially from that of the other three, it would be incorrect to add the average of the four stations together and divide by four to get and apply the result as the mean rainfall

of the whole area; it would be desirable to apply the mean of stations Nos. 1 to 3 to the area in the left of the dotted line, and the record of station No. 4 alone to the area in the right. If the difference in rainfall is attributable to change in physical features, the division of the whole area should be made accordingly.

44. Variation in annual, or seasonal, rainfall at the same station—Great variations are recorded in the annual, or seasonal, rainfall at the same station, and a rainfall record for considerable number of years is necessary to get a reliable estimate of mean annual, or seasonal, rainfallat any place. From a study of rainfall the records of a large number of different places extending over a great number of years it has been deduced that 35 years' rainfall records are required for a thoroughly reliable estimate of the mean rainfall of any place.

45 Percentage of errors in computing mean annual rainfall.—Sir Alexander Binnie has compiled a statement, from which an extract is given below, showing the magnitude of the percentage of error which may be looked for in estimating mean annual rainfall from records for shorter periods than 35 years. The statement is compiled from the rain records of 26 typical rainfall stations for which records from 50 to 60 years were available and the deviations given as a percentage of the average annual rainfall for the total period of observation.

	Five years	Ten years	Fifteen years	Twenty years	Twenty-five years	Thirty-years.	Thirty-five years
Maximum positive deviation	23 2	14 9	9 0	5 6	7 3	5 2	4 5
Do negative do	29 6	16 1	12 5	9·2	9·0	6 9	4 7
Average positive deviation ..	15 35	8 08	3 87	2 47	2 56	2 17	1 73
Do. negative do ...	14 52	8 37	5 64	4 08	2 94	2 36	1 86
Average deviation	14 92	8 22	4 77	3·27	2 75	2 26	1 79

The above gives an indication of the extent of the errors which may be made in estimating rainfall of a station from insufficient rainfall records.

46 Correction of estimate of mean rainfall in reference to records of an adjacent station.—When rain records for a sufficient period are not available for any station, and records for a long period are available for some not far distant station, in which the distribution and nature of the rainfall is similar, a reasonable correction may be made in the estimate of mean rainfall by reference to the record of the neighbouring station.

For example, if 15 years' records only were available for a station and the mean annual rainfall for that period was 30·6 inches, while for the neighbouring station the mean rainfall for the same 15 years was 34·3 inches, but for the 40 years for which records were

available it was 33·4 inches, the corrected estimate for the mean rainfall would be $334/343 \times 30{\cdot}6 = 29{\cdot}8$ inches.

47. Fluctuations of annual rainfall and their bearing on irrigation projects.—While the average rainfall on a catchment, or on a tract to be irrigated, is an important matter, it is to be recognized that rainfall is in defect of the average more often than in excess of it and moreover for all purposes of water-supply, it is in the years of defect in rainfall that artificial supply is most wanted.

In the case of projects for domestic water-supply the absolute minimum rainfall of any year and also the minimum of two and three consecutive years are matters of great importance, as these works must be so designed as to afford a sufficient supply under the most unfavourable conditions. In the case of irrigation works it is unnecessary to secure, under the worst conditions adequate supplies for the whole area normally irrigated, but it is necessary to secure such supplies in what may be termed an 'average bad year.' The rainfall of an average bad year, or season, may be taken as the mean of the lowest annual or seasonal rainfalls in a number of consecutive years, one-third to one-fourth of the whole number of years being taken for striking the average.

48. Relation of maximum and minimum to mean annual rainfall.—The question of absolute maximum and minimum annual rainfall was examined in 1892 by Sir Alexander Binnie (Proceedings of the Institute of Civil Engineers, Volume CIX) who computed the relation between mean, maximum, and minimum rainfall for a large number of stations and suggested for adoption for annual mean rainfall the following relation for computing maximum and minimum rainfall:—

Mean annual rainfall	100
Probable maximum rainfall	151
Probable minimum rainfall	60
Probable average of two consecutive years of least rainfall	69

So far as regards India the information now available shows the above proportions to be considerably short of the fluctuations in annual incidence of rainfall which may be expected.

49. Statements of mean, maximum, and minimum annual rainfall for the Madras Presidency.—The following statements have been compiled from published registers of rainfall stations in the Madras Presidency for the 40 years, 1870 to 1909. Statement I is from the statement of average rainfall in the districts of the Presidency in this period which is obtained by taking the average of each year's rainfall for all the stations in the district. The number of stations having records for so long a period as 30 to 40 years average about ten per district but in the more recent years a number of additional stations have been installed the records of which have been included in the district averages.

Statement II is a similar one compiled with reference to 22 separate typical stations scattered throughout the area of the Madras Presidency. The districts and stations are arranged in order according to the amount of mean annual rainfall.

STATEMENT I.

Yearly rainfall between 1870 and 1909 of Districts of Madras being the average annual rainfall of the rain gauge stations of each district.

Item.	Name of District.	Mean of average annual rainfalls (40 years).	Maximum average rainfall of any one year.	Proportion of maximum to mean.	Minimum average rainfall of any year	Proportion of minimum to mean.	Mean of ten driest years in 40 years.	Proportion of col 8 to col. 3.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Bellary ...	22·56	35·62	1·58	8 61	0·38	15·30	0·68
2	Anantapur ...	22·69	42·66	1·88	7·08	0·31	14·61	0·64
3	Kurnool ...	25 76	48·90	1·90	8·07	0·31	16·85	0·65
4	Coimbatore ..	26·43	35·84	1·36	17·60	0·67	20·76	0·78
5	Tinnevelly .	27·25	46·86	1·72	16·14	0 59	18·93	0·69
6	Cuddapah ...	27·84	47·91	1·72	6·84	0·25	18·23	0·65
7	Rāmnād ..	28·91	44·68	1·55	16·30	0·56	21 00	0·73
8	Madura ..	30·71	48·98	1·59	17·92	0·58	24·46	0·80
9	Guntūr	31 60	53·30	1·69	14·55	0·46	22·29	0·71
10	Trichinopoly ..	32·29	42·45	1·31	21·54	0·67	25·22	0·78
11	Salem	32·37	50·63	1·56	17·41	0·54	24·43	0·76
12	Chittoor .	32·81	53·12	1·62	13·09	0·40	23·32	0·71
13	Nellore ...	34·50	53·34	1·55	10·78	0·31	23·42	0·68
14	Kistna	35·96	61·14	1·70	22·73	0·63	25·85	0·72
15	North Arcot ..	37·78	64·07	1 70	21·59	0·57	28 73	0·76
16	Gōdāvari ...	39·08	63·00	1·61	18·00	0·46	26·50	0·68
17	Tanjore ...	44·23	61·64	1·39	28·87	0·65	32·93	0·75
18	South Arcot ...	44·86	74·35	1·66	25·98	0·58	32·35	0·72
19	Chingleput ..	45·11	73·08	1·62	19·33	0·43	29·69	0·66
20	Ganjām .	45·56	65·48	1·44	27·20	0·60	35·85	0·79
21	Malabar ...	116·69	156·54	1·34	77·77	0·67	92·35	0·79
22	South Kanara ...	145·35	183·78	1·26	98·99	0·68	116·98	0·80

STATEMENT II.

Rainfall of 40 years—1870 to 1909—at undernoted stations in Madras Presidency.

Item.	Name of rainfall station.	Mean average annual rainfall (40 years).	Maximum rainfall of any one year.	Proportion of maximum to mean.	Minimum rainfall of any one year	Proportion of minimum to mean	Mean of ten driest years in 40 years.	Proportion of col. 8 to col. 3.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Bellary ...	19·19	31 27	1·63	7·23	0·38	12·18	0·63
2	Anantapur ...	20·08	35·40	1·76	3·90	0·19	12·18	0·61
3	Coimbatore ..	22·01	30·36	1·38	13·67	0 62	16·66	0·76
4	Markapur (Kurnool Dt.).	24·09	37·64	1·56	4·03	0·17	14·82	0·62
5	Proddatur (Cuddapah Dt.).	25·44	44·82	1·76	6 25	0·25	15·00	0·59
6	Palamcottah (Tinnevelly Dt.).	27·43	43·49	1·59	14·42	0·53	18·11	0·66
7	Kulittalai (Trichinopoly Dt.).	28·84	46·45	1·61	14·83	0·51	19·82	0·69
8	Rāmnād ...	31·60	55·37	1 75	11·73	0·37	18·37	0·58
9	Guntūr	32·23	52 95	1·64	18·77	0·58	22·56	0·70
10	Gudivāda (Kistna Dt.).	34·09	67·85	1·99	20·19	0·59	23·38	0·68
11	Madura	34·81	50·76	1·45	22·24	0·64	25·49	0·73
12	Chittoor ...	36·60	66·24	1·81	14·83	0·40	22·94	0·63
13	Nellore .	37 65	62·85	1·67	11·65	0·31	22·32	0·59
14	Salem	39·23	60·12	1·53	20·76	0·53	29·23	0·74
15	Rajahmundry (Gōdāvari Dt.).	39·81	61 15	1·54	17·49	0·44	24·48	0·62
16	Arcot (North Arcot Dt.).	40·36	62·24	1·54	24·96	0·62	29·66	0·71
17	Villupuram (S. Arcot Dt.).	41·79	63·87	1·52	25·93	0·62	29·66	0·71
18	Kumbakōnam (Tanjore Dt.)	42·61	63·84	1·50	22·68	0·53	29·91	0·70
19	Chingleput ...	46·27	78·87	1·70	20·17	0·44	29·99	0·65
20	Aska (Ganjām Dt.).	48·0	70·89	1·48	28·05	0·58	36·52	0·76
21	Nilambur (Malabar Dt.).	108·67	166·31	1·53	60·57	0·56	73·50	0·68
22	Coondapoor (S. Kanara Dt.).	141·59	199·11	1·40	87·07	0·61	110·13	0·78

rom columns (5) and (9) of the above statements the following be deduced:—

(*a*) That the fluctuations of annual rainfall above and below nean are generally smaller in a large than in a small area.

(*b*) That the maximum differences from the annual mean all are generally proportionately greater for stations of small ıal mean fall and decrease as the mean fall increases. (It will oticed that Coimbatore district and station form a striking ption to this.)

50. **Average rainfall of a bad year.**—From column (9) it will be that the proportion of the mean rainfall of the ten driest years ıe mean fall in 40 years, which proportion may be fairly taken epresent the average rainfall of a bad year, does not differ tly for different districts or single stations throughout the ras Presidency, the extreme being between 0·58 and 0·8; ı this it may be deduced that for purposes of an irrigation ect it is justifiable to assume the rainfall of an average bad to be two-thirds to three-fourths the mean rainfall.

RUN-OFF.

51. **Definition of 'catchment basin'.**—The tract of land ning into any stream or reservoir is termed its 'catchment n' and the area of this tract is its 'catchment area.'

52. **Definition of 'run-off.'**—The 'run-off' of a catchment in a year (or other specified season) is the volume of water harged by the stream draining the area, or into the reservoir iving the drainage in the year (or other season). The run-off ated in terms of the equivalent depth of water over the catch- t area in inches.

53. **Irrigation supply wholly derived from 'run-off.'**—The ›ly of water for 'flow' irrigation is wholly dependent on run- and in cases where full information regarding run-off from the hment is available, the incidence of annual or seasonal rain- on the catchment is a matter of no importance The great ortance of rainfall data in respect of available irrigation sup- s is due to the fact that measurements of rainfall are much e frequently available than measurements of run-off

54. **Measurements of run-off much to be preferred to esti- es based on rainfall.**—An estimate of run-off based on even most accurate and reliable records of rainfall is liable to very t error, and it should be clearly borne in mind that even oximate gaugings of actual discharge are, wherever procurable, h preferable to estimates wholly based on rainfall data. As ever it is rather exceptional to have reliable measurements of ıal flow-off from catchments, the problem of computing run-off ı rainfall records is one which has to be constantly dealt with rrigation officers

55. **Disposal of rain falling on any catchment.**—The whole of rain falling on any catchment basin is disposed of in one or r of the following ways:—

(1) absorbed by vegetation,

(2) evaporated and passed off in the air,

(3) runs off either over the surface or percolates through the ground into streams in the same catchment,

(4) soaks into the ground, without percolating thence into surface water in the same catchment.

Considering a period of several years, and a catchment from which water does not percolate through the ground into some other catchment, almost the whole rainfall comes under the headings (1), (2) and (3), for over such a period there is normally little or no change in the amount of water impounded as sub-soil water. A very large proportion of any rain falling on a dry and porous soil may however in the first instance be absorbed into the soil by soakage although all, or very nearly all, is subsequently redistributed among items (1), (2) and (3).

Of the above items number (3) alone constitutes the 'run-off'; the balance, viz., the difference between rainfall and run-off is termed the 'rainfall loss.'

56. Factors which affect run-off.—The factors which principally affect quantity and rapidity of run-off from a catchment are:—

(*a*) the quantity and distribution of the rainfall,

(*b*) the condition of the area as regards geological formation, permeability of soil, surface slopes, wind, and temperature and humidity of air,

(*c*) the nature and quantity of the vegetation in the area, and

(*d*) the condition as regards humidity of the surface and sub-soil in the area at the time the rain falls, and the existence of surface water such as tanks, marshes, etc., which impound water.

In the absence of records of actual stream flow an estimate of run-off must be made from rainfall records, but the problem is a very complex one and the estimate is liable to considerable errors. Generally the rainfall is the only factor influencing run-off of which exact measurements are available, and the others are not susceptible of exact measurement and are stated only in general terms

57. Estimating the run-off from seasonal rainfall.—It is usual to estimate run-off from rainfall statistics in terms of a percentage of the total fall of any years, months or other periods, and to classify catchments as good, ordinary or bad, in accordance with their other run-off qualities. Different percentages of run-off are applied to catchments having different qualifications. Tables have been constructed giving percentages of run-off with reference to total rainfall of the year, or of the rainy season, having in view that the larger the rainfall the greater proportion of it will run-off. The tables represent actuals of run-off computed from the best available observations.

58. Strange's table of run-off due to monsoon rainfall.—The undernoted table extracted from Strange's 'Indian Storage Reservoirs' is suitable for estimating run-off from rainfall in the plains of South India.

able of total monsoon rainfall and estimated run-off and yield per square mile from catchment areas.

Good catchment.			Average catchment.			Bad catchment.		
Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill. c.ft.	Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill. c.ft.	Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill. c.ft.
(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0·1	0·001	0·002	0·1	0·001	0·001	0·05	0·0005	0·001
0·2	0 004	0·009	0·15	0·003	0·006	0·1	0·002	0·004
0·4	0·012	0·028	0·3	0·009	0·021	0·2	0·006	0·014
0·7	0·028	0·065	0·5	0·021	0·048	0·3	0·014	0·032
1·0	0·050	0·116	0·7	0 037	0·087	0 5	0·025	0·058
1·5	0·090	0·209	1·1	0·067	0·156	0·7	0·045	0·104
2·1	0·147	0 341	1·5	0 110	0·255	1 0	0·073	0·170
2·8	0·224	0·520	2·1	0·168	0 390	1·4	0 112	0·260
3·5	0·315	0·732	2·6	0·236	0·549	1·7	0·157	0·366
4·3	0·430	0·999	3·2	0·322	0·749	2·1	0·215	0 499
5·2	0·572	1·329	3 9	0·429	0·996	2·6	0·286	0·664
6·2	0·744	1·728	4·6	0·558	1·296	3 1	0·372	0·864
7·2	0·936	2·174	5·4	0·702	1·630	3·6	0·468	1·087
8·3	1·162	2·690	6·2	0·871	2·024	4·1	0·581	1·349
9·4	1·410	3·276	7·0	1·057	2·457	4·7	0·705	1·638
10·5	1·680	3·903	7·8	1·260	2·927	5·2	0·840	1·951
11·6	1·972	4·581	8 7	1·479	3·435	5·8	0·986	2·290
12·8	2 304	5·353	9 6	1·728	4·014	6·4	1·152	2·676
13·9	2·641	6·135	10 4	1·980	4·601	6·9	1·320	3 067
15·0	3 000	6·970	11·25	2·250	5·227	7·5	1·500	3·485
16·1	3·381	7·855	12·0	2·535	5·891	8·0	1·690	3·927
17·3	3·806	8·842	12·9	2·854	6 631	8·6	1·903	4·421
18·4	4·232	9·832	13·8	3·174	7·374	9·2	2 116	4·916
19·5	4·680	10·873	14·6	3·510	8·154	9·7	2 340	5·436
20·6	5·150	11·964	15 4	3·862	8·973	10·3	2·575	5·982
21·8	5·668	13·168	16·3	4·251	9·876	10·9	2·834	6·584
22·9	6·183	14·364	17·1	4·637	10·773	11·4	3·091	7·182
24·0	6·720	15·612	18·0	5·040	11·709	12·0	3·360	7·806
25·1	7·279	16·911	18·8	5·459	12·683	12·5	3·639	8·455
26·3	7·890	18·330	19·7	5·917	13·747	13·1	3·945	9·165
27·4	8·494	19·733	20·5	6·370	14·799	13·7	4·247	9·866
28·5	9·120	21 188	21·3	6·840	15·891	14 2	4·560	10·594
29·6	9·768	22·693	22·2	7·326	17·019	14·8	4·884	11·346
30·8	10·472	24·329	23·1	7 854	18 246	15·4	5·236	12·164
31·9	11·165	25·939	23·9	8·373	19·454	15·9	5·582	12·969
33·0	11·880	27·600	24·7	8·910	20·700	16·5	5·940	13·800
34·1	12 617	29·312	25·5	9·462	21 984	17·0	6·308	14·656
35·3	13·414	31·163	26·4	10·060	23·372	17·6	6·707	15·581
36·4	14·196	32·980	27·3	10·647	24·735	18·2	7·098	16·490
37·5	15·000	34·848	28·1	11·250	26·136	18·7	7·500	17·424
38·6	15·826	36·767	28·9	11·869	27·575	19·3	7·913	18·383
39·8	16·716	38·835	29·8	12·537	29·126	19·9	8·358	19·417
40·9	17·587	40·858	30·6	13·190	30·643	20·4	8·793	20·429
42·0	18·480	42·933	31·5	13·860	32·199	21·0	9·240	21·466
43·1	19·395	45·058	32·3	14·546	33·793	21·5	9·697	22·529
44·3	20·378	47·342	33·2	15·283	35·506	22·1	10·189	23·671
45·4	21·338	49·572	34·0	16·003	37·179	22·7	10·669	24·786

Table of total monsoon rainfall and estimated run-off and yield per square mile from catchment areas—*cont.*

Total monsoon rainfall in inches.	Good catchment.			Average catchment.			Bad catchment.		
	Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill c. ft.	Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill. c.ft.	Percentage of run-off to rainfall.	Depth of run-off due to rainfall in inches.	Yield of run-off from catchment per sq. mile in mill. c.ft.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
48	46·5	22·320	51·854	34 8	16·740	38·890	23·2	11·160	25·927
49	47·6	23·324	54·186	35·7	17·493	40·639	23·8	11·662	27·093
50	48·8	24·400	56·686	36 6	18·300	42·514	24·4	12 200	28·343
51	49·9	25 449	59·123	37·4	19·086	44·342	24·9	12·724	29 561
52	51·0	26 520	61 611	38·2	19·890	46·208	25·5	13·260	30·805
53	52 1	27·613	64·151	39·0	20·709	48·113	26·0	13·806	32·075
54	53·3	28·782	66·866	39·9	21·586	50·149	26·6	14·391	33·433
55	54·4	29·920	69·510	40·8	22·440	52·132	27·2	14·960	34·755
56	55·5	31·080	72·205	41·6	23·310	54·153	27·7	15·540	36·102
57	56·6	32·262	74·951	42·4	24·196	56·213	28·3	16 131	37 475
58	57·8	33 524	77·883	43·3	25·143	58·412	28·9	16·762	38·941
59	58·9	34 751	80 734	44·1	26·063	60 550	29·4	17·375	40·367
60	60·0	36·000	83·635	45·0	27·000	62 726	30·0	18·000	41·817

59. Estimating run-off from total monsoon rainfall.—It will be noted that the run-off in the above table is computed only for the monsoon rainfall as, in a normal year, there is no appreciable run-off from the showers which fall in the dry season. In estimating the run-off available from a catchment the actual rainfall of successive years, and not the mean rainfall, should form the basis of computation ; but when a sufficient number of years' annual rainfall is not available the yield of ⅔ or ¾ the mean rainfall may be taken as the basis for computing the run-off.

60. Objections to estimating run-off from the total monsoon rainfall.—The above method of estimating run-off is open to the grave objection, that it does not differentiate between different incidences of rain having the same total seasonal fall. It is obvious that if the season is six months and that 60 per cent falls in the first two months the run-off is likely to be greater than if the rainfall of each month were equal. Similarly the run-off from 6 inches of rain falling in two consecutive days would be more than that from 6 inches falling in 24 days in quantities never exceeding half an inch a day.

61. Strange's table of run-off from daily rainfall.—In order to allow for such differences, which greatly affect run-off, another table is given by Strange in which run-off is computed from recorded daily incidence of rainfall, and it is most desirable wherever records of daily rainfall over a catchment are obtainable, and the accuracy of estimated run-off is a matter of importance, that the daily rather than the seasonal rainfall be made the basis of the estimated run-off.

Strange's table of run-off from daily rainfall for an average catchment.

[For good or bad catchments add or deduct up to 25 per cent of yield.]

Daily rainfall in inches.	Run-off percentage and yield when original state of the ground was					
	Dry.		Damp.		Wet	
	Percentage.	Yield in inches.	Percentage.	Yield in inches.	Percentage.	Yield in inches.
0·25	...	...	...	...	8	0·02
0 50 ...	...	...	6	0·03	12	0·06
0·75	...	...	8	0·06	16	0·12
1·00	3	0 03	11	0·11	18	0 18
1·25	5	0·06	14	0·17	22	0·28
1·50	6	0·09	16	·24	25	0·38
1·75	8	0 15	19	0·33	30	0 52
2·00 . ..	10	0·21	22	0·44	34	0·67
2·50	15	0 38	29	0·73	43	1 08
3·00	20	0·60	37	1·12	55	1 65
4·00	30	1·20	50	2·00	70	2 80

It will be found convenient in applying the above, or the table in paragraph 58, to use, in preference to the table, a curve made from it, showing the relation between rainfall and percentage run-off

62. Classifying condition of catchment as regards humidity.—No hard and fast rule can be laid down for classification of the condition of catchment as wet, damp, or dry for purposes of applying the above table, this should be varied with the permeability and absorbing qualities of the soil and subsoil, and the general conditions of the catchment. The following statement of the classification conditions which have been used for application to ordinary catchments in the plains of Madras will serve as some guide to applying the tables :—

(*a*) Conditions required for transition from 'dry' to 'damp'—

¼ inch rainfall in the previous 1 day.
½ ,, ,, 3 days.
1 ,, ,, 7 ,,
1½ ,, ,, 10 ,,

(*b*) For transition from 'damp' to 'wet'—

⅓ inch rainfall in the previous 1 day.
½ ,, ,, 2 days.
1 ,, ,, 3 ,,
1½ ,, ,, 5 ,,

(*c*) Transition from dry to wet is made whenever 2½ inches have fallen on the previous or on the same day.

For instance, if 4½ inches fall in one day on a 'dry catchment the run-off would be—

	Inches.
2½ inches on a 'dry' catchment	= 0·38
(4½ − 2½) = 2 inches on a 'wet' catchment	= 0·67
Total ..	1·05

(*d*) Transition from 'wet' to 'damp'—

Rainfall less than the following, being half the rainfall in (*b*) above:—

1/6 inch	in the previous 1 day.
1/4 "	" 2 days.
1/2 "	" 3 "
3/4 "	" 5 "

(*e*) Transition from 'damp' to 'dry'—

Rainfall less than—

1/8 inch	in the previous 1 day.
1/4 "	" 3 days.
1/2 "	" 7 "
3/4 "	" 10 "

63 Run-off by measuring stream flow.—The best method of arriving at the run-off from rainfall is by measurement of actual stream flow, and as 35 years' rainfall measurements of seasonal rainfall are required for arriving at a fully reliable estimate of future seasonal rainfall, it follows that gaugings of stream flow for a similar period are desirable for a fully reliable estimate of quantity of flow It is only in very rare instances that river or stream gaugings for so long a period are available, but extremely valuable information can be deduced from the results of gaugings for much shorter periods, and this is especially the case in regard to catchments of the limited areas (say under 1,000 square miles) on which the great majority of irrigation projects are dependent

64. Importance of gaugings of stream flow even for one or two seasons.—In the case of most irrigation projects the information most required is the flow-off of the catchment in an average bad year, and the measurement of the daily flow for a single bad season, combined with the rainfall records for this season as well as for a number of other years, will often enable an accurate forecast of run-off to be framed. While the actual flow of a bad season gives more valuable information than the flow of one, or several good seasons, valuable results can sometimes be deduced from the record of a single season's measurements, even when the season has been one of abundant rainfall.

65. Example of above.—An example of the above is the forecast made by Binnie with regard to the run-off from the catchment of the Nagpur reservoir.

The information available was—

(*a*) Rainfall statistics for nineteen years, 1854 to 1872.

(*b*) Measured run-off for the years, 1869 and 1870.

(*c*)Catchment area (6·6 square miles).

The monsoon at Nagpur lasts from June to October, and during the remaining months there is no run-off. The recorded monsoon rainfall for nineteen years averaged 37·52 inches and is given below for each year:—

Year.	Inches.	Year.	Inches.	Year.	Inches.	Year.	Inches.
1854	48·40	1859	29·48	1864	28·96	1869	32·11
1855	24·04	1860	44 50	1865	38·16	1870	37·34
1856	44·33	1861	40·89	1866	41·01	1871	44·85
1857	33·46	1862	43·29	1867	53·72	1872	39·82
1858	31·87	1863	37·46	1868	19·28		

The run-off measurements available are given below:—

Year and month.	Rainfall.	Measured run-off.	Total rain of monsoon	Total run-off	Ratio of total run-off to total rainfall.
(1)	(2)	(3)	(4)	(5)	(6)
1869.	INCHES	INCHES.	INCHES.	INCHES.	INCHES.
17th June to 31st July ..	12 76	1 25	12·76	1·25	0·098
August	9 61	3·36	22 37	4·61	0·20
September ..	7·41	3·26	29·79	7 87	0 268
1870.					
June	6 77	0·32	6·77	0·32	0 047
July	12 70	2 88	19 47	3·20	0 16
August	11·82	6·59	31·29	9 79	0·31
September	7 99	5 95	39·28	15·74	0·40
October .. .	4 37	1 72	43·65	17·46	0 40

From the above Binnie made the following forecast of run-off based on the relation between the figures in columns 4 and 5:—

Total monsoon rainfall.	Probable run-off
INCHES	INCHES.
37·52	14·23
30·0	8·40
19·28	3·0

Subsequent observation has shown that these runs-off have been closely realized.

With reference to the above it is to be remarked that a similar forecast to that made is deducible from the observations of the monsoon of 1870 alone, a single season of exceptionally abundant rain.

It will be noticed that the bulk of the rainfalls in four months and that the conditions appear to have been specially favourable to computing run-off from total rainfall from commencement of the monsoon. In many places in Madras the run-off from small catchments is most of it realized between June and October and in others between September and December and the method is especially applicable to such catchments.

66. Percentages of run-off from rainfall corrected from stream flow measurements.—The above method of estimating run-off from total seasonal rainfall, modified as may be necessary to suit the character of the seasons, may be usefully applied rather to small than to large catchments. In all cases a study of the actual stream flow due to known precipitation of rain, as regards both total quantity and intensity, gives valuable assistance in constructing, or modifying, such a table of run-off as is given in paragraph 58 or paragraph 61 above, and is likely to prove of great assistance in forecasting yields of catchments. No opportunity of gauging actual flow-off of a catchment of which the yield must be forecasted, should therefore be neglected, even if time does not allow of gauging for more than a single season.

67. Method of recording stages of stream flow.—The successive stages of stream flow past any place is recorded from a register showing the level of water at certain intervals of time as read from a graduated gauge set up in the stream at that place. A gauge is usually graduated to read feet and decimals, and is fixed in a vertical position to some rock or masonry work, or to a pillar or post specially erected to carry it. The level of the surface of the water as indicated on the gauge is read and recorded at stated intervals of time, generally two or three times daily.

68. Inaccuracies due to recording flow from intermittent gauge readings.—It is obvious that, apart from neglect of the gauge reader to read punctually and correctly, the method is one susceptible to great errors in the case of rivers and streams the levels of which are liable to great and rapid fluctuations between the intervals of taking readings. It is usual to give instructions to the gauge reader that, in addition to the usual observations, extra readings during floods, and especially the actual highest and lowest flood levels should be recorded, it is nevertheless not to be expected that a gauge reader will always do this at odd hours throughout the day, and certainly will not do so in the night time. The fluctuations in large rivers and canals are not generally rapid, but the streams fed from small catchments fluctuate very rapidly and errors due to this cause are likely to be great in the case of such catchments. Gauge readings on small streams combined with the record of rainfall over the catchment are nevertheless very valuable, as rainfall registers give opportunities of interpolating floods of which the gauge readings give insufficient detail.

69 Selection of suitable positions for erection of gauges.—In selecting suitable positions for erecting a gauge it is necessary to bear in mind the precise object which is in view in erecting the gauge as both the position, and the level of the zero of the gauge, will be affected by this consideration Gauges are installed for various reasons, for instance :—

(*a*) To obtain a record of the quantity of water flowing at all times in the stream in which the gauges are to be erected.

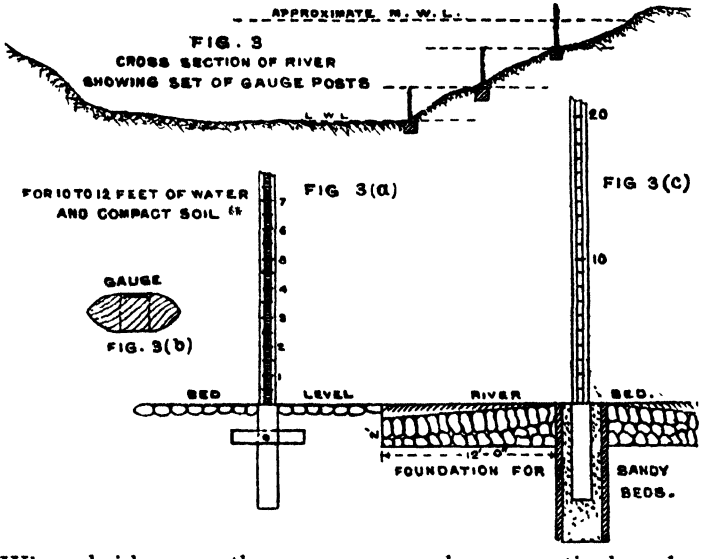
(*b*) To obtain a record of the levels of water surface in the stream during floods.

(*c*) To obtain data from which to compute the quantity of water passing over a weir or through the vents of a regulator or sluice.

70. Considerations affecting the position of gauges in rivers and streams.—A gauge should be installed in a position in which there are reasonable facilities for its being read, indeed if situated in a place where the gauge reader is not subject to constant supervision, *convenience of reading is one of the most important considerations*, in the selection of the site. The site selected should be one where the water level is not likely to be affected by local shoals or obstructions and where the water has free access to the gauge at all stages of flow of which record is required.

71. Some details of gauge erection.—When the object in view is obtaining a record of flow-off at all stages, it is most important to select, so far as is possible, a site for the gauge with stable river margins and not subject to considerable alterations of river bed level by silting or scouring at different stages of flow; the whole discharge at all stages of flow should pass through a single channel the surface level of the water in which is shown by the gauge; the above points are of more importance than selecting a site in a straight reach in which the flow is uniform and free from backwaters or cross currents, etc., although these conditions are very desirable where they can also be realized. In many places it is necessary to erect gauges in several sections and when this is so it is necessary to be very careful that the corresponding graduations on different sections of the gauge are fixed at the same level. Figure 3 shows a gauge fixed in several sections on a transverse section of a river.

GAUGE POSTS FOR REGISTERING FLOOD LEVELS.



Where bridges or other masonry works, or vertical rocks exist in suitable positions, gauges may conveniently be fixed to them, but in default of these, special wooden gauge posts, or masonry gauge pillars, must be erected and to these the graduated gauge should be secured. Gauge posts or pillars, and their foundation

must be specially designed to suit the site, having in view the depth of water, velocity of current, nature of drift and the character of the river bed in which they are to be erected. A gauge post, or pillar, must be shaped with cutwaters on its up and down stream edges as shown in figure 3-*b* and the gauge should be fixed on one side of the post intermediate between the up and down stream cutwaters and on the side on which it can most easily be read; it should be countersunk flush with the side of the post or pillar. Gauges erected in a sandy river bed frequently require a well foundation 8 to 10 feet deep such as is shown in figure 3 (*c*). Figure 3 (*a*) shows an ordinary type of wooden gauge post. In a sandy or soft river bed where there is much current a dry stone apron should be packed for 10 to 12 feet all round the gauge with the top at or below the river bed level. This is required to prevent local scour which is likely to occur in the vicinity of any obstruction projecting from the bed of a stream. The gauges used in Madras are generally made of enamelled plate iron, graduated to show feet and tenths in white on a blue ground, and are made in sections 5 feet long. When these are unprocurable, similar graduations can be painted on the gauge post. The enamelled gauges are much the best and do not, like painted gauges, require frequently repainting.

72. Zero of a gauge to be connected with bench mark.—Wherever and whenever a gauge is erected, the level of the zero of the gauge should always be connected by levelling with at least one good bench mark in the neighbourhood, and preferably with two or more; the positions and values of these with reference to zero of the gauge should be noted on the plan, or section, showing the gauge, and described in any report on the erection of the gauge which may be made. This is most important as it enables any change of the gauge zero to be checked and the gauge to be accurately replaced if in any way disturbed. If a new gauge is erected in a new site to replace an existing one, it is generally undesirable to move the old gauge, and in any case the plan and report showing the alteration should show the position of both gauges, and the level value of the zero of each with reference to a good local bench mark.

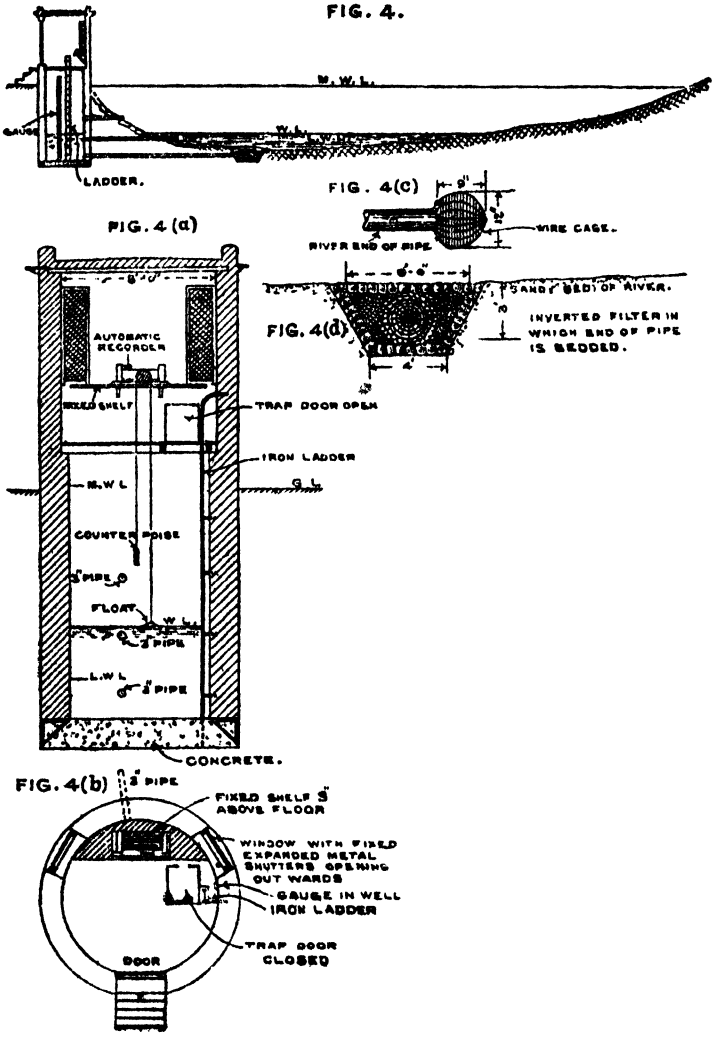
73. Gauge wells.—Gauges fixed in open and exposed water cannot be read with close accuracy, as the wash of current against the gauge pillar affects the water level and the fluctuations of level due to waves and surgings cause constant variations of depth. A 'gauge well' (sometimes called a 'stilling box') is a well which is sunk on the margin of a river or canal and provided with a gauge showing the level of water in the well. One or more pipes or openings or narrow slits in the side connect the well with the water of the stream, and thus the level of the water in the well is kept at the true level of the surface of the stream, but is not subject to fluctuations of level caused by wind or waves or the wash of currents.

A gauge well is usually open at the top and surrounded by a parapet wall, while a ladder, or steps, afford convenient access to the gauge. Wherever a gauge well is installed, there must be two gauges with their zeros at the same level, one in the well and the other in the stream, fixed on approximately the same cross section,

and the readings of both gauges must be recorded. This obviates errors which might occur, owing to the partial choking of the connection between the well and the stream, if only the level of the well gauge were recorded. In practice a good deal of trouble is experienced from this cause, especially during a low stage succeeding a flood, and wherever gauge wells are installed the readings of both internal and external gauges should be frequently checked by inspecting officers.

Pipes connecting gauge wells with a stream should not be less than 3 inches in diameter, and each pipe should be without bend throughout, so as to facilitate clearance. The lowest pipe must be wholly submerged below the level of the lowest stage of flow to be recorded. Where the outer end of the lowest pipe has to be laid

FIG. 4.

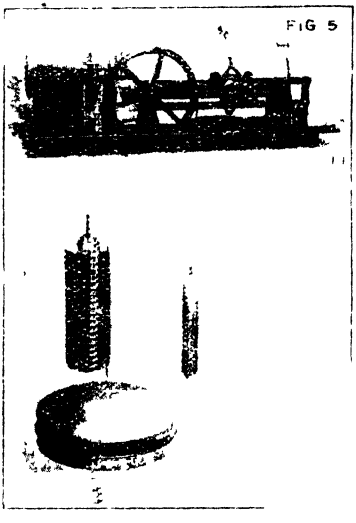


in sand, or soft soil, at or below the river bed level, it should terminate in a fixed wire cage and be laid in a cistern, or crate, containing an 'inverted filter' consisting of stones or gravel of sizes graded from about ¼ inch on the exterior to 1 inch gauge around the pipe head gauge. This is required to prevent silt partially choking the pipe.

Figure 4 shows a gauge well with three 3-inch pipes, the lowest pipe being fitted with a cage (figure 4-*c*) embedded in an inverted filter (figure 4-*d*). The well steining in this case is carried up 10 feet above ground level and roofed over to form a chamber for an automatic water level recording instrument

74. Automatic continuous water level recorders.—The inaccuracies of the records of flow based on intermittent gauge readings have led to the introduction of continuous automatic water level recording instruments, and the installation of such instruments is the only practicable means of getting accurate flow levels in rapidly fluctuating streams. Few of these instruments have as yet been installed in India but there is clearly a great want of them and it is probable that a number will be used in the near future.

The record of most of these instruments consists of a diagram traced by a pencil on a paper which is wound round a revolving drum The diagram records the level of the water at each instant of time. To effect this, in some instruments the pencil point is moved by clock work parallel to the length of the drum at a known and uniform speed thus giving time and the drum is caused to revolve by a chain passing over a pulley keyed to it, one end of the chain is attached to a float which rests on the surface of the water whose level is to be recorded, and to the other end is attached a suitable counterpoise. In other instruments the drum is revolved by clock work at a uniform speed to give time, while the pencil point moving parallel to its axis and geared to a suitable scale, reproduces on the diagram the variation of the water level. Stephens automatic water level recorder, an instrument of the latter type, is shown in figure 5.


FIG 5

The clock is driven by a weight and the intervals of time at which it requires winding depend on the heights available for the travel of the weight This type of recorder is in use at a station in South India and has worked most satisfactorily. Figures 4, 4 (*a*) and 4 (*b*) show the installation of such an instrument over a gauge well.

75. Methods of computing flow from gauge readings.—The discharge in a river or stream past a gauging station is calculated from the gauge reading by multiplying the area of the cross

section of flowing water due to each successive gauge reading by the mean velocity of flow at that stage. The area of flowing water is computed from one or more cross sections of the river taken at right angles to the direction of flow. The velocity of flow may be actually measured by timing floats, or by means of a current meter, or may be computed by Kutter's or Bazin's formula. For the latter computation the surface fall of the stream under the various conditions of flow must be ascertained and a co-efficient suitable to the river chosen. The method of calculation is described in Love's Hydraulics, Chapter VIII. This method generally gives the discharge of artificial canals and channels with reasonable accuracy, but in natural streams the surface fall, cross section area and velocity are constantly altering, and the liability to error in estimating the mean surface fall and mean hydraulic radius, and in fixing a suitable co-efficient is great; this method of computing flow should therefore only be applied to river flow where it is impracticable to obtain even rough actual measurements of velocity.

76. **Selection of site for measuring flow of a stream.**—For measuring the velocity of flow of a stream a straight reach of length exceeding twenty times the width of the stream should if possible be chosen and the part of this reach having the most regular transverse section and the steadiest flow should be selected for the gauging section or sections. It must be realized that there is no necessity for measuring the flow of a stream at the actual section where the gauge, from which readings have in the past been taken, is erected, unless this is also the best place for measuring flow. The spot chosen for actual measurement may, if desired, be at a distance several miles removed from this, and should be the reach most suitable for the purpose of accurate measurement of the flow which passes the site of the gauge. It frequently happens that a place which is the most suitable for placing a gauge to be read daily is quite unsuitable for taking the velocity and discharge measurements undertaken with a view to ascertaining the equivalent discharge for each reading of the gauge. The only limit, on the distance apart between the site of the gauge and the measuring place, is that the latter must be so situated that no material unmeasured inflow, or draw-off occurs between the two places. If the places are a considerable distance, say over half a mile apart, a temporary gauge must be erected at the measuring site and the gauges at both sites read at short intervals of time during the continuance of the measuring work. It is most desirable that the river bed in the measuring reach should not be subject to much change by silting, or scouring, during different stages of flow, and a smooth rock, shingle, or clay bed is therefore favourable, while a fine sandy bed is unfavourable. It is, however, frequently necessary to measure discharges in a reach with a changeable sandy bed, and in such cases it is necessary to frequently check the cross sections showing the bed levels, and to make any necessary corrections in the table of areas of water section due to different stages of flow, that is, to different gauge readings. In certain cases it is even necessary to take a fresh section every day.

A curve or table showing the relation between gauge reading and cross sectional area is called the ' gauge area curve ', or table.

77. Methods of measuring velocity of flow.—Velocity of flow is generally measured:—

(*a*) By surface floats.
(*b*) By velocity rods.
(*c*) By current meters.

78. Measurement of velocity by surface floats.—A length of about 100 feet in which the uniformity of flow and of the transverse section of stream is great should be selected as a float run, and the run divided into two equal lengths by transverse dividing lines, I, II, III, in figure 6, set out at right angles to the direction of flow by poles erected on the margins of the stream.

FIG. 6.

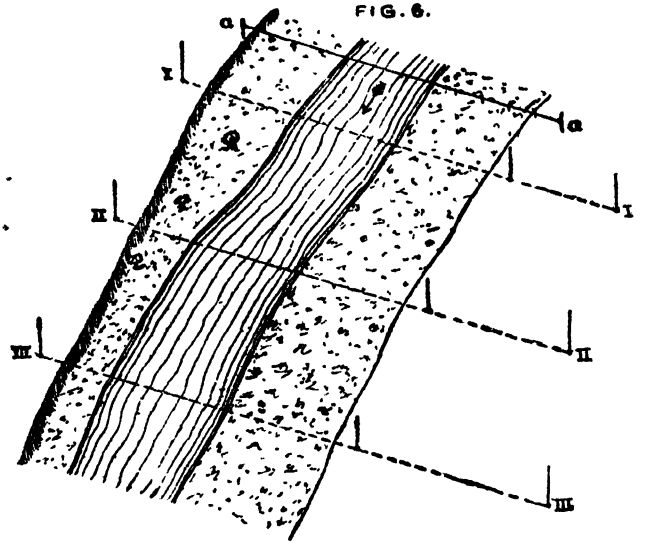


FIG. 6(a)



About 20 feet above the up-stream line (i), a cord *a a* tagged, or otherwise marked, at suitable known intervals is stretched across the stream. Floats will be released at successive points on the line *a a* and their timing as they pass in succession the lines I, II, and III will be recorded by a man stationed on the margin for this purpose and furnished with a stop watch. If only a watch with a second hand is available the length of the run may be extended.

Several floats, generally three, are let loose successively from each point, and the mean of the time of each run recorded. Where the float does not run fairly and truly down the section throughout the run or where the time of any run differs greatly from others starting from the same point the observation is discarded and an additional one taken.

In shallow streams floats are loosed by men wading, and elsewhere from a boat, bridge or cable way. Where facilities for this do not exist floats may be thrown in from the margin of the stream so as to fall roughly in the required position.

Even rough observations of this nature are likely to afford valuable information.

The spacing of the points from which floats are released depend on the facilities existing for the work, the time available and the width of the stream.

In a large river the spacing would be frequently as great as 100 feet and more, while in a narrow stream, or canal, it might be 10 feet or less, as far as practicable the points should divide the width of the stream into sections corresponding with any sudden change in the cross section of the bed.

79. Method of computing discharge from surface velocities.—As a result of observations a statement is obtained of mean surface velocities V_1, V_2, V_3 . . . . . V_n along lines spaced at distances apart l_1, l_2 l_3 l_4 . . . l_n dividing the water cross sectional area into n compartments figure 6 (*a*); the depths at each point as ascertained from the mean of cross sections of the run being d_1 d_2 d_3 . . . . . d_{n-1}.

Surface velocities are in excess of mean velocities and it has been found from numerous observations that to reduce surface to mean velocities they must be multiplied, by a factor varying from 0·75 to 0·95.

Generally for purposes of estimating irrigation supply it is safe to apply a factor of 0·8 to 0·85 according to the size of the stream or river.

If D be the total discharge from surface float observations taken as above, and F be the factor for reducing surface to mean velocity, then the total discharge can be computed from the formula

$$D = F\left\{l_1\left(\frac{d_0+d_1}{2}\right)\left(\frac{v_0+v_1}{2}\right) + l_2\left(\frac{d_1+d_2}{2}\right)\left(\frac{v_1+v_2}{2}\right) + \ldots + l_n\left(\frac{d_{n-1}+d_n}{2}\right)\left(\frac{v_{n-1}+v_n}{2}\right)\right\} \quad \ldots \quad (2)$$

The following are used for surface floats and are generally suitable:—Bottles, partially filled with water so that only the neck is visible, cocoanuts, palmyra fruits, oranges, limes, etc.

Buckley recommends circular discs of plank cut from a light wood but these are likely to be considerably affected by wind.

It is very desirable to take float observations at a time of day when there is least wind and the strength of the wind and direction with reference to the current of the stream should always be recorded.

80. Float observations with velocity rods.—Velocity rods are wood rods, or tin tubes, about 1 inch to 2 inches diameter, and of length approximately equal to the depth of water on the line of the float run. The rods are so weighted that they float nearly vertically with about 1 inch projecting above the water surface. The lower end of the rod should reach down to about ·94 of the depth from the surface of the stream.

A considerable number of rods are required to suit the varying depths of cross section.

The advantages of these rods over surface float is that they give the approximate mean velocity, and that they are not so much affected by wind.

Velocity rods can be used to great advantage in artificial canals and channels, in which a run of uniform depths can be obtained, but in natural streams the inequalities of the beds are usually so great as to preclude their use.

The general methods of taking and computing discharge with velocity rods are the same as with surface floats.

81. Measurements by current meter.—A current meter is essentially an instrument furnished with a wheel which is caused to revolve by the force of the current when immersed in a stream of water, and furnished with gear which records or indicates, the number of revolutions of the wheel.

A curve, or table showing the relation between current velocity and number of revolutions of the wheel per minute is the 'rating curve' or table of the meter. Each meter requires to be rated separately and this is generally done by towing the meter, at different known and suitable velocities, through a measured distance in still water and recording the numbers of revolutions; the results are plotted on squared paper showing the relation between velocity and revolutions and the curve which closely coincides with the observation points is the rating curve. Measurements with a meter in trained hands are more reliable and accurate than float measurements, but care and attention to detail is required to obtain accurate results.

The current meter is used to obtain the velocities at different points on a transversal section of a stream and thus to divide up the section of flow into a number of compartments, the discharge of each of which is separately computed. The general method of computation is similar to that stated in paragraph 79 above.

The meter however measures the velocity of water crossing a single section and there is no inaccuracy or complication as in float observations due to the necessity for computing the mean area of the length of the float run.

82 Methods of observing mean velocities by current meter — The meter can be used in several ways in order to get the mean velocity in the vertical at each point of observation. The most accurate as well as the most laborious method when conditions permit, is to take the velocity in each vertical at each tenth or other small fraction of the depth.

From these observations a curve showing the relation between depth and velocity can be plotted and the area enclosed between the curve and its axes divided by the depth, gives the mean velocity in that vertical.

The process is very laborious and the observations take so much time that the results are likely to be affected by change of the stage of flow during the observations.

83. Mean velocity by one and two point methods.—A very large number of vertical velocities taken as above have demonstrated that under normal conditions the mean velocity in a vertical is:—

(*a*) The mean of the velocities at 0·8 and 0·2 of the depth from the surface.

This involves errors up to a maximum of about 2½ per cent.

(*b*) The velocity at 0·6 of the depth from the surface.

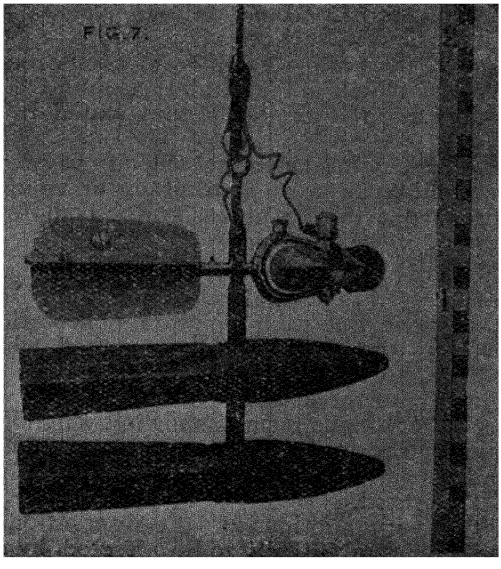
This involves errors up to a maximum of about 5 per cent.

From the above facts the practice has become usual to operate current meters by recording the velocity on each vertical either at 0·6 of depth which is called 'the one point method' or at 0·2 and 0·8 below the surface which is called 'the two point method.'

84. Surface velocity by current meter.—When the velocity is too high to allow of the meter being lowered to 0·6 of the depth without undue deflection of the suspending line or rod, the velocity at about ½ to 1 foot below the surface may be taken and a factor applied similar to that used for surface floats.

85. Methods of operating the current meter.—The current meter is operated in shallow water by a man wading and at other times either from a boat or a bridge or from a travelling car (or cradle) hanging from a wire rope which is stretched between two trestles on either side of the stream. In this cradle the observer (or two observers) sits and either is pulled from the shore or pulls himself along the rope stopping in succession at the various observing points.

The current meter most used in Madras is the Price meter (figure 7). This meter, as well as most matters in connexion with measurements of stream flow, is clearly described in Royt and Grover's book "River Discharges."



The meter is used suspended from a flexible cord with one or two weights according to the strength of the current; it is also used attached to a rigid rod. Figure 7 shows the meter suspended and with two weights attached.

86. Weirs used for measuring flow-off.—Where guage readings are available above weirs or in the case of drowned weirs both above and below, the discharges of these can be computed with fair accuracy by applying the recognized weir discharge formulæ.

In the case of very small streams, involving flow not exceeding about 10 cusecs, temporary measuring weirs, being rectangular or triangular notches made with wooden planks, may be erected to gauge discharge; these should be arranged so as to have a clear overfall at the critical stages of flow.

87. Gauge calibration curves.—While the preparation of accurate calibration curves giving the relation between gauge reading and discharge involves the taking of a large number of velocity and discharge observations, an approximate curve can be made from 4 or 5 observations of discharge, if taken at suitable stages of flow so as to give points fairly distributed along the curve.

Once a suitable curve is available, the conversion of a previous gauge reading into quantity of flow can be readily made.

The ascertaining the relation between the readings of a gauge and the corresponding discharge is termed *calibrating a gauge* and the curve or table showing this relation is the 'calibration curve' or table of the gauge.

Although gauge readings alone do not give the discharge of a stream, if the gauge readings for any period are available, the calibration of the gauge enables the discharges of that period to be ascertained, so long as the section of the river, at and about the site of the gauge, has not materially altered throughout the period.

88. Run-off computed from change of water level of an intercepting reservoir.—Where the run-off of a catchment is intercepted by a reservoir the quantity of run-off can be calculated from a record of the daily water level of the reservoir, combined with the daily draw-off, surplus and loss by evaporation and absorption. The accuracy of the result is dependent on the accuracy with which these losses are known, and the percentage they bear to the whole inflow, and also on the accuracy of the table, or curve, giving the relation between capacities and water levels in the reservoir

Under favourable conditions this method of arriving at run-off from a catchment is a very accurate one.

FLOOD DISCHARGE.

89. Maximum flood discharge from a large catchment.—The maximum stage of flow in a river or stream is produced when the rainfall of very great intensity falls at a time when the state of catchment as regards humidity is favourable to a large run-off.

When reliable records of actual flood levels embracing a long period are available they are the best guide to what may be expected in the future.

In the case of very large catchments an attempt to compute maximum flood discharge from rainfall statistics is likely to result, and in the past has resulted, in very erroneous estimates

In such cases it is almost imperative in estimating maximum flood discharge to rely on flood levels, and, in the absence of recorded flood level observations, it is necessary to place dependence on the results of local enquiries regarding past floods, which however should be checked by reference to recorded rainfalls.

As a rule villages situated near the margins of rivers are on sites above ordinary flood level but frequently leave so little margin above this that some portion of the village-site is submerged in extraordinary floods.

Enquiry in marginal villages will generally elicit fairly accurate information in respect to the highest limit to which water attained within the past 20 years or more, with the year in which such flood occurred.

Villagers will generally be able to point out on houses or walls, or with respect to fences or demarcation stones, the limits to which their fields were submerged. Careful enquiry at the several villages along the course of a river generally serves to elicit valuable information in regard to past floods.

In all such cases the various points indicated as flood limits must be connected by levelling, and comparison of levels will afford a fair indication of the reliability of the information The comparison of records of the rainfall to which such floods were due, with other heavy falls, will help in estimating to what extent the maximum flood may exceed that recorded. As a further test the computed discharge due to the flood levels as ascertained from local enquiry may be compared with the recorded rainfalls on these occasions and from this a reasonably accurate estimate of maximum flood discharge framed.

90. Floods from rainfall in moderate sized catchments.—In the case of the discharge of smaller catchments the quantity of information procurable as regards flood levels is likely to be much smaller and less reliable than in the case of considerable catchments, but fortunately in such cases the liability to error in estimating maximum flood from rainfall, although still considerable, is not so great as in large catchments, neither is the effect of erroneous estimation likely to be so serious.

The forecast of maximum discharge from small and medium-sized catchments is generally made from consideration of the maximum intensity of rainfall recorded at places in the catchment during short periods, such as one or two up to about 10 days, and of the nature of the catchment in regard to proportion of rainfall likely to run off as surface drainage.

91. Ryves' and Dickens' formulæ.—The two formulæ generally used in South India for computing maximum flood discharge are those of Dickens and Ryves and are both in the general form:—

$$D = C.M^{p}$$

Where D is maximum flood discharge,
M is the area of catchment in sq. miles,
p is a power less than unity,
C is a co-efficient fixed with reference to the nature of the catchment and the intensity of recorded rainfall and any records of actual floods in parts of the same or in similar catchments.

The formulæ referred to are—

$D = C.M^{2/3}$ ... ... (Ryves') ... . (3)
$D = C.M^{3/4}$ ... .. (Dickens') ... ... (3-a)

A similar formula has been applied with varying values of p, for the calculation of flood discharges in the United States by Fanning and Cooley, and in Australia by Karnot.

92. Table of flood discharges by Ryves' formula.—Ryves' formula is that most generally used in South India, but Dickens' has also been much used, especially in the Ceded districts. For any specific catchment it will be plain that an error in making (p) too small may be rectified by suitably increasing the co-efficient C; but unless both p and C are suitable the application of the same equation would be inapplicable to a similar catchment of different area.

The table below gives for Ryves' formula the equivalent run-off in inches over the area and in cusecs, for a number of different co-efficients and areas.

Table of flood discharges.

Ryves' formula $D = C.M^{\frac{2}{3}}$

Area in square miles	Values of C								
	200	300	400	500	600	700	800	900	1000
	Discharge in inches from area in 24 hours.								
1	7 43	11 45	14·87	18·59	22·30	26·02	29 74	33·47	37 19
5	4 34	6 52	8·69	10 86	13 03	15·20	17 38	19·58	21 75
10	3 35	5 03	6 71	8·39	10·06	11·74	13·42	15·53	17 26
25	2·54	3·82	5 09	6·36	7 63	8 90	10 18	11 45	12 72
50	2 02	3 03	4 04	5·05	6·06	7·07	8·08	9 08	10 09
100	1 60	2 40	3·20	4·00	4·80	5 60	6 40	7 21	8 01
250	1·18	1·77	2·36	2·95	3·54	4 13	4·72	5 31	3·90
500	0 93	1 40	1 87	2·34	2·80	3 27	3·74	4 22	4 69
1,000	0 74	1 11	1 48	1 85	2·22	2 59	2·96	3·35	3·72
2,500	0·54	0·81	1 09	1 36	1·63	1·90	2·18	2 47	2 74
5,000	0·43	0·65	0 87	1·09	1·30	1·52	1·74	1 96	2·17
10,000	0·34	0 51	0·69	0·86	1 03	1·20	1·38	1·55	1·73
25,000	0·25	0 38	0 51	0·64	0·76	0·89	1·02	1·14	1·27
50,000	0·20	0·30	0·40	0·50	0·60	0·70	0·80	0 91	1 01
	Discharge in cusecs (to 4 significant figures).								
1	200	300	400	500	600	700	800	900	1,000
5	584	876	1,168	1,460	1,752	2,044	2,336	2,632	2,924
10	928	1,392	1,856	2,320	2,784	3,248	3,712	4,177	4,641
25	1,710	2,565	3,420	4,275	5,130	5,985	6,840	7,695	8,550
50	2,714	4,071	5,428	6,785	8,142	9,499	10,860	12,210	13,570
100	4,308	6,462	8,616	10,770	12,920	15,080	17,230	19,390	21,540
250	7,938	11,910	15,880	19,840	23,810	27,780	31,760	35,720	39,690
500	12,600	18,900	25,200	31,500	37,800	44,100	50,400	56,700	63,000
1,000	20,000	30,000	40,000	50,000	60,000	70,000	80,000	90,000	100,000
2,500	36,840	55,260	73,680	92,100	110,500	128,900	147,400	165,800	184,200
5,000	58,480	87,720	117,000	146,200	175,400	204,700	234,000	263,200	292,400
10,000	92,830	139,200	185,700	232,100	278 400	324,900	371,400	417,700	464,200
25,000	171,000	256,500	342,000	427,500	513,000	598,500	684,000	769,500	855,000
50,000	271,400	407,200	542,900	678,500	814,300	950,000	1,086,000	1,221,000	1,357,000

93. Rainfall formulæ for flood discharge only applicable to moderate and small sized catchments.—In applying the above formula to any catchment the selection of a suitable co-efficient

has to be made. In making such selection the recorded maximum intensity of rainfall in the catchment must be considered. In a catchment of very large size very great differences will occur in intensity of rainfall in different parts and as already stated (paragraph 89) huge errors are likely to be made in forecasting maximum flood discharge from rainfall alone but fortunately other and better data are almost always procurable.

In the case of small and moderate sized catchments say under 350 square miles, the quantity and incidence of rainfall is often the best and most reliable basis on which to estimate maximum flood discharge, and it is to such catchments that the following remarks refer.

94. Standard area of maximum precipitation of rain.—It is known that rainfall of maximum intensity does not simultaneously occur over very large areas, and the records of recording stations 2 or 3 miles apart show great variations in intensity of rain, although not showing great differences as regards annual or seasonal fall

A reference to the above table (paragraph 92) shows that in selecting a co-efficient C with reference to the recorded greatest intensity of rainfall in 24 hours, it is necessary to arbitrarily fix some standard area to which this maximum rainfall may be considered to apply. It is customary in Madras to take five square miles as this standard, which area is that recommended by Mullins in the "Madras Irrigation Manual."

95. Selection of co-efficient in Ryves' formula.—In tentatively selecting a co-efficient for an ordinary gently sloping catchment the assumption is made that a suitable co-efficient will give a run-off in inches from a 5-square mile area, which will be about equal to the maximum rainfall recorded in 24 hours. For example, if the maximum recorded rainfall is 10 inches reference to the table (paragraph 92) shows that the co-efficient which would give this is most nearly 500; this however may be modified after considering other special facts in connexion with the catchment such as its shape, the slopes of the ground, the permeability of the surface and any record of actual flood which may be available.

It might be urged that this co-efficient gives, for an area of 1 square mile, an intensity of run-off of 18·59 inches, which is much more than the recorded maximum rainfall. To this objection the reply is that the rain at maximum intensity never continues to fall for 24 hours, and that the greater part of a maximum fall frequently is precipitated in 3 to 6 hours, and for a small catchment the maximum fall per hour, rather than per day should be considered. It is, however, unusual to have records giving the maximum intensity of rainfall per hour, the shortest period ordinarily recorded being rainfall per day, and if a suitable co-efficient and standard area is adopted, the formula gives a reasonable margin to allow for the heavy incidence of fall in periods of less than a day. The further assumption made in allowing for maximum run-off in 5 square miles at the rate of the whole recorded rainfall in a day, and not merely a fraction of it if, erroneous, is an error on the safe side. It is in effect the same as if the standard area were 6 square miles with a surface run-off of 84 per cent.

Again, the rainfall precipitated takes some time to reach the place where the maximum discharge is to be forecasted, and the fall over the whole area can only gradually arrive at this place. This serves to distribute the run-off and moderate the discharge in accordance with the differences in the time required for the flow of rain water, from the place at which it has fallen, to the flood discharge station.

The moderating effect of the above on floods at any place, is small in a small catchment, but increases rapidly with the area of the catchment.

96. Consideration of suitability of co-efficient with reference to data available other than the maximum rainfall in one day.—In fixing the co-efficient for the maximum recorded intensity of rainfall, consideration should be given to the fall on the day preceding and on the day succeeding the greatest fall, as the maximum fall in 24 hours might very well approximate to the latter *plus* one or other of the former.

For instance, the maximum fall recorded at Ongole was 10·45 inches on 28th October 1895, the fall on the previous and on the succeeding day being small, but there was a fall at the same place of 8·46 inches on 10th November 1903, preceded by a fall of 6·50 inches on the 9th November. Under the circumstances it would be reasonable to assume (8·46+6·50)=14·96 inches as a possible maximum, and something more than 10 45 as the basis for fixing the co-efficient for the flood discharge formula.

In many tracts in Madras there are large numbers of existing tanks, which are many of them provided with masonry surplus works, and enquiry into the history of such tanks is likely to give information important in assisting in arriving at a suitable co-efficient. If, as is frequently the case, the surplus works have been designed for a maximum flood computed from Ryves' formula, and if extraordinary falls of rain have not resulted in failures, owing to inadequate capacity of surplus works, it is evidence of the adequacy of the co-efficient used, and *vice versa.*

When finally fixing the co-efficient for computing the flood discharge to be provided for, the nature of the works in contemplation must be considered. Where underestimation of the magnitude of a flood is likely to entail very disastrous consequences, it is obvious that an estimate so liable to error should rather err on the side of liberal provision than the reverse; where, on the other hand, the cost of works is likely to be greatly increased by liberal provision for floods, and the damage due to underestimating is not likely to be great, a more conservative estimate may properly be made.

As an example the dry *taluks* of the Guntūr district north of the Gundlacamma may be cited. Here rainfall registers for over 40 years are available At one station (Sattenapalle) there was a fall of 15·20 inches on one day in November 1897 there being no rain on the previous or following day. At the nearest two rain stations (Guntūr and Narasaraopet) each distant about 12 miles, the fall on the same day was 5·70 inches at one, and less at the other. The next heaviest fall in 2 consecutive days ever registered at Sattenapalle is 6·46 inches, at Guntūr 7·5 inches, and at Narasaraopet 9·30 inches. These are fairly representative of the other stations of the area.

For purpose of ordinary tank restoration works a co-efficient of 500 in Ryves' formula has been adopted as generally applicable to designs of surplus works in this tract, but for the purpose of the Kistna reservoir Project main canal, a bad breach in which would entail widespread disaster, a co-efficient of 1,000 has been adopted

97. Effect of nature and shape of catchment on the co-efficient.—Steep or rocky catchments, involving quick run-off and small absorption, would call for an extra high co-efficient, while the co-efficient of very flat or absorbent tracts would be reduced. The shape of the catchment has also an effect on floods. A catchment of a circular shape in which the subsidiary drainages join the main one near to one another is likely to give, other things being similar, a larger flood than a long narrow catchment An exception is such a case as the above when the storm producing the flood travels down the valley of the catchment; in such a case a long narrow catchment will give a greater flood than a circular one.

Where there is a marked difference in the rainfall of different parts of a catchment, the flood discharge of the several parts must be considered separately, due allowance being made when assembling the figures together for time of transmission and for the reduction of flood due to the larger area of the whole catchment

98. Co-efficients frequently applicable in Madras.—The co-efficient to be applied must be considered in each case on its merits, but the following figures, from Buckley's Irrigation Works of India, give an idea of the magnitude of co-efficients frequently found suitable in Madras, in connexion with Ryves' formula.

C=450 in flat tracts along the coast.

„=560 in tracts with greater slope 15 to 100 mile from the coast.

„=675 in limited areas near the hills.

The co-efficient 560 may be considered as applying to the average catchment and this reduced by about 16 per cent in the case of a very flat catchment and increased by 20 per cent for areas at the foot of hills. This gives a fair idea of the way in which the slope and nature of surface of catchments affect the fixing of a suitable co-efficient.

99. Other flood formulæ.—Several other arbitrary formulæ of different form to the above, are used for computing maximum flood discharges; they however all involve large assumptions and none of them are simple of application.

100. Marking and recording flood levels and flood discharges.—The above will make it clear how uncertain are the data on which it is frequently necessary to compute the magnitude of maximum floods, and how valuable a few well established facts as regards flow-off from storms may prove in arriving at a correct estimate.

All irrigation officers should make a point of marking and recording water levels and discharges of various rivers, streams, surplus, and cross drainage works after every flood of considerable magnitude, and whenever opportunity offers should roughly

observe surface velocities of streams in flood, by throwing in floats and timing them between marks on the margin; drift wood, or rubbish will frequently serve the purpose of floats and all that is necessary to enable flood discharge to be calculated is to record the timing of the speed at which these are carried through a known distance, and to mark the corresponding flood levels and positions of the float run, and to ascertain the sectional area of discharge after the flood subsides.

It is specially important to establish flood level marks after each considerable flood along rivers or streams which have artificial flood embankments, and it is the duty of all irrigation officers in charge of works, to see that these levels are recorded by means of marks on masonry works and pegs at intervals along banks Along important embankments it is desirable to have flood gauges on all masonry works as also at suitably spaced intermediate points. If one side of such a gauge is kept whitewashed the maximum height of flood will be shown by the water mark on the whitewash which will be visible for several days after the flood subsides.

CHAPTER IV.

PERCOLATION AND EVAPORATION.

101. Percolation.—It was stated in paragraph 55 above that part of the annual rain falling on any catchment soaks into the ground and is either evaporated or absorbed by vegetation, or is stored as ground water most of which seeps slowly away through the ground to reappear as surface water in streams, tanks and swamps This water passes into and through the ground by percolation, that is, by gradual flow between and through the particles of the soil.

102. Percolation and absorption.—The term *absorption* is commonly used as interchangeable with *percolation*, and the terms are generally so used in this book and when so used include the effects of both 'absorption' and 'percolation.' Strictly speaking however 'percolation' has reference only to movement of ground water which is brought about under the operation of the force of gravity, and 'absorption' has reference to movement or retention of water, in the pores of soil or other material, by the force of capillary attractions.

Capillary movement may take upwards, downwards or laterally. Capillary movements of ground water take place in directions leading away from pore spaces wholly or partly filled with water towards those which are entirely empty or which are less nearly filled.

Immediately after ordinary rain when the surface soil is more moist than that below, capillary action aids gravity and hastens the penetration of water into the ground. As soon as the surface soils become drier than those below, the capillary currents are reversed and the flow is upwards.

Water which enters ground in excess of capillary saturation moves directly downwards till ground water surface has been reached when it raises that level and at once augments the pressure (paragraph 55) inducing lateral movement of sub-soil water.

103. Seepage.—'Seepage' is a term frequently used to denote percolation into or from the soil. 'Sokage' is used sometimes to denote percolation into the ground but is not used for percolation from the soil.

The terms 'percolation' and 'absorption' generally for the future will be used in their comprehensive sense.

104. Mean annual percolation of a catchment.—The 'mean annual percolation' of a catchment is the depth of rain water over that area (in inches) which, on an average, annually soaks away into the earth. Percolation water forms a very considerable portion of the visible flow-off of most streams and in rainless periods the whole flow of rivers and streams is derived from this source. The quantity of percolation from a given depth of rain falling on a surface in a given time, is greater the more porous the nature of the surface and subsoil, and is less the steeper the surface

slopes of the area. A steep catchment with a surface of hard sheet rock connotes very small percolation, while on a flat plain composed of deep sandy soil percolation will be great.

105. Effect of percolation on irrigation supplies.—The effect of percolation is to some extent to diminish the annual run-off of rainfall, but to a much greater extent to equalize flow-off, reducing the discharge after heavy rainfall, by absorbing part of the rain into the soil, through which it percolates at a slow rate and reappears as surface water after the direct run-off from the fall of rain has been discharged. The effects of permeable strata which drain into the valleys of a catchment are similar to those of reservoirs of capacity equal to the content of the pores of the strata, but which have outlets whose regulation is not under control.

The beds and margins of rivers, where these are freely permeable, also act in a similar way, the higher portions of the beds and the marginal strata absorbing water from the river flow on a rising river, and continuing to absorb so long as the river surface level is above the 'ground water level', which is also technically termed the 'water table.' As the river falls, the direction of percolation is reversed, the soil gradually giving back to the river or stream lower down its course the water previously impounded from its own flow.

As a striking instance of this, the Cauvery Delta system may be quoted in which the river channels near the tail end of the system (in the vicinity of the East Coast) continue to run, and flow irrigation is carried on, long after any surface flow is passing through the regulators at the head of the system. In this system the fall of river channels is unusually great, and the channels are very wide and shallow and run on beds of coarse sand through permeable strata which are saturated from surface irrigation water throughout the earlier months of the crop season, and thus the 'water table' is raised to a high level.

The permeability of soils in irrigated areas, or in the tracts through which canals are carried to feed those areas is the cause of loss by percolation into the sub-soil of large quantities of the water which enters at the head of the canal system and, in consequence of this loss, the quantity of irrigation water delivered on the surface of the fields is less than that which is passed into the canal at its head.

Such loss is generally estimated by measurement of the flow of canals and channels past different points in the distribution system; the losses so measured include both percolation and evaporation losses, and the estimates which are framed of the losses of water in transmission between the head of a canal and the fields are invariably framed to include both these factors. The actual percentage of the losses sustained in different irrigation systems from these causes will be referred to later; for the present it suffices to say that the actual losses attributable to percolation alone vary greatly with the class of soil, and in certain cases are known to be very great and to cause much injury, by loss of water which could be otherwise usefully applied to irrigation, and also by injuriously affecting the health of the inhabitants in the irrigated tracts by raising the sub-soil water levels to an insanitary extent.

106. Percolation in soils of different kinds.—For the purpose of considering percolation the soils may be divided into three classes—

(*a*) Sound rock *in situ*.
(*b*) Grit soils.
(*c*) Plastic soils.

107. Percolation through rock.—Most descriptions of stone when unaffected by decay are practically impermeable, and percolation through masses of rock of such description takes place only through cracks, fissures, bedding planes or cavities in the stone. The velocity of flow through strata composed of such stone depends on the sectional area of the fissures, and where these are open joints is dependent generally on the same conditions as regulate velocity of flow through pipes or open channels. Where the pores of stone, either owing to its original structure or to decomposition are so open as to allow percolation, the velocity of percolation will be governed by similar conditions as in the case of grit soils.

108. Grit and plastic soils.—'Grit soils' may be taken to include porous and decomposed rocks and may be described as soils, such as sand, shingle, gravel, etc., which are an aggregation of particles of hard material which do not become soft in water and which do not adhere to one another when moist. Clean sand is the most common form of pure grit soil.

'Plastic soils' when dry can be pulverized into a fine dust consisting of particles of impalpable size. When moist, such soils become plastic and the particles adhere to one another or to particles of intermixed grit. When mixed with a further quantity of water, these soils become mud and have a tendency to slip and flow.

109. Permeability of soils.—'Grit soils' are freely permeable and the degree of permeability depends on the percentage which the content of unfilled space between individual particles bears to the whole mass of the soil. This percentage depends to a considerable extent on the shape, and more still on the grading of the sizes, of the particles of grit composing the soil.

'Plastic soil' when damp is only slightly permeable. There is no soil which is absolutely impermeable, but the permeability of the best classes of clay *when in the plastic state* is so slight as to be practically non-existent. All dry plastic soils will absorb water and only become plastic when damp, while it is only in the plastic state that they are impervious.

It is common in nature to find soils composed of a mixture of grit and plastic soils, and when the latter is so disposed as to effectively fill all the interstices between the particles of grit, the soil when damp becomes impervious. Masses of pure grit such as sand and shingle, and pure plastic soil such as black cotton, and of mixed plastic and grit soil such as loam, are common in nature, the first being permeable, the second impermeable; and in the case of the last the permeability depends on the proportions of plastic and grit soils and the way these are mixed.

110. Percolation gradient.—The resistance to percolation flow through any material or soil varies in accordance with the percentage the voids bear to the solid matter in the whole content of soil.

Where there is a flow of water percolating through pervious soil from its source of supply to some other place the force moving it is gravity, and, as in the case of surface water, a certain surface slope or head is required to induce flow and the greater the slope, the greater the flow and vice versa.

The surface slope of water moving under fixed conditions through any soil is called the 'percolation gradient' and for similar soils under the same conditions the percolation gradient will be the same. The relative permeability of different kinds of soil may be compared by observing their respective 'percolation gradients' under the same conditions

111. Water table and ground water gradient.—The resistance to the passage of water through various kinds of soils varies very greatly, and through even the most pervious is so considerable as to require comparatively steep surface slopes to induce even slow movement; further the supplies of seepage water are constantly varying, and the result is that there are great differences in the level of sub-soil water in various parts of any tract of country The slope of the sub-soil water or ground-water', level in any tract is called the 'ground-water gradient', and the surface of the ground-water is often referred to as the 'water-table.'

112. 'Saturation gradient.'—In the case of earthen embankments which hold up a depth of water against one face, the bank becomes gradually saturated by percolation up to a certain level constituting a gradient falling from the surface of the water on the up-stream face of the bank at an angle which, other things being equal, is flatter or steeper in accordance as the material composing the bank is more or less permeable.

The plane of the surface of the percolation water, viz., the levels at which free water would be met if bores were sunk at a number of places in the bank, is called 'the plane of saturation' or of 'percolation', of the bank; if the 'plane of saturation', cuts the outer face of the bank, visible flow will appear along and below the line of intersection In a cross section of a bank the plane of saturation,' becomes a line and is referred to as the 'line of saturation, and the slope of this as the 'saturation gradient', or sometimes as the 'hydraulic gradient.'

The 'saturation gradient' of an embankment is synonymous with the 'percolation gradient' the terms being interchangeable. the actual gradient is the same for the same soils where the velocity of percolation is the same. The velocity of percolation is greatly influenced by the conditions of draw-off and consequently the actual percolation gradient for the same soil will vary greatly under different conditions of drainage.

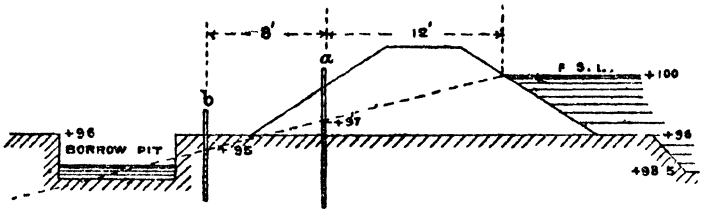
The resistance of flow through all soils is great, and the velocity of percolation is very small when compared with surface flow, and the changes of percolation or saturation gradients to suit any change in the condition of the effective head is only effected very slowly, and the less permeable the soil the slower the change. To give an idea of the rate of percolation it may be stated that, through the pores of coarse sand with a percolation gradient of one in twelve, velocity would be about 3 feet in 24 hours. Thus when observing the percolation gradients for the purpose of judging the permeability of any class of soil it is necessary to do so under conditions

in regard to effective head, which have not materially altered for a considerable time.

113. Saturation gradient in a bank shown by pipes.—If the water levels in two perforated pipes (*a*) and (*b*) sunk in the cross section of a canal bank as shown in figure 8, were found to be +97 and + 95 respectively, and the bank and sub-soil are composed of the same earth, this would indicate a percolation gradient of one in four in the bank and sub-soil. If the saturation plane intersects the surface soil either on the rear slope of the bank, or at some depression in the ground beyond the toe of the bank, percolation on the surface will take place from and below the level of such intersection If the borrow pit had been excavated as shown in figure 8, the bottom of the pit would have been covered with water percolating from the canal.

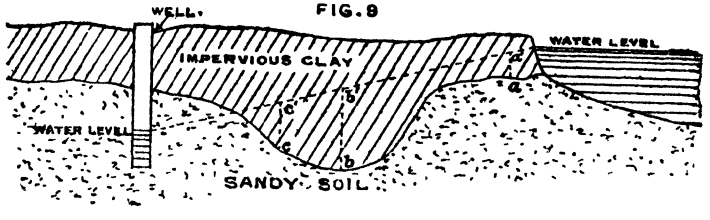
If the canal bank were of highly impervious earth, the sub-soil being unchanged, the level of water in pipe (*a*) would be unaffected because the percolation through the soil under the bank would have produced sufficient water pressure at the lower end of the pipe to raise the water in the pipe to +97 that is to the level due to the percolation gradient of the soil on which the bank stands.

FIG. 8.



114. Uplift pressure exerted by ground water.—When water has percolated into a pervious stratum which is overlaid by an impervious soil in such a way that the ground water cannot rise to the level due to permeable qualities of the lower stratum, the sub-soil water will exert a pressure on the underside of the impermeable stratum at each point equivalent to the difference at that point of the actual water level and the level of the line of percolation gradient of the permeable soil.

FIG. 9



For instance, the pressures at *a*, *b*, *c*, figure 9, are those due to columns of water of heights *aa'*, *bb'*, *cc'*.

The conditions are analogous to those of a water pipe flowing under pressure which is laid at a gradient less than its hydraulic gradient and the percolation gradient defines the virtual slope of

the ground water just as the hydraulic gradient of a pipe gives its virtual slope (see Love's Hydraulics, Chapter VI).

115. Velocity of percolation through sand.—The velocities of water percolating through sand of various grades, as regards size of individual grains, has been the subject of a considerable amount of investigation, especially in connexion with the rate of filtration in water-works, and as a result of experimental investigation it has been ascertained, that the conditions affecting the rate of percolation, within certain limits of velocity, are similar to those pertaining to the motion of water through pipes of very small sectional area, and not those pertaining to flow at ordinary velocities in pipes of normal sizes (Vide Love's Hydraulics, Chapter VI).

The matter has been treated of in considerable detail by Parker (control of water) who gives reference to much of the available experimental information on the subject published to date.

From such experiments a formula has been devised to compute roughly the velocity of water percolating through clean sand The relations are stated in the under-noted equation which is known as Hazen's formula

$$V = 210\ d^2 \left(\frac{t + 10}{60}\right) \frac{h}{l} \quad \ldots\ldots\ldots\ldots \quad (4)$$

where V is the velocity in feet per 24 hours of a solid stream of water of the same sectional area as that of the sand through which percolation takes place (not the velocity through the pores of the sand which must be 2 to 3 times greater).

d = the 'effective size' of the sand grains in hundredths of an inch.

[The 'effective size' of sand is defined as the mean diameter of a grain such that 10 per cent (by weight) of the sand is composed of smaller particles, and 90 per cent of larger particles.]

h is the head in feet, producing percolation

l is the length in feet, of the path along which percolation occurs.

t is the temperature of water in degrees Fahrenheit.

For the plains of Southern India 't' may be taken at + 80° F; the equation then becomes $V = 315\ d^2 \frac{h}{l}$ . . . . . (5)

116. Mechanical analysis of sand.—Samples of sand may be graded by shaking up the sample in water and passing it in succession through a number of sieves with meshes spaced at different distances apart.

The sand retained in each sieve is either gauged by volume, or after drying, by weight.

The importance of classifying sand by the size of its constituent grains is a matter which has been latterly coming into prominence in engineering not only in connexion with percolation, but also with reference to density of mortar and concrete.

117. Sand analysis useful for comparing designs with existing works.—It is believed that no designs of irrigation works have up to now been based on computed velocity of percolation flow based on analysis of sand or other soils, and the formula given need only be considered as a guide to the conditions which affect velocity of percolation through sand and other pervious soils,

while the practical application of sand analysis may, for the present, be confined to comparison of the sizes of sands on which existing works stand, with others forming the foundations for proposed ones.

118. Percolation past irrigation works.—The design of irrigation works such as earthen embankments, masonry dams, weirs or regulators, may aim at stopping as far as possible all percolation through or under the works, or in alternative, at restricting percolation within such limits as are required for the stability of the works.

119. Stability of works affected by percolation.—Generally speaking, flow-off from percolation through or under a work does not affect its stability unless:—

(*a*) The percolation becomes at any time of sufficient velocity to carry off with it particles of soil from the work, or from underneath it, or from bank connexions joining the work with natural features.

(*b*) The pressures developed by, and due to, the percolation are sufficient to fracture or displace some part of the work.

It is clearly necessary before designing any irrigation work subjected to hydrostatic pressure and composed of or standing on a permeable soil through which percolation must pass, to decide, as a preliminary, what is the hydraulic gradient (*s*) which will ensure that velocity of percolation flow is restricted within safe limits.

If the head (*h*) be fixed, this fixes the minimum length (l) of percolation flow for

$$s = \frac{h}{l} \quad . \quad . \quad . \quad . \qquad . \quad . \quad . \quad (6)$$

Economy of design forbids making '*s*' too small and

Security of design, making it too large.

For works on a foundation of clean sand '*s*', generally varies from 1/18 to 1/12 according to the coarseness of the sand; the coarser the sand the greater is '*s*' and this is so in spite of the fact that percolation velocity is greater in a course than in a fine sand, because the velocity required to displace the coarse sand is proportionately still greater.

120. Limiting velocity of percolation under foundations — For the superstructure of works there is generally no difficulty in designing so as to secure either freedom from percolation flow, or suitable restriction of the same, and consideration will only be given now to the conditions under which subsoil percolation takes place under works, which are in themselves free from percolation and standing on an impervious platform.

Let *a b* (figure 10) represent the section of the impervious base of the superstructure, and *a t* = *h*, the maximum head above the work.

It is assumed that this head is completely dissipated at (*b*), the down-stream toe of the work, and that percolation water is delivered there as a mere trickle at a velocity incapable of moving the soil.

The shortest possible length of the line of percolation under these conditions is that of the floor, viz., *a b*.

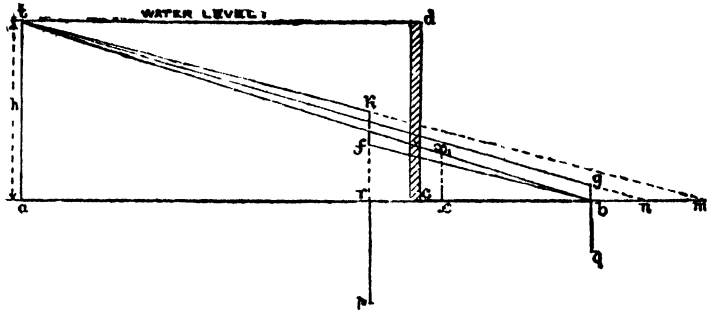
Writers on the subject are not in agreement as to the exact course of the line of flow connoting maximum velocity of

percolation under an impermeable floor through a homogeneous permeable subsoil of great depth.

Bligh considers the line of flow as one adhering to the under side of the impervious floor or to the surface of any obstruction, but the mathematical examination of the problem by Professor C. S. Slichter shows that the lines of flow probably approximate to an ellipse

In any case the length '*l*' of the line of flow cannot be shorter than *a b*. If therefore '*l*' be the length of the line of percolation which restrains the velocity of percolation within safe limits, having in view the nature of the subsoil, then safety may be secured by making *a b* equal to '*l*'.

FIG. 10



121 If an obstruction to flow such as an impervious line of piles (or wells) be interposed at *p* or *q*, it would clearly be possible to reduce the length *a b* of floor, while still retaining a length of percolation not less than '*l*'

There is some doubt as to the extent to which the value of '*l*' is affected by such obstructions If as appears probable the flow is in the form of an ellipse, the nearer to one or other end of the floor the obstruction is placed, the greater its effect in lengthening the shortest line of flow.

Bligh however maintains that the line of flow, which he terms the 'line of creep,' clings to the surface of the impervious barrier, and that therefore the effect of each line of piles (which he lays down must be at a distance apart of not less than twice their depth) would be that of twice the equivalent length of horizontal floor Under such conditions of flow the effect on '*l*' would be the same whether the piles were placed near the centre or ends of the platform.

Bligh refers to experiments as having given confirmation to the above, but does not give any details of the experiments on which he relies to support his method of computation which would need very strong confirmation before it could be accepted.

It appears on the whole advisable to take '*l*' as being the length of the shortest line connecting the ends of impervious barriers, and if both barriers in figure 10 existed, the length would be *a p q b*, and if only *p* existed, the length would be *a p b*.

122 Percolation through embankments —Where it is necessary to allow percolation flow to pass through embankments or

other works above ground level, designs must be guided by similar considerations, and the velocity of percolation suitably limited by securing adequate length for the shortest line of percolation.

123. Upward pressure on works from sub-soil percolation.—The pressures developed on the lower surface of the floor tending to displace it will now be considered. The measure of these can be best estimated by drawing a diagram of the hydraulic gradient due to percolation pressures.

In the case of percolation unobstructed by any subsoil barrier the hydraulic gradient is the line $t\ b$ (figure 10) and the line $x\ x_1$ is a measure of the pressure at any point x.

The obstruction to percolation by an impervious barrier at p, while leaving the pressures at a and b unaltered, causes a change in the slope of the hydraulic gradient by increasing the length of the line of percolation.

If the length of the line of percolation to any point x down stream of p, a vertical barrier below the floor, were computed as is done for the whole length (l) in paragraph 121 above, the length of the line representing uplift pressure at x would be $\frac{a\ p + p\ x}{l}$ and the hydraulic gradient would be formed by curvilinear lines. In view of the doubt which exists as to this matter and with a view to simplify the drawing of the hydraulic gradient, the effect of barriers in increasing the length of the line of percolation is, for the purpose of drawing the mean hydraulic gradient, computed in a manner different to that adopted in paragraph 121.

While one may hesitate to accept Bligh's proposal to consider the depth of a vertical barrier as equivalent to twice the same length of impervious floor it is considered safe to treat this depth, for the purpose of delineation of the hydraulic gradient, as equal to an equivalent length of floor and this method of arriving at the length (l), the base of the mean hydraulic gradient, is here adopted.

In the diagram (figure 10) hydraulic gradients are drawn for the same head (h) but with the length (l) of the line of percolation and with the hydraulic gradient differing in each case.

Make $b\ n = b\ q$
and $b\ m = r\ p$
join $t\ n$ and $t\ m$
and draw $b\ f$ parallel to $m\ t$

(1) With no impervious barrier under the floor ($a\ b$):—
under this condition $l = a\ b$
and the hydraulic gradient is the line $t\ b$.
Let the horizontal base of the gradient be l_1, then in all cases, the mean hydraulic gradient is $\frac{h}{l_1}$

In this case $l = l_1$ and the line of *mean* gradient is $t\ b$.

(2) With an impervious barrier $r\ p$ under the floor:—
under this condition $l = a\ p\ b$
and the hydraulic gradient is the line $t\ k\ f\ b$
In this case $l_1 = a\ b + r\ p = a\ m$
and the line of *mean* gradient is $t\ m$

(3) With an impervious barrier $b\ q$ under the floor:—
under this condition $l_1 = a\ q\ b$
and the hydraulic gradient is the line $t\ g\ b$
In this case $l_1 = a\ b + b\ q = a\ n$
and the line of *mean* gradient is $t\ n$.

124. It will be noted that no maximum net upward pressure can be developed on the section of floor upstream of the barrier $c\ d$ which holds up the water to the height h above the work, because the height of the water standing above this section of the work must more than counter-balance any upward pressure due to percolation.

Down-stream of this point the work must be so designed as to safely withstand upward pressure due to the hydraulic gradient, and the diagram clearly shows that if the length of the part down-stream of this is increased, this pressure is also increased and *vice versa*.

The total pressure on the floor is considerably increased by the down stream sub-soil barrier $b\ q$ and, although this barrier makes 'l' larger and therefore the work safer against removal of soil by percolation, which is technically termed 'piping', it entails more pressure on the floor down-stream of the superstructure.

If the sub-soil barrier had been put up-stream of the superstructure, or the floor up-stream lengthened to an equal amount, the same effect as regards percolation would be produced, but no extra strain would be placed on the floor.

125. Best position of piles or wells for checking percolation.—So far therefore as percolation alone is concerned, no sub-soil barriers or parts of floors should be down-stream of the superstructure holding up the water

As superstructural works over or through which water passes require protection immediately below them to prevent erosion by such water, these must have floors down-stream of the barrier for reasons other than those due to percolation. This matter will be further dealt with when treating of the details of design of such works.

126. Rate of evaporation.—The rate of evaporation, that is vaporization, of water varies greatly under varying climatic influences.

After a series of experiments on the subject Mr. G. Dines recorded the opinion—

(1) That no evaporation takes place from water unless the temperature of the water is greater than that of the dew point.

(2) That the amount of evaporation from water is proportional to the differences of the pressure of the vapour at the temperature of the water and at the temperature of the dew point.

(3) That the first and greatest cause of evaporation is the movement of the air.

(4) That whatever tends to increase the temperature of the water increases evaporation.

(5) That whatever tends to lessen the temperature of the dew point lessens evaporation.

127. Measurement of rate of evaporation.—Mean annual evaporation is the mean depth of water (in inches) evaporated annually from a free water surface.

Evaporation is frequently measured by means of a shallow pan floated on the body of water whose evaporation rate is to be ascertained; by this means the pan is kept at the same temperature and exposed to the same air movements as the water in which it floats.

The pan is filled with water up to a certain mark and filled up daily to the same mark, the quantity of water required for replenishment, after making allowance for recorded rainfall, being the measure of the quantity of water evaporated from the area of the surface water of the pan.

There is reason however to believe that evaporation from a pan such as described is greater than that from a large sheet of water and that in consequence records of evaporation thus measured give excessive results. Referring to this, Parker says "Modern studies show that the evaporation from the water surface of a large reservoir is considerably less than (generally about two-thirds) that which is observed in ordinary evaporation pans".

128 The following table shows the mean daily temperatures and evaporation losses observed during three years at the Assuan Reservoir on the Nile:—

Months	Temperature degrees Fahr	Evaporation Inches.	Months.	Temperature degrees Fahr.	Evaporation Inches
January ...	62·0	0 15	July ...	93 5	0·43
February ..	67 4	0·08	August ..	92 7	0·42
March	69·9	0·24	September	88·5	0·33
April	81·7	0·30	October ..	82 5	0·29
May ...	88 1	0·36	November	72 9	0·22
June	91 3	0·43	December .	62 7	0 15
			Mean for 1 year .	79 3	0·29

The losses recently observed from an evaporation pan in Red Hills Tank, Madras, are given below. The monthly losses in inches are the average of 2 or 3 observations for whole months taken in different years between October 1914 and 1917:—

	Inches		Inches.
January ..	4·66	July ...	5·73
February	4·75	August ..	6·50
March	6·80	September .	5·66
April .	7 77	October ...	4·79
May	9 00	November ...	3·97
June	6·69	December ..	4·27
		Total ..	70·59

Kennedy gives the losses from evaporation per day in the hot weather in the Bari Doab Canal as 0·18 inch per day which is smaller than would be expected.

He estimates the *minimum* losses by percolation from the same canal at 1·9 inch per day, that is, over ten times as much

Buckley states: "Loss by evaporation alone rarely exceeds 0·4 inch a day in the hottest and driest weather in India".

129. Evaporation and absorption.—Losses of water received into reservoirs and canals which are not due to visible draw-off, are attributable to evaporation and absorption, which latter includes both percolation and absorption. The loss due to these causes combined can more readily be ascertained than the loss due to either cause separately, and measurements of losses and allowances made for losses generally combine both these in one item. In tanks at times when there is no in-flow, and when the draw-off is accurately known, the combined evaporation and absorption losses can be readily and accurately determined by computations based on the daily water-levels

130. Loss from tanks.—The loss from tanks by absorption does no doubt, vary greatly with the nature of the bed, but this even if abnormal, is generally much reduced, after a tank has been in existence for some years, by deposition of fine silt all over the tank bed.

Referring to projected reservoirs Parker says:—

"A safe practical rule appears to be as follows:—Allow for evaporation and leakage combined, the quantity reported as evaporated from a free water surface, i.e., from a small evaporation pan" (*Control of Water*, page 741)

Such a rule would not of course apply to a tank the bed of which was exceptionally porous, unless the surface had been covered with a layer of impervious silt, nor to a case where a large quantity of water is lost by percolation through, or under the embankment, but it is a good working rule for ordinary cases.

131 Losses from tanks in South India from evaporation and absorption.—The monthly losses of water in tanks in South India would be about as follows:—

January	4 inches.	July	6 inches.
February	4 „	August	6 „
March	7 „	September	6 „
April	9 „	October	5 „
May	10 „	November	4 „
June	7 „	December	4 „

Yearly total 72 inches or 6 feet.

132. Losses in canals and distributary channels.—The losses from evaporation in canals and distributaries are dependent on the free water surface exposed and never amount to a considerable percentage of the whole flow; as a maximum they are unlikely to exceed 5 per cent and may be frequently less than half this.

The losses from percolation are dependent on the nature of the soil through which the canals are carried and of the sub-soil drainage conditions, that is the percolation flow.

The losses by percolation may be quite small, say, as little as 5 per cent or less, or very large, in some cases exceeding one-half of the whole water available.

From the above it follows that the great variation in losses from evaporation and percolation in canals is practically all due to percolation as distinct from evaporation, although it is usual, as already explained, to measure or allow as one item the losses due to both causes.

133. Losses from evaporation and absorption in canals and distributaries.--It has been generally assumed for purposes of calculation that percolation losses from canals vary as the wetted area and as the square root of the depth, and the formula proposed by Dyas more than half a century ago is quoted by writers in reference to this matter.

Dyas' formula is $P = C\sqrt{d}$ where P is percolation in cusecs per million square feet of the wetted surface, 'd' the depth of canal and C a constant.

As, however, more recent investigations have shown that the velocity of percolation through the interstices of fine soil varies as d, and not as $\sqrt{d}$ as was assumed by Dyas from the analogy of ordinary pipe discharge, the formula may be considered as of doubtful suitability

In practice for purposes of design of large canals it is usual to classify soils along the length of canal alignments and, neglecting depth, to allow for percolation at so much per million square feet of wetted area.

The allowances for Madras which are at present authorized per million square feet are—

1.	Decayed rock or gravel	10 cusecs.
2.	Alluvial or red soil	8 ,,
3.	Black cotton soil	5 ,,
4.	Rock	3 ,,

These are of course subject to modification according to the special conditions of each case; for instance fissured rock may leak very freely. The allowance for black cotton soils is believed to be greatly in excess of actual losses.

Any soils which are cut through at a level below the ordinary water table will seep into the canal whenever the canal water is below the water table level.

The level of the ground water table has a considerable effect on percolation losses; as soon as the space between the percolation gradient and the water table is saturated, the rate of percolation loss from the canal will be reduced (vide figure 11).

FIG. 11.



134. Percentage losses by evaporation and percolation.—For distributaries and water courses the probable losses by

percolation and evaporation are generally estimated in the form of a percentage allowance. This system is also sometimes applied to the large canals of a proposed system.

Some idea of the losses which may be incurred in a canal system is given from the following estimate made by Kennedy of those on the Bari Doab Canal in the cold season of 1881-82, viz., that out of every 100 c.ft. entering the head of the canal 20 c.ft. were lost in the main canal and branches, 6 c.ft. in the distributaries, and 21 c.ft. in the village water courses.

This left only 53 c ft. delivered on the fields and of this he estimated that about 25 c. ft. was used up by uneconomical methods of watering.

The observed losses in the Punjab in a length of 52 miles of the Bari Doab Canal were measured as 12·7 per cent of the flow of 2,114 cusecs measured at the upper end of the length ; the velocity of flow varied between a mean of 4·4 ft. per second at the upper and 2·8 ft. per second at the lower end of the length.

A second length of 63 miles down this canal where the upper discharge was 336 cusecs, showed a percentage loss of 19·1.

Observations in the Sone canals in Bengal give similar results.

The above losses are by no means of exceptional magnitude and amount to about 8 cusecs per mill. sq. ft. of wetted area.

135. In canals running in retentive soils, such as the black soils of the Gōdāvari and Kistna deltas, the losses by percolation are small.

Water was impounded in the Kistna Delta so as to stand in a length of several miles of canal in the period of closure from March to May of the year 1909 and the level measured weekly; the losses during the period were not more than what might be attributed to the effects of evaporation. On the whole the losses from canals and channels in these deltas from evaporation and percolation are not in excess of 10 per cent and are possibly considerably less.

On the other hand the losses from channels in light soils in the upland tracts throughout the Madras Presidency are likely to be quite as great as those in the Punjab, and the remark in Buckley's Irrigation Pocket Book that " losses from 10 to 40 per cent between the head of an irrigation system and the field actually watered are to be expected " may be endorsed as generally applicable in South India.

CHAPTER V.

SILT AND SCOUR.

136. Classes of silt.—Silt is the non-floating solid matter transported by flowing water in rivers, canals, etc.

With reference to silt, Parker states :—

"From an engineer's point of view two classes of silt exist in all rivers. These are :—

(i) *Bed-silt* which is rolled along the bottom of the river.

(ii) Lighter particles of silt which are carried forward in the water, as turbid matter, or *suspended silt* "

and this classification is here adhered to.

As the size of particles which can be carried as matter in suspension depends on velocity of flow, it follows that silt which may be in suspension in the steeper and more rapid part of a river's course may, in the lower and more sluggish part of the course, become bed-silt.

On the other hand as the process of rolling along the bed of a stream causes attrition between the particles tending to divide them up into smaller individual grains of matter, what may be coarse silt in the upper part of a river may by this process be converted into fine silt lower down.

137. Factors influencing the nature and quantity of river silt.—The silt carried by a river is the result of erosion by water of the rocks and soil in the catchment area of the river, and the proportion of silt to water and nature of the silt carried depend on the nature of the surface soils and surface slopes, and on the amount and distribution of rainfall in the catchment. Generally speaking the upper portions of the course of a river are steeper and with more rapid currents than the lower portions and the silt is coarser; in many places where rivers issue from the hills large quantities of heavy shingle, and even boulders, are carried along the river bed.

Further down in the plains, silt is generally confined to small shingle, gravel, sand, and earthy matter.

138. Nature of silt in Madras rivers.—In most of the rivers in the plains of Madras the 'bed-silt' consists of coarse sand and in some places of small shingle and pebbles, while the silt in suspension is finer sand and earthy argillaceous matter of a plastic nature.

139. Measurement of quantity of silt in suspension.—Computations of the quantity of silt carried in suspension by a river or canal, are made from the data obtained from measurements of samples of silt carrying water, taken from different depths and positions transversely across it.

From a measurement of the quantity of water and of solids in the samples their relative proportion becomes known, and by applying such proportion to the total discharge during the period to which each set of measurements is applicable, the total quantity of silt carried in suspension in that period can be computed.

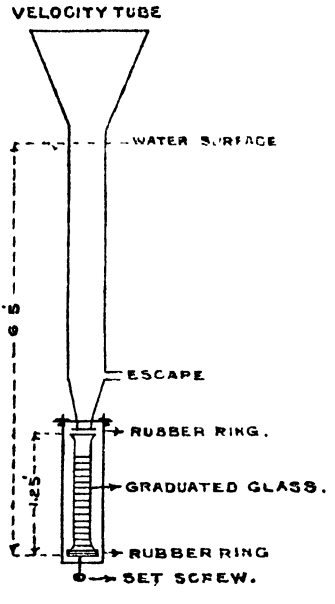
140. Taking samples for estimating quantity of silt.—The proportion of silt to water increases in varying proportions in accordance with the depth below the surface, and it is for this reason necessary, in taking samples of silt-laden water at any point in the transverse section of a river, to take these at different depths below the surface, dividing the whole depth into a suitable number of parts. If the bottom is soft, care must be taken not to disturb it in any way when taking the sample nearest the river bed. The silt in the samples is separated from the water by filtration, dried and weighed and from this the proportion by weight of solid in suspension to liquid is ascertained.

141. Mechanical analysis of nature of silt in suspension.—If desired the composition of the dry silt can be further analysed in accordance with the nature and size of the particles; thus clay may be separated from sand, and sand may be analysed into various sizes of grains.

The size of particles composing silt is of importance in the design of canals as the transporting power of a stream depends on the size and specific gravity of the particles, and while coarser particles may sink and cause obstruction in the canal bed, the finer ones will be carried in suspension through the canal and out on to the irrigated fields.

Silt can be graded according to the size of the particles composing it by passing it through a series of screens of known size of mesh (paragraph 116 above), but another and very convenient system of classifying silt has been introduced in the Punjab, and is now frequently used. The standard of measurement adopted under this system is the velocity in feet per second at which the particles sink in still water. Thus sand grains which sink 1/10th of a foot in one second would be '0·1 grade sand,' and those sinking in 1/5th of a foot '0·2 grade sand.'

FIG. 12.



142. Punjab system of sand analysis.—The apparatus used in the Punjab for grading silt is shown in the sketch (figure 12) and is termed a 'silt classifier.' The measuring glass is graduated in 14 divisions, each of volume 0·001 of a cubic foot, and each being ·07 foot in depth.

One-tenth of a cubic foot of silt is thrown into the water with which the tube is filled up to level 6 5 feet, and the intervals of time taken for the silt to settle in the measuring glass to the level of each graduation, or any required graduations are noted Thus if the heaviest sand reached the bottom of the tube in 21 seconds the rate of fall per

second would be $\frac{6\cdot5}{21}=0\cdot31$ foot, and this sand would be '0·31 grade.' If the whole sample fills the tube up to the 10th graduation in 58 seconds the lightest sand will have fallen $(6\cdot5-\cdot7)=5\cdot8$ feet in 58 seconds, the whole sample would therefore be '0·10/0·31 grade sand.' In practice it has been found that, with the usual Punjab grading of canals, sand lighter than 0·1 grade causes little inconvenience, as it is carried in suspension by canal water through the outlets and on to fields, and that it is only the sands of 0·1, and coarser grades that cause trouble by obstructing the canal flow by silt deposition.

143. Proportions of silt carried in various rivers —The results of some of the measurements made of the proportion of silt in suspension to water carried by rivers are noted below :—

	Proportion by weight
The Indus in flood	1/240
The Nile „	1/666
The Sutlej „	1/60
The Sone „	1/180
The Kistna „	1/100
The Cauvery „	1/726
The Tungabhadra in flood	1/150

While the above figures give some idea of the quantity of silt observed as being carried in suspension *in times of flood* it must be realized that the highest floods do not necessarily, or even generally, carry the highest proportion of silt and that floods of the same magnitude in the same rivers carry proportions of silt differing by more than 100 per cent Generally a rising flood carries a greater proportion of silt than a falling one, and a flood occurring after a long period of draught carries a higher proportion than other floods.

144. Silt quantities in the Kistna and Cauvery rivers.—Daily silt observations were taken in the Kistna throughout the flood seasons (June to November) of three successive years and the *maximum* proportion of silt to water measured was 1/42 by weight (1/68 by volume). The *average* for June which was the month of highest silt proportion was 1/61 by weight (1/98 by volume) For the whole period of measurement (June to November) the average was 1/297 by weight (1/475 by volume). It is clearly the last figure only which will, when applied to the quantity of flow, give a reasonably close estimate of the total quantity of silt carried in suspension during the period of observation.

Similar observations have been taken for one flow season, June to December 1909, in the Cauvery giving an average for the whole season of 1/2105 by weight (1/3370 by volume). The maximum proportion observed on any one day was 1/344 by weight.

145. Relation between measurement by weight and by volume.—The observations in the first instance give proportions by weight In order to find the relation between weight of (chemically) dry silt, and volume in the case of the Kistna and Cauvery, a sample of damp alluvial deposit as it lay in the river margins was taken as a sample volume and the weight of this known volume after drying was determined. In both cases the weight came to

between 98 and 100 lb. of dry silt per cubic foot of damp silt. The amount of absorbed water in the case of the Cauvery sample was 17·53 lb. per cubic foot.

146. Quantity of silt yearly transported to the sea by large rivers.—The total quantity of suspended silt carried yearly to the sea is enormous. The following gives some interesting details of the results of computations made in regard to this.

Sir Benjamin Baker computed that the Nile discharged 1,270 mill. c ft. of silt yearly into the Mediterranean. Humphreys and Abbot computed the Mississippi silt discharge at about 6,750 mill. c.ft.; this would suffice to raise 241 square miles 1 foot in depth annually. The Kistna silt carried amounts on an average to 4,220 mill. c.ft. yearly.

147. Exclusion of bed silt from silt measurements.—The above estimates exclude the bed silt from consideration No satisfactory method of measuring bed silt has, it is believed, been discovered. While it appears probable that bed silt carried forward is a considerably less quantity than silt in suspension it is known nevertheless to be, in many cases, very large

148. Alluvium and alluvial formations.—While river silt is derived from the erosion by running water of the rocks and soil in the catchment, it is impossible for water when not in motion to retain silt in suspension for any considerable time, and a check in velocity of a current highly charged with silt causes immediate deposit of part of the suspended particles. Thus wherever water of a river charged with silt spills and spreads over a wide surface, so as to lose a great part of its velocity, it deposits silt on that surface thus raising its level. River silt thus deposited is called 'alluvium' and the flat lands formed by deposition of alluvium are called 'alluvial formations'.

It is usual to find that rivers which flow through alluvial tracts have raised the ground surface, by deposition of their silt, to such a height that it is only subject to submersion by exceptionally great floods; if the natural denudation due to wind and rain balances the effect of silt deposition the further raising of level of the tract altogether ceases.

149. Rivers running through alluvial plains.—Rivers running through alluvial plains have generally very tortuous courses; any part of a margin which is more friable than the rest is likely to be eroded causing the river to deflect into a slight curve, and this induces a more rapid current in the concave side, and slower flow on the convex, and this again further accentuates the curvature by increased erosion on the concave side, and by silt deposition on the convex. By reflex action there is a tendency of the current after passing such a curve to form a similar concave erosion below on the other side of the river with a similar convex silt deposit opposite to it and such action is repeated throughout the course of the stream. This action has the result of making the river course a continually winding one.

150. Silt deposit in lakes.—When a river or stream enters a depression forming a lake its current is checked, and it deposits silt in a fan shape over the lake bottom, and pushes out into the water a shoal, or series of shoals, of which the surface levels are

gradually raised by silt deposit until sufficiently high to give lodgment for the growth of vegetation, which further accelerates deposition of sediment from river water, from wind, and from decay of vegetation. The shoals thus gradually become dry land through which the river in one or more branches finds its way, still pushing out into the lake further shoals. Thus in the course of time the whole lake must gradually be transformed into an alluvial plain through which the river will flow, only keeping open a water way of such area that the combined cross section area of the branches of the river are sufficient to carry the normal discharge.

151. Formations of deltas.—By similar action shoals which are gradually raised into dry land are formed in the sea by silt bearing rivers. The land so formed by the deposit in the sea of the silt carried by rivers in suspension generally takes a triangular shape, and from this originates the name 'delta' given to tracts so formed, because the form of the Greek letter bearing that name is a triangle, and resembles in shape the delta of the Nile.

A delta is formed by successive layers of river borne silt deposited in the sea in the form of gradually rising shoals until they reach the surface, when they continue to grow, partly by the seasonal growth and decay of vegetation and partly by continued deposition of silt, and floating drift, from the river, until, in course of time, a plain is formed which at its up-stream part is at such a level as only to be submerged during high floods. The seaward end however continues gradually to push out into the sea and continues to grow, unless checked by tidal or other current carrying away the silt Some rivers although carrying large quantities of silt do not form deltas and this occurs where powerful ocean currents exist, and where the rivers outfall into deep water

152. Formation of bars.—Where a river outfalls into an open sea there is usually a 'bar' which is a ridge of sand, gravel, or mud, which forms across and partially obstructs the outfall channel. The formation of this barrier is due to the conflict between the river current and the sea. The bar is formed in some places of the bed silt which the river current can push no further, while in some places it is formed by the action of the sea in piling up sand and gravel along the coast; generally both causes operate. The action of the sea in piling up detritus along the coast is well illustrated along the east and west coasts of South India where continuous lines of sand ridges and bars run for hundreds of miles along the sea coast; the sand is the coarser detrital matter carried to the sea in river water, mixed with fragments of shells, and of rocks eroded by the action of the sea on the coast. The existence of sand ridges and bars has the effect of forming along the coast long stretches of lagoons, locally termed 'backwaters', and marshes running parallel to the sea and separated from it by the sand ridges.

As a result, in many places the rivers do not discharge straight into the sea but turn, when they reach the coast, for some distance taking a course parallel to it along the backwater, until finding a weak place in the sand ridge, the water breaks through into the sea across an open bar which forms the river mouth. These bars frequently are not stationary, but the opening travels up and down the coast under the action of the prevailing seasonal

winds and ocean currents. In the dry season these openings are frequently closed altogether by the action of the sea in forming a sand ridge across them to a level above that of high tide, and they remain closed till they are reopened the next rainy season by the land water in the lagoon rising to such a height as to again breach across the ridge into the sea. Such a breach occasionally occurs in a different place from the customary one, and occasionally much local loss and inconvenience is thereby caused.

153. Distinctive features of deltaic tracts.—It will be noticed that, from the point at which a river enters a delta it generally ceases to receive tributaries, and in addition shows a tendency to subdivide into branches, which themselves in turn show a similar tendency to subdivision.

This tendency to subdivision is clearly traceable from the process of silt deposition by which the alluvial tracts, through which the river and its branches flow, were formed. The deposit of silt from the flood over-flow on either side of the rivers will naturally be heaviest close to the river margins and grow less in thickness the greater the distance away. Thus, on either side of each river, or branch, an alluvial plain is formed of which the surface slope is away from the river. The deposition of silt is specially heavy along the margins of each river branch forming local projections above the level of the plains which act like low flood banks, and these serve to retain water in the river branches even when flowing at levels above those of the marginal plains.

Wherever the main branch, by erosion in a curve or otherwise, breaches the high margins, the slope of the surface being away from the river, there is a tendency to form a separate outfall or branch, and even for the whole river to become diverted to a new course. This accounts for the number of bifurcations of rivers and their branches in the deltaic portion of their course. As each river branch forms an alluvial tract on each side of it, the splitting up of delta rivers into branches greatly assists the formation of the fan shape which such tracts usually assume. The margins of each branch are raised and the land on either side slopes away from the branch, and low ground is formed between the branches facilitating drainage These low lands drain into the lagoons and marshes stretching along the coast divided by sand ridges from the sea, and the drainage of delta lands finally reaches the sea through natural or artificial openings in the sand ridges.

Figure 13, which is a plan of the delta of the Mahanuddee river from Buckley's "Irrigation Works of India", clearly displays the typical features of a delta, viz., the triangular shape, the number of branches into which the river divides, all of which run on ridges, the absence of tributaries, and the outfalls to the sea through lagoons.

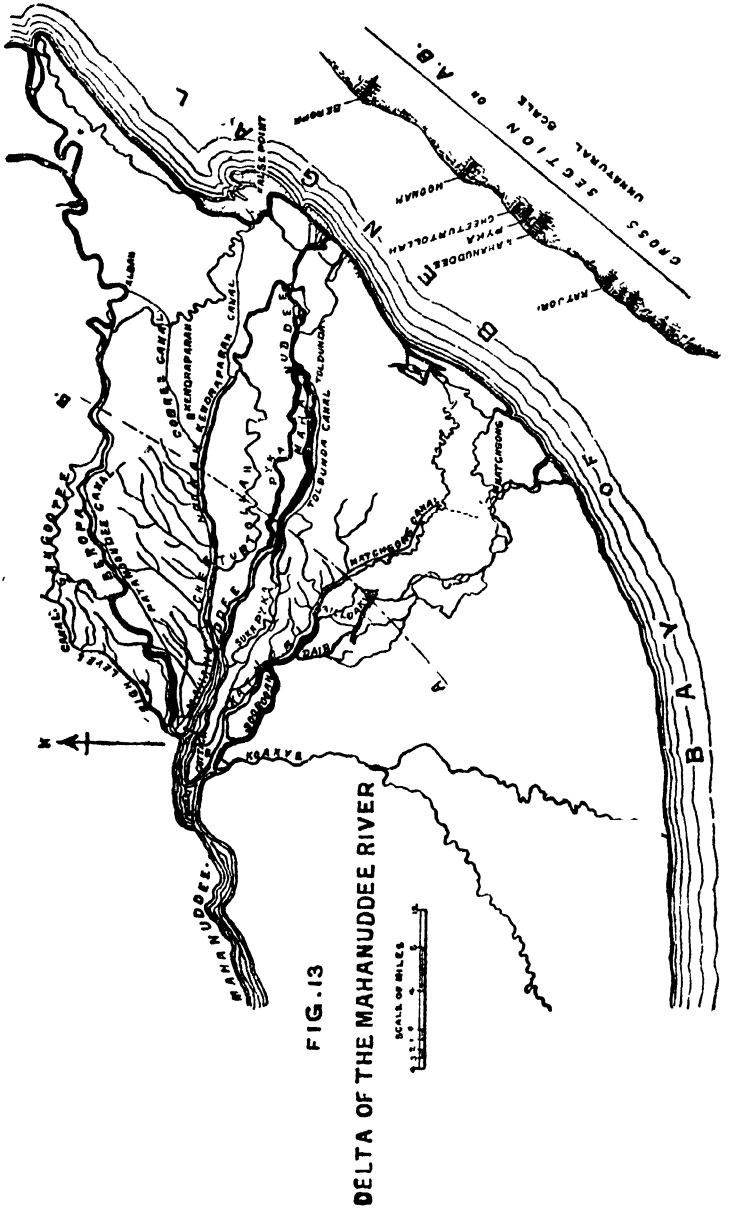


FIG. 13

DELTA OF THE MAHANUDDEE RIVER

154. Gradual flattening of surface slope of delta river.—The continuous process of formation of a delta causes the gradual pushing out of the river mouth into the sea, and as the sea level remains the same, this must entail the gradual flattening of the surface fall of the river in the lower part of its course, with a consequent tendency to diminished flood discharging capacity. Also as the sea is approached, the general level of the marginal land becomes gradually lower with reference to flood level in the river and a greater proportion of the river flood water is discharged over the margins.

On both the above grounds there is a tendency for rivers to have greater flood carrying capacities within their margins, where they enter their deltas than the combined capacities, of the rivers and their branches lower down. This tends, in the stages of delta formation to expedite the raising of the level of the lower alluvial tracts, but when these become fit for occupation it results in subjecting these tracts to inconveniently frequent submergence by floods.

155. Liability to constant change of delta river channels.—These causes also accentuate the tendency already noted for the rivers in the delta to change their course, and to throwing off branches, and also to create a tendency to change by widening or deepening the existing river channels to such dimensions as will enable them to accommodate, with an ever flattening surface fall, an increased quantity of flood discharge. This last is much increased if the throwing out of branches is checked, but, on the other hand, when a new branch is formed existing branches may partially silt up. From the above it will be understood that there is a great tendency to change in the *regime* of river branches discharging through deltaic tracts.

156. Characteristics of delta rivers.—The special characteristics of a delta and the rivers which traverse it may thus be summarized:—

(1) The branches of a river which run through the alluvial lands forming a delta run on the higher parts, or ridges, of the tract, and the general surface slope of the soil is away from the margins. The lowest lying tracts or valleys are intermediate between the river branches.

(2) Deltaic tracts are always below the maximum flood level of the river or branches through which they run.

(3) There is a constant tendency to change in delta river channels.

(4) The flood carrying capacity of the river branches within their margins are generally smaller than that of the main river at the head of the delta.

157. Irrigation facilities in deltas.—The formation of a delta offer great facilities for flow irrigation. The river water levels frequently rise above the levels of the land surface which is flat and slopes away from the river margins, the soil is usually very fertile and being soft gives facilities for the cheap construction of canals. On the other hand there is the liability to frequent and excessive submergence and injury by floods and the tendency to the change of river channels with corresponding risk of serious

damage to occupied areas. Irrigation works in deltas therefore consist of suitably controlled distribution canal systems, and works to restrict and control submergence by floods and damage to occupied areas by change of river channels The flood control works comprise, flood banks on either side of river channels leaving between them suitable sections for flood discharge, and weirs, regulators and training and protective works to direct and divert the flood discharge in suitable proportions down the various channels, and to prevent changes in these channels which are likely to cause serious damage to cultivators

158 Drainage difficulties in deltas.—While the formation of a delta gives great facilities for running river water by flow on to the lands, there are frequently great difficulties in disposing of the drainage of this water and also of the rain falling in the deltaic area. The formation of depressions between the river branches no doubt assists drainage, but there is generally a great difficulty in maintaining a suitable outfall to the sea

As has already been stated there is a tendency to the formation of bars and sand ridges all along the sea coasts, and between these and the occupied deltaic tracts lie lagoons and marshes into which the drainage water flows, while the out-flow into the sea is checked by the formation of bars, and at times altogether stopped by the closing of the same by the action of the ocean. The greatest difficulties to be encountered in connexion with the drainage of deltas are in connexion with the maintenance of effective drainage outfalls in the sea. In the earlier days of occupation of delta areas these difficulties are not accentuated, because only the higher lands are occupied, and the great extents of unoccupied low-lying lands in depressions on the margins of marshes and back waters give storage capacity into which drainage is disposed of without inconvenience; as cultivation extends and lands become more valuable, blocks lying at lower and lower levels are occupied, drainage difficulties appear and gradually become more and more accentuated until finally, a limit is reached below which it is not economically feasible to secure adequate drainage and the cultivation of lands below the limit level must be attended with risks of great losses of crop from submergence after any abnormally heavy falls of rain.

159. Silt transporting power of water.—The proportion of silt which can be transported by moving water, depends on the velocity of the water, and the nature of the silt, that is, the specific gravity and the size and shape of the particles moved.

It is a matter of great importance to design and construct earthen channels with such longitudinal slopes and cross sectional dimensions that they will neither be obstructed by the deposition of silt, or injured by erosion of their beds or marginal slopes. Design of channels should therefore be guided by the laws which govern the erosion and transportation of silt

The matter of silt transportation and the laws which govern it or by no means fully understood and there are differences of opinion between recognized authorities.

160 Kennedy's theory of critical velocities.—It is to the studies of Kennedy that we owe the rules of design in regard to silt transportation in canals which have been for the last 20 years

successfully applied in the Punjab, as also in very numerous canals and channels throughout other parts of India and Burma.

Kennedy made a study of the conditions of flow of 22 channels in the Bari Doab canal system, of very varying capacities, velocities and relations between bed width and the depth, which channels had been in existence for many years and which had come to a permanent *regime*, that is, they were neither silting nor scouring.

This does not imply absence of periodic seasonal deposits or scours, but that on the whole years' flow, and counting from year to year, there was no change in silt deposits in the beds of the channels.

Kennedy plotted a diagram showing the relation between mean velocity and depth of a number of typical sections of these channels, and found that within reasonable limits of deviation a constant relation existed between depth and velocity, which relation was defined by a curve satisfying the empirical equation

$$V_o = 0.84\, d^{0.64} \quad \ldots\ldots\ldots\ldots \quad (7)$$

Where d is the depth in feet flowing over the bed of the channel and V_o is the mean velocity.

From the facts adduced he advanced the proposition that the mean velocity and depth of a channel were the essential factors influencing the silt transporting power of flowing water, and that for the same qualities and proportions of silt to water, there is, for each depth of flow, a certain velocity (V^o), which he termed the "critical velocity" for that depth, which will just suffice to prevent silting by deposition of transported silt which enters the canal with the river water The relation between this velocity and the depth of flow is expressed by an equation in the form

$$V^o = c d^m \quad \ldots\ldots\ldots\ldots \quad (8)$$

When the soil through which the canal is cut permits of a greater velocity without erosion this is allowable, but very frequently the soil in which a canal runs is alluvium of the same quality as the canal silt, and when this is so V_o is the limit of both the non-silting and non-scouring velocity

Guided by these observations Kennedy propounded his theory in a paper published in 1895 in the Proceedings of the Institution of Civil Engineers, Vol. CXIX

The theory is in general accordance with the view which had been previously enunciated by other engineers, viz., that other things being equal, the quantity of silt carried forward per foot of width of a channel depends on the bottom velocity of flow As the bottom velocity generally bears a nearly constant ratio to the mean velocity in the vertical of the water flowing above it a similar relation would apply to mean velocity.

161. Kennedy's theory of silt transportation.---Kennedy enunciated his theory in the following terms:---

"Sediment in flowing canals is kept in suspension solely by the vertical components of the constant eddies which can always be observed over the full width of any stream boiling up gently to the surface. . . . "

"In order therefore to obtain an expression for the silt supporting power of the stream it may safely be assumed that the quantity of silt supported will be proportional to the width of the bed all

other conditions remaining the same; in fact this has often been argued on purely theoretical grounds."

"It must also vary with the velocity of the stream V say as V^{n-1}, since the greater the velocity the greater the force of the eddies, which would become zero when the velocity is zero."

"There is a third variable, the depth, but this could affect neither the number nor the force of the eddies. The amount of silt supported in the stream may therefore be expressed by $Ab\,V^{n-1}$, where A is some constant unknown, and b is the bed width. But whilst thus supported the silt is being moved forward with a velocity V so that the amount of silt transported will be equal to $Ab\,V^n$."

"It is here presumed that all the silt sediment is in suspension but there is doubtless a small portion of the heavier silt simply rolling along the bed. This amount would vary as bV instead of bV^n; so that the value of n to include rolling silt would be somewhat less than it would be if the suspended silt alone were considered"

"If it be assumed that the amount of silt supported is proportional to the upward pressure of the deflected currents of water, which varies simply as V', the expression $n = (1+2)$ or $n = 3$ is arrived at"

162 Kennedy's law considered theoretically.—Kennedy's equation $V_o = cd^m$ may also be deduced from theoretic considerations The silt quantity q carried forward per foot width of bed may be represented by the equation $q = AV^n$.

The quantity of water Q carried at a velocity V per foot width of depth d is Vd.

If p represents the ratio $\frac{\text{quantity of silt}}{\text{quantity of water}}$, then $p = \frac{q}{Q}$

and $q = pQ$

$= AV^n$

also $Q = Vd$

therefore $AV^n = pd\,V$ and $V^{n-1} = \frac{p}{A}d$

Hence $V = \left(\frac{p}{A}\right)^{\frac{1}{n-1}} \times d^{\frac{1}{n-1}}$

The above equation may be written in the form $V_o = cd^m$ . . (8)

The values of c and m experimentally deduced by Kennedy as applicable to the silt conditions of the Bari Doab canal system were $c = 0.84$ and $m = 0.64$ This value of m corresponds with a value of $n = 2.56$. The value of n is probably somewhat less than 3 because bed-silt is included.

Kennedy's Equation is

$$V_o = 0.84\,d^{0.64} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

The equation gives the relation between velocity and depth which will secure the carrying forward silt to water in the proportion (p), and if the silt and water which enter the canal during a season are in this proportion the canal will be non-silting.

163. The assumption is that p is unchanged throughout the system, and that however the discharge be varied in various canals, the relation between V_o and d must be unaltered to secure the transport of the same kind of silt and the same proportion of silt to water.

For silt of different kinds the same general relation would hold, but the value of the co-efficient c or the power m, or both may require slight alteration.

The actual values originally proposed by Kennedy have been successfully applied to the construction of new, and the remodelling of existing channels, in which the nature of the silt differed greatly from that carried in the channels originally observed, and there is reason to believe that in many cases, but little alteration is required to make formula applicable to very varying classes of silt. The relation between velocity and depth to produce a non-silting channel in any existing system may be ascertained from actual observations in existing channels of the system which have attained permanent *regime* by plotting the results on squared paper with depths as abscissae and the velocities as ordinates.

The constants of the curve of the form,

$$V_0 = cd^m$$

which most nearly suits the results may be taken as suitable to the conditions of the canals of the system.

By working in this way the following equations have been evolved for the channels in the Gōdāvari and Kistna delta systems—

Gōdāvari $V_0 = 0{\cdot}67\ d^{0{\cdot}55}$

Kistna $V_0 = 0\ 93\ d^{0\ 52}$.

164. Reduced values of Kennedy's V_0.—The following statement gives, for various values of d, the critical velocity V_0 obtained from Kennedy's formula, which is generally referred to as "Kennedy's V_0".

The formula is $V_0 = 0\ 84\ d^{0\ 64}$ . . . . . (7)

(d) Depth in feet	1	2	3	4	5	6	7	8	9	10	12	15
(V_0) Critical velocity in feet per second	0·84	1·30	1 70	2 04	2 35	2 64	2 92	3 18	3 43	3 67	4 12	4 75

165 Variation of velocity to suit different kinds of silt.—When, owing to the difference in the character of silt, it is considered desirable to modify these values, this is usually done by altering c leaving d^{64}, that is by taking a proportion of Kennedy's V_0.

Not only is this sometimes advisable in different canals but in the tail reaches of the same canal it has by experience been found that the critical velocity is reduced.

Writing on this subject in May 1907 with special reference to the Punjab rivers and after much experience had been gained on canal systems proportioned for his V_0, Kennedy states (Hydraulic Diagrams):—

(*a*) "The critical velocity (V_0) here referred to may be defined as the mean velocity which for a channel of given depth on a sand silted bed, or running fully silt-charged with what may be called the standard sand-silt found in river beds just after they leave the hills, will just keep that channel all the year round from either silting or scouring its bed. . . The following are the most common cases where some modification of V^0 is called for, and

where such is needed we may estimate its amount by the percentage or fraction of V_0 which must be substituted instead of V; for example 0·8 V_0 or 1·1 V_0, etc"

(*b*) "**Modification of V_0 due to position of offtake on the river**".—"When the offtake of the canal is far from the hills the sand being much finer is easier carried and therefore a less value than V_0 will suffice; how much is difficult to say, perhaps down to 0·75 V_0 in some cases, but the real value could be simply enough obtained by selecting a few well-defined channels in nearly permanent regime, and observing the mean velocity and depth. Then the ratio of this velocity to V_0 will give fraction of V_0 to apply in any new designs. If, on the other hand, the canal head were in the hills, and from a boulder instead of a sand bed, it would be necessary to contemplate the annual removal of shingle or even the coarsest sand and pebbles; and the head reach, where this would happen, would have to be steeply graded according to the soil. Lower down where only the standard sand would reach, V_0 would be allowed, the same as for a river with a sand bed."

(*c*) "**Effect on V_0 of changing the method of head regulation**".—"Occasionally, means may be adopted at the head of a canal to exclude the full proportion of silt in the river water especially in flood seasons Thus the river may be partially ponded-up on the offtake side by means of closed undersluices and divide walls, etc., regulation being done at the far side of the river, and the pond occasionally scoured out through the sluices. This materially reduces the sand silt entering the canal, and besides this the whole canal may be shut off during really heavy floods, so that the canal supply is not fully charged, and a less value than V_0 can be given. This may be exemplified by a reference to the Sirhind Canal, which, with normal river regulation, silted very heavily owing to too small a slope, but which, since the adoption of a change in regulation on the above lines, has given no trouble, and now runs with a value of 0·9 V_0".

(*d*) "**V_0 decreases in lower reaches of canal and distributaries**".—"When a canal gives off a branch or distributary, it occasionally happens that the offtake takes off more than its proper share of the sand silt, due for the most part to the sill of the sluices being at or near the bed level and drawing in much of the lower strata sand, which in steady flow, partly rolls along the bed The same thing may happen even if the sill is fairly high due to some persistent local eddy raising up this sand, and in such a case the offtake will be overcharged with sand and require more than V_0 and the parent channel will be left rather undercharged. On some canal branches the action is fairly well marked and the tail reaches have comparatively little sand left and can be graded to give less than V_0 –0·90 V_0 sometimes. It is on the distributaries, however that this action is certain and most marked because here the outlets are and should be at the bed level and draw off the rolling bed sand direct, with little or no stirring up action of the main body of the flow, which would otherwise equalise the distribution of the sand from surface to bed. The result is that the greater part of the heavy sand is drawn off by the outlets in the first few miles of the head reach of the distributary, so that the

lower reaches can be graded to much less than V_0 and this accounts for the fact that the grade near the head of a distributary must always be and is much steeper than farther down. The fraction of V_0 to be graded for, therefore should usually decrease gradually from V_0 at the head to 0·95, 0·90 and 0 85 V_0 and in some very long channels even 0·80 V_0 may be safe; and this is fortunate since in very flat country it would be otherwise impossible to get the requisite slope. Moreover to grade for V_0, the whole length would in some cases of poor soil cause erosion simply because the water having little or no charge of its own, can pick up a local supply."

(*e*) "**Increase of V_0 and erosion from excessive velosity**".— "Quite a common case is where a canal reach has been graded too steep or when the banks are eroding from too great depth and velocity, thus overcharging the supply to such an extent that the branches or distributaries lower down are unable to carry the excess sand. The remedy here is of course to stop the cause, though very often this may go on for years before the evil is recognized and many more may elapse before the remedy can be applied. In many soils an excess grading giving only 1·1 V_0 will cause this trouble (especially if depth is great) raising endless difficulties farther down, and this all points to the necessity of correct grading in the first place, and the danger of tampering with an existing well-established regime especially when we remember that such mistakes cannot be remedied at once, but must wait opportunity, often for years."

(*f*) "**Effect of ponding-up in regulating supplies**".- "Sometimes where a canal bifurcates, one branch may require a heading up of the supply to a greater height than the other, and when each branch is run alternately in rotation, what happens is that there is alternate ponding-up accompanied by sand collection on the bed and then lowering of the level with a consequent scour, carrying all the accumulated sediment into the lower level channel. Thus the lower channel becomes overcharged, and the higher undercharged. This action can only be remedied by so designing all regulators that the offtakes are all similar and at one level of sill with no ponding-up ever necessary. To facilitate this, all double regulators should be built as one work and not some distance apart, as is the case with many of the older works."

(*g*) "**Effect of Kutter's Non value of V_0**".—"When a channel is silting badly and the cause seems inexplicable, after eliminating the possibility of any of the abovenoted elements, reference will no doubt be made to the longitudinal section, and it may seem that the full V_0 obtains, and that the diagrams show there should be no such trouble. This is, however, tacitly assuming that the value of Kutter's N is really 0 0225, whereas it is a not uncommon occurrence that from long neglect the side slopes are in very bad order, and the real value may in such cases be 0 025, or in a very bad case 0·0275, which, of course, would by no means give V_0. For this reason, if the real ratio of V_0 is wanted in any particular case, it will not be enough merely to consult a drawing, actual observation is necessary, and not at one site but at many."

166. Deposit of heavy silt near canal heads.—While the correct grading of canals in accordance with Kennedy's principles

does much to minimize troubles from silting, this grading alone will not in most cases enable the coarser bed-sand transported by a river to be carried through the canals on to the fields. The course of canals is frequently for long distances through friable alluvium, most of it composed of material silt deposited by the river from which the irrigation water is derived, and in such a case a velocity sufficient to transport the heavier river-bed silt would be more than sufficient to erode the margins of the canal cut through such soil.

Much trouble is therefore still experienced from the sanding up of canal heads by silt deposit in the first few miles below the heads of canals.

This should be minimized by design of a canal head and head sluices with a view to prevent the entry of bed silt as far as possible. The methods by which this may be effected will be described in the chapters dealing with design of head works.

The difficulties may also be dealt with by provision of scouring sluices in the canal, some miles below the head, through which, when water is available, some of the heavy silt may be scoured out of the canal back into the river Where effective arrangements of the above nature cannot be provided, it is frequently necessary to resort to periodic excavation and removal of the silt from the head reach of a canal which is a remedy which tends to become more and more expensive the longer it is carried out, owing to gradually increasing lead and lift.

167. Silt deposit in tanks —The statement made in paragraph 150 above of how silt carried by a river is deposited in the form of shoals in a lake, is clearly also applicable to silt deposit in tanks

Where a tank is large in comparison to the annual run-off from the catchment area, all the water-borne silt carried by the run-off water will be deposited in the bed of the tank The heavier silt will be deposited where the feeder streams outfall into the tank, but the fine silt, which takes a long time to settle, will be spread more evenly all over the tank bed, and frequently the thickest deposit of such silt will be below deepest standing water, that is, in the vicinity of the bund A small amount of the silt deposited in the tank may be removed by the water drawn off through outfall sluices, but such action cannot be other than very local

When the run-off of the catchment of a tank is large in comparison to the tank capacity, so that the tank surpluses freely for long periods, it is possible that a considerable proportion of the lightest silt in suspension may be passed off with the surplus water, but the bed silt, and all but the lightest kind of silt in suspension will be deposited in the tank bed

The rate at which a tank which wholly intercepts the run-off of a catchment will lose capacity by deposit of silt, depends very greatly on the proportion which the run-off bears to the capacity of the tank.

For instance, if the annual run-off into a tank of capacity 500 mills. c ft. has silt to water a proportion of 1/1000, and averaged 1,000 mills c.ft. the yearly silt deposit would be about one mill. c ft., and the tank would be half filled with silt in 250 years, but if the tank were only 80 mills. c.ft. capacity it would be nearly half filled up in 40 years.

From the point of view of silting it is desirable to locate a tank so that the area of catchment intercepted is limited to the area which will give, on a bad year, a run-off sufficient to fill and replenish the tank in accordance with its irrigation requirements. Intercepting a run-off disproportionate to the requirements is a defect.

The chains of tanks so common in South India constitute an arrangement likely to result in the unduly rapid silting of the upper tanks in which a disproportionate quantity of silt is deposited.

The deterioration of the tanks of South India by silt deposit is continuous and very serious Many of the tanks have been impounding silt for centuries and have lost very considerable proportions of their original capacities to the great injury of the crops under them No effective remedy for this has been discovered and the problem is one which becomes increasingly serious as time goes on.

168 Regulation of impounding to avoid silt deposition.—Suppose that, in the case instanced above in which the tank half filled with silt in 40 years, a set of powerful deep sluices were provided in the tank bund at the stream crossing through which the run-off could be discharged without any heading up as quickly as it was received, then, for so long as no heading up took place, the silt would be discharged with the water through the sluices, and it would not be deposited in the tank, which might be filled from the low flow of comparatively clear water received after the floods If the nature of the run-off and the demands for water were such as to render this a suitable system of working the tank, its serviceable life could be greatly prolonged by such a method.

With this object in view, this method of impounding is resorted to in the case of the great Assouan Reservoir across the Nile in Egypt. Were it not for this system of regulation of impounding the reservoir would silt up rapidly, as the Nile floods carry much silt, and the total annual flow is many times greater than the reservoir capacity.

169 Scour.—Scour is the process of erosion and removal of matter by the action of running water.

The scouring power of water depends on the velocity of its flow where in contact with the eroded material, and also on the nature of the material

Scour is also greatly affected by the direction of the flow with regard to the surface exposed to erosion and by any sudden changes in direction or velocity of flow The reason for this is, that such changes have the effect of greatly increasing the number and the power of eddies which are the main causes of scour and which give to water its silt transporting properties.

Eddies are produced wherever sudden obstructions are interposed in the way of a current, and if the surfaces on which the current acts are not sufficiently hard to be capable of resisting the extra eroding power due to these eddies, local scours in the vicinity of the obstructions will be formed.

Scour is thus produced by excessive velocity of flow, or by sudden change of velocity or direction of flow producing turbulence

and cross currents, resulting in eddies, abnormal both in number and energy.

Owing to the effects of marginal scour the courses of rivers running through alluvial soil are nearly always very tortuous, and in addition their beds vary greatly in depth, in some places deep pools are scooped out by eddies in floods which are caused, either by sudden bends or obstruction in the course of the stream, or by excessive velocity in the upstream reach suddenly checked and dissipated in cross currents and back flow.

170. Irrigation works and scour effects.—Scours in the vicinity of irrigation works built in rivers, or scours in any parts of artificial canals, are usually harmful, and failures of works are not infrequently due to this cause

Water passing over weirs, or through regulators and sluices, is frequently necessarily subjected to sudden changes in velocities of flow, and also to changes of direction of flow either horizontal or vertical, and the beds and margins of rivers and canals in the vicinity of such works are therefore specially liable to scour.

The changes in local condition of bed and margin produced by scour are in themselves likely to accentuate any sudden and local changes in water currents, and thereby react in producing proportionately more rapid scouring action

171. Nature of damage done to irrigation works by scour—Excessive scour in the vicinity of masonry or other works founded in the bed of a river or canal is likely to affect the stability of the works by the undermining of foundations causing settlements, slips or other failures.

Continuous scour of beds of canals causes lowering the level of water flowing in it, and thereby entails loss of command of land. Scour of margins at any point of a canal tends to produce, by reflex action, scour at successive places on alternate sides of the canal for great distances below the place where the original scour occurred.

172. Providing protection against scour—In the case of masonry works damage by scour should be guarded against by arranging, so far as is practicable, for avoiding sudden changes of velocity and direction of flow, and where these must occur, by providing suitable protection for the beds and margins of the rivers and canals for safe distances above and below the works.

In the case of canals and channels generally, the velocity of flow must be such as will, as far as practicable, carry forward in suspension the silt entering the head of the canal with the irrigation water, but which will not be sufficiently rapid to cause erosion of bed or margins in the absence of sudden changes of direction. Where such changes occur, special protective works must be provided.

It is to be realized that Kennedy's non-silting and non-scouring velocities are applicable to channels in steady regular flow over comparatively smooth beds, and round regular gradual curves, and that local interruption of such flow will result in local silt or scour, which will frequently give rise by reflex action to repetition in successive places for considerable distances below the site of the original disturbance.

173. Eroding power of clear water—As flow at a certain velocity and depth connotes power of transportation of a certain percentage of silt of certain quality, it follows that where a canal, flowing at a velocity which will transport a proportion (p) of silt to water, is supplied at the head with water with a less proportion of silt, there will be a tendency to pick up and transport from the canal bed or margins the balance of silt required to make up this proportion. It is found that this takes place and that silt, deposited in the upper reaches of canal when river waters are heavily charged with silt, is picked up and carried forward when the river water becomes clear, and that while there may be great seasonal difference in the turbidity of water near the head of a system modelled on Kennedy's system, there is little seasonal difference towards the tail end of the channels

Thus a canal which may not scour with silty water may do so with clear water flowing at the same depth and velocity

CHAPTER VI.

HEAD WORKS.

174. Irrigation Head Works.—From an engineering aspect head works may be divided into two classes :—

(*a*) Diversion works.

(*b*) Storage works.

Class (*a*) works are generally associated with direct flow irrigation and class (*b*) with tank irrigation.

There are, however, many storage works which receive supplies diverted into them by means of class (*a*) works and the majority of class (*b*) works serve the combined purposes of storage and diversion.

175 Diversion works —These have their site in the vicinity of the point of off-take of the head of the canal system in the river from which the water supplies are drawn

The works usually consist of.—

(*a*) A weir across the river of sufficient height to maintain a water level above the work suitable for the supply of the canal system, that is, a level which will give 'command' of the area to be irrigated, and which will give sufficient depth of flow in the canal to carry the required supply

Weirs across rivers are in South India frequently termed *anicuts*, this being the Tamil word for a river weir.

(*b*) A canal taking off from one or both of the sides of the river above the weir, and capable of carrying sufficient water for the irrigated area at a level which ensures supply by direct flow

(*c*) A 'head sluice', frequently termed a 'head regulator', by means of which the quantity of water admitted into the canal is limited and controlled.

(*d*) Weir scouring sluices.

(*e*) Flood banks or other protective works sufficient to secure the weir from being outflanked by floods in the river

Of the above, the canal from below the head sluice is part of the distribution system which will be dealt with later under that head.

176 Level of weir crest.—A weir is essentially a rough stone or masonry barrier or wall built across the river, by means of which water level upstream of the work is raised up to the crest, or top level of the weir, before any of it can pass down the river below the work. Modern weirs are frequently fitted with some form of crest shutters which can during floods be laid flat with the crest, or raised clear of water level, and these shutters enable water to be headed up to the shutter crest level without obstructing the river by so large a barrier as would be necessary to secure the same heading-up by a solid weir without shutters. The term 'shutter crest level' signifies the level of the shutter top when standing erect on the crest, or sill, of the weir or regulator.

The crest of the shutters or, where there are no shutters, of the solid weir must be at such a height as to secure adequate command,

that is, to enable a sufficiently high water level to be maintained in the canal to irrigate the required area by flow.

The weir also must be high enough to ensure passing into the canal adequate quantities of water to supply the seasonal requirements of the crops, up to the limit of the full flow of the river.

177. Disadvantages of a solid weir —The erection of a solid weir across a river generally entails serious inconveniences :—

(1) Raising flood levels above the work.

(2) Causing accumulation of silt and rise of river bed and frequently the formation of shoals above the work.

(3) Causing scours of the river bed below the work

(2) and (3) above are a minimum in a river with a rocky bed, and are aggravated in a river with a soft bed over which much bed silt is transported

The greater the proportion of the whole water way which is blocked by a solid barrier, the greater the evils entailed, and it is therefore undesirable to block more of the water way of a river by a solid weir than is necessary to effect the object in view; from this consideration the advantages to be derived from crest shutters become at once apparent

178. Regulator substituted for a weir —Occasionally, in place of a weir, a regulator is built across a river to effect a similar purpose This consists of a number of piers with a suitable flooring between them and with grooves in each pier in which are installed lift shutters of suitable height capable of being raised and lowered.

These shutters are lifted by gear installed on the tops of the piers, or on a bridge platform connecting the heads of the piers. The advantage of a regulator as compared with a weir is, that it effects the purpose of a weir without greatly obstructing the water way.

179 Head sluice.—The head sluice must be capable, with water at crest level of the weir or weir shutters, of passing through it the full supply required for irrigation, and also of shutting out excess river flow from the canal.

In silt bearing rivers it is necessary to arrange the site and design of the work so as to exclude as far as possible river-bed silt from being carried into the canal along with the irrigation water.

The means taken to secure exclusion of bed silt frequently entail considerable increase of cost of the work, and the extent to which these are justifiable must be decided for each case separately on its merits.

With a view to exclude bed silt the sill of the head sluice should be at a level higher than the deep bed of the river, and the sluice shutters should be in tiers, so as to enable the canal to be fed from water drawn from near the surface of the river at times when the water is heavily charged with silt. In operating the shutters, the water should be passed through the sluices at the lowest possible velocity, that is, should be drawn off at the highest level practicable. The total sluiceway provided should be sufficiently large to enable the supply to be passed at moderate velocities through the sluices.

The position of the off-take, and the condition of draw-off, should be arranged so as to avoid as far as possible causing sudden changes in the direction or velocity of flow which, as has been explained in Chapter V, entails the formation of eddies which lift bed silt, and cause it to be carried into the canal.

180. Scouring sluices.—Sluices with sills at about the deep bed level of the river generally form part of the subsidiary works of a weir. The function of these sluices is, to keep open a suitable low water channel to conveniently feed the canal head sluice, and to deepen the river bed in front of the head sluice with a view to the exclusion of bed silt from the canal. The scouring sluices are placed on the extreme flank of the weir above which the canal takes off, and the head sluices is generally placed with its face close up to the margin of the river, so that the scouring sluices will draw directly across the face, and keep the river bed in front of it scoured out to a considerable depth below the sill level, of the head sluice.

181. Divide groyne.—In order to form a definite leading channel to the head sluices and scouring sluices along which the draw of the latter will be concentrated, it is desirable to build a divide wall or groyne running up stream from the junction of the scouring sluices with the weir. This work is termed a 'divide wall', or 'divide groyne', and it should extend a considerable distance above the upstream vent of the head sluice.

182. Plans of typical head works —The head works of the Sirhind canal system at Rupar on the Sutlej (figure 14) show works in relative positions described above. The profiles of the scouring sluices and head sluices are given in figures 56 and 57, pages 177 and 180.

HEAD-WORKS, SIRHIND CANAL.

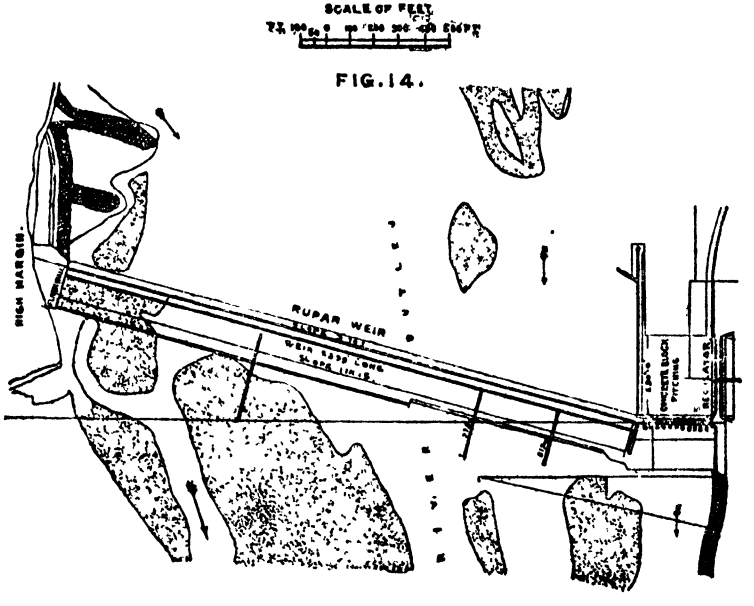


FIG. 14.

Figure 55, page 175, gives a plan showing the relative positions of the Toludur regulator and head sluice, now under construction on the Vellar in South Arcot district.

183. System of regulation of head and scouring sluices.—The aim of regulation is to pass water through the head sluices in quantities required for irrigation, which while containing the normal proportion of silt in suspension carries as little bed sand as possible into the canal.

The *rôle* of the scouring sluices is to keep open through the river bed a deep channel leading to the head sluices, and by occasional scour to keep the bed of this channel where it passes in front of the head sluices at a depth lower than the head sluice sill and so minimize the amount of bed silt passing into the canal.

It has been found in practice that the simultaneous working of head and scouring sluices, although effecting the scour of the river bed in front of the canal sluice, also so stirs up and lifts the heavy silt that much more bed silt passes into the canal when the scouring sluices are open than when they are closed.

It is therefore desirable not to open the scouring sluices when the head sluices are drawing water and, if practicable, the latter should be closed before opening the former for the purpose of scouring the bed of the approach channel. When this channel is deep and when the scouring sluices are closed, a pool having but little velocity is formed opposite the head sluices, from which pool water should be drawn from a high level so as to give the requisite supplies without drawing bed silt.

It is thus desirable to have head sluices with ample ventway so as to allow of the irrigation water being drawn at low velocities from as high a level as possible, and to have scouring sluices which are capable of being kept altogether closed under most conditions of river discharge, but which, when opened, will pass water with great and unobstructed velocity, thus affording the maximum of scouring effect.

The discharge capacity of undersluices should be large in comparison to that of the approach channel so that silt may be readily moved along it, and the sluices should be capable of being operated under all conditions of river flow.

The above system of regulation has been developed of late years in Northern India, and has been highly successful in reducing silt deposit near the heads of canals. The troubles experienced from this cause have been far more serious in Northern India than in Madras

It is frequently impracticable, on account of the requirements of supply, to close canal head sluices at all times when the approach channel requires scouring, and it is then for the officer in charge to decide, whether to open the scouring sluices in spite of the extra silt deposit involved, or to wait to scour until local rain or other conditions permit the closing of head sluices. No hard and fast rule can be made, and in each case, the nature of regulation must be decided with reference to local conditions.

184. Flood banks.—The effect of building a solid weir, blocking a portion of the waterway of a river, is to raise the high flood levels above the work and it is necessary when designing a weir to compute the height of the maximum flood level. If the natural

level of the country to which the bank connexions of the weir on each side of the river are joined are not at such a height above flood level as to prevent any spill passing from the up-stream to the down-stream side of the weir round the flanks of the work, it is necessary to prevent this by flood banks connecting the weir flanks with high ground.

Even where there is no danger of the weir flanks being thus turned, the rise in maximum flood level may cause serious injury by submergence to marginal lands, houses, etc., and protective flood banks may, on this ground, be a necessary adjunct to head works.

A spill round the flank of a weir generally entails serious risk of a deep channel being cut round the flank through which all or a large part of the river flow may be diverted.

This is one of the causes of failure of weirs, and instances of weirs being out-flanked by omission to properly maintain the protective bunds are by no means uncommon. The adequacy and proper maintenance of such banks is a matter to be specially attended to, as breaches in these are likely to entail serious failures, while the upkeep of bank connexions of weirs frequently entails but little work, and in consequence is likely to be overlooked.

185. Sites for diversion works.—The selection of the best site for head works involves considerations of many factors and is frequently a matter of considerable doubt.

If a definite area has been selected for irrigation the head works must be placed, so that full 'command' may be attained by a weir of reasonable height, and so that the combined cost of the construction of the head works, and of cutting the canal from the weir to where irrigation commences, shall be as small as is consistent with the efficiency of the works. Generally the higher the head is placed up the river, the greater the cost of canal but the less the height of weir necessary. The nature of the soil through which the canal head reaches must be cut, and the nature of the foundations of the head works, must be considered; on the one hand it may happen that a most favourable site for head works cannot be adopted, because it entails large quantities of rock cutting in construction of the canal, while on the other a favourable line of canal may have to be abandoned on account of the cost entailed in construction of head works to suit this alignment.

It is rare that a site can be obtained which does not entail several unfavourable conditions, and the best site is that which will give fully effective works at the lowest cost having in view both construction and maintenance charges.

186. Favourable conditions for sites of head works.—The following are favourable conditions of site for head works:—

(1) A straight length of river in which the flow is fairly uniform and generally parallel to the axis of the bed, and which entails little cost in flood banks and training works to keep the river in its course and check marginal erosions.

(2) A reach not very wide or encumbered with shoals; for, not only does a wide reach entail extra length of weir but it is subject to aggravated troubles from bed silt.

(3) Good foundations; rock or clay are especially desirable,

(4) Low cost of cutting the offtake canal from the weir to the point whence it commences to irrigate.

(5) Proximity to materials required for construction, and good communication with the sources of supply.

It is generally impracticable to select a site in which all the above conditions are combined and the selection must be made by carefully weighing the advantages and disadvantages of each possible site.

187. Direction of a weir with reference to river axis.—As a rule a weir should be built at right angles to the axis of the river; an oblique direction, besides entailing a greater length of weir, is likely to develop face scours, i.e, scours parallel and close to the upstream face of the weir.

On rock, boulders, or hard gravel, scours need not be feared and it may be economical to build an oblique weir in order to take advantage of local conditions as regards foundations.

Buckley (Irrigation Works of India) says that a weir inclined towards the offtake channel head is found to be an advantage in rivers with beds of boulders and shingle as with this feature the low water stream is directed to the head of the offtake channel, while in such a river bed danger from face scour is small.

188. The details of design of the component parts of diversion works, viz, of weirs, head and scouring sluices, etc, are dealt with in detail in succeeding chapters

STORAGE WORKS.

189. Storage works as headworks.—When a canal or irrigation system has a head sluice drawing its irrigation supplies directly from a reservoir, then the reservoir itself is the headworks of the canal or system.

An irrigation reservoir generally combines the functions of storage and diversion, the latter being effected by drawing water headed up by the dam from a level suitable for the command of the area to be irrigated under the canal supplied from the reservoir; occasionally however the reservoir is used only as a storage work, to impound or supplement the direct flow of a river, and separate works are constructed to divert the water on to the land to be irrigated.

190. Component works of a reservoir.—The component works constituting an irrigation reservoir are :—

(*a*) A dam which impounds the water

(*b*) One or more outlets, or supply sluices, through which irrigation water is drawn from the storage work.

(*c*) Surplus works to pass off any inflow in excess of what can be safely impounded.

191. Reservoir dams.—The dam is generally either of masonry or earth, but such works have been constructed of steel and also of stone filling, with a water-tight face or hearting, which has been made of earth, concrete, or wood or steel sheeting.

In the case of storage works the impounding of considerable quantities of water makes it necessary to hold up against the upstream face of the dam depths of water generally considerably

greater than in the case of diversion works. The dams of tanks in South India are generally of earth and are termed *bunds* or banks. Masonry dams can only be built when sound rock foundations are available, and are generally restricted to sites where considerable depths of water have to be impounded. A masonry dam may combine the functions of a storage and a surplus work when, in addition to impounding a quantity of water, it is designed to pass surplus over its crest. Such a work is technically a high weir, and may be called an 'overflow-dam', or a 'weir-dam' to distinguish it from a dam over the top of which water is not passed.

Under no circumstances should water be passed over the top of earthen dams; the topping of an earthen dam by the impounded water almost invariably causes the breaching of the bank.

192. Reservoir supply sluices.—The outlets or supply sluices from a tank are generally located in the main dam. In the case of large earthen dams, outlets are occasionally made through deep cuttings or tunnels round the flank of the bund so as to avoid building a sluice through the bank; this, however, is unusual, and it is customary to carry the outlet through the bank. The sluices are necessarily provided with regulating arrangements which give control of the quantity of water drawn off through them.

193. Reservoir surplus works.—These usually take the form of a weir on one or both flanks of the tank. The crest of the work is usually at the full tank level (F.T.L.), and the discharging capacity sufficient to dispose of the computed maximum surplus water without the water rising above a fixed level, which is termed the maximum water level (M.W.L.). As in the case of diversion weirs, reservoir weirs are sometimes provided with some form of crest shutters, with the object of securing a lower M.W.L. without sacrificing the storage capacity, that is, without reducing the F.T.L.

Occasionally surplus sluices are provided either in place of or to supplement the surplus weir. In the case of earthen dams surplus sluices are generally near one flank of the dam, but in masonry dams they are placed in any position convenient having in view the facilities for working the sluices and disposing of the surplus water. In the case of overflow-dams, the dam itself forms the surplus weir.

194. Favourable conditions of site for reservoirs.—The following are favourable conditions of site for reservoirs:—

(1) The supplies of water at the site during the flow season should be fully adequate, but not very greatly in excess of normal requirements.

(2) The capacity required can be secured with a dam of reasonable maximum height and total content.

(3) The nature of the foundations of the dam should be suitable and should not entail very costly foundation works.

(4) The disposition of material required for the work should be convenient and communications to the site good.

(5) The site should give facilities for the economical and safe disposal of surplus water.

(6) Where the tank is to be fed from an artificial canal, the construction of this canal and its headworks should not entail great expenditure.

(7) Economical line of canal should be available for distribution of the stored water.

(8) The bed of the proposed reservoir should be but slightly permeable.

This last condition is specially important when the water feeding the tank carries but a small percentage of plastic silt.

As in the case of the site for diversion works, the advantages and disadvantages of each site in respect to the above must be carefully weighed, and that site which entails least cost in the construction and maintenance charges for safe and effective works should be selected.

195. There are in many irrigation systems large numbers of diversion works and storage works in positions far removed from the heads of the systems and which are not classed as or considered as 'headworks'. Such works are however in effect the headworks of the canals or channels which take off from them, and from an engineering point of view, their treatment should be the same as works of a similar nature which are actually classed as 'headworks'.

The design of storage works, including earthen and masonry dams and subsidiary works, is dealt with in detail in subsequent chapters.

CHAPTER VII.

HEAD WORKS—MASONRY DAMS.

196. Definition of masonry dam and weir.—*A masonary dam* is a wall which upholds on one side of it (the upstream side) a mass of water to a certain limit of level not greater than that of the top of the dam.

A masonry weir is a wall which not only upholds water as does a dam, but is designed with a view to water passing over its top and when this occurs there will be water, with surfaces at different levels, both on its upstream and downstream sides.

Masonry structures over which water passes if of considerable height (say 30 feet and upwards), are generally referred to as 'dams', or more particularly as 'overflow-dams', and the rigid distinction in nomenclature between dams and weirs, such as is made in the above definitions, is not preserved, but for purposes of examination of conditions of stability, the distinction is necessary.

Masonry dams are divided into two classes:—

(1) *Gravity dams* in which the water pressures are resisted by the weight of the dam only.

(2) *Arched dams* which are built in the form of horizontal arches, in which the water pressures are resisted by the reaction of the abutments at each end of the arches

GRAVITY DAMS.

197. Distribution of pressures in a masonry structure.—In the theory of stability which relates to the distribution of pressures along the line of any joint of dams and other masonry structures, it is *assumed* that the change of intensity of pressure along any horizontal joint is uniform, and this implies that the curve of pressure distribution along a joint is a straight line The mean intensity of pressure along a joint of unit width is the total of the whole pressure divided by the length of the joint supporting it, and, under the assumption made, the maximum or minimum intensity of pressure must be at the extremities of the joint Under such conditions it can be proved that:—

if S_1 = mean stress

and S = maximum or minimum intensity of stress.

$$\text{then } S = S_1 \left(1 \pm \frac{6c}{b} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

where b is the length of the joint, and c is the distance from the centre of this length to the centre of pressure, that is, to the point where the line of resultant pressure cuts the joint

198 From equation (9) the relations between mean and maximum intensity of pressure on any joint can readily be ascertained, if the position of the line of resultant pressure is known

Applying this to particular cases:—

(*a*) If the line of resultant pressure passes through one extremity of the middle third of the base—

In this case $c = \frac{1}{6} b$ and the maximum pressure is

$$S_1 \left(1 + \frac{b}{b} \right) = 2\, S_1 = \text{twice the mean pressure,}$$

and the minimum pressure is--

$S_1 \left(1 - \frac{b}{\delta}\right) =$ nil.

(*b*) If the line of resultant pressure were in the extreme position of cutting an end of the base, in this case $c = \frac{b}{2}$ and :--

the maximum pressure is $S_1 \left(1 + \frac{3b}{\delta}\right) = 4\ S_1$

the minimum pressure is $S_1 \left(1 - \frac{3b}{\delta}\right) = -2\ S$

Here the maximum pressure is 4 times the mean and is developed at one end of the base, and a negative pressure, that is, a tension, having an intensity of twice the mean pressure, is developed at the other end of the base.

(*c*) If the line of resultant pressure cuts the base somewhere within the middle third, say, at ¼ of its length from one end of it,

then $c = \frac{l}{12}$

the maximum pressure is $S_1 \left(1 + \frac{b}{2b}\right) = 1\frac{1}{2}\ S_1$

the minimum pressure is $S_1 \left(1 - \frac{b}{2b}\right) = \frac{1}{2}\ S_1$

(*d*) If the resultant pressure cuts the base in the centre then $c = 0$ and the maximum and the minimum stress is equal to S_1 the mean stress

199. Diagram of distribution of pressure in a joint.--The distribution of pressures on the base of a masonry dam resulting from the position of the centre of pressure can be conveniently shown diagramatically.

If AB in figure 15 be the base, trisected at D and E, and bisected at C, draw semicircles AOE and BOD intersecting at O and with radius 1/3 the base.

The points of intersection of the resultant pressures with the base which are shown for purposes of illustration are :—

R_1 Intersects the base at E one extremity of middle third.

R_2 ,, M between centre of base and one extremity of middle third.

R_3 intersects the base at centre C.

R_4 ,, K outside the middle third.

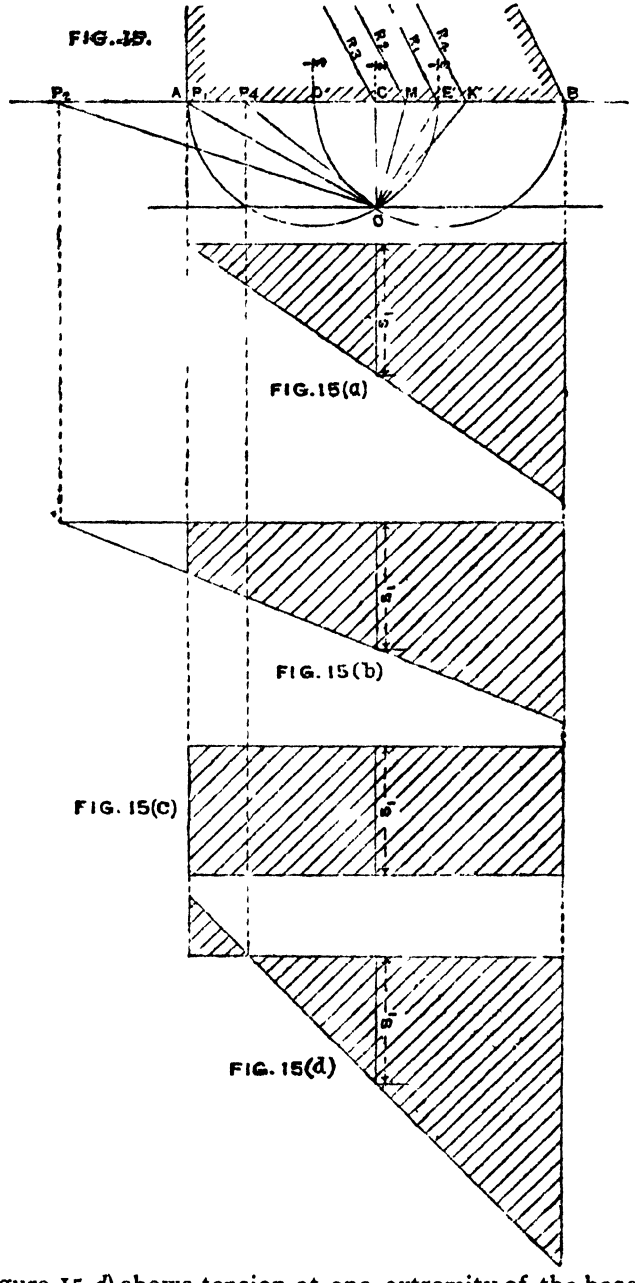
Join the intersections of the resultants and base with O, and from O draw perpendiculars to these lines cutting the base in P_1, P_2, P_4. These points are neutral points, where no pressure, either compressive or tensile, is developed on the base. The distribution of stress along these joints is shown by the shaded areas in the figures 15 (*a*), (*b*), (*c*), (*d*), the stress at any point along the base being proportional to the length of the perpendicular from that point to the boundary line of the shaded area.

In each case the mean stress S_1 is the same and is that at the centre of the base.

R_1 (figure 15-*a*) shows the case typical of the elementary profile described in paragraph 205 below.

R_3 (figure 15-*c*) shows the case where pressure is uniform throughout the base.

R_2 (figure 15-*b*) is intermediate between R_1 and R_3 and exhibits a maximum stress less than in case R_1.



R_4 (figure 15-*d*) shows tension at one extremity of the base; this is a condition which should not be developed in a well designed dam.

200. Deductions in respect to distribution of pressure.—From the above it is clear:—

(*a*) That when the centre of pressure passes through one extremity of the middle third of the base, or other horizontal joint of a dam, the stresses vary uniformly from zero, at that end of the joint most remote from the centre of pressure, to a maximum, at the other end, which is equal to twice the mean stress (figure 15-*a*).

(*b*) That if the centre of pressure is outside the middle third, *tension* is exerted for some distance along the end of the joint most remote from the centre of pressure (figure 15-*d*).

(*c*) That if the centre of pressure falls within the middle third, there is compressive stress throughout the joint, which stress is smallest at the end remote from the centre of pressure and greatest at the other end; the nearer the centre of pressure is to the centre of the joint the nearer the maximum stress is to the mean stress, that is, for the same total pressure, the smaller is the maximum stress. It is by designing so as to induce this condition that the pressures developed in high dams are kept within safe limits

201. Stability conditions of a gravity dam.—A masonry dam must be designed so as to be safe against failure:—

(1) By over-turning.
(2) By rupture from tension.
(3) By sliding.
(4) By crushing.

202. 'The middle third rule'.—As masonry is unsuitable for withstanding any considerable tensional stresses, dams should be designed so that no such stresses are developed. From what has been shown in paragraphs 198 and 199 above, it is clear that, to be assured of this, the centre of pressure must fall *within the middle third* of the base and of every horizontal joint; this constitutes a guiding principle of design of masonry dams, and satisfies conditions (1) and (2) above, and is known as 'the middle third rule'.

The most economical condition under which this *proviso* is fulfilled is when the line of resultant pressure under the condition of greatest stress cuts the base at one extremity of the middle third. Where this occurs condition (4) will be satisfied if the masonry is capable of safely bearing a compressive stress (S) equal to twice the mean stress (S_1); and subject to this the shortest base and series of horizontal joints, and therefore the most economical dam, will be obtained when the centre of pressure, under extreme conditions of stress, passes through the extremity of the middle third of each joint.

This condition must obtain both with the 'reservoir full', when the maximum water thrust (P) is exerted on the upstream face of the dam, and with the 'reservoir empty', when there will be no water thrust and the centre of pressure of any joint will lie in a vertical through the centre of gravity of the part of the dam above that joint.

203. The minimum profile of masonry wall which, when acted on by its own weight only, fulfils the 'middle third' condition, is

a triangle with either the angles at both extremities of the horizontal base acute angles or one of these angles acute and the other a right angle. In the latter case the centre of gravity of the masonry passes through one extremity of the middle third of the base and of each horizontal joint in the structure.

Similarly the minimum profile which fulfils the middle third rule when subjected to the pressure of water with its surface at the top of the dam is a triangle, and it can be shown that the minimum section of a dam of impervious masonry which fulfils this rule is a right angled triangle with the water face vertical.

This proposition holds only so long as the weight of the masonry per cubic foot is not less than twice that of water.

The minimum length of base required for such a profile is computed below :—

Let b be the length of base required.

Let p be the specific gravity of masonry with respect to water considered as unity.

Let R (figure 16) represent the line of resultant pressure which cuts the base (b) at the downstream extremity of the middle third.

Let P be the total water pressure against the face of the dam.

Let W be the weight of the dam resting on the base.

Then referring to figure 16,

$$W = \frac{H\,bp}{2} \text{ and } H = \frac{2\,W}{b\,p}$$

$$P = \tfrac{1}{2}\,H^2$$

$$\frac{P}{W} = \frac{\tfrac{1}{3}\,b}{\tfrac{1}{3}H} = \frac{\tfrac{1}{2}H^2}{\tfrac{1}{2}Hbp}$$

Hence $b^2p = 2$

and $b = \frac{H}{\sqrt{2p}}$ ..... (10)

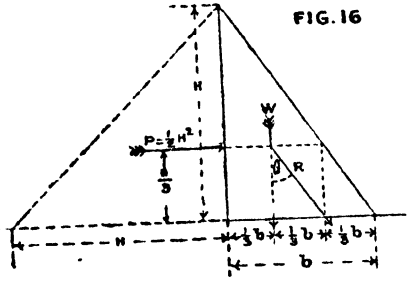


FIG. 16

204. Security against sliding.—As regards liability to sliding at a joint this depends on the friction and cohesion between the surfaces meeting in the joint.

The cohesion between mortar and stone is considerable but very varying, and this item is neglected in computation and remains as a factor of safety against failure at a joint by sliding.

The co-efficient of friction (tan a) between dry masonry surfaces is generally taken between 0·67 and 0·75 where a represents the angle of friction; a thus varies between 34° and 37°.

For safety therefore P, the resultant between the horizontal thrust of water above any joint, and W the weight of the dam *plus* the vertical component of water thrust above the joint, must not be inclined to the vertical at a greater angle than from 34° to 37°.

Condition (3) therefore entails that the angle θ in figure 16 shall not exceed 34 to 37 degrees according to the class of masonry.

Care should be taken in building a dam not to have smooth joints, but to leave at each level an irregular surface with large number of

stones projecting from it so as to increase the resistance of the joint to sliding and if this is done a gravity dam which fulfils the other conditions of stability will always also fulfil this one.

205. 'The elementary profile'.—It has been shown that the correct theoretical economic profile of a masonry dam is a right angled triangle with the upstream face vertical and with a width of base expressed by the equation.

$$b = \frac{H}{\sqrt{p}} \qquad \ldots\ldots\ldots\ldots \qquad (10)$$

where b=the width of base

H=the height of vertical side of triangle and

p (rho)=the specific gravity of the masonry.

This figure is termed by Bligh (Practical Design of Irrigation Works) the 'elementary profile' or 'elementary triangle' and is most useful in affording a rapid means of preliminary examination of the proportions of any masonry dam.

A dam in the form of the elementary profile (figure 17) fulfils the following conditions :—

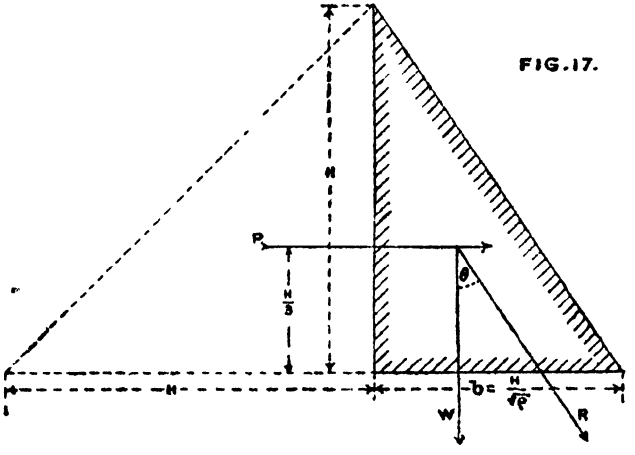


FIG. 17.

The resultant pressure at all times falls within the middle third of the base. When the reservoir is empty the centre of pressure of the weight (W) of the dam passes through the upstream extremity of the middle third, and when the reservoir is full, with water at level of the apex of the triangle, the resultant of the combined forces of the water pressure (P) and the weight of masonry (W passes through the down-stream extremity of the middle third The profile is thus the most economical which fulfils conditions (1) and (2). It also normally falls within the limits of condition (3).

206. Weight and specific gravity of masonry —Masonry dams or weirs are usually built of stone masonry or stone concrete.

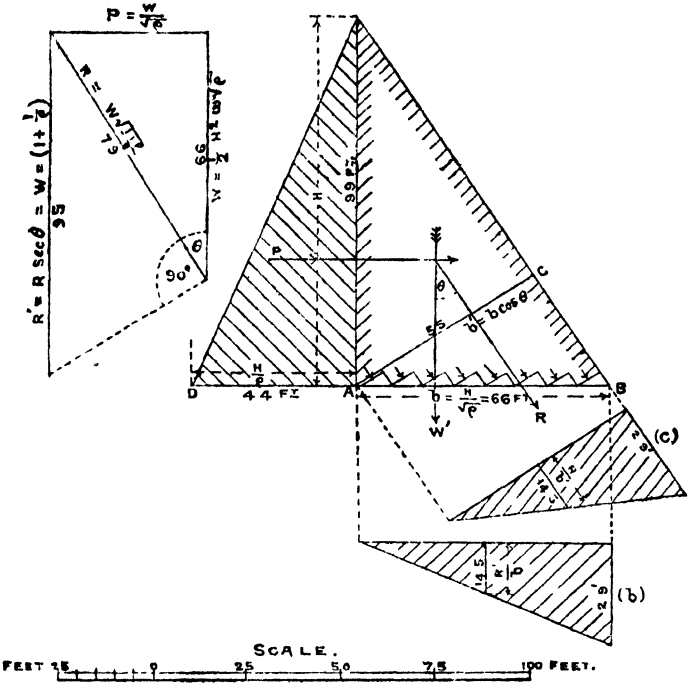
Modern concrete dams are frequently built with large numbers of very heavy stones bedded in the concrete, which has the effect of raising the specific gravity of the concrete.

The specific gravity is the weight of the material compared with that of water considered as unity. If the weight of water per cubic foot is distinguished by the letter 'w'

$w = 62\frac{1}{2}$ lb.

$= \frac{1}{35}$ of a ton approximately.

FIG. 18.



The weight of masonry per cubic foot is therefore $w\rho$ The specific gravity of stone masonry dams may generally be taken at $2\frac{1}{4}$, which is $\frac{1}{16}$ of a ton (140 lb.) per cubic foot. Good brick work may be taken as of specific gravity 2 which is about $\frac{1}{18}$ of a ton ($124\frac{1}{2}$ lb. per cubic foot).

207 Maximum stress due to resultant pressure.—Referring to the elementary profile figure 18, it will be seen that (R) the resultant pressure, reservoir full, is in a direction oblique to the base, and that a mean intensity of pressure greater than that at right angles to the base could be developed along a series of indentations along the base, of which one face of each indentation is at right angles to the direction of R, and the other, which would not expose any surface to pressure, parallel to R.

The aggregate length of base thus exposed to pressure would be equal to the line A C$=b'=b\cos\theta$,

where θ is the angle the resultant R makes with the vertical.

Thus the mean intensity of pressure in a direction at right angles to that of the resultant pressure, reservoir full, is

$$\frac{R}{\cos\theta} = \frac{R}{b'} \quad . \quad . \quad .

If $R'=R \text{ Sec } \theta$ the above equation may be expressed in the form

$$\frac{R}{b'} = \frac{R \text{ Sec } \theta}{b} = \frac{R'}{b} \quad \ldots \quad (12)$$

but $R = W' \text{ Sec } \theta$ and therefore the last equation may also be written as $R' = W' \text{ Sec}^2 \theta$ . . . . . (13)

where W′ is the vertical component of all forces 'reservoir full' that is the weight (W) of the masonry *plus*, the vertical component of water pressure on the face of the dam; when the face is vertical $W = W'$.

208. It is generally convenient, when representing these forces graphically, to plot from the expression in the form of equation (12), that is, instead of plotting the resultant force as R and the base as $b \cos \theta$, to represent the resultant as R Sec θ and the base as b.

Plotted either way the same result will be given as regards mean and maximum pressure which can be developed. Both forms of plotting are shown figures 18 (*b*) and (*c*)

209. Equations giving relations between pressures and stresses.—From the form of the elementary profile (figure 17) we can derive the following relations:—

$$P = \frac{H\,w}{2} = \frac{W}{\sqrt{\rho}}$$

$$W = \tfrac{1}{2} H \times \frac{H}{\sqrt{\rho}} \times w\rho = \frac{H^2 w\sqrt{\rho}}{2} \quad \ldots \quad (14)$$

$$R^2 = P^2 + W^2 = \left(\frac{H^2 w}{2}\right)^2 (1+\rho)$$

$$\therefore R = \frac{H^2 w}{2}\sqrt{1+\rho} = W\sqrt{\frac{1+\rho}{\rho}} \quad \ldots \quad (15)$$

$$R' = R \text{ Sec } \theta = W\left(\frac{1+\rho}{\rho}\right) = W\left(1 + \frac{1}{\rho}\right) \quad (16)$$

$S\tilde{e}$ = maximum pressure on base 'reservoir empty.'

$$= \frac{2W}{b} = \frac{H^2 w\sqrt{\rho}}{H} \times \sqrt{\rho} = H\,w\rho \quad (17)$$

S_f = maximum pressure on base 'reservoir full.'

$$= \frac{2R}{b \cos \theta} = \frac{H^2 w\sqrt{1+1}}{\frac{H}{\sqrt{\rho}}} \times \frac{\sqrt{\rho+1}}{\sqrt{\rho}} = H\,w\left(1+\rho\right). \quad (18)$$

$$\text{Cos } \theta = \frac{W}{R} = \frac{H^2 w\sqrt{\rho}}{H^2 w\sqrt{\rho+1}} = \frac{\sqrt{\rho}}{\sqrt{\rho+1}} \quad \ldots \quad (19)$$

210. Unit for computation of pressures in force diagrams.—It will be seen that, in figure 17, the area of the triangle of base H represents the water pressure in units of cubic foot of water, each of weight w.

Similarly the weight of the masonry is the area of the elementary triangle in units of cubic feet of masonry, each of weight $w\rho$.

If the base of the water pressure triangle were plotted in the same pressure units (viz., 1 cubic foot of masonry) as the profile of the dam represents, then the base of this triangle would be $\frac{H}{\rho}$ instead of H, and the triangle area would represent water pressure in units

each equal to the weight of 1 c.ft. of masonry; the same applies to the profile of the dam. This method of plotting all pressures on a basis of *unity equals* 1 *c.ft. of masonry*, is adopted for convenience in all diagrams, and thus the forces P and W are to each other as the area representing water pressure and area profile of masonry respectively.

In cases where the heights of these are the same the areas vary as the mean widths, and members of force diagrams can in such cases be directly plotted from the widths of the masonry and pressure profiles.

To reduce to tons the results, in force diagram plotted to such units, the lengths must be multiplied by the weight of 1 cubic foot of masonry $= wp = \frac{\rho}{36}$ tons.

In cases where heights of profiles have been eliminated as common, the results must also be multiplied by the eliminated height to arrive at the pressures in tons.

Thus in figure 18 the water pressure at A is plotted as H/ρ and, in the diagram of forces figure 18-*a*, the water pressure P is plotted as equal to AD, and the vertical force W as equal to AB.

Thus lengths taken directly from the profile are applied to the construction of the diagram of forces.

211. Stresses worked out in tons per square foot.—If it is required in this case to give the results in tons, the results in the lineal scale must be multiplied by $\frac{1}{2}$ H wp, the common height of the areas being H.

The reason for the $\frac{1}{2}$ is that the forces have for convenience been represented as the bases of the triangular figures, whereas the areas of the triangles are equal to half the bases multiplied by their heights.

As an example, definite dimensions will now be applied to the profile figure 18.

Let H = 99 feet and let $\rho = 2\frac{1}{4}$, then $\sqrt{\rho} = \frac{3}{2}$
and A B = b = 66 feet; $\frac{1}{2}$H wp = 3 $\frac{3}{8}$ $\frac{3}{2}$

A scale is attached to the figure on this basis. Scaling from the diagram of forces:—

R = 79 feet and 79 × 3 $\frac{3}{8}$ $\frac{3}{2}$ = 244 tons.
R′ = 95 feet and 95 × 3 $\frac{3}{8}$ $\frac{3}{2}$ = 294 tons.

b' = 55 feet, therefore $\frac{R'}{b}$ = $\frac{9\,5}{6\,6}$ = 1·44 also $\frac{R}{b'}$ = $\frac{7\,9}{5\,5}$ = 1·44

1·44 × 3 $\frac{3}{8}$ = 4 45 tons.

In alternative by applying the formulæ from paragraph 209 above.

From formula (15) $R = \frac{H^2 w}{2}\sqrt{\frac{1}{4}}$ = 136 × 1·8 = 245 tons.

From formula (16) $R' = R \sec\theta = R\frac{\sqrt{\rho+1}}{\sqrt{\rho}} = \sqrt{\frac{1}{8}\frac{2}{9}}\,R = 294$ tons.

$$b' = b\cos\theta = \frac{66\times\sqrt{\rho}}{\sqrt{\rho}+1} = \sqrt{\frac{1}{1}\frac{8}{3}}\times 66 = 55 \text{ feet.}$$

In plotting the diagrams of distribution of pressure figure 18-*b* and *c* the mean pressures are on a scale 10 times that of figure 18-*a*.

The maximum stress measured from these diagrams is 2·9 feet, i.e., $2{\cdot}9 \times 3\frac{3}{32} = 9$ tons.

212. Pressures in an elementary profile of specic gravity 2¼.—It will be convenient now to tabulate the pressures in tons, worked out from the equations of paragraph 209, for masonry of a specific gravity of 2¼ which is usually the approximate weight of stone masonry in a dam:—

Number of equation.	Equation.	Reduced to tons with $\rho = 2\frac{1}{4}$.
(14)	Weight of elementary profile $W = \frac{H^2 w \sqrt{\rho}}{2}$	$\frac{H^2}{48}$
(15)	Resultant pressure reservoir full $R = W \sqrt{\frac{1+\rho}{\rho}}$	$1\,2\,W = \frac{H^2}{40}$
(17)	Maximum unit stress reservoir empty $Hw\ \rho$,,	$\frac{H}{16}$
(18)	,, reservoir full $Hw\ (\rho + 1)$	$\frac{H}{11\,1}$

213. Limit of depth of elementary profile.—If the maximum safe unit stress on the masonry is determined, from equation 18, one can compute the limit of height H_λ of a dam designed on the basis of the elementary profile.

If the Greek letter *lambda* (λ) is this stress then

$$\lambda = H\lambda w\ (\rho + 1)$$

$$\therefore H_\lambda \text{ the limit height} = \frac{\lambda}{w\ (\rho + 1)}$$

where λ is tons per square foot
and ρ is 2¼

$$H_\lambda = 11{\cdot}1\lambda \quad . \quad . \quad . \quad . \quad (20)$$

The limit of stress allowable in stone masonry dams depends on the nature of the material used, and varies from about 6 up to 16 tons per square foot, involving a range of height from 66½ to 177½ ft. The tendency of late years especially in America is towards allowing higher stress.

214. 'High' and 'low' dams.—Where it is required to build a gravity dam to hold up water to a depth greater than what the limit of unit stress would permit when determined on the lines indicated by the elementary profile, the part of the dam below the limit depth must be designed so that the centres of pressures, reservoir full and empty, shall fall nearer to the centre of the base than the extremities of the middle third; the nearer these are to the centre of the base the nearer the maximum stress approaches to the mean stress. This is clearly illustrated in paragraph 199 above

Such dams are frequently referred to as 'high' dams in contradistinction to 'low' dams in which the limitations of allowable stress on masonry do not affect the design.

For 'low dams' the basis of design should be that of the 'elementary profile', the aim being throughout each section of the dam, so far as is practicable to keep the lines of resistance, reservoir full or empty, within the middle third of each horizontal joint, and as close as possible to the extremities of outer boundaries of this zone.

215. Crest width of a masonry dam —In actual practice it is of course impossible to rigidly adhere to the elementary profile as a dam must have a certain crest width, while also it requires some 'free-board' above the maximum water level.

The crest width (a) of a dam is fixed from considerations of the special requirements of each case such as the nature of traffic, required to pass over the top of the dam, the nature of the crest shutters, if any, and the space required for them, etc.

Bligh suggests the following empirical relation between H and a which is generally suitable :—

$$a = \sqrt{H} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

216. Free-board of a dam —The 'free-board' of a dam that is, the height of the crest above water level, is generally fixed from consideration of the height of the waves likely to be raised at the dam site,and this depends on the 'fetch,' that is, the longest straight length from the dam face of water surface exposed to wind

Stephenson's rule for finding the height (h feet) of wave with a 'fetch' of F miles is given in the equation

$$h = 2.5 + 1.5\sqrt{F} - \sqrt[4]{F} \qquad \ldots\ldots\ldots \quad (22)$$

Generally the top of a dam would be raised above M.W.L. to a height somewhat greater than h and the free-board would seldom be less than 4 feet

The water level for purposes of stability should be taken either (1) as the top of the wave that is to h feet above M.W.L., or (2) as the level of the crest of the dam

217. Profile for a 'low' dam.—The necessity for having a crest of a certain width and height above water level, alters the position, of the lines of resultant pressure from those of the elementary profile, bringing that with the reservoir full (R) further towards the centre of the base, and thus increasing stability, but throwing that with the reservoir empty (W) a little outside the middle third.

The adjustment required to rectify this is to give a slight projection outside the elementary profile to the lower part of the upstream face

The adjustment required is worked out by Tudsbery and Brightmore (Principles of Water Works Engineering), and the resulting profile is shown in figure 19, in which the parts of the adjusted profile lying outside the elementary triangle are shaded.

The adjustment required to the up-stream face to balance the top thickness is only $\frac{1}{16}$ of the latter and in many cases may be omitted as unnecessary.

When such balancing masonry up-stream of the vertical face the elementary profile is introduced, it is frequently given as slight and continuous batter starting either from the top of the da or from some point below water level.

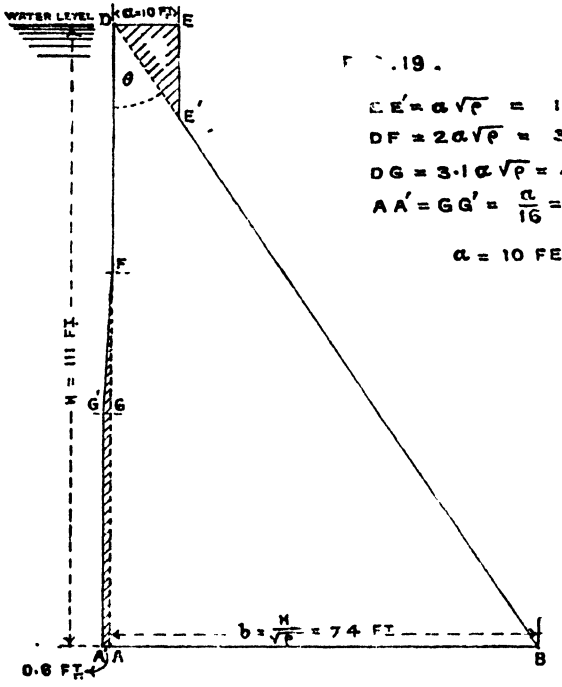
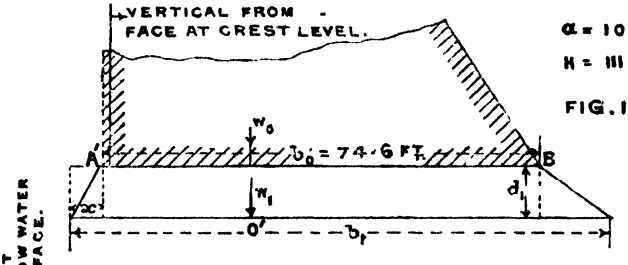


FIG. 19.

CE' = α√P = 15 FT.
DF = 2α√P = 30 FT.
DG = 3·1α√P = 46·5 FT.
AA' = GG' = α/16 = 0·6 FT.

α = 10 FEET.



α = 10 FT.
H = 111 FT.

FIG. 19 (a)

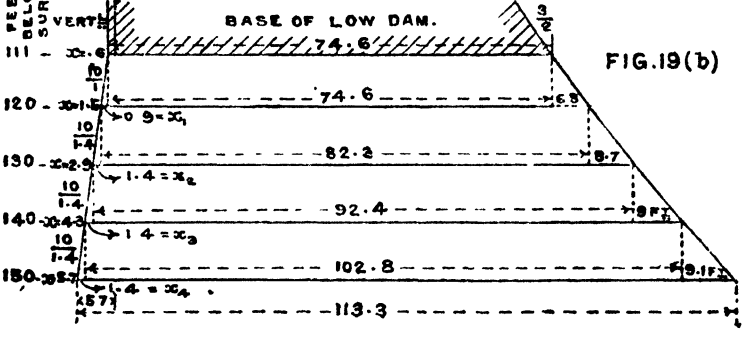


FIG. 19 (b)

218. All the information necessary for the design of the profile of a 'low dam' of the gravity type has now been given.

The profile (figure 19) clearly displays the type and the divergence from the elementary profile required on account of the crest width, and this profile constitutes a suitable design for a low dam where the masonry is of specific gravity $2\frac{1}{4}$, and the limit of safe compressive stress not less than 10 tons a square foot.

If this is the limit the dam as shown is the maximum height which comes in the category of 'low dam'.

In cases of dams which need not be calculated for stability with water as high as the crest level, the base width down-stream of the vertical from the water face at water level would be as before $\frac{H}{\sqrt{p}}$ where H is the depth of water and thus would be slightly less than that in figure 19. In consequence of this the up-stream projection of the face to counterbalance the weight of the top would have to be slightly increased to get the best theoretical profile. This may be done by making a trail profile and testing this for stability by computation or by a diagram in the manner subsequently described for a 'high-dam'

High Masonry Dams

219. Design of a 'high' dam.—The design of a 'high' dam is a much more complex process than that of a 'low' dam, and the distribution of material in the profile while, following the same rules as those necessary for low dams, is governed by limitations in respect to maximum stresses in masonry. In addition to this it is most undesirable to increase the width of base in the lowest layers of a dam profile by flat slopes as this weakens the toe or heel of the dam. It is not desirable to have any slope flatter than 1 to 1 on either face.

Such limitation introduces new factors, and in practice in designing a high dam after arriving by calculation at a theoretical profile conforming with the standard rules, it is usual to fix a final profile which is a practical modification of the former, and to test this profile for stability and stress limit by calculation and stress diagrams In the following paragraphs is described a method of computation for arriving at the theoretical profile, and methods of testing the derived practical profile, both by arithmetical computation and by diagram are also given.

In important cases both methods may be employed as each forms a check on the other.

220. Formulæ applicable to design of 'high' dams.—The following method of designing the theoretical profile of a 'high' dam below the limit depth of water suitable for a 'low' dam is due to Messrs Tudsbery and Brightmore ("Principles of Water Works Engineering"). The design of the dam from water surface to the low dam limit is assumed to be that shown in figure 19, the form of which is also due to the same authors.

The method depends on the following propositions:—

If u is the distance of the centre of pressure from that end of the horizontal base at which the stress is greatest, then, from equation (9)—(paragraph 197 above), the following can be deduced:—

$$u = \frac{b}{3}\left(2 - \frac{s}{2s_1}\right) \quad \ldots\ldots\ldots\ldots \quad (A)$$

Let λ = be the limit stress in masonry in tons per square foot.
H = the depth of water.
w = the weight of water per cubic foot, in tons.
W = the *vertical* force due to, weight of masonry and of any water lying over any sloping section of the up-stream face.
b = the width of the base.

Then from the above equation (A) the following relation can be derived (the proof of this is given in "Water Works Engineering").

$$b = \sqrt{\frac{wH^3}{\lambda}\left(1 + \frac{w^3H^4}{4W^2}\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (B)$$

If W be known even approximately the value of b which will keep maximum stress within the limit λ can be deduced with close accuracy.

Having found the length of b, from equation (B), it has to be ascertained how it is to be placed under the masonry dam, that is, how much (x) of its length is to project up-stream of the vertical from the up-stream edge of the crest of the dam.

Referring to figure 19-*a* if O′ is the point where W acts on this base the length of which is termed b_1 then the conditions require that O′ shall not be closer to the up-stream face than $\frac{b_1}{3}$.

The base b_1 should be so placed that O′ is at a distance $\frac{b_1}{3}$ from its up-stream extremity.

If moments be taken round O′ of the vertical forces on either side of it, the sum of such moments must equal zero.

From this the following equation is derived:—

$$\frac{\rho w d_1}{24}\left\{3b_0^2 - b_1^2 + 6x_1(b_0 + b_1) + 2b_0b_1\right\}$$

$$- \frac{wx_1}{12}(H + H_1)(2b_1 - 3x_1) - W_0\left(\frac{b_1 - b_0}{3} - x_1\right) = 0 \quad . \quad . \quad (C)$$

where

H_1 = depth of water at base $b_1 = H + d_1$

d_1 = vertical depth of new strip added below the base b_0.

x_1 = the projection of the end of the base b_1, above base b_0, that is, up-stream of A^1.

Thus referring to figure 19-*a* and the known base A′B of the low dam, the value of W_0 is the resultant vertical pressure due to W, the weight of masonry *plus* the vertical component of water pressure, and this is known.

To find the length b_1 of the base at a depth d_1 below A^1B (the base of the low dam), the value W_1 can be approximately ascertained by continuing the profile with the same side slopes for a vertical distance d_1 below the base A′B and computing the additional weight.

The method of procedure is demonstrated in the following example:—

221. Example of design of a 'high' dam.—As an example, a case of design is taken from "Water Works Engineering" and suitable actual dimensions assigned to the low dam figure 19, and lower layers will be designed to a, depth of 150 feet below the

crest level, which, it is assumed, is also maximum water level.

The top width $a = 10$ feet.

$\rho = 2\frac{1}{4}$ and limiting pressure $\lambda = 10$ tons.

The limit depth of the low dam is $11{\cdot}1\ \lambda = 111$ feet.

$$\text{Also A'B} = b_0 = \frac{111}{\sqrt{\rho}} + \frac{10}{16} = 74{\cdot}6 \text{ feet.}$$

It is required to design the lower portions of the dam down to 150 feet below water level.

To do this the vertical space from 111 feet to 150 feet will be divided into four horizontal laminæ, numbers 1, 2, 3 and 4 with bases of length b_1, b_2, b_3, b_4 at levels below water,

$H_1 = 120$ feet.
$H_2 = 130$,,
$H_3 = 140$,,
$H_4 = 150$,,

Correspondingly will be denoted the vertical depths d_1, d_2, d_3, d_4, of each lamina, and the projection of each base up-stream of the one above it x_1, x_2, x_3, x_4 and the vertical, component of weight of masonry and water above each base W_1, W_2, W_3, W_4.

First Lamina—

$W_0 = 265$ tons, $H = 111$ feet. $b_0 = 74{\cdot}6$, $x_0 = 0{\cdot}6$ feet. Approximate weight of lamina produced from 111 to 120 feet below water in tons

$$= \frac{9 \times 74{\cdot}6 \left(1 + \frac{120}{111}\right)}{2 \times 16} = 44 \text{ tons.}$$

Then as a first approximation $W_1 = 265 + 44 = 309$ tons

Substituting values in equation (B).

$$b_1 = \sqrt{\frac{(120)^3}{36 \times 10}\left(1 + \frac{(120)^4}{4 \times (309)^2 \times (36)^2}\right)} = 82{\cdot}5 \text{ feet.}$$

Next step is to find x_1 from equation (C).

$$\frac{9}{16 \times 24}\left[3 \times (74{\cdot}6)^2 - (82{\cdot}5)^2 + 6 \times 157{\cdot}1 \times x_1 + 2 \times 74{\cdot}6 \times 82{\cdot}5\right]$$

$$- \frac{x_1}{36 \times 12}\left[231\,(165 - 3x_1) - 265\left(\frac{7{\cdot}9}{3} - x_1\right)\right] = 0$$

$$\therefore 1{\cdot}6\,x_1^2 + 199\,x_1 - 177 = 0$$

$$\therefore x_1 = \frac{-199 + \sqrt{39{,}601 + 1{,}113}}{3{\cdot}2} = \frac{3}{3{\cdot}2} = 0{\cdot}9 \text{ foot.}$$

From the approximate values of b_1 and x_1 thus found, make a second calculation of the weight of the lamina *plus* that of the water on its up-stream face (viz., 44·2 + 2·8), and W_1 will be found to be 265 + 47 = 312 tons.

Substitution of this value in equation (B) gives a revised value $b_1 = 82{\cdot}3$.

The difference is too small to affect the value of x_1 which thus remains 0·9 foot.

The results are therefore—

$W_1 = 312$ tons, $b_1 = 82'3$, $x_1 = 0'9$ foot.

$$x = x_0 + x_1 = 0'6 + 0'9 = 1'5 \text{ feet.}$$

Second Lamina—

$$H_2 = 130 \text{ feet.}$$

The weight of the lamina produced from profile above (firs approximation) *plus* overlying column of water

$$= \frac{10}{16}\left\{82'3 + \frac{10}{9} - \tfrac{1}{2}(82'3 - 74'6)\right\} + \frac{120 + 130}{2 \times 36} \times 1'0 = 58 \text{ tons.}$$

W_2 (first approximation) $= 312 + 58 = 370$ tons.

Applying this to equation (B)

we get $b_2 = \sqrt{\frac{(130)^3}{360}\left(1 + \frac{(130)^4}{5184 \times (370^2)}\right)} = 92'5$ feet.

Applying this value of b_2 to equation (C)

we get $x_2 = 1'4$ feet.

From these first approximations of b_2 and x_2 recompute weight of second lamina *plus* water.

$$W_2 = 312 + 59 = 371 \text{ tons.}$$

Recalculation of equation (B) with this value of W_2 gives

$$b_2 = 92'4 \text{ feet.}$$

The values at base of second lamina are therefore

$W_2 = 371$ tons. $b_2 = 92'4$. $x_2 = 1'4$

$\therefore x = '6 + '9 + 1'4 = 2'9$

Third Lamina—

$W_3 = 371 + 67 = 438$ tons.

$b_3 = 102'8$ feet

$x_3 = 1'4$.

The above are a first approximation.

The assumed values from prolonging the slopes of the trapezoid above are so nearly correct that no second approximation need be worked.

For third lamina the base dimensions are

$W_3 = 438$ tons $b_3 = 102'8$ $x_3 = 1'4$ feet.

$$x = 2'9 + 1'4 = 4'3 \text{ feet.}$$

Fourth Lamina—

Similarly the fourth lamina gives

$W_4 = 513$ tons $b_4 = 113'3$ feet.

$x_4 = 1'4$ feet, $x = 4'3 + 1'4 = 5'7$ feet.

The lower laminæ fulfil the following conditions for 'reservoir full': –

(*a*) Weight of masonry plus water lying on up-stream face 'reservoir full' acts at the up-stream extremity of the centre third.

(*b*) Maximum stress is limited to 10 tons per square foot.

It does not, however, necessarily follow that the maximum stress 'reservoir empty' might not exceed 10 tons and this will be examined by the method of moments in paragraph 226 below.

222. Practical profile based on the theoretical profile.—From the profile of the laminæ as drawn figure 19-*b* it will be seen that the sides of the trapezoids have slightly different slopes, and in drawing a practical section the profile would be simplified so as to have as few changes of slope as economically possible.

For the dam in the aboye example, the practical constructional profile may be as in figure 20.

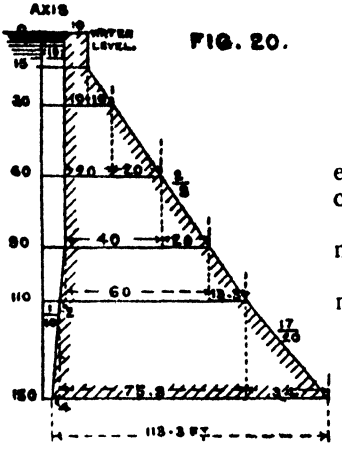


FIG. 20.

The practical profile must now be examined to see if it fulfils the required conditions, viz.—

(1) Lines of resistance within the middle third.

(2) Maximum stress of masonry not to exceed 10 tons per square foot.

223. Essential divisions of a joint of a dam.—Before proceeding further the distribution of the different essential parts of each horizontal joint in a dam will be examined:—

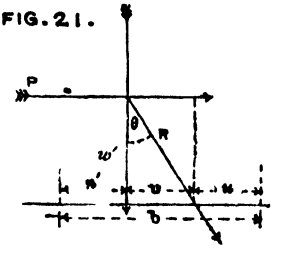


FIG. 21.

If 'b' be the length of the horizontal joint under examination (figure 21) this may be divided into three parts, by the centres of pressure reservoir full, such that $b = n' + v + u$

Let H be the depth of water above the joint.
" M be the moment,
" P be the total horizontal water thrust above the joint, and
" W' be the weight of the dam *plus* weight of water on the slope (if any) of the up-stream face.

Take moments round the intersection of R with the base—

$$\tfrac{1}{3}\,PH = M = W'v$$

$$\therefore v = \frac{M}{W'} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(D)}$$

$$= n' + v + u$$

$$\therefore u = b - (n' + v).$$

224. Formula for stress in the joint of a dam.—Again if the centre of pressure is within the middle third of the base and distant 'u' from the end of maximum stress.

We have from equation (9), paragraph 197 above.

$$S = S_1\left(1 + \frac{6c}{b}\right) \quad \ldots\ldots\ldots\ldots \quad (9)$$

$$\text{also } c = \left(\frac{b}{2} - u\right)$$

$$\text{and } S_1 = \frac{W'}{b}.$$

Substituting in (9) for c and S_1

$$\text{then the maximum stress } S = \frac{W'}{b}\left(1 + \frac{3b - 6u}{b}\right)$$

$$S = \frac{2W'}{b}\left(2 - \frac{3u}{b}\right) \quad \ldots\ldots\ldots \quad (E)$$

This stress is the maximum *vertical* stress on the horizontal base, but if the resultant R is inclined at an angle θ to the vertical the resulting maximum stress S_1 (as already explained in paragraph 207 above) becomes

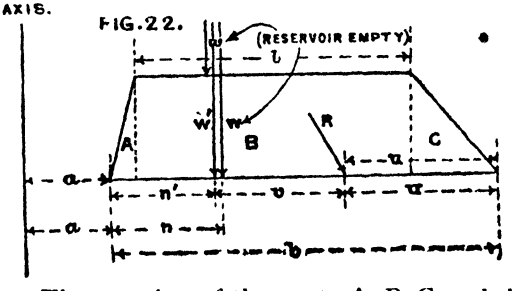
$$S_1 = S \sec\theta = \frac{2W'}{b}\left(2 - \frac{3u}{b}\right)\sec\theta \quad \ldots\ldots \quad (F)$$

225. Method of moments for testing stability.—The test of the practical profile, referred to in paragraph 222 above, will now be undertaken by the method of moments. The method can be applied to weirs, regulators, and other structures, as well as to dams.

Divide the dam up into a suitable number of horizontal laminæ.

Consider one of these as in figure 22.

FIG. 22.



Take moments round a vertical axis any convenient distance from the up-stream face.

It is assumed that the weight (W) and its moment about the axis 'reservoir empty' is known from the working out of the upper *laminæ*

The weights of the parts A, B, C and their individual moments about the axis can be computed.

Reservoir empty.—The sum of the moments of W, A, B, and C round the axis gives the moment of the whole weight (W) on the base b about the same axis and this moment divided by W will give the length $(a + n)$ of the lever arm 'reservoir empty'.

Thus n (reservoir empty) becomes known.

Reservoir full.—If the up-stream face of the dam from the water surface down to the joint is vertical, the weight (W') 'reservoir full' will be the same as 'reservoir empty' (W). If, however, the up-stream face is sloped, separate moments must be taken for the condition 'reservoir full' and the values found of—

n' reservoir full as well as of

n reservoir empty.

From equation (D), paragraph 223 above.

$$\therefore \quad v = \frac{M}{W} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \quad (D)$$

From this u will be found by equation

$$b = n' + v + u$$

$$\therefore u = b - (n' + v) \qquad \ldots \ldots \ldots \ldots \quad (G)$$

226 Example of test by the method of moments.—The practical profile, figure 20 of paragraph 222 above, will now be tested throughout the whole height of the dam for 'middle third' and limit of stress (10 tons per square foot), by taking moments round an axis 10 feet from the vertical up-stream face of the dam. In this example the unit of weight and pressure is taken as the weight of one cubic foot of masonry. The dam will be divided into laminæ above bases at levels below water surface 30, 60, 90, 110, 150 feet.

(1) Section to 30 feet below water surface ($b = 20$ feet).

Reservoir empty and full.	Weights.	Lever arm	Moment.
Rectangular portion above level 30	300	15	4,500
Triangular ,, ,,	75	23·33	1,750
Total	375	.	6,250

$$n = n' = \frac{6250}{375} - 10 = 16{\cdot}66 - 10 = 6{\cdot}66$$

Moment of water thrust $\frac{4}{9} \times \frac{30 \times 30 \times 30}{2 \times 3} = 2{,}000.$

$$v = \frac{M}{W} \quad \frac{2000}{375} = 5{\cdot}33.$$

$\frac{1}{3} b = 6{\cdot}66$

$n = n' = 6{\cdot}66$ ft.

$v = 5{\cdot}33$ ft.

Total 11·99 ft.

$b = 20{\cdot}00$

$u = 8{\cdot}01$ ft.

(2) Section to 60 feet below water ($b = 40$ feet).

Reservoir empty and full	Weights.	Lever arm	Moment.
Portion of dam above level 30	375	...	6,250
Rectangular portion between levels 30 and 60	600	20	12,000
Triangular ,, ,,	300	36·66	10,998
Total ...	1,275	.	29,248

$$u = n' = \frac{29248}{1275} - 10 = 22{\cdot}9 - 10 = 12{\cdot}9.$$

Moment of water thrust $= \frac{2 \times 60 \times 60 \times 60}{27} = 16{,}000$

$$v = \frac{16000}{1275} = 12{\cdot}55$$

$\frac{1}{3} b = 13{\cdot}33$

$u = n' = 12{\cdot}9$

$v = 12{\cdot}55$

$25{\cdot}45$

$b = 40{\cdot}00$

$u = 14{\cdot}55$

(3) Section to 90 feet below water level (b = 60 feet).

Reservoir empty and full.	Weights.	Lever arm.	Moment.
Portion of dam above level 60 ...	1,275	...	29,249
Rectangular portion between level 60 and 90.	1,200	30	36,000
Triangular „ „	300	56·66	16,998
Total ...	2,775	...	82,246

$$n = n' = \frac{82246}{2775} - 10 = 29{\cdot}6 - 10 = 19{\cdot}6$$

Moment of water thrust $= \frac{2 \times 90 \times 90 \times 90}{27} = 54{,}000$

$$v = \frac{54000}{2775} = 19{\cdot}46$$

$\frac{1}{3} b = 20$

$n = n' = 19{\cdot}6$

$v = 19{\cdot}46$

$39{\cdot}06$

$b = 60{\cdot}00$

$u = 20{\cdot}94$

(4) Section to 110 feet below water level (b = 75·3 feet).

Reservoir empty.	Weights.	Lever arm	Moment.
Portion of dam above level 90 ...	2,775	...	82,246
Dam between levels 90 and 110.			
Triangular portion up-stream of vertical face.	20	9 33	186·6
Rectangular portion down-stream.	1,200	40	48,000
Triangular „ „ ...	133	74·44	9,900·5
Total ..	4,128	...	140,333·1

$$n = \frac{140333}{4128} - 8 = 34 - 8 = 26$$

Reservoir full.	Weights.	Lever arm.	Moment.
Reservoir empty as above	4,128	...	140,333
Whole water on slant up-stream face	88	8·9	783
Total ...	4,216	..	141,116

$$n' = \frac{141116}{4216} - 8 = 33{\cdot}5 - 8 = 25{\cdot}5$$

Moment of water thrust $= \frac{2 \times 110 \times 110 \times 110}{27} = 98{,}592$

$$v = \frac{98592}{4216} = 23{\cdot}4$$

$\frac{1}{3} b = 25{\cdot}1$

$n' = 25{\cdot}5$

$v = 23{\cdot}4$

$48{\cdot}9$

$b = 75{\cdot}3$

$u = 26{\cdot}4$

(5) Section to 150 feet below water level (b = 113·3 feet).

Reservoir empty.	Weights.	Lever arm.	Moment.
Portion of dam above level 110 ..	4,128	..	140,331
Triangular portion up-stream of vertical face between levels 110 and 150	80	6·66	

Reservoir empty.	Weights.	Lever arm.	Moment.
Rectangular portion down-stream.	3,012	45·6	139,147
Triangular portion	680	94 6	64,328
Total ...	7,900	...	344,338

$$n = \frac{344338}{7900} - 4 = 43{\cdot}6 - 4 = 39{\cdot}6$$

Reservoir full.	Weights	Lever arm.	Moment.
Weight of dam and water on the slant face above level 110 ..	4,216	...	141,116
Masonry only between levels 110 and 150	3,772	..	204,007
Water on slant face between levels 110 and 150	231	5·95	1,374
Total .	8,219	...	346,497

$$n' = \frac{346497}{8219} - 4 = 42{\cdot}2 - 4 = 38\,2$$

$$\text{Moment of water thrust} = \frac{2 \times 150 \times 150 \times 150}{27} = 250{,}000$$

$$v = \frac{250000}{8219} = 30{\cdot}4.$$

$$\frac{1}{3} b \quad 37{\cdot}7$$
$$n' = 38{\cdot}2$$
$$v = 30{\cdot}4$$
$$68\,6$$
$$b = 113{\cdot}3$$
$$u = 44{\cdot}7$$

Maximum stress on masonry in tons.

Reservoir empty - vide equation (E).

$$\text{Maximum stress} = \frac{2 \times 7900}{16 \times 113{\cdot}3}\left(2 - \frac{3 \times 39{\cdot}6}{113{\cdot}3}\right) = \frac{7900}{906{\cdot}4} \times 0\,95$$

$$- \frac{7505}{906{\cdot}4} = 8{\cdot}3 \text{ tons}$$

Reservoir full.

$$\text{Maximum vertical stress} = \frac{2 \times 8219}{16 \times 113{\cdot}3}\left(2 - \frac{3 \times 44{\cdot}7}{113\,3}\right)$$

$$= \frac{8219}{906{\cdot}4} \times 82 = \frac{6739\,6}{906{\cdot}4} = 7{\cdot}4 \text{ tons.}$$

$$\tan\theta = \frac{\text{Water thrust}}{\text{Weight Reservoir full}} = \frac{150 \times 150 \times 4}{2 \times 9 \times 8219} = 0{\cdot}609$$

$\theta = 31°20'$, $\text{Sec}\,\theta = 1\,17$, $\text{Sec}^2\theta = 1{\cdot}36$

Maximum stress $= W' = 7\,4$, $\text{Sec}^2\theta = 10{\cdot}06$ tons.

The actual vertical pressures reservoir empty (W) and reservoir full (W') are $\frac{7900}{16} = 493$ tons and $\frac{8219}{16} = 513$ tons showing that the weight of water resting on the up-stream sloping face when the reservoir is full is 20 tons.

227. Graphical construction for pressure on up-stream sloping face.--Before making a graphical representation of the

distribution of centres of pressure and stresses in a 'high dam' a method will be described of delineating graphically the water pressures on the up-stream faces of dams which are inclined at an angle to the vertical.

Let the up-stream face ECB of the dam profile ADECB (figure 23) be vertical for a depth EC = H_1 from the water surface which is level with the top of the dam and at a slope for the rest of the total depth (H). Draw BB′ at right angles to CB, the sloping portion of the face and equal to $\frac{H}{\rho}$. Produce BC to meet the water level in F and join B′F. As BB′ represents the water pressure at B the triangle FBB′ represents what the total pressure on the face of the dam would be if the whole face were sloping up to water surface (like FB) and each perpendicular to any point in the face such as CC′ gives the pressure at that point.

Therefore the area of the trapezoid CB′ represents P_2 the total water pressure on CB which is applied in a direction perpendicular to CB through the centre of gravity of the trapezoid.

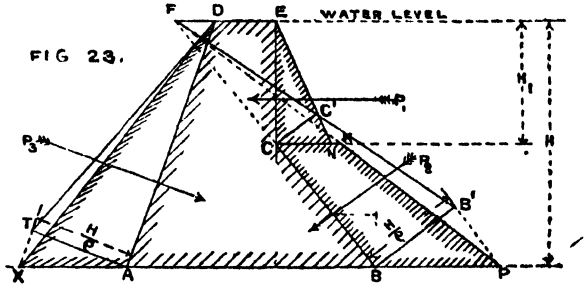


FIG 23.

Through B′ draw B′P parallel to CB and join FP.

Through C draw CK horizontal. The area of the triangle FBP is equal to that of the triangle FBB′ being on the same base and between the same parallels and the area of the trapezoid CKPB bears the same proportion to the triangle FBP as the trapezoid CB′ bears to the triangle FBB′. Therefore the areas of the trapezoids CC′ B′B and CKPB are equal to each other and to the water pressure on CB The latter area equals the mean width in a horizontal direction multiplied by the vertical height.

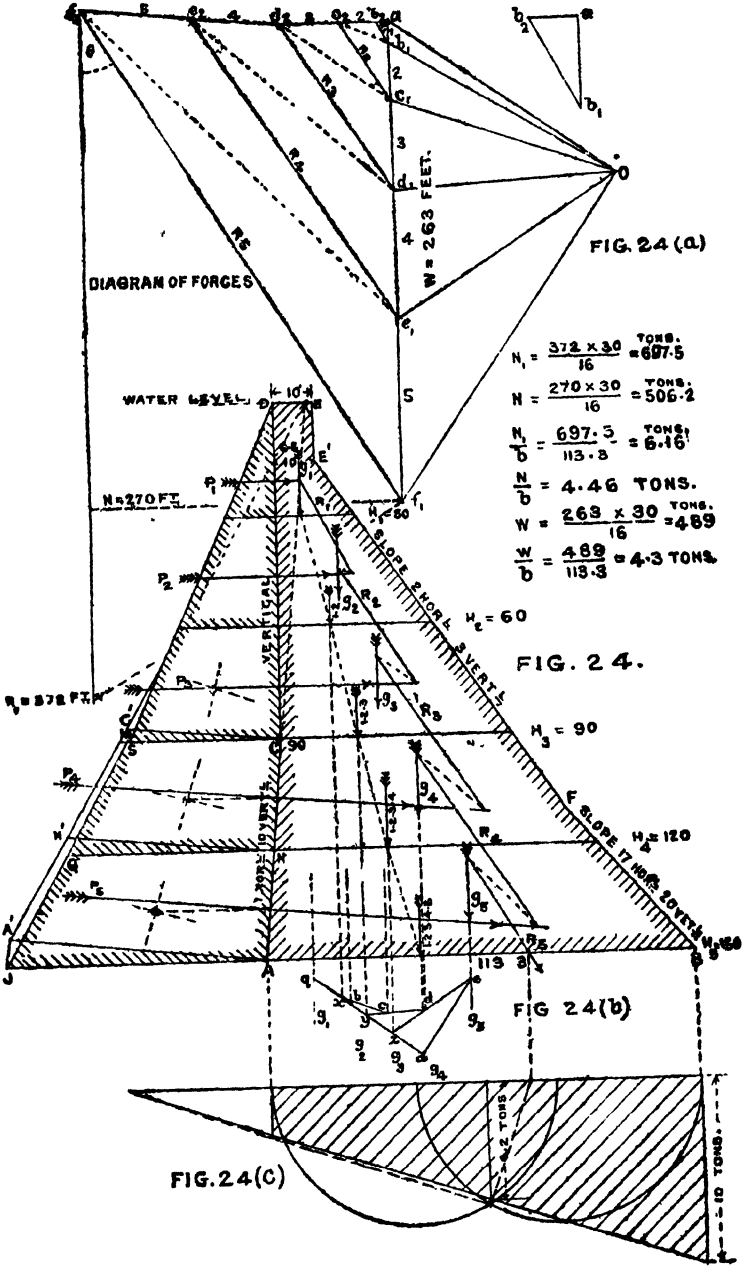
The direction of the pressure is at right angles to the face and the centre of pressure is the point at which a horizontal through the centre of gravity of the trapezium CKPB cuts the face. Make CN equal to $\frac{H_1}{\rho}$, join EN. The pressure (P_1) on the vertical part of the face EC is represented by the area of the triangle ECN and acts horizontally at ⅓ EC above C.

If the water face was AD instead of ECB the water pressure (P_3) would be represented by either of the equal triangles DAX or DAT applied at ⅓ of AD from A and at right angles to the face AD.

228. Stability diagram of a dam.—The practical profile in figure 20 will be taken as a concrete example for examination of

stability and stress intensity of a dam by graphical methods. The procedure is as follows :—

Draw to a suitable scale the dam profile DEE'FBA (figure 24). From A draw AA' at right angles to AC and make AA' equal to



$$N_1 = \frac{378 \times 30}{16} = 697.5 \text{ TONS.}$$

$$N = \frac{270 \times 30}{16} = 506.2 \text{ TONS.}$$

$$\frac{N_1}{b} = \frac{697.5}{113.8} = 6.16 \text{ TONS.}$$

$$\frac{N}{b} = 4.46 \text{ TONS.}$$

$$W = \frac{263 \times 30}{16} = 489 \text{ TONS.}$$

$$\frac{W}{b} = \frac{489}{113.8} = 4.3 \text{ TONS.}$$

$\frac{H}{\rho}=(\frac{4}{9}\times 150) = 66{\cdot}66$ feet Draw A′J parallel to CA. Produce AC to meet the water line at K; join JK and A′K. Draw CM and CC′ parallel respectively to AJ and AA′ to meet JK and A′K in M and C′ respectively. From CM cut off $CS = \frac{90}{\rho} = 40$ and join SD. The area of the triangle DSC gives the water pressure on DC and that of the trapezoid MCAJ the water pressure on CA.

Divide (H) the whole depth of the dam into 5 *laminæ* of equal depth by horizontal base lines distant $\frac{H}{5}= 30$ feet apart. The weights, centres of pressure and stresses on these base lines are to be delineated by graphic statics.

From what has already been stated it will be clear that the pressures P_1, P_2, P_3, P_4, P_5, on the up-stream faces of the laminæ are proportionate to, and may be represented by, the mean horizontal widths of the shaded pressure triangles or trapezoids opposite to them, and will act at right angles to the face of the dam through the centres of pressure, which are the points of intersection of horizontals through the centres of gravity of the shaded pressure areas and the face of the dam.

Similarly the weight W_1, W_2, W_3, W_4, W_5, of each lamina may be represented by the mean horizontal widths of the laminæ of the profile each of height $\frac{H}{5}$, while the pressures due to the weights of each of the laminæ will act vertically through their centres of gravity in the vertical lines g_1, g_2, g_3, g_4, g_5.

A diagram of forces (figure 24-*a*) may now be constructed by drawing a vertical line from any convenient point *a* along which the mean widths ab_1 b_1c_1, c_1d_1, d_1e_1, e_1f_1, of successive *laminæ* of the dam are set off. For convenience the scale is half that of dam profile.

Again from the point *a* in a horizontal direction set off the mean widths ab_2, b_2c_2, c_2d_2, of the areas of figures representing P_1, P_2, and P_3. From the extremity (d_2) of this line, draw a line d_2f_2 parallel to the direction of P_4 and P_5 and set off along it d_2 e_2 and e_2f_2 the mean widths of the areas representing these water pressures.

Draw R_1, R_2, R_3, R_4, R_5 which represent the resultant pressures of the masonry weights and water pressures of each lamina.

Draw the four dotted cross lines b_1 c_2, c_1 d_2, d_1 e_2, e_1 f_2, joining the lower end of one resultant with the upper end of the next.

Take any convenient point o as a pole and draw the converging rays from o to each of the divisions of the vertical line a f_1.

Take any convenient point '*q*,' figure 24-*b*, in the vertical g_1 which passes through the centre of gravity of the uppermost lamina of the dam and produce the verticals g_2, g_3, g_4, which pass through the centres of gravity of the other laminæ.

(1) Through '*q*' draw *qa* and *qb* parallel to the rays o*a* ob_1, the latter cutting g_2 in *b*.

(2) Draw *bc* parallel to the ray o c_1 cutting *qa* in *x*.
(3) „ *cd* „ „ o d_1 „ *qa* in *y*.
(4) „ *de* „ „ o e_1 „ *qa* in *z*.
(5) „ *ea* „ „ o f_1 „ *qa* in *a*.

Then a vertical through *x* passes through the centre of gravity of W_1 and W_2.

A vertical through *y* passes through the centre of gravity of W_1, W_2 and W_3.

A vertical through z passes through the centre of gravity of W_1, W_2, W_3 and W_4.

A vertical through a passes through the centre of gravity of W_1, W_2, W_3, W_4 and W_5.

The intersections of these verticals with the bases of the laminæ give the intersections on these bases of the resultant pressures 'reservoir empty.'

The lines of resultant pressure 'reservoir full' R_1, R_2, R_3, etc., are drawn in on the dam profile parallel to the like line in the diagram of forces, R_1 is drawn from the intersection of P_1 with g_1 and is produced to intersect P_2

From this intersection draw a line parallel to the dotted line b_1 on the diagram of forces being the resultant of P_1, P_2, with W_1. From where this intersects g_2 draw R_2 on profile parallel to R_2 on the diagram of forces.

Similarly in succession R_3, R_4, R_5, are drawn on the profile and where these points cut the base lines of the laminæ are points of centre of pressure 'reservoir full'

The curve of pressure 'reservoir full' is drawn by joining up the points where R_1, R_2, etc, cut the bases of the corresponding laminæ while the curve of pressure 'reservoir empty' is obtained by joining up the points where the vertical through the centre of gravity of all the masonry above the base of each lamina cuts that base.

The resultant total stress R_5 is acting on a horizontal base. If the stress on a plane at right angles to this is required, this may be found from the intersection at N_1 in the diagram of forces of a vertical line from the upper extremity of R_5 and a line at right angles to R_5 from the other extremity. This gives the value of $N_1 = R_5 \operatorname{Sec} \theta$.

To reduce N_1 to tons the length in feet has to be multiplied by the common height factor $\frac{H}{5} = 30$ ft and by $w\rho = \frac{I}{16}$.

Scaling from the diagram $N_1 = 372$ ft.

$$N_1 = 372 \times 30 \times \frac{I}{16} = 697{\cdot}5 \text{ tons.}$$

$$\text{Mean pressure } \frac{697{\cdot}5}{113{\cdot}3} = 6{\cdot}2.$$

A diagram, figure 24-c, below the base of the dam profile is constructed in the manner described in paragraph 199 and figure 15.

Setting off the mean pressure of 6·2 tons and constructing the diagram the maximum pressure resulting is 10 tons 'reservoir full.'

229. Analysis of results of the above example.—Analysing the results, it will be observed that the curve of resultant pressure reservoir empty falls slightly outside the middle third from 30 feet to about 100 feet below water level. This was to be expected from the fact that the batter of the water face only commences at a depth of 90 feet below water level.

The tensional pressures developed would however be so small that the profile need not on this account be rejected.

In all other respects the profile is suitable and the maximum pressure just reaches the limit 10 tons which is the basis of the design.

It will be seen that the graphical results correspond as closely as can be required to those given by arithmetical computation; this method of examining the distribution of stresses has the advantage of more clearly depicting the forces than the more exact arithmetic method, while at the same time involving less labour.

230. Another graphic method.—A simpler method of applying the force diagram to the dam profile is adopted in figure 25, which gives a profile of the *Wigwam Dam* (*United States*).

The dam was designed in 1893 for a water depth of 90 feet, but was only built to a height of 75 feet, the proposal being to subsequently raise it when more water is required.

The digram is drawn for the ultimate height with a top width of 12 feet and water level reaching the top of the dam. It will be seen, by reference to figure 25, that the dam has been divided for purposes of examination into 3 laminæ and in the method here adopted instead of computing the pressure on each lamina separately as in the previous case, this has only been done for the top lamina giving a resultant R_1. Then W_2, the sum of the weights of Nos. 1 and 2 laminæ acting vertically through their combined centre of gravity, is taken, the line of action being ascertained by graphical construction as in the last case,

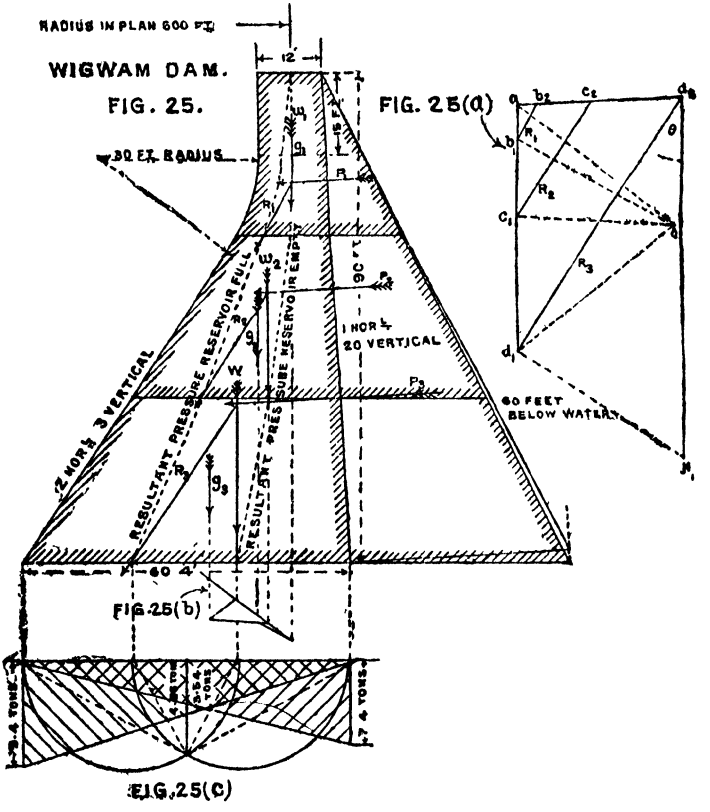


figure 25-*b*. The intersection of W_2 with P_2, the line of resultant water pressure on the faces of the two upper laminæ, gives the point from which R_2, the resultant, is drawn parallel to R_2 in the force diagram. Again P_3 is the total water pressure on the face of the dam acting at ⅓ its height and the intersection of this with the vertical W through the centre of gravity of the whole gives the point through which the resultant R_3 should be drawn

As before the pressure N_1, which is $R_3 \operatorname{Sec} \theta$ may be found by construction and reduced to pressure in tons by multiplying by $\frac{H}{3} = 30$ feet and by $wp = \frac{1}{16}$.

The line d_2 N_1 scales 130 feet. Therefore $N_1 = 130 \times 30 \times \frac{1}{16} = 245$ tons. The mean stress on the base at maximum loading is therefore $\frac{245}{60{\cdot}4} = 4{\cdot}56$ tons. The maximum stress 'reservoir full' is found graphically as before (figure 25-*c*) and equals 9·4 tons.

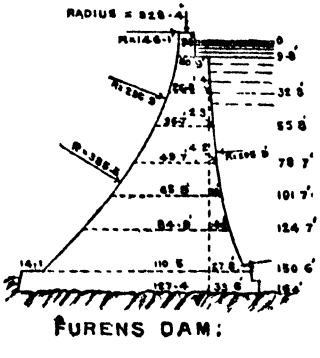
The pressure on the base 'reservoir empty' in the force diagram is $ad = 99$ feet and $99 \times 30 \times \frac{1}{16} = 185{\cdot}6$ tons.

Mean stress on base $\frac{185{\cdot}6}{60{\cdot}4} = 3{\cdot}54$ tons and the maximum stress reservoir empty is found graphically and is 7·4 tons (figure 25-*c*).

231. Analysing the results it will be seen that if the water level is raised to a height of 90 feet there will be tension on the upper part from about 15 to 45 feet depth. Below this level the dam is suitable as a gravity dam.

It should, however, be mentioned that the dam, although designed as a gravity dam, is curved in plan to a radius of 600 feet and the designer possibly relied on the support which arch action will give to the upper part of the dam if the water level is raised to the top of the masonry. The nature of this support will be described in dealing with arched dams. It appears however more probable that surplusing arrangements will make it impossible for the water to ever rise to the top of the dam and the maximum water level is 5 or 6 feet below the top.

FIG. 26.



FURENS DAM.

The method employed in this diagram is superior to the previous one in that each resultant is independently drawn for the whole mass above the base of each lamina, so that errors are not cumulative, and each result is independent of the others

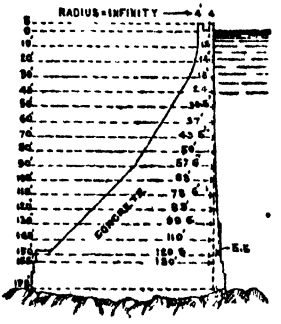
The method is however only applicable when the water face is one slope throughout.

232. Examples of existing high dams.—A few profiles of gravity dams which have been erected are now given.

(*a*) *The Furens dam, France* (*figure* 26). Built 1862—66. This is the first dam built conforming to modern ideas as regards proportions of profile.

It was designed by M. Delocre and is built of rubble masonry.

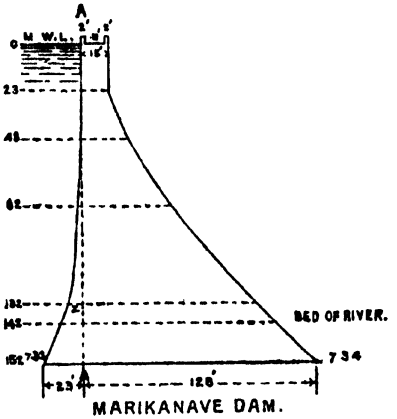
FIG. 27.



PERIYAR DAM.

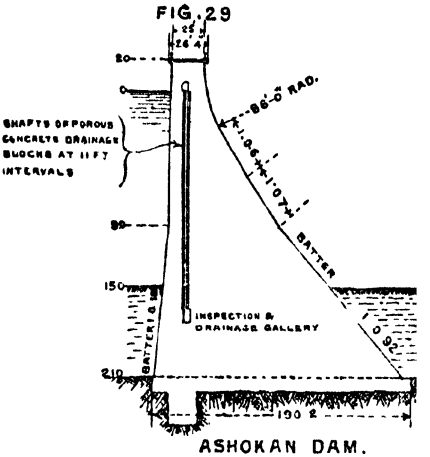
(*b*) *The Periyar dam, Madras* (*figure* 27). Built in 1889—96 of stone concrete faced with rubble masonry. The mortar used was made of kunker lime, surki, and sand.

FIG 28



MARIKANAVE DAM.

FIG. 29



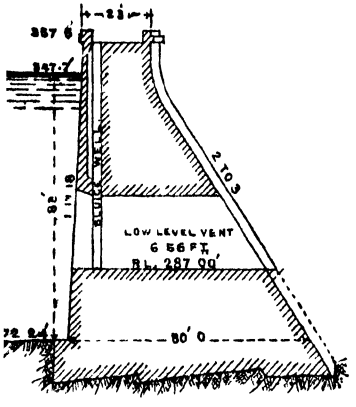
ASHOKAN DAM.

(*c*) *The Marikanave dam, Mysore* (*figure* 28). Completed in 1904 of rubble masonry. The mortar was kunker lime and surki; no sand was used.

(*d*) *The Ashokan or Olive Bridge dam, New York* (*figure* 29). Completed in 1912, Cyclopean Masonry faced with concrete blocks. The mortar was Portland cement and sand. A special feature of this dam is the introduction of drainage shafts, and an inspection gallery which is also a drainage gallery with outfalls at intervals leading to the down-stream face. This provides for the escape of any water percolating through the up-stream face and is a feature coming into use in modern dams.

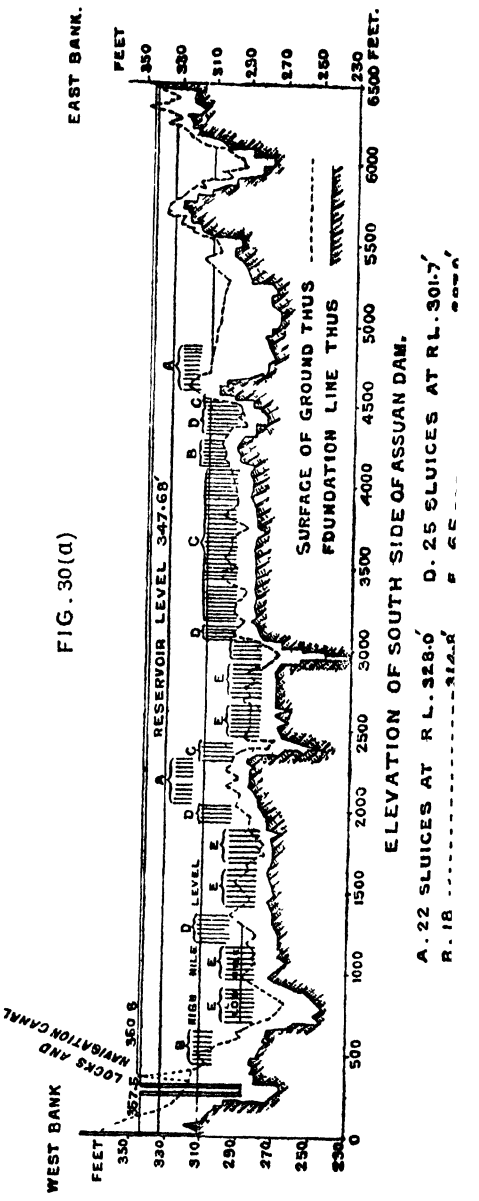
233. The Assuan dam.—The Assuan dam across the Nile (figures 30 and 30-*a*) was completed in 1903.

FIG. 30



ASSUAN DAM.

FIG. 30(a)



The dam has a very massive profile which was adopted because of the large number of sluice-ways traversing the body of the dam through which the whole flood discharge of the Nile has to be passed. This is the first reservoir dam which provides for passing

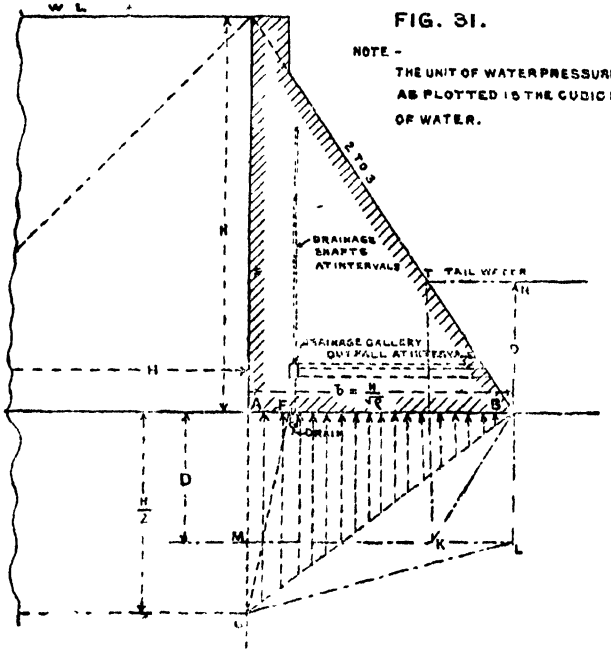
the whole flood discharge of a large river through sluices in the dam but the method is likely to be widely followed in the future.

The method of regulation at Assuan is to pass through the sluice ways, without impounding, the freshes of the river during the early part of the season when these are most heavily charged with silt and thus prevent the bulk of such silt being deposited in the reservoir bed. Impounding is resorted to later in the season when the river water carries only small quantities of silt.

234. 'Uplift' and 'flotation'.— If a masonry dam is built of impervious masonry on a foundation of impervious rock devoid of cracks and fissures, there is no hydraulic pressure developed under the base, or in the joints, tending to lift the masonry.

In building large masonry dams great precautions are taken to secure impermeability of the foundation and of the water face of the work, but the attainment of this aim is a matter of difficulty and uncertainty, and it is now generally recognized that, in spite of all precautions, some percolation generally takes place, and the trend of modern design is to make provision against any great development of uplift pressure under the foundation or in horizontal joints or cracks in a dam, by draining the down-stream portion of the base and body of the dam.

The uplift on any joint is the sum of the upward pressures due to the hydraulic gradient between the up and down stream end of



the joint; this has been already explained in paragraphs 123--125, pages 52, 53 above, in connexion with the uplift on an impervious floor overlying a pervious foundation soil.

Referring to figure 31 where AB is the base of length (b) of a masonry dam, with a depth (H) on the up-stream face, and assuming that the foundation is not fully watertight and that half the up-stream head (H) is absorbed as 'head of entry', then the uplift head is ½ H at A, and it is *nil* at B, where all the head is dissipated. The total uplift on the base is therefore $\frac{1}{2} \times \frac{H}{2} \times b = \frac{Hb}{4}$, and the centre of this pressure acts on the base at $\frac{b}{3}$ from its up-stream end (vide triangle AJB, figure 31). If there was no loss of head on entry, the uplift pressure at A would be H, and the whole uplift on the base ½ Hb, and this is called the 'full uplift' due to the head water level.

If drains are introduced in the masonry (as shown in figure 31) giving a free outlet at F to percolation, from a width of say $\frac{b}{6}$ from the up-stream face, and if the base of the dam is connected with such a system of drains, then the uplift head on the base is dissipated from the point F, and the width of base on which uplift acts is reduced from b to ⅙ b, and the uplift will become ⅓ Hb (the triangle AJF).

The provision of body and base drainage is a feature in many modern gravity dams, of which an instance is given in figure 29 above, and, although the losses by leakage are thereby likely to be considerably increased, uplift is to a great extent eliminated.

Other precautions against uplift are either, (1) to make the foundation and masonry impermeable or, (2) to make provision for the force of uplift in designing the profile of the dam. In many cases the former has been attempted, but generally without fully successful realization. The latter is occasionally allowed and is an absolute safeguard if 'full uplift' is provided for.

In the elementary triangle full uplift involves an increase of base from $\frac{H}{\sqrt{\rho}}$ to $\frac{H}{\sqrt{\rho - 1}}$ which, with a specific gravity of 2¼, is an increase of about one-third the quantity of masonry.

Such an increase would involve very great additional cost, and careful staunching or cutting out of cracks and fissures in the foundation, and watertight masonry on the up-stream face, combined with drainage is generally resorted to.

From the point of view of uplift it is a distinct advantage in a dam that the down-stream portions of the masonry profile should be somewhat permeable, if strength is not thereby impaired, and the up-stream face for 6 feet or 8 feet in width should be of the most impermeable masonry that can be built; in large dams it is usual to employ a richer and more impervious mortar for a few feet from the water face than for the remainder of the profile.

235. It is here convenient when discussing uplift to consider the effect of a certain depth of tail water lying against the down-stream face of the dam. This seldom occurs in dams, unless, as in the case of the Assuan dam, there are large sluices in the body wall, but it nearly always does in weirs.

It is clear that, under such conditions, drainage cannot be effected below the level of the tail water, and that either head or tail

water, or both, may exert uplift. If the down-stream bulk of the profile or foundation were permeable and the up-stream watertight, uplift due to the depth of the tail water only would be developed.

If the dam foundation is slightly permeable throughout, and tail water is standing against the dam as represented in the figure 31 by a chain dotted line, the uplift will be considerably greater than when there was no tail water, there being no drainage below this level, for 'full uplift' will be developed up to level of tail water. The area of four sided figure AJLB represents the uplift on the base AB, the assumption being as before that a head $\frac{1}{4}H$ is lost on entry. The full uplift due to tail water level is represented by the area AMLB, the unit being one cubic foot of water; of this total the triangle KBL is neutralized by the reverse pressure of the equal triangle TBN, representing the vertical component of the back pressure on the dam.

Thus the net uplift due to the tail water level is the figure AMKB, which is equal to AETB, the volume of masonry below the tail water level. If therefore 'flotation' for this volume be allowed it is equivalent to full uplift due to the tail water level.

As there is absolutely no drainage below the level of tail water full uplift due to this level will be gradually developed in this portion of a dam wherever the foundation and the masonry are not practically watertight.

Unless really impervious masonry and foundation can be secured, some uplift for all masonry below the tail water level should be allowed for in design, and it will generally suffice to allow for the full flotation due to any tail water level which is maintained for a considerable consecutive period. In most cases where this is done there will be no necessity to make any further allowance for the uplift due to the head water level, but in the case of a considerable difference of level, drainage of the masonry profile should also be provided for.

The term 'flotation' is frequently used as synonymous with 'uplift' It implies the reduction of the weight of the immersed material per cubic foot by the weight of a cubic foot of water. If the specific gravity of the material is ρ, the effective weight per cubic foot of masonry subject to flotation, is $(\rho - 1)$.

In figure 31 the full uplift due to the tail water level is represented by the area AMLB whereas 'flotation' is represented by the area ETBA which equals AMKB. The balance of the uplift (the triangle KBL) is counterbalanced by the triangle TNB, the vertical component of the water pressure on the face of the dam

It will be realized from this that *where 'flotation' is taken to represent uplift on a dam or other structure*, the verticle component of water pressure on any face of the structure is being used to counterbalance uplift and *only the horizontal components of water thrusts remain to be balanced.*

ARCHED MASONRY DAMS.

236. Arched masonry dams.—In suitable sites masonry dams are frequently built arched in plan, the abutments of the horizontal arch which the dam constitutes being the rock forming the sides

of the gorge which is spanned by the structure. Such structures are designed to sustain by arch action the water pressures brought on them, the pressure being transmitted to the abutments through each horizontal arch ring.

The lowest lamina of such a dam rests on the natural rock foundation and its movement as an elastic masonry arch must be restrained by adhesion to the immoveable foundation and it cannot act as a true arch but must be subjected to oblique strains. In spite of this, experience has shown that a dam computed as an arch taking all water pressure by arch thrust is a stable structure, and, this being so, the fact that near the base part of the pressure is resisted by transverse stress may be neglected.

The weight of the masonry is distributed over the base as in a gravity dam and the dam must be designed so that the distribution of this weight, with the 'reservoir empty', does not cause excessive compressive stress on the masonry.

237. Stress in arches under water pressure.—The pressure of water acts normally to the surface in contact and therefore the pressure on the extrados of an arch dam is radial to the arch, and the line of pressure through the arch corresponds with its curvature.

The whole pressure transmitted through a horizontal arch ring exposed to water pressure over the extrados, which is the water face, is $R \times P$, where P is the water pressure per square foot and R the radius to the centre of the arch ring

R is, in practice, frequently taken to the extrados (the water face of the arch ring) which is an inaccuracy on the side of safety.

238. Formulæ giving stress in arched dams.—Let S and S_1 be the maximum and mean stresses in a horizontal arch ring, then from the above are derived the simple equations :—

$$S_1 = \frac{R\,H\,w}{b} \qquad . \qquad . \qquad . \qquad (23)$$

$$b = \frac{R\,H\,w}{S_1} \qquad . \qquad \ldots \qquad . \qquad (24)$$

where H is the depth of water, w the weight of water per unit, and b the width of the lamina or arch ring; where b is small in comparison to R, the average stress S_1 differs but little from the maximum S and $S_1 = S$.

Where this is not so, if R and R_1 be the radii of the up and down stream faces of the arch ring, the relation between mean and maximum stress is given by the equation

$$S = S_1 \times \frac{2R}{R+R_1} = \frac{R\,H\,w}{b} \times \frac{2R}{R+R_1}$$

and $R_1 = R - b$

herefore if the limit stress is λ then :

$$\lambda - S = \frac{2\,H\,w\,R^2}{2\,b\,R - b^2} \cdot$$

$$\lambda = S = \frac{2\,H\,w}{\frac{b}{R}\left(2 - \frac{b}{R}\right)} \qquad . \qquad . \qquad . \qquad . \qquad . \qquad . \qquad . \qquad . \qquad (25)$$

also

$$b = R\left(1 - \sqrt{1 - \frac{2\,H\,w}{\lambda}}\right) \qquad . \qquad . \qquad . \qquad . \qquad . \qquad . \qquad . \qquad (26)$$

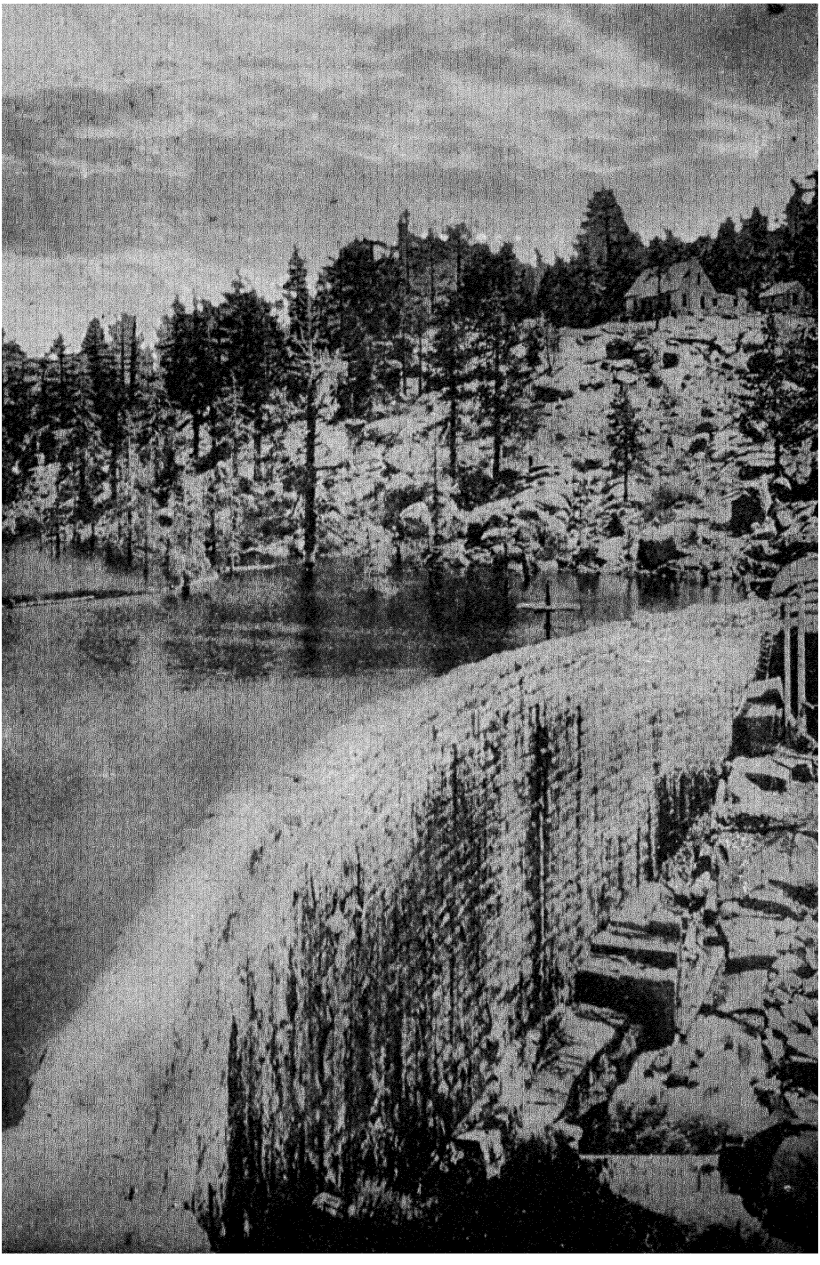


Plate I.—The Bear Valley Dam.

239. Theoretic profile, a triangle.—As in the case of the gravity dam the theoretic economic profile of an arched dam would be a triangle with the apex at water level. Also the upstream face should preferably be vertical unless the distribution of pressures on the base 'reservoir empty' renders it necessary to give this a batter in order to bring the maximum stress within a suitable limit.

240. Crest width of arched dams.—Practically the crest of the arched dam must be given a certain thickness determined from general or local considerations. This would seldom be less than $3\frac{1}{2}$ feet but is generally considerably less than the top width given to a gravity dam.

Bligh proposes a top width (a)

$$a = \frac{\sqrt{H}}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

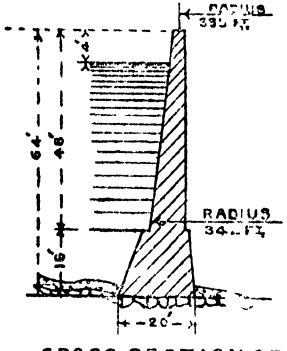
being half the width he suggests for a gravity dam.

241. Maximum stresses allowed in arched dams.—It is usual to allow higher stresses for an arched dam than for a gravity dam and this has been justified by successful practice.

The weight of the masonry of the arch although not taken into consideration in calculating stability of the dam does assist towards this and tends to reduce the computed arch pressures, and for this reason it is desirable to keep the water face either vertical or with as small a batter as conditions permit of.

242. The Bear Valley dam.—The most remarkable example of such a structure is the Bear Valley Dam (figure 32) constructed in California in 1885. Its top length is 300 feet, top thickness $2\frac{1}{2}$ to 3 feet and thickness 48 feet below the crest is $8\frac{1}{2}$ feet. At that level it rests on a curved base 13 feet wide but of a slightly different radius so that the offsets on either side of the $8\frac{1}{2}$ feet lamina are varying.

FIG. 32



CROSS-SECTION OF BEAR VALLEY DAM.

The full reservoir level is 4 feet below the crest, or 44 feet above the base offsets, while the radius to the water face at this level is 342 feet.

$$\text{From this } S = \frac{RHw}{b} = \frac{342 \times 44 \times 2}{17 \times 36} = 49 \text{ tons.}$$

The dam is constructed of uncut granite laid coursed in cement mortar with a hearting of rough rubble. The work is regarded as one of the engineering wonders of the world and, while it cannot be recommended for imitation, its success justifies the greater pressures usually allowed in arched, as compared with gravity dams.

It may here be remarked that Parker computes a reduction of arch stress in this dam owing to the assistance given by the weight of the masonry acting as a gravity dam as equal to about 17 per cent (Control of Water, page 408).

243. Australian arched dams.—Several arched masonry dams designed on the basis of equation (24) have been completed in Australia in recent years and have proved successful and economical works

Examples among these are: –

The Barossa Dam (figures 33 and 33-*a*)
and the Mudgee Dam (figures 34 and 34-*a*).

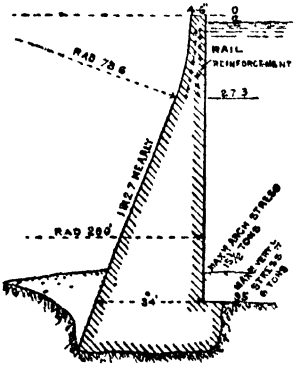
The maximum arch stress on the former is

$$\frac{95 \times 200}{34 \times 36} \text{ 15½ tons and}$$

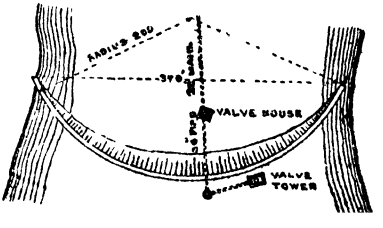
on the latter $\frac{50 \times 253}{18 \times 36} = 19\frac{1}{2}$ tons.

BAROSSA DAM.

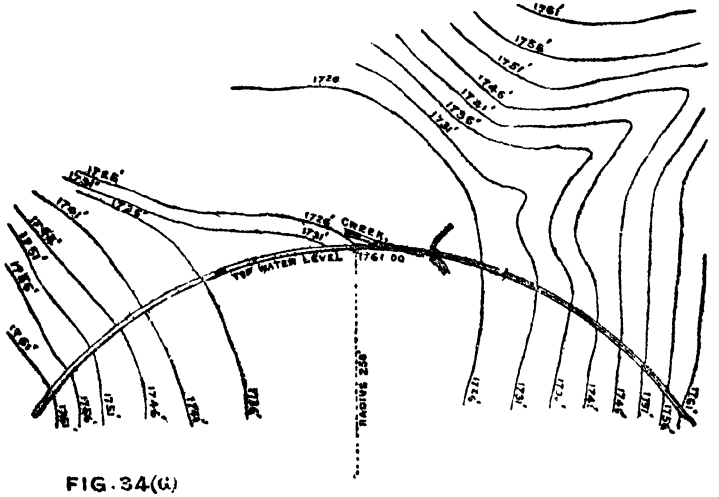
SECTION. FIG 33.



SITE PLAN. FIG. 33(a)



SITE PLAN OF MUDGEE DAM.
FIG. 34.



Worked as a gravity section for masonry of specific gravity 2·4, the resultant pressure 'reservoir full' falls just outside the extremity of the toe of the base and worked out on this basis the maximum stress (S) would be

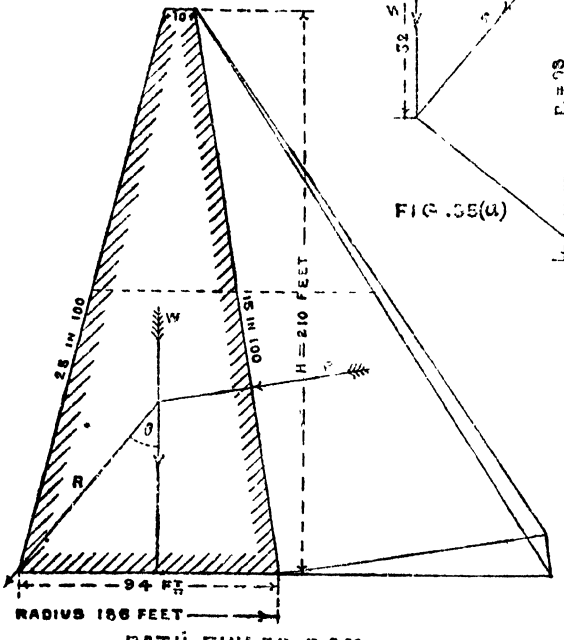
$$4 \times \frac{HR'w\rho}{b} = 4 \times \frac{210 \times 98}{94 \times 16} = 54{\cdot}7 \text{ tons per square foot.}$$

$$\text{The maximum arch stress} = \frac{2Hw}{\frac{b}{R}\left(2-\frac{b}{R}\right)} = \frac{210 \times 2 \times 1/36}{\frac{94}{210}\left(2-\frac{94}{210}\right)}$$

$$= \frac{11{\cdot}66}{{\cdot}694} = 16{\cdot}8 \text{ tons per square foot.}$$

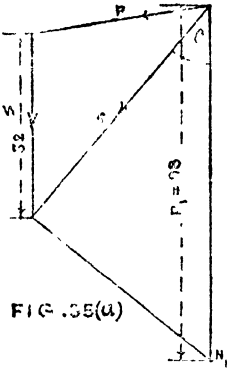
The actual arch stress would however be considerably less owing to the resistance afforded by the weight of the dam acting as a gravity dam.

FIG. 35.



PATH FINDER DAM.

FIG. 35(a)



245. Arched dams only applicable to narrow gorges.—It is clear from the stress formula for pressure that there is no economy in constructing an arched dam with a long radius and therefore it is only across narrow gorges that such dams can suitably be constructed. With a limit of pressure of 10 tons per square foot and a radius of 270 feet, the volumes of masonry required for an arched dam and for a gravity dam are approximately equal. If the pressure is raised to 20 tons the radius which brings about a similar state is 500 feet.

It has been shown by Captain Garrett, R.E., that the most economical curve for a given span is one with a central angle of about 120 degrees (Government of India Technical Paper 170 on Arched Masonry Dams); when working with small spans (say under 150 feet) it is desirable to design for a smaller central angle as a certain thickness is desirable to prevent percolation. For this reason the arches of buttress dams are generally given smaller central angles.

BUTTRESS DAMS.

246. Buttress dams—A buttress dam consists of a number of buttresses or piers, dividing the space to be dammed into a number of spans. To hold up and retain the water between these buttresses, panels are constructed consisting of horizontal arches, or ferro-concrete beams or slabs, or steel beams and planting.

A buttress dam thus bears a resemblance to a bridge laid on its side, with arches or other road-bearing platform vertical and constituting the water face of the dam, and the piers laid on their up river faces constituting the buttresses. Where the space between the buttresses is spanned by arches these dams are termed 'arched buttress dams'

247. Stability of buttresses.—In considering gravity dams, the stresses and stability are examined with reference to a slice, or profile of the dam, one foot in width, but in the case of a buttress dam it is necessary to consider the thrusts and weights of one buttress and two adjacent half spans With this exception, the methods of examining the forces on the buttress are generally similar to those employed for the gravity dam.

248. Panelling between buttresses—If the panelling between the buttresses consists of masonry arches the stresses in these are those in the arches of an arched dam

Where ferro-concrete or steel panelling is employed the designs may be worked out from the loading due to water pressure in a similar way to the road-bearers and platform of a bridge.

There appears to be little scope, under existing conditions, in the plains of South India for construction of ferro-concrete buttress dams, for ordinary masonry in lime mortar is, comparatively to Portland cement masonry, so cheap in India that the uses to which ferro-concrete is put in other countries are here frequently economically inapplicable.

In the case of dams and many other irrigation works weight in the structure is a feature tending to stability, and thus ordinary masonry, even at slightly higher prices, is preferable in such works to ferro-concrete or steel structures.

249. Arched buttress dams.—The arch buttress dam is the only type of buttress dam likely under existing conditions to come into extensive use in India and the design of this type alone is dealt with in detail.

It is desirable in such dams to build the arch with an axis inclined to the vertical at from 30° to 45°.

While this increases the total pressure on the face of the arches it does not increase the intensity of stress and it has the effect

of depressing the intersection of the line of resultant of the water pressure and weight of masonry, and of making the resultant to take a steeper angle to the horizontal, and thus increases the frictional resistance to sliding which is a danger to which this type of dam is specially liable.

250. Spacing of buttress for arched dams.—The spacing of the buttresses will depend on the depth of water and the nature of the foundations, and should be considered separately for each case. Generally the spacing in the clear would not be less than three-fourths of the depth of water or more than twice the depth. This only refers to arched panelling, and when ferro concrete facing slabs or other panelling is used, the buttresses would be closer together than three-fourths of the depth. The width of the buttress should be sufficient for the skew backs of the arches to rest on.

The arch may be such as to subtend an angle at the centre of 100 to 120 degrees.

251. Stresses due to the weight of masonry of an inclined arch.—Part of the weight of each arch ring of an inclined arch is carried by arch action and supported by the reaction of the abutments, and the remainder is carried from each arch ring to the successive rings below and is supported, as in a gravity dam, by the reaction of the foundation.

If A be the angle with the vertical of the axis of an arch, R_c the radius to the centre of the arch ring, then, for each cubic foot of masonry of weight $w\rho$, a part equal to $w\rho \sin A$ is supported by arch action, and a part $w\rho \cos A$ is supported by the reaction of the foundations.

If S_a be the stress in the arch induced by the weight of the arch itself

$$S_a = w\rho \sin A \times R_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

and S_a must be added to the stress induced on each arch ring by the water load to get the total thrust on the arch ring. The stress on the base or any joint of the arch at right angles to the axis is found as in the case of a gravity dam but allowing $w\rho \cos A$ instead of $w\rho$ for the weight of each cubic foot of masonry in the profile above the joint.

252. Example of a design of an arched buttress dam.—An example of design of an arched buttress dam subject to the following conditions will now be worked out. (Vide figure 36.)

Conditions—

Depth of water H to the top of the dam 40 feet

Arches to be stone concrete and buttress rubble masonry of specific gravity $2\frac{1}{4}$.

Design of arch.—The axis of the arches to be inclined at 30° to the vertical and the limiting stress in arch masonry to be 10 tons.

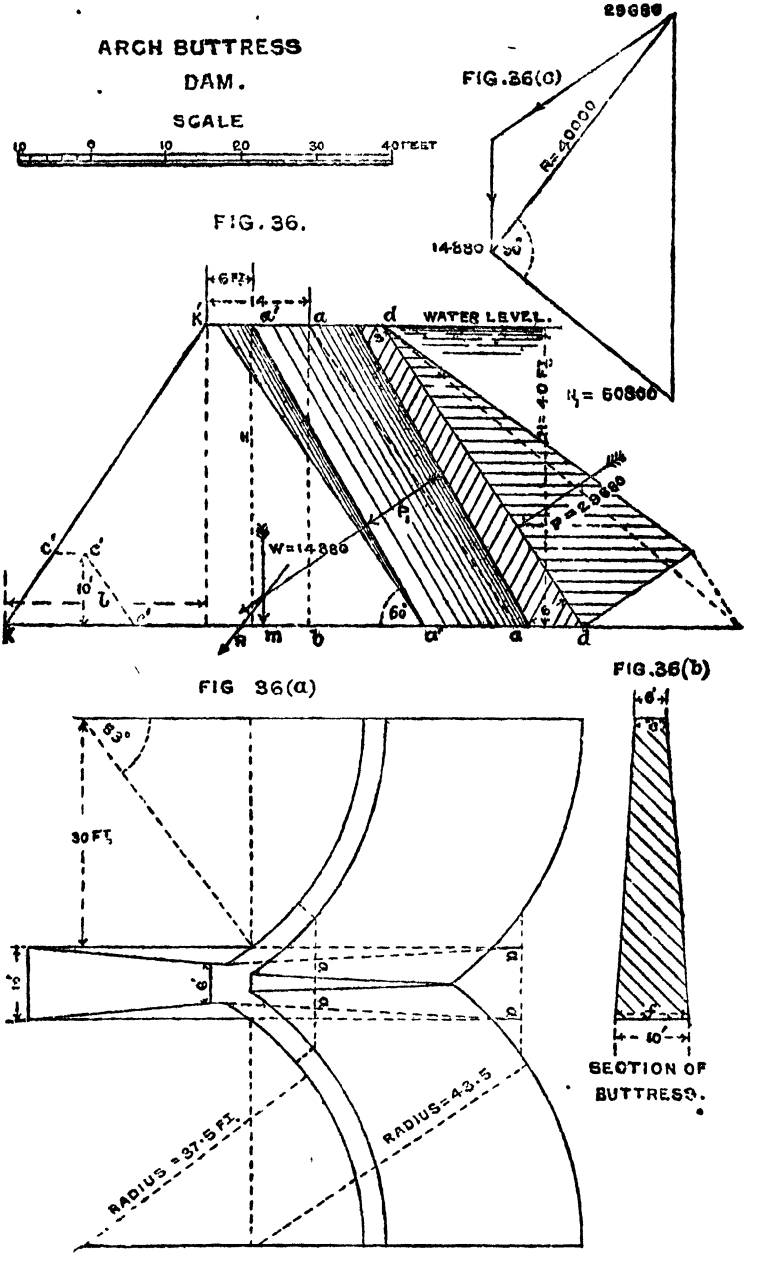
The spacing between buttresses in the clear will be made $1\frac{1}{2}$ times the depth = 60 feet.

The radius of the arches to the intrados will be made $37\frac{1}{2}$ feet which gives a central angle of 1068 and a versed sine of .

The top width of the arch will be 3 feet, this being about $\frac{1}{2}\sqrt{H}$ from equation (28), paragraph 251 above.

The slopes of the up and down-stream faces will be the same.

The bottom width (b) to give the limiting stress will be somewhat more than that given by equation (26), paragraph 238 above, because of the extra stress due to the weight of the arch.



H suitable width has been found by trial to be 6 feet and the suitability is tested as follows

The value of (b) being considerable in reference to the radius the long formula equation (25) must be applied to calculate the arch stress due to water pressure.

$$S = \frac{2Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)} = \frac{2 \times 40 \times \frac{1}{3.6}}{\frac{6}{43.5}\left(2 - \frac{6}{43.5}\right)} = 8.7 \text{ tons.}$$

The arch stress due to weight of arch from equation 28 is

$$S_a = wp \sin A \times R_c = 1/16 \times \tfrac{1}{2} \times (37.5 + 3) - 1.3 \text{ tons.}$$

Therefore the total arch stress $S + S_a = 8.7 + 1.3 = 10$ tons

Design of buttress.—The buttress may be made 10 feet thick at the base and batter at 1/20 on each side (figure 36-*b*).

The general method of designing the buttress is by trial tested by computation, or graphic statics, in a similar way to a gravity dam profile.

There is however an indeterminate factor which must be discussed before proceeding to design the buttress, viz., *to what extent the weights of the arches may be expected to contribute to the stability of the buttress*

It is evident that they must do so to a considerable extent as the buttresses can neither slide nor overturn without the same happening to the arch, or at any rate to a part of it The greater the angle at the centre of the arch the greater will this assistance be and vice versa

In dealing with this question Bligh considers the whole of the two half arches on either side of a pier as constituting an up-stream prolongation of the pier split into two diverging branches

He plots the equivalent weight of the two half arches distributed as a subsidiary part of the pier extending up-stream to a plane tangential to the up-stream faces of two adjacent arches. This appears to be an over-bold assumption, but it is considered safe to include ⅓ of each half arch and consider these as constituting a split up-stream prolongation of the buttress terminating at *a*, *a*, *a*, *a*, (in figure 36,) the intersection with the prolonged pier of the plane intersecting the arches at ⅓ their length along the extrados

The longitudinal section of this prolongation of the pier is the portion *á*, *á*, *a*, *a*. It is assumed that the prolongation is of the same side batters and top and bottom width as the rest of the buttress. This is a fair assumption as the content of the ⅓ length of the two half arches about correspond with the content of prolonged buttress.

Under this assumption the up-stream slope of the buttress is *a a* in section (figure 36) and a similar slope is given to the down-stream end of the buttress. This latter is merely a matter of convenience and any other suitable slope may be given.

If l is the base of each of the sloping ends of the buttress, and the thickness at bottom and top f and c respectively, then the weights of the sloping parts of the buttress up and down-stream are equal and each of volume $\frac{lH}{6}(2f + c) = \frac{30 \times 40}{6} \times 26 = 200$ c.ft.

In this case as the sloping ends of the buttress are exactly the same up-stream and down-stream, the centre of gravity of the whole buttress must pass through the centre of the portion of the buttress of which the top is horizontal

The volume of this central portion per foot length is $\frac{H}{2}(f+c)=$ 320 cubic feet.

The buttress should be made of such length as to bring the centre of pressure near to the centre of the base so as to make the maximum pressure on the foundations nearly equal to the mean pressure.

The water pressure (P) is the area of the triangle of pressure (figure 36), multiplied by the distance (70 feet) between buttresses centre to centre and as taken from the diagram equals 29,680.

This is applied along P in direction at right angles to the extrados slope *d*, *d*, at a point along *a' a'* one-third of its length from the base.

It is found by trial, by means of the diagram of forces, that by giving a length of 14 feet, measured from *a*, to the horizontal from top of buttress, the line of resultant comes near to the centre (*m*) of the base *a k* and it is therefore so designed.

The weight of this portion of the buttress is $320\times14=4,480$ units.

The whole weight of the buttress is thus $10,400+4,480=14,880$ units. Plotting this in a diagram of forces (figure 36-*c*) the resultant equals 40,000, and cuts the base *a k* three feet down-stream of the centre.

The base scales 69 feet long. The mean pressure on the base due to N_1 is $\frac{50800}{69\times10\times16}=4{\cdot}6$ tons

The centre of pressure is 3 feet from the centre of the base.

Therefore the maximum pressure $=S_1\left(1+\frac{6c}{b}\right)$

$$=4{\cdot}6\left(1+\frac{6\times3}{69}\right)$$

$$=4{\cdot}7\times1{\cdot}26$$

$=5{\cdot}9$ tons per square foot which is suitable.

253. Quantity of masonry as compared with gravity dam.—The amount of masonry per span works out to approximately,

	14,560	cubic feet, archwork
	10,733	cubic feet, buttress
Total	24,293	cubic feet per span,

that is $\frac{24293}{70}=347$ c.ft. per foot run.

A gravity section with a 6 feet top width and a section of the elementary profile has a content of 559 c.ft. per foot run.

The arch section has therefore a great deal less masonry.

The cost however of all masonry and especially the arch work would be more per unit for the buttress dam than for a gravity one.

On the whole there would frequently be saving in cost by using this type of dam which is one likely in the future to come more largely into use.

CHAPTER VIII.

RIVER WEIRS.

254. Definition of a weir.—A weir is a dam over the crest of which water is discharged. From this it follows, that a certain depth of water is usually standing against the down-stream face of the dam when it is acting as a weir, and also that, unless the course of the outfall channel receiving the overfall water is hard rock, protective works, in the form of masonry or pitched aprons, or water cushions, are necessary to prevent erosion of the bed of the channel by the water discharged over the weir.

Where the weir stands on hard rock, the dam or weir wall alone constitutes the principal work, but where the stream bed is friable, the weir wall is frequently but a minor item in the whole work required for the construction of the weir.

The general principles of design of the weir wall are the same in all cases, but certain modifications which will be described later are necessary when the structure stands on a permeable foundation. It is assumed that the weirs referred to in paragraphs 255 to 264 are founded on impervious rocks.

255. Functions of weirs.—Diversion weirs are usually of far less height than masonry dams, and generally run from 10 to 30 feet height and their primary object is the raising of the level and the diversion of water from rivers and streams into canals. There are however a considerable number of masonry weirs which combine the function of diversion with that of storage while there are others which are primarily dams for storage of water while throughout a part or the whole of their length the dams are used as surplus escapes for passing off excess flow which cannot be stored.

Some of these works have been built to a great height and are termed 'overflow dams' or simply 'dams' and not weirs, but these are for the present purpose considered and referred to as weirs.

256. Elementary profile for a weir wall.—The elementary profile of the gravity weir is a triangle with the apex at the maximum water level, the triangle top being cut off at the crest level of the weir. The base of this triangle would, as in the case of the gravity dam, be $\frac{H_1}{\rho}$ where H_1 is the depth of the base below maximum water surface. Generally as in the case of the gravity dam the up-stream face of the economical profile would be vertical

In practice it is however frequently desirable to keep the down-stream face of a weir wall at a steeper slope than a vertical up-stream face would admit of, and the up-stream face of a weir is therefore frequently built with a batter.

In figures 37 and 37-*a* a weir profile and a force diagram are drawn showing the main and back pressures of water on the weir faces.

If the height of the weir wall be H and the depth of water passing over it be *d*, the base (*b*) of the trial section may be $\frac{H+d}{\sqrt{\rho}}$

Here the elementary triangle is *a b s*.

FIG. 37

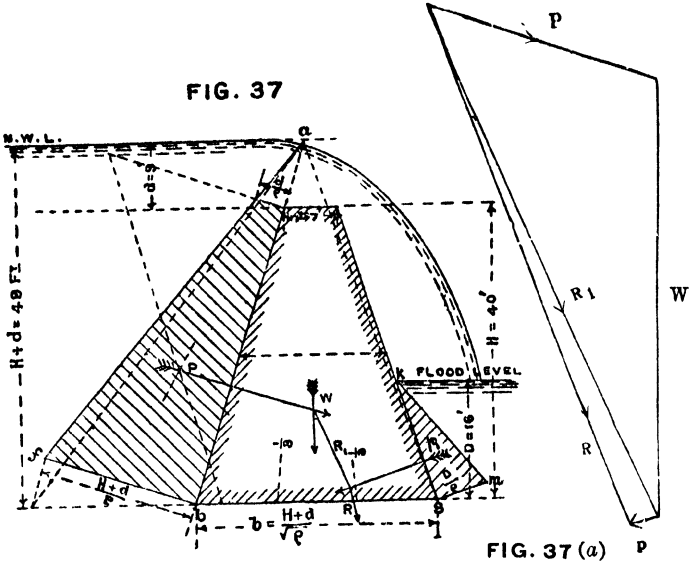


FIG. 37 (*a*)

The water pressure on the upstream side is shown by the shaded portion of the triangle *abf* the line *bf* being equal to $\frac{H+d}{p}$

while the piece cut off the top has a base $\frac{d}{p}$

The back pressure from the tail water is represented by the shaded triangle *ksm*. It will be seen that the resultant pressure (R) just falls within the middle third of the base

257. Formulæ for moment of water pressure on the face of a weir —

Let P be the whole pressure on the face of the weir (as in figure 37).

„ P_h and P_v be the horizontal and vertical components of P.

„ U = the length of the inclined face of the weir.

„ d = the depth in feet of water over the crest.

„ r = the slope of the face.
= the horizontal component of the slope for each vertical foot.

Then taking the units of length and force, as one foot and the weight of one cubic foot of water—

$$P = \tfrac{1}{2}\{(H+d)+d\} \times U$$

$$P = \frac{U}{2}(H+2d) \quad . \quad . \quad .$$

Similary

$$P_h = \frac{H}{2}(H+2d) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

$$\text{and } P_v = rH\left(\frac{H}{2}+d\right) = \frac{rH}{2}(H+2d) \quad . \quad . \quad (29\text{-a})$$

The centre of pressure, that is the point of application of the horizontal force P_h, is at a distance above the base equal to $\frac{H}{3}\left(\frac{H+3d}{H+2d}\right)$. Hence if M be the moment of horizontal pressure about a point in the base

$$M = \frac{H^2}{6}(H+3d) \quad \ldots\ldots\ldots \quad (30)$$

If the water surface is at the level of the weir crest,

$$M = \frac{H^3}{6} \quad \ldots\ldots\ldots \quad (31)$$

258. Effect of velocity of approach on a weir.—The extra pressure on a weir due to the velocity of flow of the water is provided for in stability diagrams or calculations by adding to the depth of the up-stream water a depth equal to the head (h_v) due to velocity of approach $\left(h_v = \frac{V^2}{2g}\right)$ so that the depth (d) over the crest is $(d_1 + h_v)$ where d_1 is the actual computed, or observed depth of flow

It may be added that d_1 is not the actual depth passing over the crest but the difference between the crest level and the level of water a short distance up-stream of the crest clear of the draw of the weir (vide figure 37).

259 Crest width of a weir.—The crest width of a weir must be fixed with reference to the following theoretical and practical considerations :—

(a) The theoretical considerations are the depth (d) due to the water over the weir crest and its velocity, the specific gravity (p) and co-efficient of friction (μ) of the masonry.

The width (a) of a horizontal joint at crest level to ensure against sliding would be $a = \frac{d}{\mu p}$

and assuming ⅔ as a safe value for the co-efficient of friction this becomes $a = \frac{3d}{2p}$

and the weir crest width should not be less than this.

(b) The practical considerations are—

The height of the weir.

The nature of the drift carried by the stream.

The width required for crest shutters or for traffic of any kind.

Bligh suggests the following empirical rule for the crest width which subject to the minimum $a = \frac{3d}{2p}$ may be adopted under ordinary conditions.

$$a = \sqrt{H} + \sqrt{d} \quad \ldots\ldots\ldots \quad (32)$$

260. Condition of maximum stress on weir walls.—In the case of a dam the condition of maximum stress is when the water level above the base is a maximum, and this condition also connotes the state in which the head of water on the dam is greatest.

In the case of a weir the greater the discharge the smaller is the difference of level between the head and tail water surfaces and the condition of maximum water level above the work generally connotes that of minimum head on the work; the condition of maximum head on the work is that in which the head water is at the level of the crest of the weir, or weir shutters, and no water is passing over the weir.

The condition of head water at a maximum level is generally that of maximum stress in high weirs, and in low weirs the condition of maximum stress is not infrequently that of head water at weir crest level, but there are many weirs in which the maximum stress is induced by some condition of flow intermediate between these two and it is necessary to ascertain for each weir what is the condition of greatest stress.

261. Different states of water pressure on a weir wall.—To find the maximum overturning moment on a weir it is necessary to consider it under two different states

State I—When the up-stream water, or 'head water,' is at crest level, or, if there are crest shutters, at shutter crest level.

In this state the down-stream water, or 'tail water,' will be at about low water level (L.W.L.) and the conditions of water pressure on a weir will be similar to those on a dam.

In this state the head on the weir, that is the difference of levels of the head and tail water, is a maximum.

State II.—When water is passing over the weir crest.—As the depth of water passing over the weir crest increases, the head on the weir decreases and the level of the tail water continually rises.

Two conditions of discharge occur under state II—

II (*a*) When the weir is submerged.

II (*b*) When the weir is discharging with a clear overfall.

262. Overturning moments on a weir.—

Let H be the height in feet of the weir wall above the base.

„ *a* be the crest width in feet.

„ *b* be the width of the base in feet.

„ s be the height in feet of the crest shutters.

„ D be the depth of the tail water above the base.

„ Mo be the overturning moment on the weir, the units of weight and length being one cubic foot of water and one foot.

The overturning moment will be considered as the difference between the horizontal water moments on either side of the weir, the vertical components of water pressure being considered as part of the weight resisting overturning.

State I.—This state connotes the maximum overturning moment on the weir when no water is passing over the crest and this is given by the equation (31).

$$Mo = \frac{H^3}{6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31\text{-}a)$$

If the weir has crest shutters s feet high the maximum moment, state I, is—

$$Mo = \frac{(H+s)^3}{6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31\text{-}b)$$

State II (a).—If h is the difference of level of head and tail water then the height of the tail water above the weir crest is $d-h$

$$M_o=\frac{H^2}{6}\left\{H+3d-H-3(d-h)\right\}$$
$$=\tfrac{1}{2}H^2h$$

and this is a maximum when h is a maximum, viz.:—
when the tail water is just at weir crest level, that is when the state II (*a*) merges into II (*b*) and $d=h$

The maximum overturning moment under state II (*a*) will thus occur when the weir is on the point of becoming submerged.

State II (b).—If D be the depth of the tail water above the base then from equations 30 and 31 above

$$M_o=\frac{H^2}{6}(H+3d)-\frac{D^3}{6}$$
$$M_o=\tfrac{1}{6}(H'+3dH^2-D') \quad \ldots\ldots\ldots \quad (33)$$

263. To find the condition of maximum overturning moment of a weir when water is passing over the crest.—In order to find when M_o is a maximum it is necessary to know with reference to each weir the relative values of d and D, for values of D varying up to a maximum of D = H.

In order to arrive at these values accurately it would be necessary to ascertain by calculation, or by actual gauging, the river discharges at the site of the proposed weir for several stages of flow.

From the results a curve can be drawn giving the relations between gauge readings and discharges; from which the river discharge corresponding to each water level will be known. The discharges of the proposed weir for various values of (d) in the limits of flow would also be computed and from the results the value of d corresponding to any specific value of D would be ascertained.

When the weir extends the whole width of the river and the latter is of considerable width in comparison to its depth, say width not less than 30 times the depth, the value of d is roughly proportionate to that of D.

Under these conditions

$d=KD$ and $K=\frac{d}{D}$ where K is a constant.

In such a case, if the river water level is known for any one discharge, the value of D for that discharge is known, and the corresponding value of d can be calculated, and thus K can be ascertained with sufficient accuracy for the purpose required

If possible the value of K should be fixed from a value of D connoting:—

(*a*) For a submersible weir, the level at which the weir would become submerged (viz., D = H).

(*b*) For an insubmersible weir, the highest flood of which a reliable water level and discharge can be ascertained.

Having found the value of K as described above, the condition of flow producing the maximum overturning moment can be

ascertained by the following method which is due to Mr. S. V. Kanakasabai Pillai of the Madras Public Works Department. (Vide "Design of Masonry Weirs" by the above author published by Solden & Co., Madras, 1919.)

Substitute KD for d in equation (33).

then $M_o = \frac{1}{6}(H^3 + 3\,KD\,H^2 - D^3)$.

this expression is a maximum when

$D = H\sqrt{K}$.

This gives the condition of tail water level connoting the maximum overturning moment of State II.

The base of a weir frequently stands on a platform above the low water level of the river and the tail water is not raised to the level of the weir base until a certain depth of water is passing over the crest.

Let t be the depth of water passing over the weir when the tail water is at the weir base level.

Let d be the depth of water passing over the weir when the tail water is at a height D above the base.

In this case $d - t = KD$.

$$\therefore d = KD + t \text{ and } K = \frac{d-t}{D}$$

When M_o is a maximum $D = H\sqrt{K}$.

and $D^2 = H^2K = \frac{H^2(d-t)}{D} \therefore D^3 = H^2(d-t)$.

and $d = KD + t = K^{\frac{3}{2}}H + t$.

Substituting for D^3 and d in equation (33)

When M_o is a maximum

$$M_o = \frac{H^3}{6}(1 + 2K^{\frac{3}{2}}) + \tfrac{1}{2}H^2t \quad \ldots \quad (34)$$

264. Condition of greatest stress of a weir built on impervious soil.—When a weir is founded on an impervious soil and constructed of impervious masonry the weight of the masonry is not affected by variations in the water levels above or below the weir and the moment of resistance to overturning is practically constant.

It is true that the moment of the vertical component of the water standing over the slope, if any, of either face or passing over the crest forms part of the total moment but its effect is so small that the above statement may be accepted as applicable to all ordinary cases.

Thus the condition of maximum stress on a weir built an impervious foundation soil is that which connotes either:—

(I) Head water at the crest level of the weir shutters.

(II) Tail water at maximum flood level, or at the level $H\sqrt{K}$ above the base of the weir, whichever is the lower.

The weir must be so proportioned that its moment of resistance about the down-stream extremity of the middle third of its base must not be less than the overturning moment under (I) or (II).

If there are no weir shutters State II is always that of maximum stress.

265. Varying moment of resistance of a weir built on pervious soil.—When either the foundation soil or the masonry composing the weir is pervious the effective weight of the masonry is

reduced by uplift, as explained in paragraph 234 above. The exact incidence of uplift depends on indeterminate factors such as loss of the head on entry, the permeability of the river-bed soil, and also on the nature and thickness of the foundation platform and of the aprons up-stream of the body wall and the level of the tail water.

In designing irrigation weirs and regulators on permeable soil Bligh provides for uplift by treating all layers of masonry below the level of the tail water under the condition considered, as subject to full flotation, that is of effective weight $(\rho - 1)$ per cubic foot, and all masonry above this level as having effective weight ρ per cubic foot. This allowance is a suitable and sufficiently accurate provision for uplift in all ordinary cases, and is adopted in this text book.

In cases where a weir is built on very pervious soil and where its crest is much above the maximum level of tail water the allowance will be insufficient, and in such cases partial flotation for the whole profile may be allowed as well as full flotation for the portion of the profile below tail water level.

266. Condition of greatest stress of weirs on pervious soil.—When full flotation up to tail water level is allowed for, the moment of resistance of a weir to overturning diminishes as the tail water rises and is a minimum when the tail water is at or above crest level. Under such condition it does not follow that the water levels giving the maximum overturning moment of State II connote also the condition of greatest stress, and as a fact, in most cases the condition of greatest stress, in a weir liable to submergence is when the tail water is at crest level, that is when $D = H$. There are some rare exceptions to this and under certain circumstances the condition of greatest stress is when $D = H\sqrt{K}$.

For ordinary cases occurring in practice it is safe to assume that *in weirs subject to flotation* the condition of greatest stress of State II is when the tail water is either at crest level, or at its maximum flood level if this is below crest level.

267. Examination of stability of a weir by the method of moments.—The stability of weirs may be examined either by arithmetical computation or by diagrams and an example of the former method in respect to the bodywall of the weir shown in figure 38 is worked out below by the method of moments which method has already been described in paragraphs 223 and 226 above, which may be referred to for the symbols used for the divisions of the base.

The weir of the figured dimensions is built of brick (sp. gr. 2) and is subject to flotation and also to submergence When the tail water is at crest level the observed depth of the head water (d_1) is 4 feet over the crest. The base of the weir is at low water level. The weir is provided with crest shutters 3 feet high.

State I.—(figure 38).

$$M_0 = \frac{(H + S)^3}{6} = \frac{(10 + 3)^3}{6} = 366.$$

State II.—(figure 38-*a*).

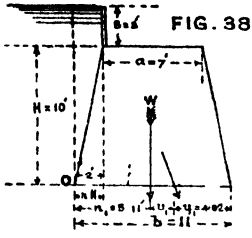
From the data we have $K = \frac{d_1}{D_1} = \frac{4}{10} = 0{\cdot}40$ and $\sqrt{K} = 0{\cdot}6$.

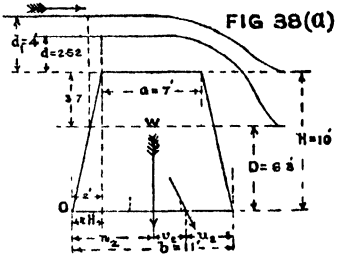
If d and D are the values connoting crest and tail water levels due to the maximum overturning moment (M_0), then

$$D = H\sqrt{K} = 6{\cdot}3 \text{ and } d = KD = 2{\cdot}52.$$
$$M_0 = \tfrac{1}{2}[10^2 + (3 \times 2{\cdot}52 \times 10) - (6{\cdot}3)^2] = 251.$$

If the weir were not subject to flotation State I would be that of maximum stress for which the profile has to be tested for safety. The weir however is subject to flotation and the condition of maximum stress is still unknown and stability will be examined by the method of moments both in States I and II

State I (with head water at shutter crest level—figure 38).





Taking moments round O

	Weight	Lever arm.	Moment.
Masonry profile $\frac{1}{2}(7+11) \times 10 \times 2$	180	5 5	990
Water on slant face ... $\begin{cases} 3 \times 2 \times 1 \\ \frac{1}{2} \times 2 \times 10 \end{cases}$	6 10	1 0 0 67	6 6 67
Total ..	196	.	1,002 67

$$n_1 = \frac{1{,}002\ 67}{196} = 5{\cdot}11$$

Horizontal moment of water thrust = 366.

$$v_1 = \frac{366}{196} = 1{\cdot}87$$

$$u_1 = b - (n_1 + v_1) = 11 - (5{\cdot}11 + 1{\cdot}87) = 4{\cdot}02.$$

State II.—(1) With tail water at weir crest level (figure 38-*a*). Taking moments round O—

	Weight.	Lever.	Moment
Masonry profile with flotation 9 × 10	90	5·5	495.

Note. The vertical component of water thrust is eliminated by taking up lift in the form of flotation. (Vide paragraph 235 above.)

$$n_2 = 5{\cdot}5$$

Horizontal moment of water thrust = $\frac{d\,H^2}{2} = 200$

$$v_2 = \frac{200}{90} = 2{\cdot}22.$$

$$u_2 = b - (n_2 + v_2) = 11 - (5{\cdot}5 + 2{\cdot}22) = 3{\cdot}28.$$

As $\frac{1}{3}b = 3{\cdot}67$ the centre of pressure in this case falls outside the middle third of the base. State II is that of greatest stress.

(2) Moments will also be taken with the tail water at level $H\sqrt{K}$, that is, when D = 6·3 and d = 2·52 (figure 38-a).

	Weight.	Lever arm.	Moment
Weight of profile with flotation	90	5·5	495
Extra weight above tail water level ½ (7 + 8·48) × 3·7	28·6	5·5	157·3
Water on slant face above tail water level { ½ × 3·7 × ·74	1·37	1·50	2·05
{ 2·52 × ·74 ...	1·86	1·63	3·03
Total ...	121·83	.	657·38

$$u = \frac{657{\cdot}38}{121{\cdot}83} = 5{\cdot}39.$$

the moment of horizontal water thrust is 251

$$v = \frac{251}{121{\cdot}83} = 2{\cdot}06$$

$$\therefore u = b - (u + v) = 11 - (5{\cdot}39 + 2{\cdot}06) = 3{\cdot}55.$$

This is a larger value of u than with the tail water at the crest, and it will be found that this will, as a rule, be the case.

In the above example State I would be the condition of greatest stress if the weir were not subject to flotation and the centre of pressure would also be within the middle third.

It is only in exceptional cases that weirs are considered as free from up-lift and it is desirable to allow for flotation as above except where the weir is founded on hard rock free from cracks and fissures, or on impervious clay.

268. Formulæ for the moment of resistance of a trapezoidal weir with flotation to level of the tail water.—Let Mr be the moment of resistance about a point at the down-stream extremity of the middle third of the base and let the other symbols be as in paragraphs 257 and 262 above.

State I.—Head water at shutter crest level:—

$$Mr = \tfrac{1}{6}\{\rho\, H\, b^2 + [\rho\, H\,(a - r\, H) + 2\, r\, H\,(H + 2\, S)]\, b - \rho\, a\, H\,(a + 2\, r\, H) - r^2 H^2 (H + 3S)\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

If the up-stream face be vertical, then—

$$Mr = \frac{H\rho}{6}(b^2 + ab - a^2) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35\text{-}a)$$

If both faces of the weir have the same slope—

$$Mr = \tfrac{1}{12}\{[(\rho + 1\tfrac{1}{2})\, H + 2\tfrac{1}{2}\, S]\, b^2 + a\,(\rho\, H - H - S)\, b - \tfrac{1}{2}a^2 (H + 3\, S)\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35\text{-}b)$$

State II.—(*a*) With tail water at weir crest level—

$$Mr = \frac{H\,(\rho - 1)}{6}\left\{b^2 + (a - rH)\, b - a\,(a + 2r\, H)\right\} \quad . \quad . \quad . \quad . \quad (36)$$

This is the general formula for the moment of resistance of a trapezoidal weir subject to 'full flotation' and fulfilling the 'middle third' rule. From this equation it will be seen that, for the same moment of resistance, 'b' is a minimum when 'r' is

zero, that is, the up-stream face vertical gives the most economical profile—

If the up-stream face be vertical

$$Mr = \frac{H\ (\rho - 1)}{6}\ (b^2 + ab - a^2) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36\text{-}a)$$

If both faces of the weir have the same slope:—

$$Mr = \frac{H\ (\rho - 1)}{12}\ (b^2 + ab) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36\text{-}b)$$

II (*b*) With the level of tail water "*h*" feet below the weir crest:—

$$Mr\ \tfrac{1}{6}\ \left\{\left[H\ (\rho - 1) + \frac{h^2}{H}\ \left(2 - \frac{h}{H}\right)\right] b^2 + \left[H\ (\rho - 1)\ (a - rH)\right.\right.$$

$$+ 4\ ah + \frac{h^2}{H}\left(\frac{2h}{H}\ a + 2\ rh - 3\ r\ H - 5a\right) + 2rh\ (h + 2d)\Big]\ b$$

$$- a\ H\ (\rho - 1)\ (a + 2\ r\ H) + \frac{ah^2}{H}\left(3\ a + 3r\ H - 2\ rh - \frac{h}{H}\ a\right) - 3\ ah\ (a$$

$$\left.\left. + 2\ rH - rh) + (r\ h)^2\ (2\ h + 3\ d) - 3\ rh\ r\ H\ (h + 2\ d)\right\}\right. \quad . \quad (37)$$

If the up-stream face be vertical:—

$$Mr = \tfrac{1}{6}\left\{\left[H\ (\rho - 1) + \frac{h^2}{H}\ \left(2 - \frac{h}{H}\right)\right]\ b^2 + \left[H\ a\ (\rho - 1)\right.\right.$$

$$+ 4\ a\ h + \frac{a\ h^2}{H}\left(\frac{2\ h}{H} - 5\right)\Big]\ b + \frac{a^2\ h^2}{H}\ \left(3 - \frac{h}{H}\right) - a^2\ H\ (\rho - 1)$$

$$\left.\left. - 3\ a^2\ h\right\}\right. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (37\text{-}a)$$

269. Finding the base width of a weir by calculation.—It is usual to arrive at a suitable section for a weir wall by trial, selecting a suitable crest width, taking as a trial base the width of base of the elementary triangle modified from considerations of flotation, crest width, etc., and to test for stability as described in paragraph 267, or by diagrammatic methods described later and modify the profile as the results may indicate.

If, however, the crest width and the slope of the up-stream face is fixed, the base width required to bring the resultant pressure to the down-stream extremity of its middle third may be found by equating the moment of resistance given in paragraph 268 with the corresponding overturning moment, and solving the quadratic equation for *b*, the base width.

For instance, in the case of the weir forming the subject of paragraph 267 above, the length of base as arrived at by this method is 11·75 with an up-stream batter of one in five; with a vertical up-stream face the base width necessary is reduced to 10 feet, a reduction in the area of the profile of nearly 10 per cent.

270. Stability diagram of a weir.—In paragraph 267 above a weir was examined for stability by arithmetic computation. An example is now given of construction of a stability diagram.

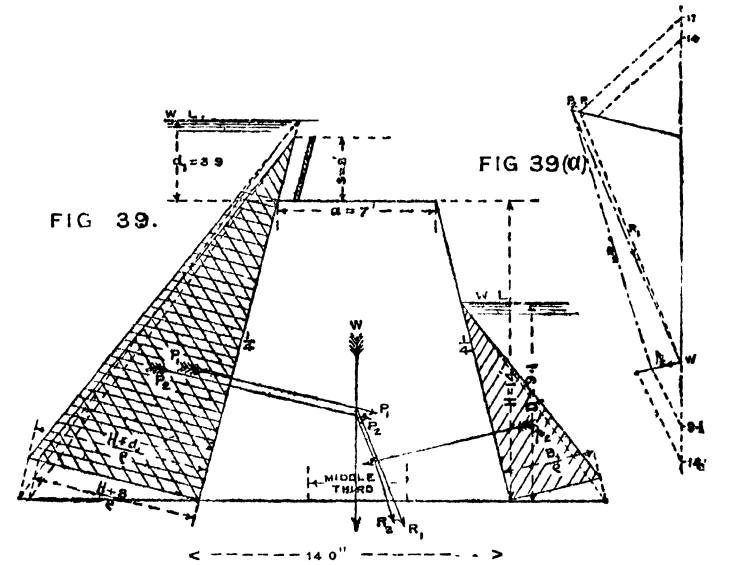
The weir profile is shown in figures 39 and 39-*b* and is 14 feet high with 7 feet crest width, while the conditions of flow are such, that when the tail water is at weir crest level the head water level is 6 feet above the weir crest. The specific gravity of the masonry is 2¼ and both faces have a batter of one in four.

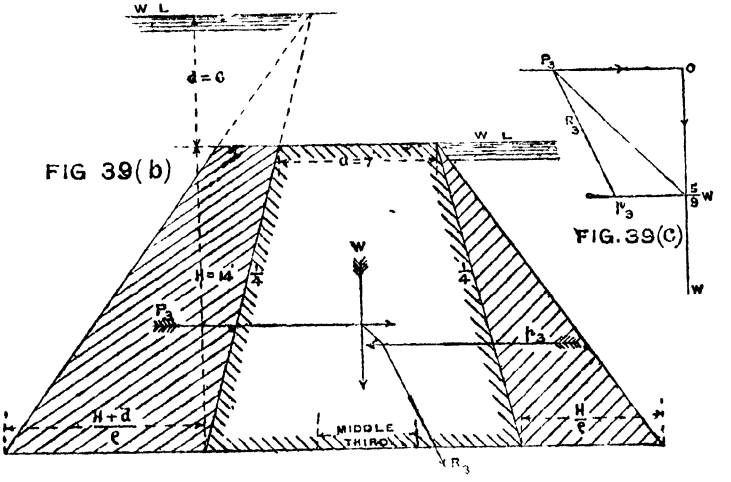
From the above we have $K = \frac{d}{D} = \frac{6}{14} = \cdot 43$

$$H \times \sqrt{K} = D_1 = \cdot 65 \times 14 = 9 \cdot 1$$

The lines of pressure and weight are plotted on the profile and in the force diagrams.

State I.—With head water at shutter crest level.—The forces and resultant are P_1, W, R_1 (figures 39 and 39-*a*).





State II.—(*a*) Supposing the weir standing on an impervious foundation with tail water at 9·1 feet above base.

In this case $d_1 = \cdot 43 \times 9 \cdot 1 = 3 \cdot 90$ feet.

The forces and resultant are P_2 p_2, W, R_2 (figures 39 and 39-*a*).

(*b*) Supposing the weir to be on a pervious foundation with tail water at crest level.

The forces and resultant are P_3 p_3, $\frac{5}{9}$ W, R_3 (figures 39-*b* and *c*).

The force diagrams are made taking the mean widths for the areas of the figures, the common height being H = 14. In figure 39 (*a*) there is an exception to this in the case of the triangle

representing the pressure P_1 which is of height $(P + S) = 17$. Thus the force is represented by $\frac{17}{14}$ of the mean width of the triangle. Similarly the triangle representing p_2 is of height only 9·1 feet and the force is represented by 9·1/14 of the mean width of the pressure triangle.

In figures 39-*b* and *c* only the horizontal components of water pressures are plotted, the vertical components being absorbed in resisting uplift as explained in chapter VII, paragraph 235

From the above example it appears that without flotation state I is that of greatest stress and that the profile is of ample base width if founded on impervious rock. If on pervious soil state II is the condition of greatest stress and the base width is insufficient.

The required base width in this case where both weir faces have the same slope may be readily ascertained by substituting in equation 36-*b* of paragraph 268 above, the known values of *a*, H, and ρ.

Then $M_1 = \frac{14 \times 14}{12} (b^2 + 7b)$

The overturning movement $M_0 = \frac{H \cdot d}{2} = \frac{14 \cdot \times 6}{2}$

and equating these $b^2 + 7b + (3\frac{1}{2})^2 = \frac{14 \times 3 \times 12 \times 4}{5} + (3\frac{1}{2})^2$

$= 403{\cdot}2 + 12{\cdot}25 = 415{\cdot}45.$

$b = 20{\cdot}38 - 3{\cdot}5 = 16{\cdot}88$ feet.

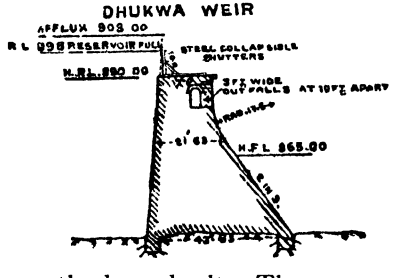
This shows that on a pervious soil a batter of one in three on both faces is required.

271. Importance of recording the depth of head water on weirs on the point of submersion.—The above shows that it is important in regard to a submersible weir to record the depth of the head water when the weir is on the point of submergence as this information is necessary to enable the stability of the weir to be examined. The information should be recorded on the plan of the weir.

272. Dhukwa and Sunkesala weirs.—A few sections of weirs built on rock foundations will now be given—

(*a*) *The Dhukwa weir across the Betwa river in Bundelcund* (*figure* 40).

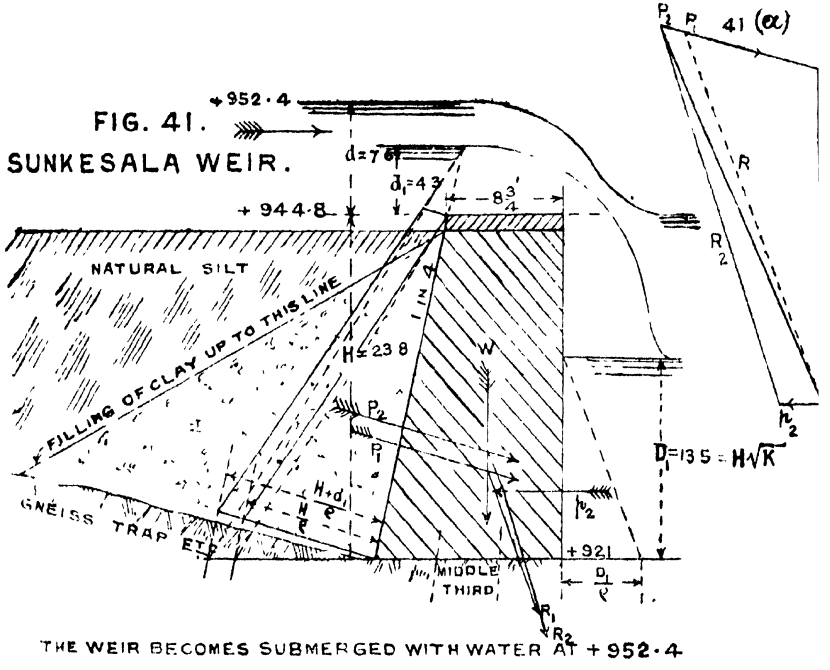
FIG. 40.



This weir has recently been built. The passage underneath is used for lowering the crest shutters which can be released by moving lever arms the ends of which project into the passage ; it is also useful as a means of communication. The profile approximates to the elementary triangle with masonry of specific gravity 2¼ and it is clear that the weir has been designed without

allowance for flotation. The foundation is hard rock and it is probable that little uplift is developed. It is noticeable that foundation stop walls at both the up and down-stream extremities of the base have been provided.

(*b*) *The Sunkesala anicut across the Tungabhadra* (*figure* 41).—This is a weir across the Tungabhadra river at the head of the Kurnool-Cuddapah canal. It was originally built (1866) three feet higher than shown, but it breached in 1882 and the crest was then cut down.



The stability diagram shows that, even now, the lines of resultant pressures (R_1 and R_2) fall outside the middle third both under state I and state II without any allowance for 'flotation.'

It is to be noticed that when building the weir a bank of clay was formed up to nearly crest level against the up-stream face and the continued existence of this work is probably due to the reduction of hydraulic pressure caused by this. The utility of a bank of impervious clay up-stream of a masonry dam or weir in reducing both direct and uplift pressure should be noted.

273. Austin weir.—Figure 42 gives the section of the Austin weir dam across the Colorado river (Texas, U.S.A.).

This dam failed during a high flood in 1900, it is believed, by sliding on the laminated horizontal layers of limestone on which it stood and through which leakage is known to have passed. Thus while the dam was subject to uplift there was probably little cohesion between the layers of rock on which it stood The rounded top is shaped with a view to coincide with the natural trajectory of the falling water while the reverse curve at the bottom

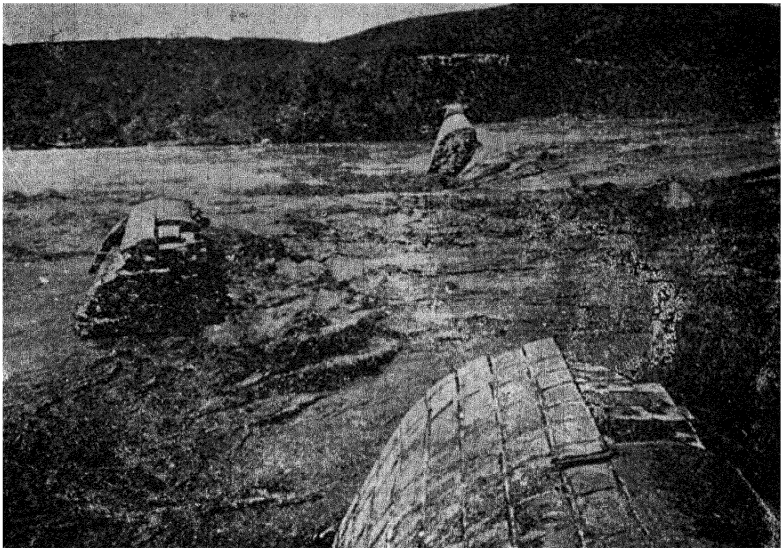


Plate II (a).– Austin Dam, Texas. View taken during flood a few minutes after the break.

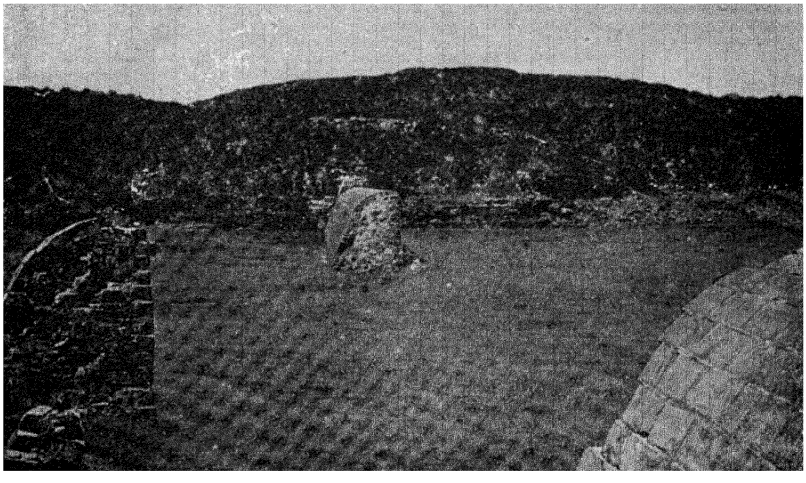
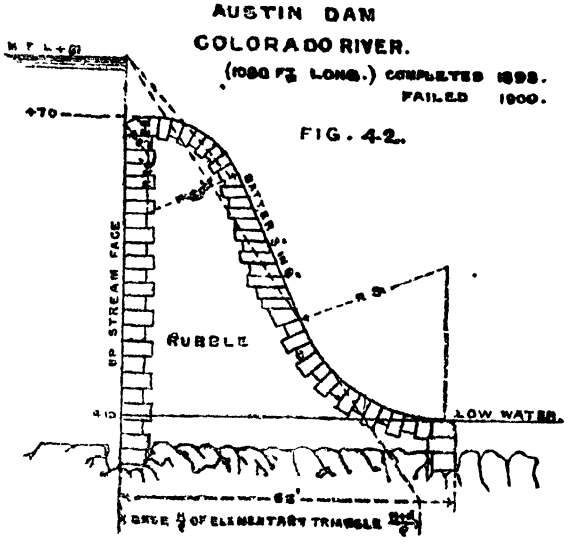


Plate II (b).—Austin Dam, Texas, after subsidence of flood of April 7, 1900. Showing section of masonry moved bodily down-stream.

is given to guide the water from a nearly vertical to a horizontal course when leaving the masonry at the foot of the dam.

Such a shape is termed an ogee curve and was at one time much used in India, but this form has now been abandoned for low weirs as it was found to induce scours in the margins and in the bed of the run-off channel owing to the very high velocity with which the water is projected horizontally from the foot of the fall.



The practice now adopted in India is to dissipate as far as practicable the velocity of the water moving down-stream from the toe of the work and this can be effected better by an overfall into a water cushion or on to an apron. The ogee shape is however much used in America for weirs on rock foundations

For high weirs (overflow dams) it is probable that a rounded crest is the best form.

274. Buttress weirs.—Buttress weirs and arched buttress weirs may be designed and constructed on similar lines to those described for this description of dams.

In the case of weirs, this type of work gives special facilities for the cheap construction of a water cushion, the cushion walls of each bay resting on the down-stream ends of the buttresses forming a series of cushion cisterns between each pair of buttresses. A section c' c' c' of such a cushion wall is shown in dotted lines on the section of the arched buttress dam (figure 36 above).

WEIRS ON SAND.

275. Weirs on sand and other friable foundations.—When weirs are constructed on sand or other soft soil it is necessary to protect the foundations of the masonry weir wall from injury by masonry aprons down-stream of the wall and sometimes also up-stream. Sand is the soil on which such weirs are most commonly constructed being, in the absence of rock, the usual soil in river beds,

and the weirs now treated of are weirs founded on sand, but the general principles and methods of design apply equally to weirs founded on other soils such as gravel, loam and clay, etc.

When the soil is impermeable the special features introduced in design to guard against injury to the work by percolation are uncalled for, but protection against scour is required in all soils other than rock.

Weirs on sand or permeable soils are generally low weirs, and those on sand are almost invariably diversion dams as contrasted with storage dams. They vary in height generally from 8 to 14 feet and are frequently works of great length and magnitude. Most weirs across rivers with beds of sand are 'drowned weirs,' that is, they are subject to submergence.

276. Causes of failure of sand foundations —Sand is incompressible and forms a good foundation for masonry works if it can be protected against erosion or disturbance, and the protective aprons of a weir are designed to effect this.

There are two ways in which sand foundations under a weir can be disturbed—

(*a*) By percolation.
(*b*) By scour or erosion.

The effect of percolation in moving sand, and the methods of guarding against such movement under an impervious platform have already been described in Chapter IV, paragraphs 120 and 121, and the measures required for protection against scour will now be treated of.

The discharge passing over a weir crest at a level much above that of the tail water involves shock, turmoil and great increase of velocity over the normal flood velocity of the river, and if no protection immediately below the weir wall were provided, the erosive power of the water would inevitably excavate a deep hole close under the wall and thereby the foundation soil would be undermined and the work would fail.

A protective apron overlying the sand immediately down-stream of a weir is therefore a structural necessity.

277. Apron down-stream of the weir wall.—The apron down-stream of the weir wall fulfils two functions :—

(1) When there is an overfall on to it, some of the excessive velocity due to the head over the weir wall is destroyed by impact.

(2) The locality of the scour in the river bed is transferred from immediately below the weir wall to down-stream of the tail of the apron.

The width of the apron should therefore be sufficient to ensure that any scour caused below the weir wall shall be sufficiently far removed from it as to leave the soil on which it stands unaffected.

It is impracticable to make the apron wide enough to ensure that the flow-off of water from the tail end shall be of such a nature as regards limit of velocity and absence of eddies and cross currents as to prevent any scour of the sand immediately down-stream of the toe of the apron. The down-stream section of the apron is therefore made of loose stone or blocks of concrete which will conform to any deepening of the sand in the river bed below it with the corresponding slipping of the sand on which this loose

apron lies, to form a gradually sloping side to the cavity caused by scour.

The tail apron stone settles with, and conforms to, the face of the sand slope and forms a protective covering for the same, preventing the scour working back up-stream towards the solid apron and body wall above.

The greater the velocity and turmoil of the water at the toe of the apron, the deeper the toe scour is likely to be, and the greater the width and depth required for the loose stone of the tail apron or *talus*.

278. Types of river weirs or anicuts —From the above considerations the following alternative methods of dealing with the water passing over the weir crest are indicated :—

(1) It can be dropped from the crest on to a solid apron built at, or nearly at, the low water level of the river below.

By this means much of the excessive velocity is dissipated at once, and a section of flow sufficient to carry the discharge with normal river velocity is formed close up to the body wall, and thus the action down-stream of the toe of the work is much reduced.

(2) The whole width of the apron can be formed as a sloping surface from crest level to the dry packed down-stream apron, the level at the lower extremity of the slope being about the low water level of the river

A sloping apron may be formed either by a solid masonry sloping platform, or by filling in of stone over the river bed to the required levels the surface being of large stones dry packed at the required slope.

Bligh has divided weirs on sand foundations into three types and his classification will be here followed :—

Type A.—Vertical drop on to a horizontal floor (vide figures 44 and 45).
Type B.—Sloping masonry apron (vide figure 48)
Type C.—Rock fill sloping apron (vide figures 50 and 51).

He subdivides type A into two types.

In type A_1 the surface of the apron is at or below L.W.L. (figure 44).

In type A_2 the apron is partly or wholly above L.W.L (figures 45 and 46).

279. Apron up-stream of an anicut —In addition to a down-stream apron the body wall of a weir not infrequently requires protection from scour on the up-stream side.

Such scours are chiefly due to cross currents, any current running in a direction parallel to the face of the work being specially dangerous as causing deep scours. These currents are most likely to be formed where the weir is situated at a part of the river where the course is crooked, or obstructed by shoals, or where the main current is not parallel to the axis of the river, or the weir not at right angles to the current.

Protection against scour above a weir is generally given by a flat or sloping rough stone or block apron ; where cross currents are developed either above or below the weir they are checked by ' hanging groynes.'

280. Hanging groynes.—These are groynes of rough stone run out from the work at right angles to its length, the top level of the groynes would generally be within 2 or 3 feet of the masonry crest level if up-stream of the work, and at about low water level if down-stream of it.

281. Considerations governing design of aprons.—The lengths and thickness of the aprons required for the protection of weirs from the effects of bed scour are clearly not susceptible of determination by calculation, and they must be fixed with reference to, and by study of, the dimensions, conditions, and history of existing successful works and of failures.

In comparing works, the conditions as regards height of crest, the velocity and depth of water passing over the work, the nature of river bed, and the nature and specific gravity of the masonry of which they are composed, must be carefully noted, as it is obvious that all these items must have an effect on the design of the protective aprons required to render the works secure.

In preparing designs of a weir to be built on sand, it is necessary to consider and combine the above requirements with those necessary for protection against dangerous velocity of percolation or dangerous uplift pressures; these are susceptible of definite calculation, as described in chapter IV, paragraphs 119 to 125 above, if the classification of sand as regards its powers of safely withstanding percolation without piping is known.

282. Classification of river bed sand —If $\frac{1}{C}$ is the steepest slope of the percolation gradient which ensures safety against 'piping' in any particular soil, then the length (l) of the shortest line of percolation under a head (H) should not be less than $H \times C$.

Bligh has divided river bed sands into four classes and assigned to each class a value for C as below:--

Class I.—Light sand and mud (as in the Nile) C = 18.

Class II.—Fine micaceous sand (as in the Himalayan rivers) C = 15.

Class III.—Coarse-grained sands, as in the Central and South India (this is the commonest variety) C = 12.

Class IV —Boulders or shingle and gravel and sand mixed C varies from 9 to 5

Using the above classification the length of impervious apron necessary to prevent 'piping' and the distribution and intensity of up-lift pressures can readily be computed.

From this, the weight and the thickness of any impervious apron necessary to resist uplift can be calculated.

283. Formulæ for computing widths of aprons.—From the above considerations, and taking the Narora weir as a standard, Bligh has evolved a series of equations giving data for computing suitable widths and thicknesses of different parts of the aprons of weirs founded on sandy soils.

These formulæ have been tested by comparison with existing successful works and found to generally agree with such standards.

Bligh's formulæ with some slight alterations have been adopted in this manual and are given below, the notations used being—

l = shortest length of percolation.

l_1 = length of percolation for computing uplift pressures on the apron (vide chapter IV, paragraphs 123-124).

C = the classification number of the bed sand.
H = The static head under the condition under investigation, that is, the difference of level ot the water up and down-stream.
H_s = Difference of level between the crest of weir shutters and the surface of the main apron.
H_b = Difference of level between crest of masonry body wall of weir and lowest water level.
H_s = Difference of level between the top of the crest shutters and lowest water level.

NOTE.—Where there are no weir shutters H_b will be substituted for H_s in any formula.

q = computed maximum discharge in cusecs per foot run of weir crest.

(*a*) *The minimum percolation length* is found from the equation

$$l = C H_s \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (38)$$

(*b*) *The width of the main (impervious) apron* (W) down-stream from the body wall is a proportion of 4C varying as the root of H_b and taking Narora weir (H_b = 10 feet) as a standard.

$$W = 4C\sqrt{\frac{H_b}{10}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

(*c*) *Total width of apron down-stream of the body wall.*—The total width (L) of the rear apron (viz., the main masonry apron *plus* the loose apron) would be affected by the height of the obstruction (H_b) and also by the discharge (q) per foot run over crest of weir.

From this is framed the formula for type A weirs.

$$L = 10\,C\sqrt{\frac{H_b}{10}} \times \sqrt{\frac{q}{75}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (40)$$

For type B or C weirs L should be increased by 10 per cent.

The formula represents that L varies as the square root of H_b and of q which is not unreasonable, but this and the last equation are admitted to be purely empirical.

Bligh gives a list of a number of existing Indian weirs and shows that the actual value of L agrees fairly with the results given in equation

The width of the loose talus is the difference between L and W, and as the width of any impermeable floor required up-stream of the body wall is reduced by increasing W, it will therefore occasionally be economical and desirable to increase W beyond the width given by equation (39), thereby correspondingly reducing *both* the width of the talus and of the up-stream apron.

284. Thickness of aprons.—When considering the question of thickness of aprons it will be realized that there will be a practical minimum thickness required for stability under the action of falling or rushing water while the main apron which is impervious masonry (types A and B) must have such additional thickness as may be necessary to render it safe against blowing up by the uplift pressure acting on its undersurface.

The minimum thickness.—

(1) The main apron will be generally not less than 3 to 4 feet thick the latter being a minimum for type A in respect to the part of the apron on which the overfall impinges.

(2) The dry packed apron or talus will vary from 3 to 5 feet in thickness according to the nature of the river bed and velocity of

current and type of weir. Weirs of type B require a heavier talus than type A.

(3) The up-stream apron, as a rule, should consist of a layer of impervious clay over which should be packed a protective layer of rough stone or concrete blocks or brick of about 1 to 2 feet in thickness.

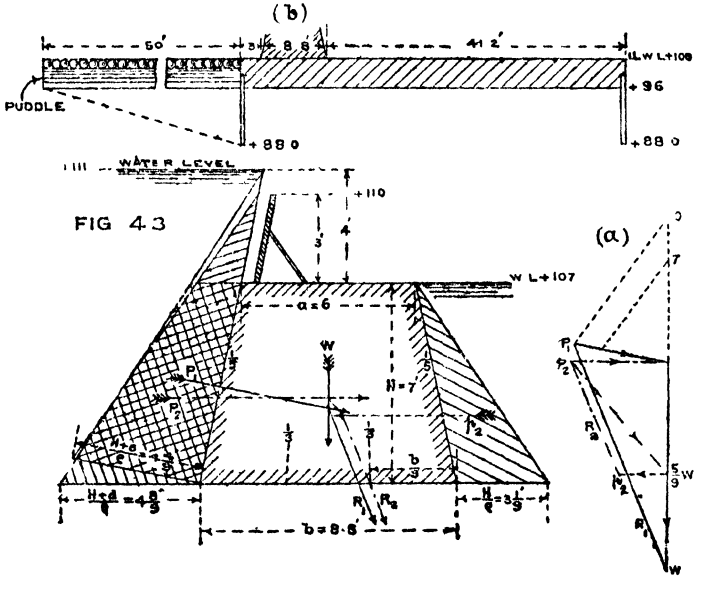
If a solid masonry apron is used up-stream of the body wall it may be 1½ feet thick or upwards.

The body wall design may follow the lines of the design of weirs on rock foundations with the exception that allowance for flotation should always be made.

285. Uplift pressures on the main apron.—There remains to compute the extra thickness, if any, required for the main apron to withstand uplift, and this is best done by first drawing a section of the weir having dimensions computed from the considerations and equations above stated, and then testing the aprons so designed for uplift stability, making any alteration shown by such examination to be desirable.

The section of the weir required for this examination comprises the body wall, main apron and impervious up-stream floor, together with any lines of piles or any wells proposed as foundation work below the bottom surface of the impervious floors.

286. Example of weir design.—An actual case will be now taken up as an example:—A weir is to be constructed on coarse sand (C=12). The weir wall will be of rubble stone masonry 7 feet high above L W.L. surmounted by 3 feet falling shutters.



The weir will be of type A with the main apron surface at L.W.L. The maximum discharge per foot of weir is computed at 70 cusecs, and the maximum depth of water passing over the crest

at 8 feet and at such time the afflux is 2½ feet; the weir becomes submerged when 4 feet of water is passing over the crest

In this case $H_n = 10$ feet, $H_b = 7$ feet, $H_s = 10$ feet, and $q = 70$.

287. The body wall.—The body wall requires a 6 feet top width to leave suitable space for working the falling shutters.

The weir will be built with the same slope on both faces and the base width should be that required to bring the resultant pressure under states I or II to the down-stream extremity of the middle third

The required base width in each state may be calculated as below:—

Here $H_b = 7$, $a = 6$, $S = 3$, $d = 4$ and $\rho = 2\frac{1}{4}$

State I.—Substituting the above values in formula 35-*b*, paragraph 268—page 134,

$$1/12\left\{[(2\tfrac{1}{4} + 1\tfrac{1}{2}) \times 7 + 2\tfrac{1}{2} \times 3]\, b^2 + 6\,(2\tfrac{1}{4} \times 7 - 7 - 3)\, b - 18\,(7 + 9)\right\} = M_0 = \frac{(7+3)^3}{6}$$

$$33{\cdot}75\, b^2 + 34{\cdot}5\, b - 288 = 2{,}000$$

$$b^2 + 1{\cdot}02\, b + (0{\cdot}51)^2 = \frac{2{,}288}{33{\cdot}75} + 0{\cdot}26 = 68{\cdot}06$$

$$b = 8{\cdot}25 - 0{\cdot}51 = 7{\cdot}74 \text{ ft.}$$

State II.—Substituting the above values in formula 36-*b*—

$$\frac{7 \times \frac{b}{4}}{12}\left\{b^2 + 6b\right\} = M_0 = \tfrac{1}{2}\, d\, H_b{}^2$$

$$b^2 + 6\,b = \frac{4 \times 7 \times 7 \times 12 \times 4}{2 \times 7 \times 5}.$$

$$b^2 + 6\,b + (3)^2 = 134{\cdot}4 + 9 = 143{\cdot}4.$$

$$b = 11{\cdot}98 - 3 = 8{\cdot}98$$

For stability, state II requires the widest base and a suitable slope is one in five giving a base width of 8·8 feet which will be adopted

A stability and a force diagram (figures 43 and 43-*a*) are for purposes of illustration drawn for both States I and II instead of only for the latter—

State I.—The forces are P_1 and W and the resultant R_1.

State II.—The forces are P_2, ρ_2 and W and the resultant R_2.

The diagram confirms the result obtained by calculation and the profile is suitable

288 General proposals for foundations and apron.—It is proposed to found the body wall on a platform of concrete being the up-stream continuation of the main apron and not less than 4 feet thick. This will extend 3 feet up-stream of the face of the wall and 1 foot from the up-stream and down-stream ends two rows of steel or ferro-concrete, sheet piling will be provided to be driven to 12 feet below L W.L

The percolation length $l = CH_s = 12 \times 10 = 120$ feet

From equation (39) the length (W) of the main apron is—

$$W = 4\,C\sqrt{\frac{H_b}{10}} = 48\sqrt{\frac{7}{10}} = 40{\cdot}3.$$

Making the floor 41·2 feet, this with the base of the body wall will give a length 41·2 + 8·8 = 50 feet from the up-stream heel of the body wall to the down-stream end of the main apron. The above as will be seen from figure 43-*b* will give a percolation length

4 + 53 + 12 feet = 69 feet. This makes no allowance for the up-stream piles, and leaves an additional length of (120 − 69) = 51 feet to be provided. The up-stream apron may be made 50 feet being 2½ feet of puddle clay with a covering of 1½ feet dry packed rubble stone.

The percolation length $l = 4 + (50^2 + 8^2)^{\frac{1}{2}} + 53 + 12 = 119{\cdot}6$ feet.

The effect of the up-stream piles in lengthening the line of percolation is insignificant being $(\sqrt{50^2 + 8^2} - 50) = 50{\cdot}6 - 50 = 0{\cdot}6$ feet.

The piles would theoretically be better placed at the up-stream end of the apron so far as percolation is concerned, but placed as shown they afford protection to the body wall foundations in case of scour displacing the apron and puddle in any part of the length of the work. The whole width (L) of the down-stream apron is found from equation (40)

$$L = 10C\sqrt{\frac{Hb}{10}} \times \sqrt{\frac{q}{75}} = 120\sqrt{\frac{7}{10}} \times \sqrt{\frac{10}{75}} = 120 \times {\cdot}8$$

= 96 feet.

The down-stream loose apron = 96 − 41·2 = 55 feet

289. Testing the main apron for uplift by a diagram.—It is now necessary to plot the section of the impervious weir floor and draw the hydraulic gradient representing the pressures, on each part of the down-stream impervious apron

A profile of the work is drawn figure 44 and below it figure 44-*a*, the diagram of the hydraulic gradient and effective weight of the flooring

This only affects the impervious part of the floor from *a a*, the up-stream end of the up-stream aporn, to *c c*, the down-stream end of the main apron

The security of the latter from uplift by the upward pressures developed on its lower surface will now be examined

These pressures should be counterbalanced by the weight of the masonry of the apron with a margin for safety of about 20 per cent

Construct figure 44-*a* showing the upward and downward pressures on the section of the work; this figure is drawn on a base *a n* representing low water level, for greater clearness the vertical scale is made twice the horizontal.

Mark off *a c* = 103 feet the length of the impervious floor

In prolongation of this length commencing from the down-stream end, mark off in order from *c e* the vertical heights of the obstructions below the river deep bed level, which is in this case L W.L; that is make *e h* = 12 feet, viz, the thickness of floor, plus the depth of piles at *e e'*.

Make *h m* = 8 feet = projection of the pile at *b* below the under surface of floor.

Make *m n* = 4 feet = depth of floor at up-stream end *a a* Erect the vertical *a a'* equal to (H_8) 10 feet Join *a' n* This line gives the *mean* slope of the hydraulic gradient

From *m* draw a line parallel to *a' n* cutting verticals from *a* and *b* in *a''* and *b'* respectively.

From *h* draw a similar parallel cutting verticals from *b* and *e* in *b''* and *e'*

Then the hydraulic gradient is the firm line a', a'', b', b'', e', e.

Owing to the downward pressure of water up-stream of the shutter, there is no resultant upward pressure up-stream of line $e\ e'$ but down-stream of this, the level above L.W.L. of any point in the gradient line gives the pressure head at the corresponding point on the underside of the floor and this upward pressure must be counterbalanced by the weight of the floor at that point.

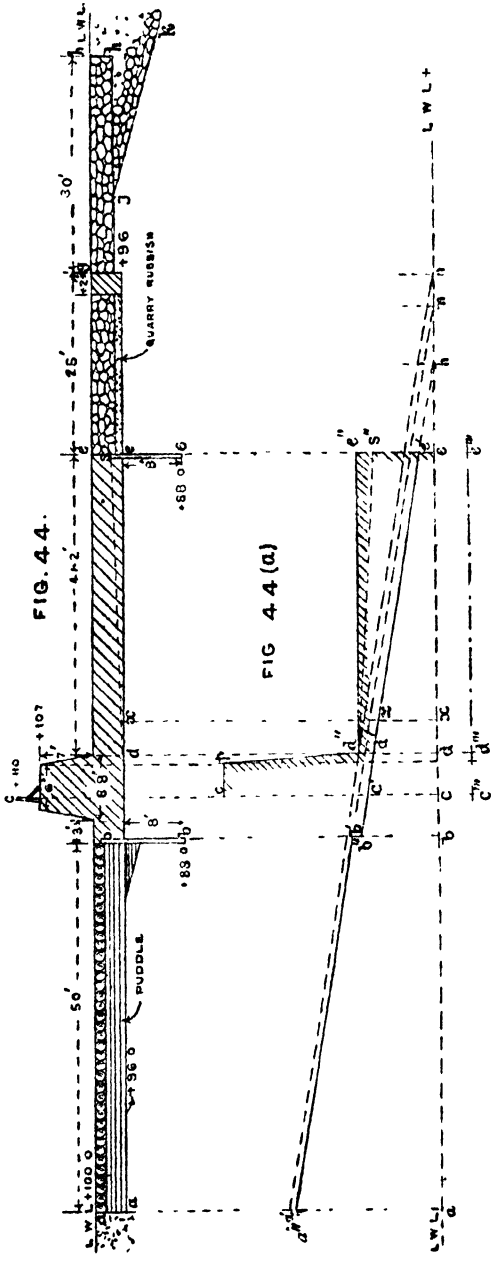


FIG. 44.

FIG. 44 (a)

290. Effective weight of the masonry of the apron—In plotting the weights of the masonry available to balance the upward pressure it is important to remember that the whole weight per cubic foot of any masonry above the down-stream water level of the condition under examination (generally the L.W.L.) as compared with water is (ρ), the weight of any masonry lying below that level is by flotation, that is by uplift, reduced to $(\rho - 1)$

In the case under discussion $\rho = 2\frac{1}{4}$ and $(\rho - 1) = 1\frac{1}{4}$ and all the masonry below floor level is also below L.W.L., and therefore weighs $1\frac{1}{4}$ compared with water and that above L.W.L., as in the bodywall weighs $2\frac{1}{4}$ per cubic foot.

291. Diagram of effective weight of apron.—The net weight of masonry lying above each point between c c

and $e\,e$ is therefore now plotted in figure 44-*a* as vertical lines above the base which is at L.W L.

$$c\,c'' = (7 \times 2\tfrac{1}{4} + 4 \times 1\tfrac{1}{4}) = 20\tfrac{3}{4}.$$
$$d\,d'' = e\,c'' = (4 \times 1\tfrac{1}{4}) = 5.$$

and the height of each point on the resulting line $c''\ p\ d''\ e''$ above the base represents the effective downward pressure of masonry in terms of feet of water heights, or what is the same thing the level of each point on it is the pressure head which the masonry can balance and this should be above 20 per cent above the head as shown by the hydraulic gradient It is unnecessary in practice to draw the weights of the bodywall or any other walls where it is obvious that these provide adequate weight; this has been drawn in this case merely for the purpose of clear demonstration of the method.

The effective weight of the floor being uniform all along the line $d\ e$ and the upward pressure gradient being sloping the critical section is d at the toe of the bodywall The diagram shows a margin of excess length of $d'\ d''$ over dd' slightly more than 20 per cent; and hence the weight of the floor is adequate.

At $e\ e''$ the weight of floor is excessive and might be reduced if the limit of thickness imposed by other considerations were not specified as 4 feet

292. Balancing head of a masonry apron —The thickness actually required to resist uplift would in this, and in most cases, give a trapezoidal section to the floor. In this case it would not be unreasonable to make the floor thickness diminish gradually from 4 feet at d to 3 feet at e by sloping the uuderside of the floor upwards as shown by the dotted line $d\ s$. In such case the line of balanced heads would be $d''\ s''$ (figure-44-*a*)

Reference to the above figure shows that, as the hydraulic gradient $c'\ e'$ represents vertical heads at each point, viz.:—the level to which water would rise in a pipe passed through the floor at that point, the counter balancing weight gradient $c''\ p\ d''\ e''$ similarly represents at each point the level of water which would just balance the effective weight of the masonry at that point, and may be termed the balancing head of the masonry.

293. Alternative diagram of effective weight of apron.—If the level of the under surface of the impervious flooring were laid at a level above $d'\ e'$ throughout no upward pressure would come on the floor.

The pressure thus depends not only on the floor weight but on its level.

The base level in this pressure diagram has been taken as L W.L., whereas the underside of the floor is 4 feet below this.

This has been compensated for by deducting 4 feet head for 'flotation' from the total weight of masonry lying below L.W.L ; and the result would have been precisely the same if the full weight of masonry had been allowed and the underside of this floor plotted at its true level $c'''\ e'''$; because $c\ c'''$ and $e\ e'''$ each are equal to the thickness of the floor in feet and the lines d'' d''' and $e''\ e'''$ are each equal to $4 \times \rho$; that is no deduction is to be made for flotation if the weight of masonry is plotted on a base line coinciding in level with the underside of the floor.

In the case of weirs of types A_2 and B this method of plotting is frequently the simplest and when adopted *no deduction is made for flotation.*

294. Combined effect of level and weight of apron on its 'balancing head'.—The clear grasp of the influence of the level of the underside of the floor is of importance in computing the effective weights of floors which are wholly or in part above the down-stream water level which is the base line from which the hydraulic gradient is plotted. In every case what has to be considered is the 'balancing' water level due to the combined effect of the level and weight of the floor with reference to the *virtual* water level given by the hydraulic gradient.

295. Design of talus and 'inverted filter'.—In respect to the profile in figure 44 of the above example a few remarks are now added in regard to the loose apron (the talus). It will be seen from the section that this has been divided by a longitudinal dwarf wall 2 feet wide into 2 parts, 25 and 30 feet wide respectively The up-stream part is provided with 3 feet stone packing over one foot of quarry rubbish while the second part is 3 feet of packed stone only.

The section shows the settlement of the tail of the latter which would gradually be caused by scour down-stream of the toe of the apron. The stone of this tail apron would be added to from time to time, if it appeared desirable, the necessity being judged from periodic surveys and soundings of the depth and positions of the scours below the weir.

The up-stream section is expected to remain intact and the quarry rubbish acts so as to quickly dissipate any remaining pressure at the down-stream toe of the main apron, also checking any tendency to piping or blowing of sand upwards through the open stone packing This arrangement is similar to that used in a sand filter bed to prevent sand being removed by percolation but is inverted, the sand being in this case below and not above the gravel and for this reason it is termed 'an inverted filter.'

The arrangement is economical and effective and constitutes an additional safeguard against 'piping.' Broken brick or stone, shingle or gravel are also suitable materials for an inverted filter

296. Design of up-stream apron.—The up-stream apron is 2½ feet of puddle, covered by 1½ feet of dry rubble The puddle is thickened at the junction with the piles to make a good joint. This is *most* important as failure in this respect would make the up-stream apron useless so far as percolation and uplift is concerned.

297. Example of a weir of type A_2.—An example of a weir of type A_2 is now given.

The conditions are assumed to be as follows :—

Body wall crest above L.W.L. = H_b = 12 feet (vide paragraph 283); crest shutters 4 feet high : therefore H_s = 16 feet, crest width 7 feet.

Equal face slopes of ¼ up and down-stream.

This proposed profile (figure 45) may be tested as was done in the previous example ; it is here *assumed* to be suitable. The main

apron floor is 7 feet below the body wall crest; the minimum thickness of the main floor is tentatively fixed at 4 feet.

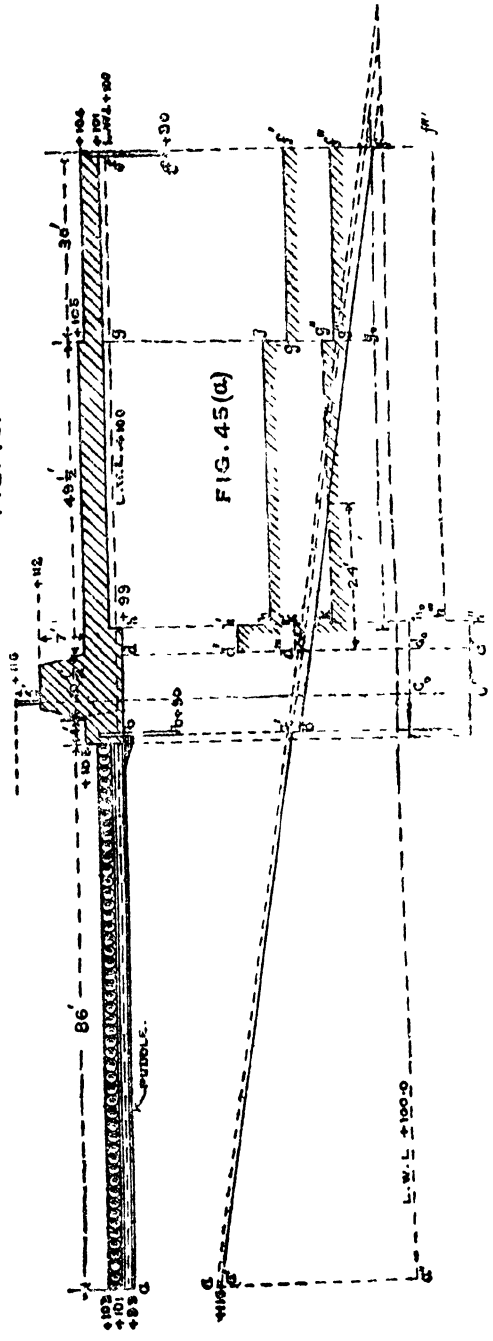


FIG. 45.

FIG. 45(a)

The value of the maximum computed flood discharge of the weir per foot run is 75 cusecs = q. The weir is to be built on coarse sand (class C = 12). The L.W.L. and deep bed of the river is at level + 100 and the type of foundations will be similar to the last example, the details being as in the profile (figure 45)

The minimum length of percolation is found from equation (38), $I = 12 H_s = 12 \times 16 = 192$ feet.

The minimum length of main apron (W) from equation (39)—paragraph 283

$$W = 4C\sqrt{\frac{H_s}{10}}$$

$$= 4 \times \sqrt{12} = 52{\cdot}58, \text{ say, } 52\tfrac{1}{2} \text{ ft.}$$

The length of the main apron is here only $49\frac{1}{2}$ ft as the length is prolonged by an impervious apron 30 feet long and 3 feet thick. This prolongation saves 30 feet of talus and also 30 feet of up-stream apron, and is therefore not extravagant while giving some little additional security The total length (L) of the down-stream apron from equation (40)—paragraph 283.

$$L = 10C\sqrt{\frac{H_s}{10}} \times \sqrt{\frac{q}{75}}$$

$$= 10 \times 12\sqrt{\frac{12}{10}} \times \sqrt{\frac{75}{75}}$$

$$= 132 \text{ feet}$$

Therefore the width of talus is (132—79½) = 52½ feet. From figure 45 the length of underside of the impervious main apron as prolonged under the body wall is

(14½ + 49½ + 30) = 94 feet
The down-stream piles = 10 ,,

Total . . 104 ,,

Allowing, 1 foot for the up-stream piles and 1 foot for the depth of the bottom of the puddle apron below the deepest bed level the length of puddle apron up-stream must be 192 — (104 + 2) = 86 feet.

The profile of the whole impervious apron with piles and body wall must now be completed (figure 45) and the diagram of hydraulic gradient a, a', b', b'', f_0, f showing virtual water levels must be drawn (figure 45-*a*) as in the previous case on a base at L.W L.

The actual level of the underside of the aprons $c_0\ h_0\ h\ f_0$ is plotted in the diagram and in parallel lines (chain dotted) drawn at a distance from it of the thickness of the base multiplied by (ρ), giving the lines $d'' h'$, $h\ g$ and $g' f'$ which show the balancing water levels due to weight of aprons.

These show that the apron is amply safe with a good deal of margin.

298. The up-stream apron and the talus —The up-stream apron has been fixed at a level a little below L W L. so that the puddle clay might never become dry as puddle in this condition cracks and becomes ineffective

The talus of this weir has not been plotted in the section.

299. Comparison of types A_1 and A_2 of weirs —If the weir had been type A_1 and the upper main apron surface had been at L W.L , the underside of the aprons constituting the base would have been on line $c''' h'' h' ' f'''$, 5 feet lower than plotted, and the balancing levels would be obtained by plotting the full weight of masonry apron and water overlying it, on the base $c''' h'' h''' f'''$.

For the 4 feet apron the weight ordinate would be 4ρ and for the 3 feet apron $(3\rho + 1)$ as 1 foot of water overlies the apron surface. This balancing level is plotted (figure 45-*a*), viz , the lines $d''' k' k g''$ and $g''' f''$ (chain dotted).

These show that *under these modified conditions* for a length of about 24 feet from the body wall the main apron would require additional thickness without making any allowance for factor of safety.

This brings out clearly the effect of *level* of the apron on the uplift pressures and the advantage in having the apron above L.W.L.

Considerable saving in pumping and contingent expenses in actual construction results from being able to lay the apron without unwatering A weir of type A_2 also has the advantage over type A_1 that there is a smaller vertical fall on to the apron; for these reasons the former type is frequently preferable to the latter. On the other hand the raising of the floor restricts the waterway over it and thus increases velocity at the down-stream toe of the work.

300. Two methods of drawing diagrams of uplift forces.—Two methods of plotting pressure diagrams have been above described —

(*a*) That of plotting both upward water pressure and equivalent downward apron weights from a common base at the level of the down-stream water of the condition under examination (generally L W L.) and allowing the masonry weights as ρ or $(\rho-1)$, respectively, for each unsubmerged and submerged foot of masonry

The method is not of universal application and is unsuitable unless the down-stream impervious aprons are horizontal and either (1) wholly submerged or (2) with the underside horizontal and at the level of the down-stream water

(*b*) That of plotting the upward water pressure as in (*a*) but plotting the weight of the impervious floor on a base at the level of the under surface of each length of that floor

In this method the length of the weight ordinate at any point should be ρ multiplied by the thickness in feet of the masonry of the floor *plus* the depth of water standing over the surface of the floor at this point

This method is of universal application but a little less convenient than (*a*) for wholly submerged horizontal aprons.

301. Computing uplift without drawing a diagram—It is possible to verify the safety of a weir against uplift from the section of the work without drawing a separate diagram

Referring to the last example (figure 45) the percolation length l_1 from the section of the work is—

$$l_1 = (1 + 86 + 9 + 14\tfrac{1}{2} + 49\tfrac{1}{2} + 30 + 10) = 200 \text{ feet.}$$

The mean hydraulic gradient is therefore $\frac{16}{200}$ or $\frac{1}{12\,5}$. Starting from L W L at the up-stream end of the apron, where the maximum head is 16 feet and working along the underside of the base at every $12\frac{1}{2}$ feet of percolation length the uplift head is reduced by one foot with reference to the level of L W.L. as datum level The uplift head can thus be computed for the critical points in the profile, viz., vertically below the down-stream toe of the body wall (*d*), at any change in thickness of the apron (*h*) and (*g*), and at the tail end just up-stream of the piles (*f*). Thus in figure 45 the uplift heads reduced to the level of L W. are—

At (*d*) the uplift head is $16 - \frac{1 + 86 + 9 + 14\frac{1}{2}}{12\,5} = 16 - \frac{110\,5}{12\,5} = 7$ feet (about) The masonry apron is here 6 feet thick of which 1 foot is below L.W.L., and therefore subject to flotation The uplift head it could balance above L W.L is therefore $(5 \times 2\frac{1}{4} + 1 \times 1\frac{1}{4}) = 12\frac{1}{2}$ feet. There is thus ample thickness against uplift.

At (*h*) the uplift head is $\left(16 - \frac{116\,5}{12\,5}\right) = 6\frac{1}{2}$ feet (about) from L.W.L As the underside of the floor is one foot above L.W.L. the uplift head on it is only $5\frac{1}{2}$ feet. The thickness of masonry above (*h*) is 4 feet and the balancing head is $4 \times 2\frac{1}{4}$ feet $= 9$ feet. The floor is therefore amply thick.

At (*g*) the uplift head on a surface at a level one foot above L.W.L is $\left(16 - 1 - \frac{162}{12\,5} \right) = 2$ feet The balancing head due to masonry is $2\frac{1}{4} \times 3 = 6\frac{3}{4}$ feet.

At (*e*) the uplift head on the under surface of the apron would be $\left(16 - 1 - \frac{192}{12\,5} \right) = -\frac{9}{25}$ that is, there is no uplift

302. The floors and bank connexions of all works holding up a head of water to be designed for security against piping.—The methods described above of designing weirs of type A on permeable soil so as to provide a platform of a length which will give a safe percolation gradient, that is, a sufficient length of *creep* and also so as to make the flooring safe against uplift must similarly be applied to other types of weirs and also to other irrigation works which hold up the water above them to a higher level than that of the water below. These works include regulators, sluices and drops.

Not only must the length of *creep* under the floor be sufficient, but a hydraulic gradient suitable to the marginal and embankment soils must be provided round the wings and bank connexions so as to ensure a sufficient minimum length of *creep* against failure by percolation round the flanks of the works having in view the maximum head to which the work will be subjected

303. The Palar anicut (North Arcot district)—The Palar anicut breached badly in 1903 under the action of the heaviest recorded flood in that river and a considerable length of anicut was carried away. Figure 46 gives a section of the anicut before it breached, and figure 46-*b* the section after restoration

From the levels and dimensions given in the figure we get

$$H_t = H_b = 10.$$

The maximum discharge per foot run of crest is computed at 97 cusecs.

The class of sand gives C = 12. If we assume that the grouted rubble forming the main apron of the original profile which failed is *an impervious floor* the value of *l* for the profile is 91 feet against 120 feet required as found from formula $l = CH_b$. The length of the main apron (W) is 46 feet against 48 feet given by equation (39) paragraph 283, and 50 feet given by the new profile

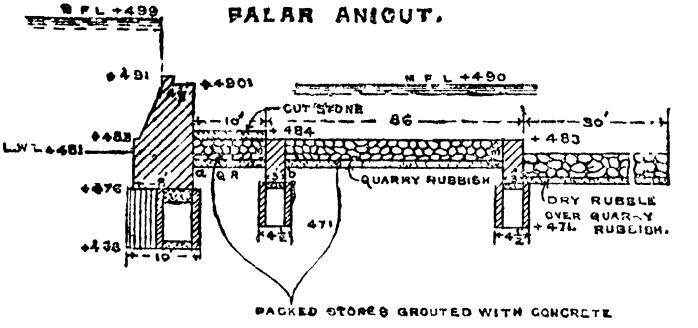
The total width (L) of the whole apron downstream of the body wall is 76 feet, compared with 136 feet given by equation (40), and 100 feet given by the new profile.

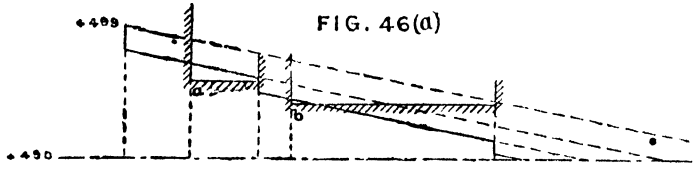
Examining the uplift conditions it must be observed that an apron grouted from the top after laying cannot be a satisfactory impervious apron as *at the best* the water penetrates to three-fourths thickness of the apron before it meets the impervious grouting. In this examination, however, the grouted apron is assumed as masonry of specific gravity $2\frac{1}{4}$.

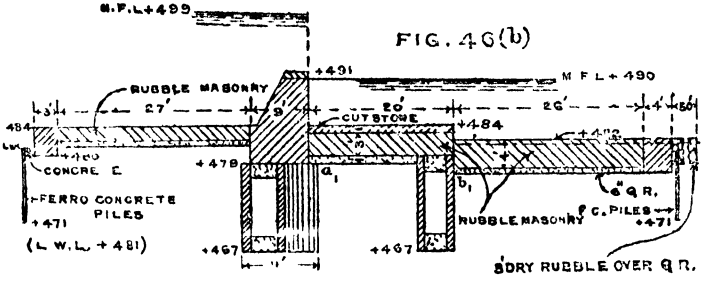
The actual head observed during this flood was 9 feet according to the up and down-stream levels reported which are shown on the profile.

The total gradient length (l_1) is 91 feet giving a hydraulic gradient of $\frac{1}{10}$. From this the uplift head on the under side of the apron at the down-stream toe of the body wall and with reference to a base at +490 (the level of the water below the work) is $9 - \frac{25}{10} = 6\frac{1}{2}$ feet The apron is submerged and has weight $(\rho - 1) = 1\frac{1}{4}$ units per cubic foot and the balancing head of the apron is $1\frac{1}{4} \times 4 = 5$ feet Thus even with a solid masonry floor the apron would not have been safe against uplift

FIG. 46.







Similarly the uplift head at b is $(9 - \frac{48}{10}) = 4{\cdot}2$ feet. The balancing head is $1\frac{1}{4} \times 3 = 3{\cdot}75$ feet and is insufficient. The same thing is shown graphically in figure 46-*a*. This is quite enough to account for the failure of the weir apart from 'piping'.

Possibly however that both causes contributed to failure. This is a case where the submergence of the aprons makes all the

difference between safety and danger as the apron of this weir would be safe against uplift when, as is in fact usual at this anicut, little or no water is passing over the crest.

Examining the new profile for uplift at a_1 and b_1 the percolation length $l_1 = 39 + 14 + 20 + 13 + 30 = 116$ feet In computing this the depth of the wells only has been counted, the piles being omitted from the computation as it is not considered reasonable in this special case to add the depth of all the vertical barriers The hydraulic gradient is $\frac{9}{116} = \frac{1}{13}$. The uplift head at a_1 with reference to the down-stream water level (+ 490) is $\left(9 - \frac{53}{13}\right) = 5$ feet

The balancing head is $1\frac{1}{4} \times 4 = 5$ feet The apron is just sufficiently thick to resist uplift without any margin At b_1 the uplift is $\left(9 - \frac{86}{13}\right) = 2\frac{1}{2}$ feet.

The balancing head is $1\frac{1}{4} \times 4 = 5$ feet and this is amply strong.

The design may be criticised in the light of the formulated system of design as follows :- The lower main apron might be 3 feet and the upper one 5 feet thick and the masonry up-stream apron might be replaced by puddle covered with dry stone which would be equally effective against percolation and less expensive. The piles were put in chiefly as a protection against scour by up-stream cross currents to which the failure of this anicut has been attributed by some engineers.

The inverted filter made by a layer of 1 foot of quarry rubbish under the talus may be noted This practice has been employed in Madras for many years and is also noticeable in the old profile The quarry rubbish below the impervious aprons does not appear desirable as being likely to increase creep on the underside of the floor

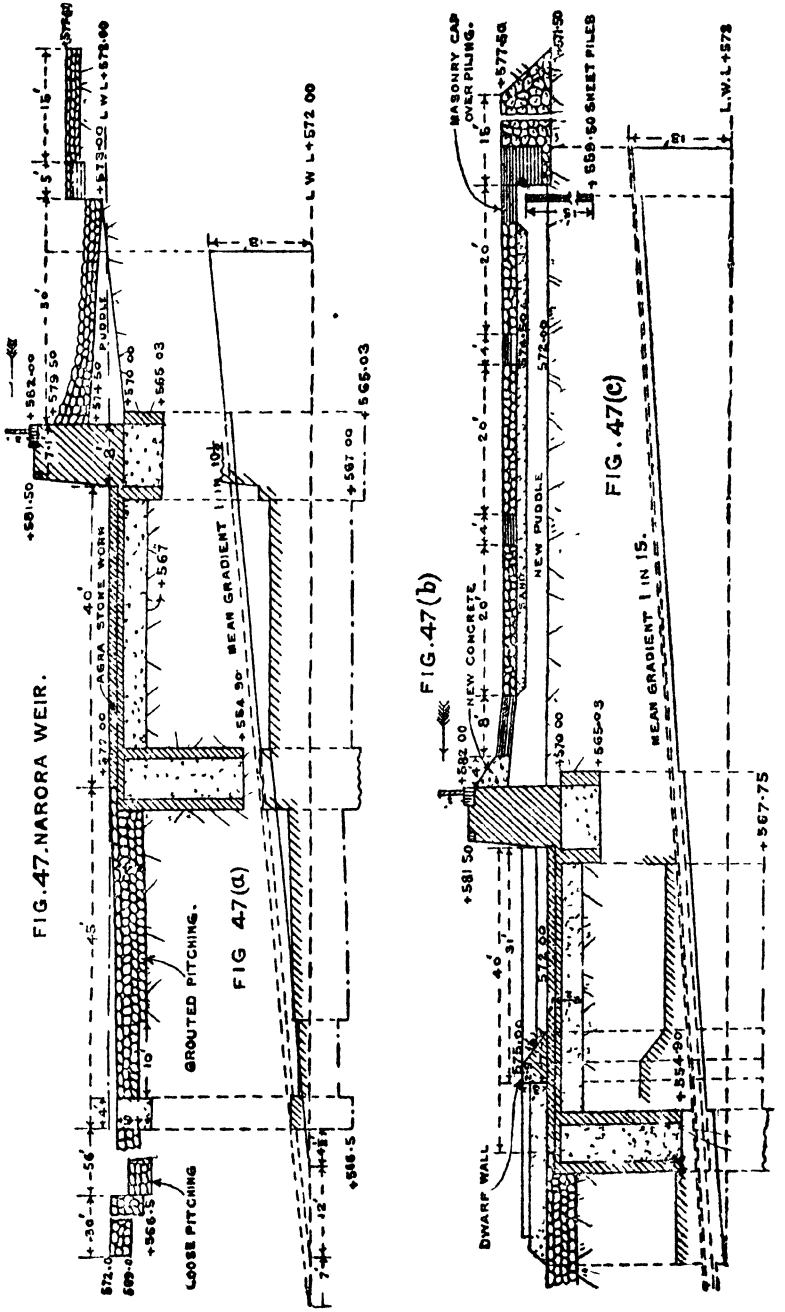
304 Condition of greatest uplift on weir aprons --In weirs of type A_2 the condition of greatest uplift is frequently that at which the tail water is just at the level of the main apron surface and this is generally, if not always, the case when there are no crest shutters. The uplifts examined for the Palar anicut under such a condition must approximate to those above computed

305. Failure and restoration of Narora weir --This weir failed in 1898 by the 'blowing up' of 350 feet of the main apron Part of the pitched and grouted apron was blown up at the same time. At the time of the failure the stability of the floor was under investigation, pipes passing through the main apron which had been inserted a few days before, showed an unsafe uplift pressure on the underside of the floor.

The weir as originally constructed is shown in figure 47 ; it is provided with crest shutters 3 feet high.

At the time of the failure the up-stream apron with the underlying puddle had been disturbed in places so that it was giving little protection to the work against percolation.

Pressure diagram, figure 47-*a*, shows the condition before the up-stream apron was displaced. The masonry is brickwork (sp. gr 2) and the grouted pitching has been counted as of sp. gr. 1¾.



The weir is built on fine sand (C = 15).

The diagram shows that the apron was insufficiently thick and that the mean percolation gradient was somewhat steep even with an intact up-stream apron.

The displacement of the puddle brought matters to a crisis and the main apron failed by uplift.

The restored weir, figure 47-*b*, and its pressure diagram, figure 47-*c*, show the present condition of the weir which has shown no further signs of weakness.

It will be noticed that the down-stream impervious apron is shorter than before failure, thus reducing pressure on the down-stream main apron while the increased length of percolation is secured by the increased length of the up-stream apron. An extra weight of main apron is secured by the water cushion now provided.

The studies made regarding the failure of this weir led to investigations by Clibborn, Beresford and other engineers on which is based the method of design of weirs on sand described in this chapter and first formulated by Bligh in his "Practical Design of Irrigation Works."

306. The Lower Chenab weir at Khanki examined as an example of type B weir.—This is a low weir with 6 feet crest shutters, the crest of the body wall being only 7 feet above L.W.L. It stands on sand of grade C = 15.

The work failed three years after construction by the collapse of the main apron

The profile (figure 48) gives the section subsequent to restoration.

Prior to the collapse of the main apron the up-stream apron consisted of dry stone packing 50 feet wide and 2 feet thick without any puddle or other impervious floor material.

In restoration the sloping part of the talus of dry stone was increased from 3 to 4 feet thick and grouted while a new up-stream apron was constructed on a puddle base 50 feet wide and a line of wells 8 feet wide and 20 feet deep with their tops 7 feet above L.W.L

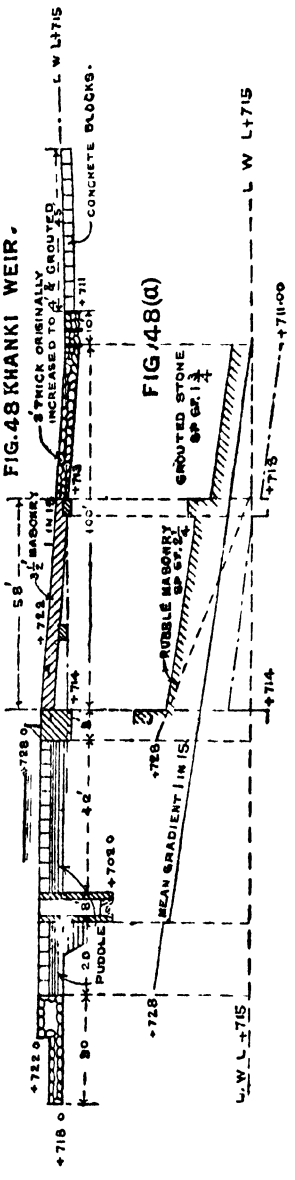
The additions have been successful and the weir has worked satisfactorily for the last fifteen years.

In the pressure diagram, figure 48-*a*, the grouted rough stone apron is taken as effective with a sp. gr of $1\frac{1}{4}$ The masonry apron has a sp. gr. $2\frac{1}{2}$.

The diagram shows in dotted lines the hydraulic gradient before restoration with a mean slope of $\frac{13}{65} = \frac{1}{5}$ (nearly) and after restoration in continuous lines with a mean slope of $\frac{13}{193} = \frac{1}{15}$ (approximately).

The original distance was clearly utterly inadequate and the weir probably failed by 'piping,' the sand being washed out from under the masonry apron which collapsed.

It will be noted that in the original profile the apron weight was sufficient against uplift. The balancing level of this apron is shown hatched in the diagram.



In the present profile it leaves a large margin of safety.

307 The type C or "rock-fill" weir.—This type of weir is common throughout South India and rests on all kinds of foundations, rock, clay and sand, etc., and is formed across rivers and streams of all magnitudes Many of these works are of great antiquity and the type, being merely a barrier or bund of rough stone with a flat down-stream slope, is one which would naturally be evolved in the earliest stages of weir construction. The width of the works vary with the nature of the river bed, the more easily the bed is eroded or undermined by percolation the wider the work

On a hard rock foundation such a dam would generally not be as economical as a masonry weir wall, while on a clay or soft rock foundation weirs of type A would generally in these days be cheaper both to construct and maintain; the same remark is applicable even to sand foundations Type B or type A weirs are generally cheaper to construct and almost always cheaper to maintain than type C.

In the past, many and important type C weirs have been constructed and irrigation engineers, in South India, have to deal with this type of anicut in existing works more often than with any other type.

In its inception it is a rough stone *bund* with the surface of heavy stones carefully packed and wedged up to prevent displacement. In many old works the crest stones have been tied together by iron cramps.

In most, but not by any means in all of the old works there is a body or face wall of rubble or brick in mortar which serves to prevent percolation.

If the work is on an impermeable foundation, such a wall will altogether stop leakage, but on permeable foundations as in the case of type A or B weirs it will only be checked.

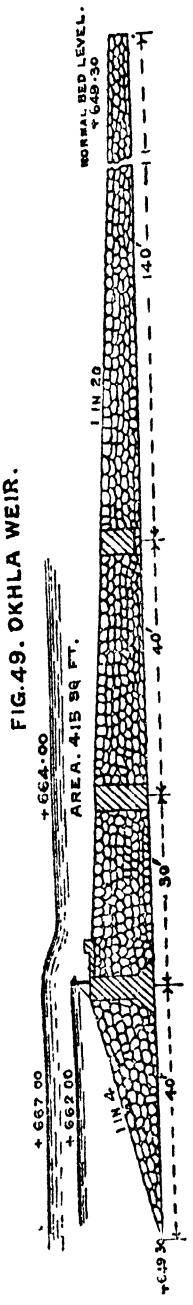


FIG. 49. OKHLA WEIR.

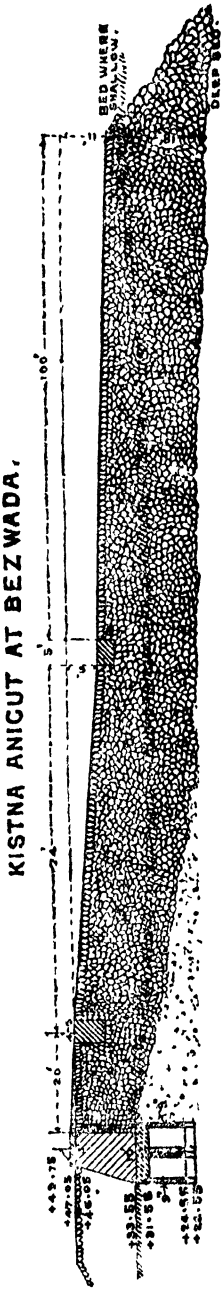


FIG. 50. KISTNA ANICUT AT BEZWADA.

In modern works of type C a body wall is always built and generally also one or more curtain walls are provided parallel to, and downstream of, the body wall, thus dividing the width of the apron into compartments These walls serve to check percolation and to localise damage by checking the extension of injury caused by displacement of any of the surface stones of the apron.

308 Body wall of a type C weir—The practice in respect to the foundations of the body and curtain walls varies; in some cases, as in Okhla weir (figure 49), no excavated foundations have been given and the walls are built up from the level of the deep bed of the river; in others, as in Bezwada and Sangam weirs (figures 50 and 51), either the body wall or curtain walls, or both, have well foundations carried to a considerable depth below the river bed

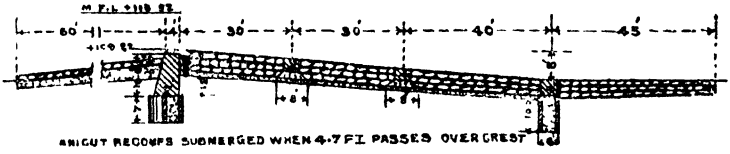
While the success of Okhla weir, which is built on fine sand (C = 15), shows that foundations are not necessary, the existence of well foundations for the body wall reduces percolation and also, in cases in which the main apron has been displaced and seriously scoured, such foundations have saved weirs from breaching

A notable instance of this occurred in the case of Bezwada anicut in 1894.

Well foundations to the tail curtain wall, such as have been provided in Sangam anicut, afford a protection to the main apron, in the case of deep scours being formed down-stream of the apron, but there does not appear to be sufficient reason to justify the cost of well foundations for intermediate curtain walls. In some cases shallow binding walls, only 3 or 4 feet in depth and founded on the rough stone filling, are built instead of intermediate curtain walls. These serve to restrict extension of damage to the apron but will not be much use in checking percolation

FIG. 51.

SANGAM ANICUT ACROSS THE PENNER.



309 Dry stone aprons of type C weirs -- The most essential feature in which type C differs from type B weir is that there is no impervious apron in the former, and the percolation under the body wall is free to escape through the interstices between the stones of the apron. The flow through these is no doubt somewhat obstructed, especially if these interstices have been filled, as they should be during construction with chips of stone and quarry rubbish, but at the best this line of flow will offer less resistance than that through the sand Thus there must be an upward percolation through the spaces between the stones which raises sand off the bed and generally deposits it in the interstices of the stones. The foundation sand is thus moved up into the stone and the stone settles into the sand. The surface of the apron must be made up as it settles by adding more stone and this makes the maintenance of this type of apron expensive so long as settlement continues

Gradually, as this settlement progresses, the interstices of the lower layers of stone become filled with sand, and the mass of stone though still permeable comes to a state giving as great or greater resistance to percolation than the foundation sand. When this happens if the apron is of sufficient width, settlement of the apron will cease and the conditions of the weir will not differ materially from that of a B type weir

There is obviously a danger especially during the initial stages of settlement to failure of the body wall by cavities being formed underneath it ; experience however shows that serious failures do not occur and the conclusion to be drawn is that the 'runs' in the sand are local and constantly forming and filling in under the body wall assisted in this by the weight of the loose stone in the up and down-stream aprons.

310. Remarks on the settlement of aprons of type C weirs.— In practice it is found economical to grout with concrete the interstices between the surface stones of the sloping down-stream apron

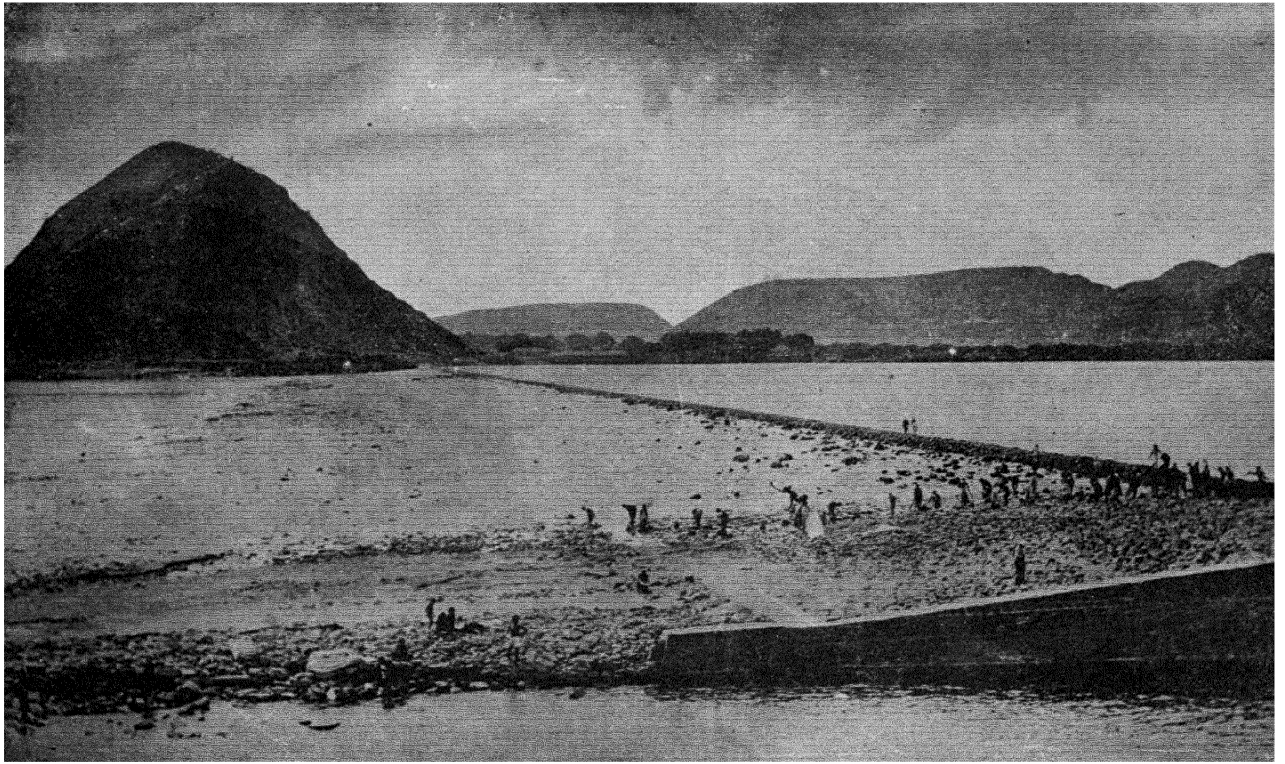


Plate III.—Kistna Anicut.

of weirs of this type as this saves much cost in maintenance by reducing displacement. Such grouting should not be carried out until the work has stopped settling, but in an old weir of this type it is almost always desirable. Grouting makes the surface impermeable and the stones of the surface layer liable to uplift as in the case of a B type weir

The amount of uplift would much depend on the slope of the apron surface and on the nature and density of the silt accumulated between the stones below.

In some type C weirs as in Sangam anicut (figure 51) a platform of broken stone or quarry rubbish is laid over the foundation sand and the stone filled in over this, such construction probably reduces the settlement of the apron, by forming an inverted filter through which but little sand can be uplifted. On the other hand it would delay if not altogether prevent the gradual staunching of the apron and with it the percolation past the weir. The cost of maintenance of Sangam anicut has as a fact not been heavy.

From the above considerations it would appear desirable in placing the rock filling to fill in the interstices between lower layers of stone with clay into which chips of stone might be thrust thus forming an impervious or nearly impervious mass. In places where clay or plastic silt is not at hand this would be costly but even sand filling, which would cost little, would stop most of the settlement of the apron and by so much reduce the cost of maintenance which is one of the most serious objections to this type of weir when built on sand.

311. Widths and slopes of aprons of type C weirs.—The great quantity of stone required for this type of weir not only for original construction but for filling up sinkage, makes it an undesirable one except where stone and unskilled labour are cheap and abundant and skilled labour and mortar expensive as it requires more stone, but less skilled labour and less cementing material, than any other type.

The latter conditions obtained in the case of the Okhla, Bezwada, and Sangam weirs.

It is however not so likely that these conditions will recur so frequently in the future as in the past and type A and B weirs will generally be found preferable to type C.

The apron slopes of type C weirs may be similar to type B and generally may be about $\frac{1}{C}$,where C is the class of the river bed sand ; on less perviuos foundation the slope may be limited to $\frac{1}{8}$. The slope is sometimes given in the form of steps and it is common to have a drop of from 1 to 3 feet immediately below the body wall. This is generally the result of raising the crest subsequent to original construction.

It is generally desirable to construct the apron immediately below a step exceeding $1\frac{1}{2}$ feet in depth of solid masonry.

The total width of down-stream apron of a type C weir may be the same as for type B; this includes the horizontal loose apron intended to settle to the slope of the scours below the work.

The upstream apron width will depend on the conditions of site and the liability to cross currents. It would generally be desirable to make this apron on a bed of puddle as in the case of type A

and B weirs. This adds but little to the cost and greatly reduces percolation.

If settlements of the main apron are excessive these may be reduced by making (or extending) an impervious up-stream apron, which will reduce velocity of percolation and therefore also reduce settlement, unless the latter is due to excessively deep scours below the toe, in which case the down-stream apron must be extended.

312. Advantages of weirs of types A and B contrasted.—In contrasting types A and B the former entails on the main apron greater stresses from the impact of falling water than the latter the apron of which is not subjected to any overfall. On the other hand the sloping apron type does not offer any check on the velocity of the water flowing down it and this entails greater velocities and greater depth of scour down-stream of the toe of the weir and generally heavier maintenance charges.

If the river bed is moderately hard kunkur, soft rock, gravel, or clay, and the overfall moderate, say not exceeding 5 or 6 feet, type A is probably the best. It is also probably the best even on very friable soil for a small overfall say, up to 3 feet.

If the difference of level (H_b) between the bodywall crest and L W.L. is considerable, and the foundation is of sand, type B is generally preferable.

When type A is adopted over a sandy bed it is generally desirable to keep the underside of the main apron at about L.W.L. so as to diminish cost of works and height of overfall, as well as uplift on the apron.

313. Water cushion aprons.—This forms a modification of a type A weir, the main apron being a cistern, or water cushion into which the water passing over the crest falls, the force of the impact on the masonry being reduced by the depth of the water standing over its surface.

A water cushion is seldom a suitable type of apron for a river weir where the bed soil is sand as the features of design generally necessitate laying the floor of the cushion at a considerable depth below L.W.L. which in freely permeable river bed entails great expense in dealing with water during construction. When river bed soil is but slightly pervious this type of apron may occasionally be suitable. The effective weight of the floor of the cushion reinforced by the weight of the water in the cushion must be sufficient to resist uplift.

When river weir is built on a bed of rock insufficiently hard to resist the impact of the falling water an apron may be dispensed with by building a low subsidiary weir across the river below the main work thus forming a protective water cushion immediately below the main weir.

314. Crest shutters and open weirs.—The tendency of modern design of diversion weirs is to obstruct the flood water-way as little as possible and to hold up as much of the total head as is economically feasible by some form of crest shutters which in floods do not obstruct flow. Thus the height of the solid obstruction required is reduced by the height of the shutter with corresponding reduction during floods in the afflux and in the velocity of flow over the apron and at the toe of the work.

The depth of accumulation of silt on the river bed above the weir is also correspondingly reduced and with it the trouble in the form of bed silt carried into the canals.

The Khanki weir (figure 48, page 158) above is a good example of this, the crest shutters being 6 feet high and the body wall crest only 7 feet above L.W.L.

Crest shutters have been installed on some of the largest anicuts in South India ; on Bezwada anicut 3 feet shutters and on that at Dowlaishweram 2 feet shutters have been fitted as an alternative to raising these weirs in solid masonry ; as a result the increased depth required in the canals have been obtained without putting any extra flood stresses on the structures.

When bridges exist at the sites of weirs, crest shutters may suitably be lift shutters moving in grooves in the upstream ends of the piers, the lifting gear being worked from the bridge. The whole heading up of water necessary for a river diversion work may be effected by means of such lift shutters and in such case the work is termed an 'open weir' or 'river regulator', or 'barrage' which type of work is dealt with in the next chapter, or the heading up may be effected partly by a solid weir and partly by lift shutters forming a combined weir and regulator. Several of the large anicuts of Madras have been improved in recent years by lowering the body walls and installing lift shutters on the lowered crests of the works. For instance the Upper and the Lower Coleroon anicuts were remodelled about the year 1900 by cutting down the masonry body walls 4 and 6 feet respectively and installing on them 6 feet lift shutters.

CHAPTER IX.

HEAD WORKS.

River Regulators, Scouring and Head Sluices.

River Regulators.

315. Regulators or sluices.—A regulator or sluice is essentially a bridge spanning the waterway to be regulated, in the openings of which shutters are installed capable of adjustment so as to regulate the amount of waterway left free for the passage of water.

The bridge platform may either be adapted for purposes of public traffic, or merely a platform from which the gear for regulating the shutters can be operated.

316. Regulator shutters.—Regulator shutters are usually made of wood or steel, and fitted to move in vertical grooves in the piers and abutments of the regulator superstructure; they are raised or lowered by means of winches, or screw gearing, operated from the bridgeway above.

317. River regulators, scouring and head sluices.—Regulators are occasionally made across rivers or streams so as to serve the purpose of weirs without causing the obstruction which is inseparable from a solid weir. These are termed 'river regulators' or 'open weirs.'

It is usual to construct a regulator on one or both flanks of a solid weir for the purpose of scouring the river channel feeding the canal head and keeping the deep river channel in *regime*, such regulators are termed 'scouring sluices.'

Regulators across the heads of canals through which the irrigation water is passed to the systems are termed 'head regulators' or in South India more frequently 'head sluices.'

318. Roadway over regulators.—The roadway of regulators or sluices is, as in ordinary road bridges, generally carried by masonry arches or by beams of steel, wood, or ferro concrete. The relative cheapness of masonry in India, and the stability resulting from the extra weight of masonry arches makes this form of construction generally the most suitable.

319. Span of regulator openings.—The span of the bridge openings of a regulator are limited by the width of shutter to be installed; the general introduction in recent years of shutters moving on rollers have resulted in very great increases in the width of shutters, and in correspondingly greater spans for regulators, and also in the more extensive use of regulators in place of weirs for river diversion works. Before the introduction of rollers the width of regulator shutters was generally restricted to 10 or 12 feet, and as a bridge with such small spans must cause great obstruction in the waterway of rivers, weirs, with or without crest shutters, were almost universally used for river diversion works.

The spans of shutters for river regulators and for large scouring sluices are now generally from 16 to 40 feet and bridges with such spans will seldom cause undue obstruction to river discharge.

320. Design of regulators.—When the shutters of a regulator are not raised clear of the ventway any pressure of the water on the shutters is transferred to the piers (or abutments) which must be designed, as in the similar case of a buttress dam, to resist such pressure without sliding, or the development of tension or undue intensity of pressure in any joint, or on the foundation soil.

Unless the work is founded on hard rock a floor will be necessary and this must be designed of weight and strength sufficient to resist displacement by impact, and the impervious portion of it must be of length sufficient to prevent 'piping' and of thickness sufficient to withstand uplift.

Bligh has, as in the case of river weirs, propounded empirical rules of design for regulators based on successful practice as ascertained from existing works, and these rules form a convenient basis of design subject to the proviso, that each such design must be verified for stability of piers and weight and length of floor, etc., by the methods already described and applied to the piers and foundations of buttress dams, the floors of weirs, etc.

321. Formulæ for design of regulators.—The formulæ recommended for preliminary design of regulators are stated below:—

1. *Thickness of piers for spans of 'S' feet.*—It should generally not be less than $\sqrt{S}$ and vary from 1/5 to 1/3 of S . . . . (41)

The thickness should incline towards the larger proportion in great depths of water especially for small spans, and tends to decrease for lower depths of water, or for large spans.

Thus for a depth of 15 feet with a span of 10 feet Bligh suggests a proportion of ·25 but with a span of 25 feet, a proportion of 0·21. Again for a depth of 30 feet and a span of 10 feet he proposes 0·31 and for a span of 25 feet 0·28.

Where, as in the case of a head regulator, the work is subject to a great difference of head of water up and down-stream the proportion may be raised to 0·4.

2. *The thickness (t) of segmental masonry arches at crown.*—Formula is $t = \cdot 45\sqrt{R}$ . . . . . (42)

where R is the radius of the intrados of the arch. For spans of over 20 feet increased thickness towards the abutments will be necessary.

3. *Abutments.*—Trautwine's empirical rule may be followed—

$$\text{Thickness at springing} = 2 + \tfrac{2}{10}R + \frac{\text{rise of arch}}{10} \quad . \quad . \quad . \quad (43)$$

$$\text{Back slope of abutment} = \frac{\text{Span of arch (horizontal)}}{25 \times \text{Rise of arch (vertical)}} \quad . \quad . \quad . \quad (44)$$

The above are suitable for tentative dimensions for arches of road bridges.

4. *Height of piers.*—The springing of arches should not generally be below the M.F.L. up-stream so that as little obstruction to flow as possible may be caused, and also so that the weight

of the arch, spandrel filling, and roadway may not be reduced by flotation. Subject to this a public roadway should be at the most convenient level to suit the approaches, etc.

The points of suspension of the shutters whether on a level of the bridge roadway or not, must be at such a height that the shutters when fully raised shall have their lower edges above maximum flood level.

When the regulator is not used for traffic this consideration alone will determine the height of the piers and platform.

5. *The length of the piers.*—This is fixed by the necessary width of roadway and by consideration of the requirements of stability Where a public road is combined with the regulator and is carried by a masonry arch, the length of pier required to give adequate width of roadway is generally sufficient to give requisite stability under all conditions of flow, or of regulation. This however must be tested by stability computations or diagrams.

When the structure is restricted to use as a regulator or when the roadway is a light one carried on beams, the lengths of piers have frequently to be increased above what would suffice to carry the bridge in order to fulfil the requirements of the regulator as regards stability.

6. *Flooring and aprons.*—The design of these generally follows the rules given for solid anicuts.

322. Example of working out of the design of a regulator.—An example is now worked out of the design of the pier of a river regulator

A regulator not required as a public road way of ten spans of 20 feet clear is to be built across a river the following being the conditions:—

Mean bed level of the river and level of sill of regulator + 50.
Maximum flood level before construction + 61·5.
Computed M.F.L. above the regulator after construction + 62.
Height of top of shutter when resting on sill + 58.
Foundation soil, coarse sand of class C = 12.

The roadway is to be carried on rolled steel beams with wood decking and to be 6 feet wide. The top of the piers requires a length of 6 feet down-stream of the groove to give room for working the shutters.

The masonry of the pier is brick of sp. gr. = 2 and one cubic foot of masonry is the unit of weight.

Make the thickness of pier 1/5 of span = 4 feet. The weight of the girders and roadway, etc., is computed at 2 tons per span, that is 2 × 18 = 36 units of weight.

The weight and support given by the semicircular cutwaters up and down-stream, and by the masonry up-stream of the back of the shutter grooves, are neglected. This is the usual practice but it is quite legitimate to consider the masonry for a reasonable distance (say ¼ its height) up-stream of the down-stream face of the grooves as contributing to stability.

The level of the top of the pier must be at least the height of the shutter (8 feet) above M W.L. and to give a working margin it may be made 9 feet above this, viz. + 71.

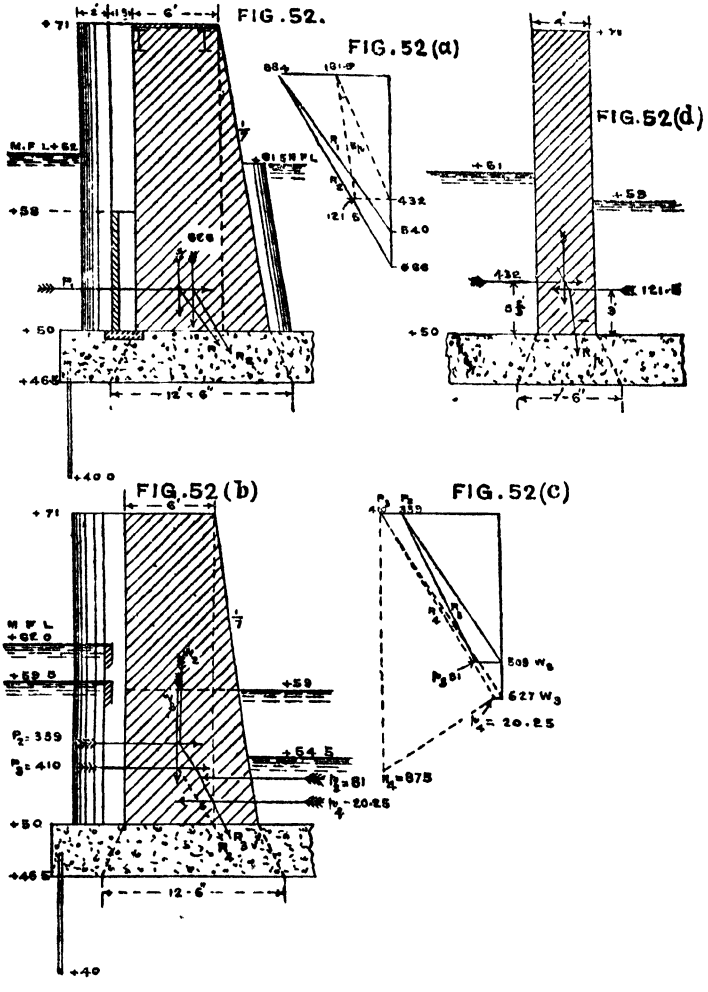
As the length of the pier top dow n-stream of the groove is 6 feet the minimum cubic content of the pier is 21 × 4 × 6 = 504 units in weight.

Adding weight of roadway, the total weight per span is 540 units.

The water pressure with shutter resting on the sill and water at the level of the top of the shutter ... $= \left(\frac{I}{\rho} \times \frac{8^2}{2} \times 24\right) = 384$ (units)

and this acts at ⅓ of 8 = 2⅔ feet above the floor.

These forces plotted in figures 52 and 52-*a* give a resultant (R_1) cutting the base about 1·2 feet from the extremity and thus outside the centre third of the base.



It is therefore necessary to add to the length of the pier by giving either increased length to the whole pier or by giving a batter to the down-stream face of the pier, or both.

The batter is most economical and a slope of 1 in 7 may be tried nd is drawn in figure 52.

This will give extra masonry of weight $\frac{1}{2}$ (21 × 3 × 4) = 126 units and a total weight of masonry and platform of 666 units with the centre of gravity 3·75 feet from the up-stream face.

Completing the force diagram (figures 52 and 52-*a*) the resultant (R_2) cuts the base well inside the middle third; it thus may be possible to economise by making a steeper batter to the down-stream part of the pier; it is however first necessary to consider the possible variations in working conditions of the regulator and the stresses these involve

323. Different conditions of stress on a regulator.—When the regulator shutters are partially raised, if the water on the up-stream side is kept to the top of the shutter, and the down-stream water does not back up so as to partly or wholly submerge the down-stream face of the shutter, the pressure on the shutter is $\frac{1}{2}\left(20 \times \frac{8^2}{2}\right) = 320$ units that is the same as under the condition already examined, but the centre of pressure is raised by the height to which the shutter is lifted off the sill and thus exercises a greater overturning moment on the pier.

Further when water down-stream rises over the floor level and submerges the pier the effective weight of the part submerged is reduced to (ρ—1) per cubic foot.

It is thus clear that while the condition already examined is that under which the greatest head is developed, this does not necessarily connote the condition of greatest stress and in the case of open regulators of this nature it seldom is so. This has been already demonstrated in the somewhat similar case of weirs.

The maximum stress under working conditions must in each case be ascertained by trial as follows:—

After preparing a calibration table or curve showing the discharges which correspond to successive levels of river water at the site of the proposed work, the successive openings of shutters which with the water level at the tops of the shutters will give corresponding discharges must be worked out.

From the above the condition of greatest stress under working conditions can be ascertained.

324. For instance in this example there are 10 vents of the same width (20 feet) and the same sill level (+ 50). The maximum discharge is ascertained to be 12,000 cusecs; the down-stream discharges from under noted water levels are computed (or observed) to be as below: —

Discharge of river	Corresponding level of river.	Corresponding discharge of regulator per foot run of sill.
12,000 cusecs ...	.. + 61·5 (M.F.L.)	... 60 cusecs.
8,000 „	.. + 58·75	.. 44 „
4,000 „	+ 54·5	... 20 „
1,000 „	... + 50·0	. 5 „

From the above the following relations between water levels up and down-stream of the work are calculated, the water being except in case (1) level with the top of the shutters :—

Case.	W.L. above.	W.L. below	Head in feet	Shutters raised off sill in feet.	Total discharge in cusecs.
1	+ 62·0	+ 61 5	0·5	Clear	12,000
2	+ 62 0	+ 59 0	3·0	4 0	8,800
3	+ 59 5	+ 54·5	5·0	1·5	4,000
4 ...	+ 58 0	+ 50 0	8·0	Closed.	Nil.

The condition of stress under case (1) does not call for investigation and that under case (4) has been already examined. It remains to examine those under (2) and (3).

Case 2.—(*a*) The water pressure on each shutter is 320 − back pressure $\left[\frac{1}{2}\left(\frac{5^2}{2}\times 20\right)\right]$ = 320 — 125 = 195 units, applied at level (4 + 3·3) = 7·3 feet above floor.

(*b*) Water pressure on pier face = $\frac{1}{2}\times 4\times\frac{(12)^2}{2}$ = 144 units applied at level 4 feet above floor.

(*c*) The pressure (*a*) + (*b*) in the down stream direction = 339 units applied at 5·9 feet above floor level.

(*d*) Back pressure on pier = $\frac{1}{2}\times\frac{9^2}{2}\times 4$ = 81 units. This acts at 3 feet above floor level.

(*e*) Weight of pier and platform allowing for flotation below + 59 = $666-\frac{1}{2}\left(\frac{7\cdot7+9}{2}\times 4\times 9\right)$ = 509 acting 3·61 feet from the up-stream end of the base.

Plotting the above in figures 52 (*b*) and (*c*) the resultant (R_s) falls about 1 foot outside the middle third.

Case 3.—W.L. above 59·5. Shutters raised 1½ feet.
„ below 54·5. Head = 5 feet.
Velocity through vents = 14·3 f.s.
Pressure on shutter = units 320.

This acts at (1½ + ⅓ × 8) feet = 4·17 feet above the floor. As the velocity of the water passing under the shutter would be sufficient to form a standing wave there would be no back pressure on the shutter in this case.

Pressure on pier = $\frac{1}{2}\times 4\times\frac{(9\cdot5^2)}{2}$ = 90.

This acts at 3·17 feet above the floor.

The resultant pressure is 410 units and acts at 3·95 feet above the floor.

The back pressure on the pier is $\frac{1}{2}\times 4\times\frac{(4\cdot5)^2}{2}$ = 20·25 units and it acts at 1·5 feet above the floor.

The weight of the pier and platform reduced by flotation below level + 54·5 is $\left\{ 666 - \frac{1}{2} \left(\frac{8{\cdot}36 + 9}{2} \times 4{\cdot}5 \times 4 \right) \right\} = 627$ units.

Plotting the above in figures 52 (*b*) and (*c*) the resultant R_4 (dotted) is obtained; this cuts the base just at the extremity of the middle third.

325. Limitation of working conditions of regulators.—There is generally no gain to irrigation obtainable by permitting water to rise to any great height above the level of the 'crest level' of shutters of river regulators and the rules of regulation should generally enjoin that water level should, *so far as is possible*, not be allowed to rise to more than 1 foot above the shutter crest level or to fall to more than ¼ foot below this level. When taking out stability diagrams the proposed limits of regulation must be considered and the design made accordingly.

In this example the design is stable with the shutters raised 1½ feet and the water up to level + 59·5 and the rules of regulation should limit the water level to + 59 as maximum, unless the shutters are raised clear. The stability of the pier is therefore sufficient and the down-stream slope of 1 in 7 is suitable and will be adopted.

326. Water not allowed to spill over shutters.—In Madras it is usual to enjoin that under no circumstances is water to be allowed to overtop regulator shutters, but in some works the practice of passing water over shutters is under certain circumstances allowed; when this is permitted, stability diagrams for these conditions must be drawn, and designs both of masonry and shutters made to suit.

327. Pressure on foundations.—The pressure on the foundation of the regulator piers must also be examined

The floor is proposed to be of concrete 3½ feet thick. The maximum inclined pressure on the pier base is shown by the length of the line N_4 in the force diagram (figure 52-*c*) and equals 875 units, or 875/18 = 48·5 tons. This is $\frac{48{\cdot}5}{4 \times 9} = 1{\cdot}34$ tons per square foot on an average and the maximum intensity of pressure is twice this, viz., 2·68 tons which is a moderate stress.

For pressure on the foundation soil, the effective width of base of the piers may be taken to spread at a slope of ½ to 1 through the platform slab on which it rests and is equal to 7½ feet (figure 52-*d*).

Similarly the effective length of base of the pier excluding cutwaters, etc., as shown in dotted lines in figure 52-*b* is 12½ feet. The total vertical pressure on each pier is 666 units and to this must be added the effective weight of the concrete foundation slab. If lowest water level is, as here assumed, at floor level, the weight of the slab is reduced by flotation and equals (3½ × 94) ½, that is, 164 units. The total weight is thus, 830 units or 46 tons and the mean pressure on the foundation soil $\frac{94}{46}$ = say, ½ ton. This is well within the limits of safety.

If, however, the pressure was found to be excessive the part of the foundation slab under each pier, should be made thicker, the extra thickness being carried out at each side and at each end of the pier for a width of one-half the total thickness of the foundation slab. In alternative, wells can be sunk as foundations of the piers.

328. Cross thrust on Piers of Regulators.—It is further necessary in regulators to consider if the piers are stable laterally supposing the adjacent vents to be under different conditions of regulation.

The conditions of maximum overturning in such cases must be considered supposing :—

(*a*) Water up to top of shutters and one shutter fully opened the remaining shutters closed.

(*b*) Flood conditions with one shutter closed; as for instance if the gear were broken, and all the others lifted clear.

The pressure may be considered as confined to a length of pier of 6 feet, which with superstructure weighs 540 units. In this example:—

Condition (*a*).—The mean level of water over the floor of the open span will be assumed as + 57 and there would be no appreciable depth of water on the floor of the closed spans

The water pressure $= \frac{1}{2} \times \frac{4^2}{3} = 12{\cdot}25$ units.

This would act at $2\frac{1}{3}$ feet above the floor. It is obvious that under these conditions there is no danger to the pier and it is unnecessary to plot this.

Condition (*b*).—This is generally the condition which brings most cross pressure on the piers. The water level under the closed span cannot be accurately determined and must be assumed; the difference of head will be considerably less than under condition (*a*), but it will be more than that due to the up-stream and down-stream maximum flood levels. In this case the difference of head may be taken to be 2 feet or 4 times the computed difference of head in a maximum flood; and the levels in the adjacent vents may be taken as + 61 and + 59.

The weight of the pier will be $36 + 12 \times 4 \times 6 + \frac{1}{2}(9 \times 4 \times 6) = 432$ units.

The water pressures are $\frac{1}{2} \times \frac{12^2}{3} \times 6 = 181{\cdot}5$ and $\frac{1}{2} \times \frac{9^2}{3} \times 6 = 121{\cdot}5$ acting at heights $3\frac{2}{3}$ and 3 feet above the floor. This is plotted in figure 52 *d*.

The resultant falls just outside the middle third; as the conditions assumed are extreme ones and the head of the pier gets some support from the platform girders the design does not require altering by thickening the piers.

If masonry arches were used to carry the gangway the thrust of these would give great support to the piers against overturning.

329. Toludur Regulator across the Vellar.—There are a number of regulators across rivers in the Madras Presidency especially the Cauvery and the Vellar, but nearly all are lift shutters on the crest of a solid anicut forming combined weirs and

regulators. A river regulator with its floor at mean river-bed level is now being built across the Vellar at Toludur and this is cited as a fair example of such a work and is shown in figures 54 and 55.

The regulator is 16 spans of 30 feet with brick arches carrying a main road across the river.

The flood discharge of the river is 80,000 cusecs.

The main offtake sluice which is to feed a reservoir is at the left flank and the three end vents of the regulator on this flank have their sills 2½ feet below the sills of the remaining vents. These will serve as scouring vents with a view of keeping silt in front of the head sluice at a low level. These three spans are founded on soft rock, but the ordinary spans to the right of these are founded on coarse sand.

The section of an ordinary span is given in figure 54.

330. Design computations, Toludur Regulator.—The following data are noted regarding Toludur Regulator:—

The superstructure is of brick-work of sp. gr. 2. Span 30 feet clear—Piers 6 feet wide and arches 2 feet thick with rise ⅙ span.

Levels—

Sill of shutters + 269·75, floor + 269·5.
Sill of scouring vents + 267·25, floor + 266·0.
M.F.L. up-stream + 285·85.
M.F.L. down-stream + 284·00.
Down-stream W.L. when shutters closed + 267.
Springing of arch + 286·5.
Shutter platform level + 299.
Shutter platform scouring vents + 300·5.
Level of metalled roadway + 295·0.
Height of shutters 8½ feet.
Height of scouring vents 11 feet.

It is assumed that the concrete filling over the spandrels and road metal up to roadway have the sp. gr. of brick work, viz., 2. In taking out stability, the weights of the cutwaters and of the portions of the piers up-stream of the down-stream end of the shutter groove will be omitted. The assumption is made that the rules of regulation will be such that water will not be allowed to rise above level + 279·25 without raising all shutters clear of the water.

Stability of piers.—Back pressure on the shutter is not allowed for as the velocity would produce a standing wave down-stream of the shutter and therefore there would be no back pressure developed on it. The left end span of the regulator has its sill 2½ feet lower than that of the ordinary vents and has a shutter 11 feet high and the floor at + 266 but with the same section of pier and arch as in the ordinary spans.

The stability diagram will be plotted for this span with W.L. at + 279·5 and water up to the top of the shutter. The level of water below the vent is taken at + 270 and the shutter is raised 1½ feet off the sill.

The area of the open segment of circle between the springing level and the intrados of arch is 101 sq. ft.

Weight over each span = (30 × 8·5 − 101) × 19 = 154 × 19 = 2,926 units.

Weight of pier and masonry above from floor at (+ 266) to roadway level (+ 295) = 6 × 29 × 19 = 3,306 units.

Total weight	...	6,232 ,,
Deduct for flotation of pier (6×4×19) ½	...	228 ,,
Effective weight	...	6,004 ,,

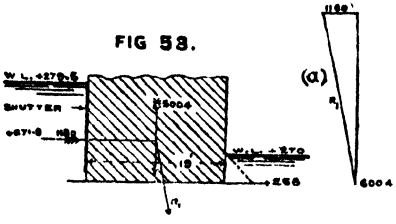
The water pressure on the shutter is $\frac{1}{2} \times 30 \times \frac{11^2}{2} =$ 907 units and acts at level $\left\{ + 279{\cdot}5 - \left(\frac{2}{3} \times 11 \right) \right\} = + 272{\cdot}17$.

The water pressure on the pier is $\frac{1}{2} \times 6 \times \frac{13\,5^2}{2} = 273$ units and acts at level + 270·5.

The total water pressure is 1,180 units with centre of pressure at level + 271·8.

The diagram (figure 53) shows that the resultant pressure (R_1) falls far within the middle third of the base.

FIG 53.



The scouring vent piers in this case are founded on soft rock while the ordinary spans are on coarse sand (C=12).

331. Formulæ for width of aprons of river regulators.—The width (W_1) and thickness of the main apron and the width (L_1) of the main apron and talus, in the case of an open regulator are computed on similar lines to those for an anicut of type A, the empirical formula for these expressions being altered by substituting $\left(\frac{H_a}{13}\right)$ for $\left(\frac{H_b}{10}\right)$ in the formula for main apron and $\left(\frac{H_s}{13}\right)$ for $\left(\frac{H_b}{10}\right)$ in that for the apron and talus (vide Chapter VIII, paragraph 283, formulæ 39 and 40).

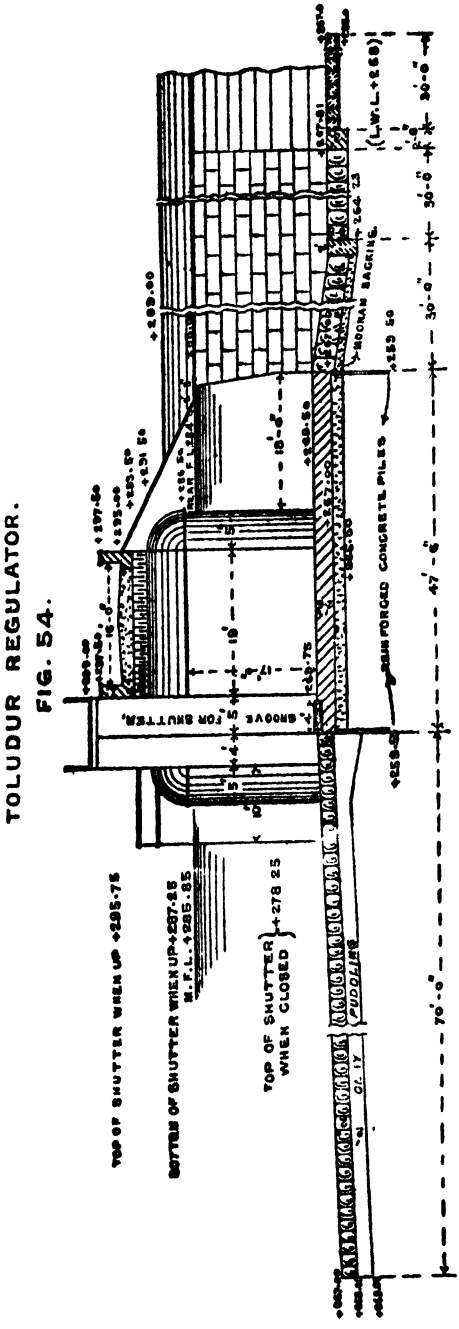
Hence for river regulators (open weirs) we get the empirical formulæ:—

$$W_1 = 4\,C\sqrt{\frac{H_a}{13}} \quad \ldots\ldots\ldots\ldots\ldots \quad (45)$$

$$L_1 = 10\,C\sqrt{\frac{H_s}{13}} \times \sqrt{\frac{q}{75}} \quad \ldots\ldots\ldots\ldots \quad (46)$$

332. Aprons of Toludur river regulator.—The dimensions of the apron of the ordinary spans of Toludur regulator will now be examined with reference to the formulæ applicable to such a case.

From the section of the regulator figure 54—



TOLUDUR REGULATOR.
FIG. 54.

(*a*) Length of line of percolation $= l_1 = 2 + 70 + 6 + 47{\cdot}5 + 6 = 131{\cdot}5$.

$H_s = 278{\cdot}25 - 268 = 10\frac{1}{4}$ feet. $C = 12$.

H^a = height of shutter crest on weir above main apron $= 278{\cdot}25 - 269{\cdot}5 = 8{\cdot}75$ feet.

(*b*) Main apron floor width computed from formula $45 = 4\,C\left(\frac{H^a}{13}\right)^{\frac{1}{2}} = 48 \times \left(\frac{8\,75}{13}\right)^{\frac{1}{2}} = 39\frac{1}{2}$ feet.

The length as designed is $47\frac{1}{2}$ feet.

(*c*) Uplift head just below the shutter is $\left(1 - \frac{78}{131\,5}\right) \times 10\frac{1}{4}$ feet $= {\cdot}41 \times 10\frac{1}{4} = 4{\cdot}2$ feet.

The apron is $1\frac{1}{2}$ feet of concrete of sp. gr. 2, which when submerged will balance $1\frac{1}{2}$ feet head of water, and $2\frac{1}{2}$ feet of rubble masonry of sp. gr. $2\frac{1}{4}$.

The L.W.L. is + 268 and the lowest foot of rubble masonry is submerged and will balance $1\frac{1}{4}$ feet of uplift while the upper $1\frac{1}{2}$ feet is not submerged and will balance $3\frac{3}{8}$ feet of uplift.

The total uplift which the apron can resist is thus $(1\frac{1}{2} + 1\frac{1}{4} + 3\frac{3}{8}) = 6\frac{1}{8}$ feet.

This will balance the computed uplift of 4·2 feet and there is 30 per cent to spare as factor of safety.

(*d*) Width (L_1) of talus and apron, q the maximum discharge per foot run being 166 cusecs, is from formula (46).

$$L_1 = 120\sqrt{\frac{10{\cdot}25}{15}} \times \sqrt{\frac{16}{7}}$$

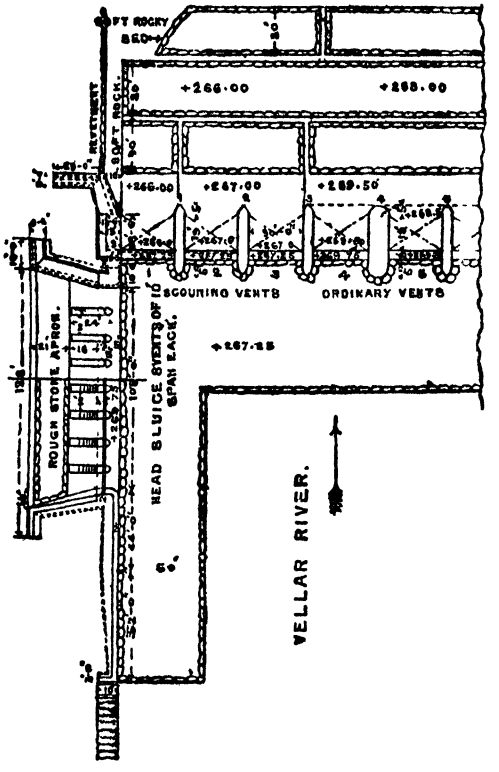
$= 106{\cdot}5 \times 1{\cdot}5 = 159{\cdot}75 =$ 160 feet.

The actual width as designed is 140 feet and it may be found necessary in the future to prolong the *talus* by further pitching.

SCOURING SLUICES.

333. Scouring Sluices in Weirs.—These are placed on the flank of the weir on the side of the river from which the canal off-takes. They are works subsidiary to solid weirs and their functions are—

PLAN LEFT FLANK
TOLUDUR REGULATOR.
FIG. 55.



(*a*) To keep open during low water season a channel through the river bed past the head sluice.

(*b*) To keep the bed of the above channel at a low level with reference to the sill of the head sluice and thus as far as possible prevent bed silt entering the canal.

334. Sill level of scouring sluices.—The sill of scouring sluices should be at as low a level as can conveniently be arranged and generally at about the deep bed level of the river. A deep sill greatly facilitates their success in operation.

335. Size of scouring sluice vents.—The main object of the sluices being scour it is clear that large spans and unobstructed depth of flow are an advantage, and the practice of building them of small spans and orifices of very restricted vertical height is detrimental to their efficiency. It is desirable that the spans should be large and the space between the piers unobstructed from floor to M.F.L.; any arch or platform over the piers should, when practicable, be wholly above flood level and the working platform should be accessible under all conditions of flood These conditions are frequently not attainable, but it is apparent that in many existing works their desirability was not clearly recognized.

It is desirable to keep the head-sluices closed whenever the scouring sluices are open, but it is generally impossible to close the head sluices unless there has been heavy rain in the irrigated

area, and thus, even when there is a flood in the river it is not infrequently desirable to keep the scouring sluices closed. In order to make this practicable there must either be a face wall between the piers of the scouring sluices from the crest level of the scouring sluice shutters to M.F.L. or water must be allowed to pass over the crest of the shutters. It is not usual in Madras to allow water to spill over the tops of lift shutters but there are considerable advantages to be derived from allowing this in the case of scouring sluice shutters.

Where there is no face wall to the scouring sluices it is clearly essential that the crest level of the sluice shutters should be at least as high as the crest of the weir or weir shutters.

336. Formulæ for the widths of aprons of scouring sluices.—The conditions under which scouring sluices are operated connote passing great volumes of water at high velocities and the kinetic action on the floor and apron below the shutters is likely to be greater than on the floor and apron of the weir of which they form a part.

The widths of the down-stream floor and of the talus is computed by Bligh from formulæ similar to those he applies to the weir apron but allowing proportionately greater widths.

The width (W_2) of the main floor of the scouring sluices is affected by the crest level of the weir shutters, or, where there are no shutters, of the weir crest because the sluices will frequently be worked when the water is being maintained at this level. Bligh's formula for the width of the floor of scouring sluices may be adopted and is as follows:—

$$W_2 = 7\ C \sqrt{\frac{H_s}{13}} \quad \ldots\ldots\ldots\ldots \quad (47)$$

where H_s is the difference between the crest level of the weir shutters and the level of the scouring sluice floor.

For the width (L_2) of the whole apron below the shutters, Bligh gives a formula which depends on the height (H_b) of the solid weir crest above the floor and *not* on the height (H_s) of the weir crest shutters above L.W.L.; this appears illogical in respect to scouring sluices although quite logical in respect to a solid anicut, and a formula on the same lines as Bligh's but using $\frac{H_s}{13}$ for $\frac{H_b}{10}$ is substituted:—

$$L_2 = 15\ C \sqrt{\frac{H_s}{13}} \times \sqrt{\frac{q}{75}} \quad \ldots\ldots\ldots \quad (48)$$

The width of the impervious apron required up-stream of the sluices depends, as in weir design, on the width required to make the length of the line of percolation not less than CH_s.

The thickness of the down-stream main apron should not be less than 3 or 4 feet and should also be safe, with 20 p.c. to spare, against uplift due to a head H_s.

337. Conditions of working of scouring sluices.—It will be noted that the empirical equations for scouring sluice aprons while following the same lines as those for solid and open weirs have larger constants as multipliers. The reason for this is that the scouring sluices are frequently worked at their maximum discharging capacity when there is little or no water being passed by the

weir and these conditions entail a much lower level of tail water and greater heads and velocities and consequent stresses on the apron than when large quantities of water are being passed over the weir.

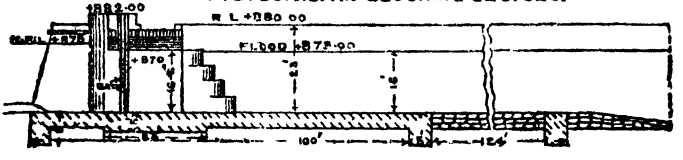
It is on this account that, other things being equal, the aprons of scouring sluices require greater protection than those of solid weirs or river regulators.

It frequently happens however that the scouring sluices are founded on more compact soil than the main length of weir and suitable allowance for this results in reduction in the length of apron.

338. Scouring vents of Toludur regulator.—In the case of the Toludur regulator the apron of the scouring vents is no longer than that of the central vents because the former are built on soft rock and the latter on sand The relative positions of the scouring vents, the open weir and the head sluices shown in figure 55 are fairly representative of what these conditions should be, but in this case no divide groyne has been provided because the natural deep channel of the river lies on the head sluice side and the conditions of regulation require that during low stages all the flow will be passed through the scouring vents to the lower anicuts on this river, which have prior rights to such water Under these special conditions a divide groyne may not be necessary but if one is found desirable it can be constructed at any future time.

339. Scouring sluices of Rupar weir.—Figure 56 shows the part plan and section of the scouring sluices of Rupar Weir, Sirhind Canal System of the head works of which a site plan has already been given in figure 14, paragraph 182. This work may be taken as fairly representative of a modern type of scouring sluices applicable to large works. In this case the work is founded on hard clay and boulders and the foundation soil may be considered of a class in which C = 9. The superstructure consists of piers 5¼ feet wide and 20 feet apart spanned by masonry arches with springing clear of flood level.

FIG. 56. RUPAR SCOURING SLUICES.



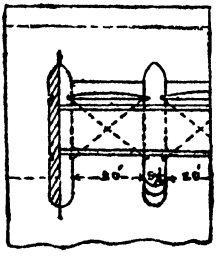
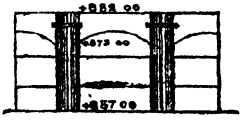


FIG. 56(a)



The shutters are in two tiers each moving in separate grooves on fixed rollers.

The level of the tops of the shutters when fully lowered are at + 870 the crest level of the weir shutters being + 872. This is a peculiar condition from which it would appear that water is permitted to pass over the tops of the shutters as failing this it must be necessary frequently to open the scouring sluices when the head sluices are open.

340. Face walls of scouring sluices.—As already stated it is desirable to have the crest level of the scouring sluice shutters as high or higher than that of the weir shutters and if conditions do not permit either of this, or of passing water over tops of the shutters then a face wall permanently closing the space between the piers from the level of the crest of the scouring sluice shutters should be provided in spite of the diminution of scouring capacity thereby entailed.

Such a face wall forms a feature of design, of nearly all the scouring sluices in the Madras works, but it should be clearly recognized that it is an undesirable one and should only be adopted where economic or special conditions make it desirable. This however is not infrequently the case more especially in connexion with small works.

Head Sluices.

341. Requirements of a head sluice.—A head sluice should be designed with a view to passing into the canal had at moderate velocities all the water required for irrigation, while as far as practicable excluding bed silt, it must also be capable of excluding from the canal any water which may not be required.

The fulfilment of these requirements calls for a work built across the canal head to a level above that of the maximum flood and of sufficient stability to support water standing at M.F.L. against its up-stream face when there is no water in the canal below it.

While the object in view in scouring sluices is passing of water at high velocity, that in head sluices is passing the water at a moderate velocity and while excluding bed silt to take into the canal the normal proportion of silt carried in suspension by the river.

342. Arrangements for drawing surface water through head sluice.—The passing of supply with low velocity and the exclusion of bed silt are best attained by drawing the supply from the film of water nearest the surface, and in recent years several of the head regulators of the systems in Northern India which suffered badly from silt troubles have been remodelled with this object in view.

In the original designs of such works, in Northern India as also in Madras the above points were not lost sight of, but experience has shown that where silt troubles are accentuated, the system of drawing water at a high level with a low velocity of entry should be carried much further than was originally proposed.

In Madras the troubles from silt are far less than those experienced in Northern India and it has not been found necessary to

remodel the head sluices of any of the larger irrigation systems in Southern India on this account.

The drawing of surface water at low velocity involves great width of ventage and necessitates arrangements which permit of varying the level of the sill of the orifice, or notch, over which the water is taken (into the canal) to suit the varying levels of the surface of water in the parent river. The above implies extra cost, and it is a matter for consideration in each case, to what extent such extra expenditure on original construction is justifiable in view of the reduction of silt troubles and probable savings in future expenditure for silt clearance.

343. Head sluices in Madras —In the Madras Presidency it has been the practice for many years to make the sill of the head sluices 1½ to 3 feet higher than the sills of the scouring sluices and to design head sluice shutters in 2 or 3 tiers working in separate grooves so as to be capable of admitting water at different levels; it has been usual to keep the lower shutters closed as long as it is possible to pass the supplies required for irrigation through the upper ones. There is no great advantage in having large span shutters for head regulators, unless, as is frequently the case, the required length of ventage can be more economically given with large spans of 16 to 20 feet than with comparatively short ones of, say, 5 to 10 feet; in Madras the head sluices generally have vents of widths not exceeding 10 feet.

344. Sites of Head sluices.—The position of the head sluices should be with the face in line with the river margin so that the scouring sluices should draw directly across the front of the sluice. A mistake has been frequently made in placing the head sluice a short distance down the canal thus leaving a recess, out of the draw of the scouring sluices, from which silt deposit cannot be scoured, and which moreover is likely to engender eddies and swirls which lift the silt off the river-bed whence it is carried into the canal.

If it is necessary to construct head sluices some distance down the canal the scouring sluices should be built in the leading channel just above the head sluices and at right angles to them.

345. Head Regulator of the Sirhind Canal at Rupar —Figure 57 shows the elevation of part of the head regulator at Rupar of the Sirhind Canal as originally constructed.

This consisted of 39 spans of 5 feet. The work consisted of 13 main spans of 21 feet with main piers 4½ feet wide carrying a roadway, and each of the main spans was divided by jack piers 3 feet wide into 3 spans of 5 feet in the clear. The sill level of the vents was 2 feet above that of the scouring sluices.

The head sluices of several of the large systems of Madras are of the same type, and no aggravated troubles from silt deposit in these canals have been suffered. In the case of Sirhind Canal, however, great silt troubles were experienced which have been almost completely overcome by—

(*a*) Installing 6 feet crest shutters on the weir crest thereby raising ordinary supply level in the river at the head regulator from + 866 to + 872.

(*b*) Removing the jack piers and 5 feet span shutters and installing in the head regulator 21 feet shutters in two tiers moving on rollers.

(*c*) Building a breast wall 7 feet high on the floor of the regulators across the arches thus raising the sill by no less than 7 feet.

Figures 57-*a* and *b* show in elevation and section the revised arrangements from which it appears that the lower shutter can be both raised or lowered behind the breast wall but the upper shutter can only be raised.

FIG.57. SIRHIND CANAL HEAD REGULATOR. FIG.57(a)

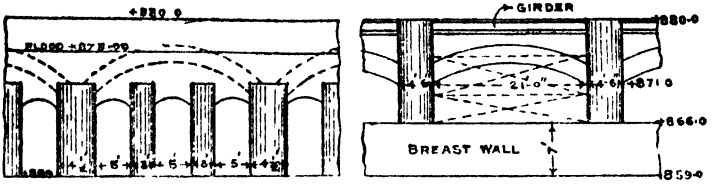
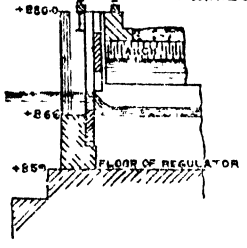


FIG 57(b) RAILS FOR TRAVELLER.



346. **Head sluices of the Gōdāvari Eastern and Kistna Western main canals.**— In figures 58 and 58-*a* are shown part plan and section of the sluice at the head of the Gōdāvari Eastern Delta main canal. The arrangement of main and jack arches follows generally that of the original Rupar regulator. The shutters are in three tiers but the upper ones cannot be lowered, and a modification allowing of this would have been an improvement.

FIG 58 DOWLAISHWARAM HEAD SLUICE.

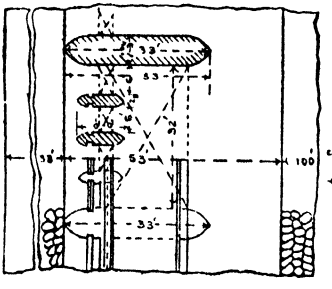
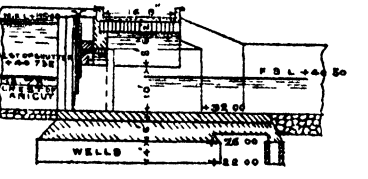
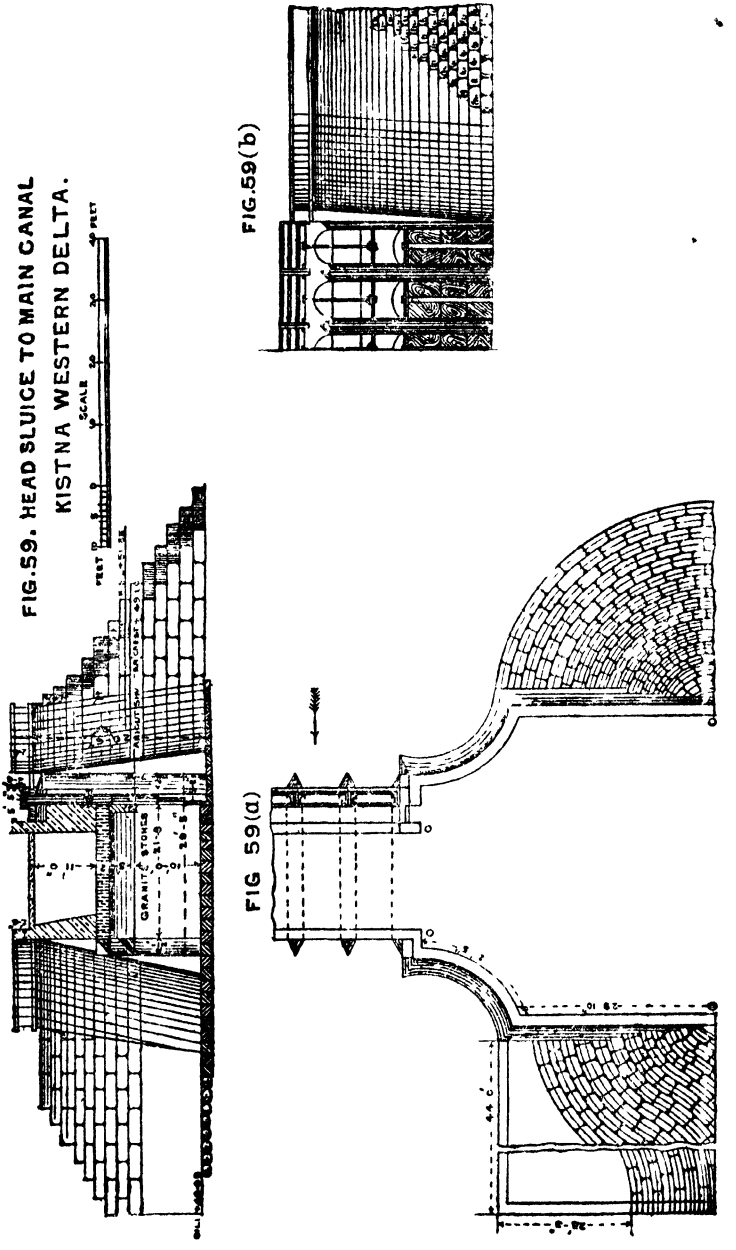


FIG.58(a)



In figures 59, 59-*a* and *b* is shown the head sluice of the Kistna Western Delta main canal on the right flank of Bezwada anicut. The shutters of this are in three tiers as in the previous case. Both these works are the original sluices built about the middle of the last century and are still in operation.



It will be noticed on comparing them that in the Gōdāvari sluice, vents are merely a panelling opening out into the wider waterway of the main arches, while in the Kistna sluice the narrow archways are carried as main arches under the whole width of the work, and a great deal of filling between the face walls above the arch solid to road level is necessary. While this assists stability by adding weight to the work the type has two great disadvantages and generally the Gōdāvari type appears preferable.

The disadvantages are—

(1) That the restriction of waterway and the consequent high velocity is carried right through the width of the arch work with the resulting increase in turmoil below the work.

(2) That the type does not lend itself to alterations of size of vents which is, as has been proved in the case of Rupar regulator, a marked advantage which is possessed by the panel type.

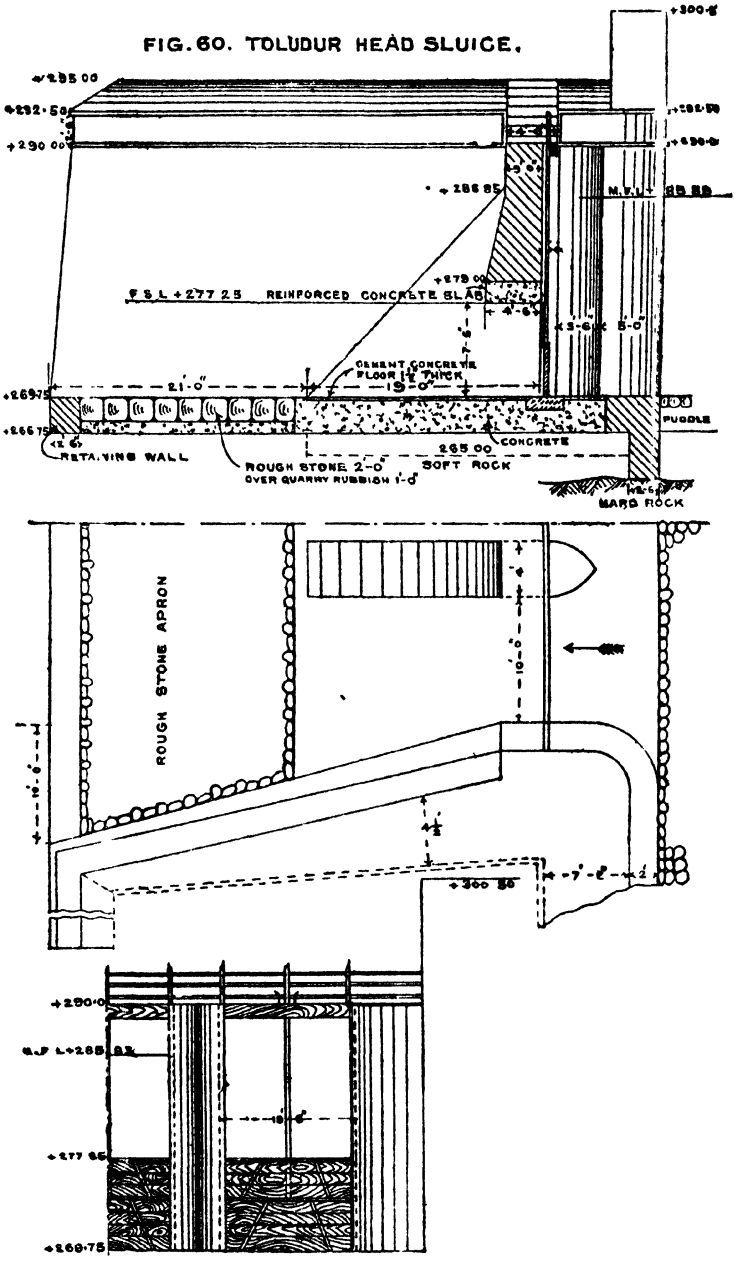
Generally speaking, for sluices carrying a roadway it is advisable to adopt the panel type where much extra expense is not thereby incurred; even with single vent it is frequently desirable to keep the springing of the main arches at a higher level than the top of the vent and close the face of the arch with a breast wall and subsidiary arch rather than to carry a low narrow archway throughout

This is a point frequently overlooked and has necessitated rebuilding, which would not otherwise have been necessary, of many sluices when the areas under them have been increased.

Where there is no roadway or embankment passing over a headsluice the superstructure may be restricted to sloping piers

and a breast wall. Figure 60 gives the design of Toludur Head Sluice which is a work of this type.

FIG. 60. TOLUDUR HEAD SLUICE.



347. Foundations and flooring of head sluices.—Better foundation soil is usually available for head sluices than for the

connected anicut and scouring sluices and thus the foundation work is frequently less costly; it is however to be realized that head sluices are subjected to greater statical heads than the weirs or scouring sluices and that, where the foundation soil is similar uplift and percolation are likely to be greater under the head sluice floor which must be designed accordingly. It must also be specially noted that the draw of scouring sluices will operate across the face of the head sluices and undue scour must be guarded against by a curtain wall or suitable apron up-stream of the face of the head sluice.

As water is not passed with undue velocity on the down-stream side of the vents the protective aprons of head sluices other than those required to resist piping or uplift, need not be nearly so extensive or heavy as in the case of river regulators, or scouring sluices which are subjected to high velocities and uncontrolled magnitude of flow

348. Head sluices of Minor Canals.—Many irrigation channels have their heads in rivers and streams at places where there is no anicut to prevent the river water falling below a convenient level. In such a case the head sluice only constitutes the permanent head work of the channel.

The sill of such a sluice is generally made to suit the level of the bed of the irrigation channel at the head which in turn is at the lowest level from which the irrigated land can be commanded. Where the off-take is from a river with a sandy bed, the sill is generally above the level of the deep bed of the stream both as a precaution against excess of silting, and in order to get command of higher land. In such channels the irrigation supplies are generally very fluctuating and the channels and head sluices must be designed for supply at a low duty to make up for the intermittent character of the supplies

In the case of many such channels it is customary during the low water season to construct temporary diversion works in the stream bed to raise the level of the flowing water and divert it into the channel head. Such temporary diversion works are termed '*korambus*' in South India They are generally made of sand and earth with brush wood and *nanel* grass. Sometimes they are carried right across the stream bed and sometimes they merely project into it sufficiently for diversion purposes, leaving a by pass round the end for excess flow. Where the construction of such works is customary, the local workmen are as a rule skilful in their construction and in making the best use of the locally available material. Whenever a fresh comes down the river the korambus are washed away, and an unreasonable fresh, far from improving irrigation supplies of korambu channels, is frequently the means of depriving them of necessary supplies.

The head sluices of such river channels should have vents large enough to allow full supply to be passed with a head of ¼ of a foot or less. Wherever regulating establishment is to be maintained the head sluices should have the regulation shutters in tiers so as to allow of supply in flood time being drawn from surface water.

connected anicut and scouring sluices and thus the foundation work is frequently less costly; it is however to be realized that head sluices are subjected to greater statical heads than the weirs or scouring sluices and that, where the foundation soil is similar uplift and percolation are likely to be greater under the head sluice floor which must be designed accordingly. It must also be specially noted that the draw of scouring sluices will operate across the face of the head sluices and undue scour must be guarded against by a curtain wall or suitable apron up-stream of the face of the head sluice.

As water is not passed with undue velocity on the down-stream side of the vents the protective aprons of head sluices other than those required to resist piping or uplift, need not be nearly so extensive or heavy as in the case of river regulators, or scouring sluices which are subjected to high velocities and uncontrolled magnitude of flow

348. Head sluices of Minor Canals.—Many irrigation channels have their heads in rivers and streams at places where there is no anicut to prevent the river water falling below a convenient level. In such a case the head sluice only constitutes the permanent head work of the channel

The sill of such a sluice is generally made to suit the level of the bed of the irrigation channel at the head which in turn is at the lowest level from which the irrigated land can be commanded. Where the off-take is from a river with a sandy bed, the sill is generally above the level of the deep bed of the stream both as a precaution against excess of silting, and in order to get command of higher land In such channels the irrigation supplies are generally very fluctuating and the channels and head sluices must be designed for supply at a low duty to make up for the intermittent character of the supplies.

In the case of many such channels it is customary during the low water season to construct temporary diversion works in the stream bed to raise the level of the flowing water and divert it into the channel head. Such temporary diversion works are termed '*korambus*' in South India. They are generally made of sand and earth with brush wood and *nanel* grass. Sometimes they are carried right across the stream bed and sometimes they merely project into it sufficiently for diversion purposes, leaving a by pass round the end for excess flow. Where the construction of such works is customary, the local workmen are as a rule skilful in their construction and in making the best use of the locally available material. Whenever a fresh comes down the river the korambus are washed away, and an unreasonable fresh, far from improving irrigation supplies of korambu channels, is frequently the means of depriving them of necessary supplies.

The head sluices of such river channels should have vents large enough to allow full supply to be passed with a head of ¼ of a foot or less. Wherever regulating establishment is to be maintained the head sluices should have the regulation shutters in tiers so as to allow of supply in flood time being drawn from surface water.

Figure 61 shows a design for river head sluice with shutters in 2 tiers moving in separate grooves. The upper shutter is capable of being either lowered or raised so that water can be drawn either over it or between it and the lower shutter. The level of the sill and the ordinary bed level of the channel at the head is + 6 being about 1 foot above the mean bed level of the river. The floor of the sluice would under ordinary circumstances be at the same level but, for the reason given in the next paragraph it is designed 2 feet lower The length of impervious apron gives a percolation

FIG.61

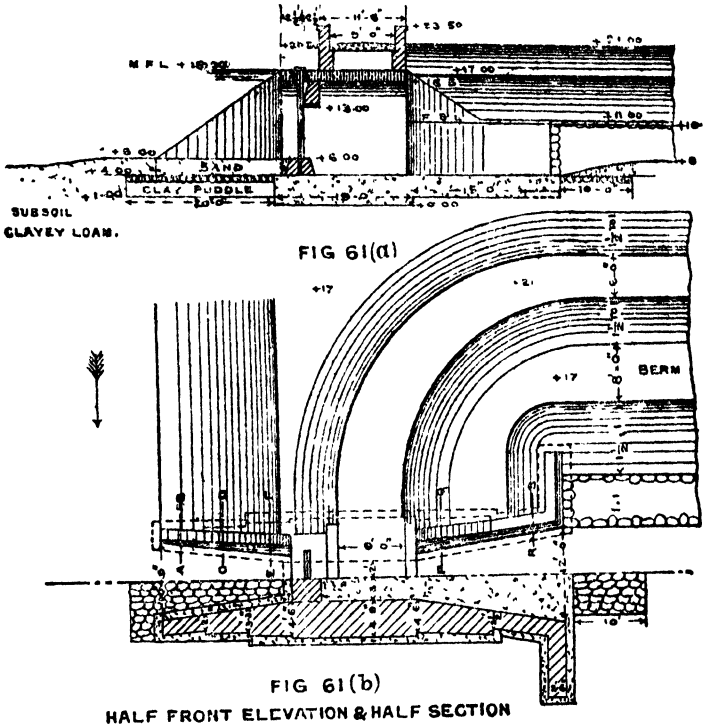


FIG 61(a)

FIG 61(b)

HALF FRONT ELEVATION & HALF SECTION OF SLUICE BARREL

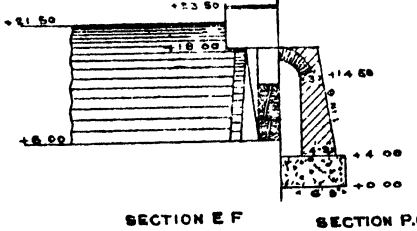
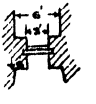


FIG.61(C)



SECTION E F

SECTION P.Q

SECTION.C.D

SECTION.A.B.

SECTION.R.S.

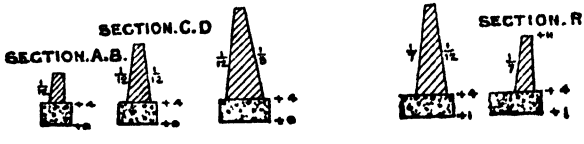


Figure 61 shows a design for river head sluice with shutters in 2 tiers moving in separate grooves. The upper shutter is capable of being either lowered or raised so that water can be drawn either over it or between it and the lower shutter. The level of the sill and the ordinary bed level of the channel at the head is + 6 being about 1 foot above the mean bed level of the river. The floor of the sluice would under ordinary circumstances be at the same level but, for the reason given in the next paragraph it is designed 2 feet lower. The length of impervious apron gives a percolation

FIG. 61

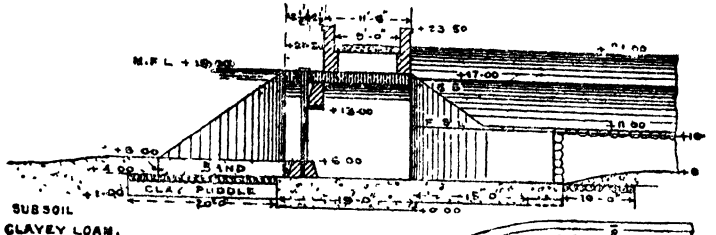


FIG. 61(a)

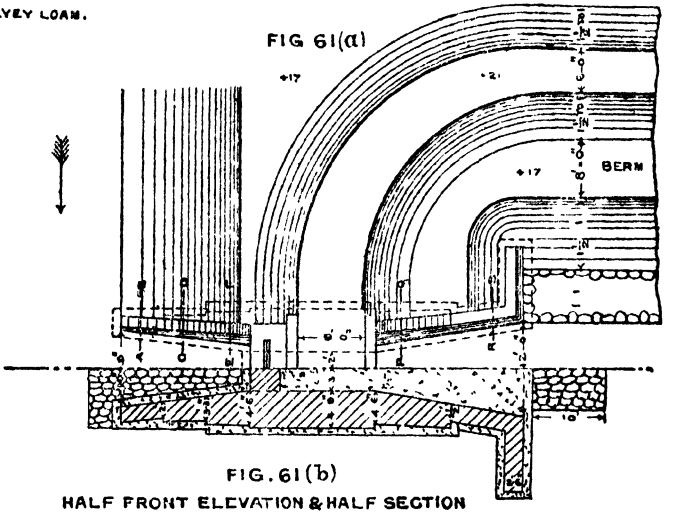


FIG. 61(b)

HALF FRONT ELEVATION & HALF SECTION OF SLUICE BARREL

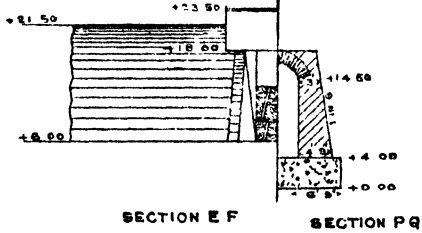


FIG 61(c)



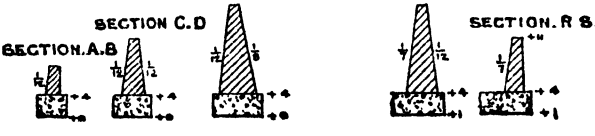
SECTION E F

SECTION P Q

SECTION C.D

SECTION A.B

SECTION R S.



gradient in maximum flood of 1 in 5 with the head sluice closed. The maximum flood level submerges the river margins and as there are no river flood banks the flood water is kept out of the channel by the banks of the channel itself.

349. Spring channels.—It is a common practice in South India to draw supplementary water supplies for minor channels having their heads in rivers with sandy beds, after surface flow in the river has ceased, by excavating a channel in the sand of the river-bed running up-stream from the canal head and with a bed slope considerably flatter than that of the river. The level of the bed of such a channel gets deeper below the river-bed the higher up the river it is taken. Such a channel draws sub-soil water by percolation from the river-bed throughout any part of its course which is excavated below the existing river-bed water level. These channels are called 'spring channels' In some cases the bed level of a spring channel at the canal head will be the same as the normal bed level of the channel and in such cases spring water will flow over the head sluice sill without any special arrangements; frequently however the spring channel where it enters the canal head has a bed level lower than that of the canal, and the spring channel is excavated in the canal bed and carried down it at a flatter bed slope than that of the canal till the bed levels of the two coincide.

Special provision for the spring channel has to be made in designing head sluices for such a canal and this can be done by cutting a notch through the sill wall of the sluice of section and depth to suit the dimensions and level of the spring channel Such a notch is kept closed during the flow season by a plank inserted in grooves which is removed during the low water season while the spring channel is in operation. The arrangement is shown in figure 61 the plan of notch in the sill wall being shown in figure 61-*c*.

Either the floor of the sluice must be at the bed level of the spring channel, as is shown in the design, or a small channel to carry the spring water must be made through the floor.

The main sill of the sluice might be made lower than the normal channel bed level in order to provide for the spring channel but this would increase the passage of bed silt into the canal head.

CHAPTER X.

HEAD WORKS.

STORAGE WORKS WITH EARTHEN DAMS.

350. Tanks.—Storage works are generally called 'Tanks' in South India, but a few of exceptionally large size are referred to as 'reservoirs'. There is no technical difference between a reservoir and a tank and the application of latter term is confined to Indian works.

Earthen reservoir dams are generally referred to as tank banks or bunds and this class of works is probably more numerous throughout South India than anywhere else in the world. The great majority of these works are of small depth comparatively with the works described in standard engineering treatises dealing with the subject of earthen dams. The general run of tanks in South India has a *maximum* depth of less than 15 feet, while some are as deep as 25 to 30 feet and in a few quite exceptional cases the maximum depth of water exceeds 35 feet The publications describing earthen dams and the precautions to be taken in their construction generally contemplate depths of 40 feet and upwards and prescribe a standard of work which would be economically impracticable of realization in reference to most of the tank bunds of South India. It is not proposed in this work to describe in detail the requirements of design in the case of earthen dams of great magnitude, for which reference may be made to Strange's "Indian Storage Reservoirs" and other works, but to describe the practice of design of small earthen dams which are dealt with in the common daily practice of irrigation in South India. The general principles of construction of earthen bunds for small works are however the same as for large ones although in practice the same precautions cannot always be applied to the construction of small as of large works.

351 Types of earth bunds—Earthen dams may be formed of material disposed in several ways in the profile of the work thus constituting separate types:—

Type A.—Of homogeneous material throughout the section.

This is the most usual method of formation and in it the soil used for the bund is the same throughout the section.

Type B.—A 'hearting' of plastic material supported by a casing' of more stable material containing a larger quantity of grit.

Type C.—A bund with an impervious core wall which is generally of puddle, but in exceptional cases may be of masonry.

352. Causes of failure of earthen dams.—The design of earthen dams cannot, as in the case of masonry dams, be based on a mathematical examination of their stability, and while the pressure entailed on the foundation soil by the weight of the super-incumbent earth must not be left out of consideration the design of the profile is based on practical considerations and on the experience

derived from success and failure of similar works and of their working conditions.

The causes of failure of earthen dams are—

(1) Percolation
(2) Slipping.
(3) Overflow, or topping of the bund.
(4) Erosion by waves.

353. Failure by percolation —Failure by percolation is due to water which percolates either through the bund or under its foundations carrying with it particles of soil, thus forming tunnels or runs which, when, once formed, rapidly enlarge forming cavities into which the bund sinks thus enabling the water to top it and carry it away by erosion

The cause of failure by percolation may be due to 'piping' and this is due to the insufficient length of the path of percolation and a resulting percolation velocity higher than the soil can safely sustain. Such failure would be attributable to inadequate profile or want of an effective stop wall to obstruct the path of percolation.

More frequently failure by percolation is attributable to the opening through the bund, or under its base, of a passage for water either by cracking of the soil, or by the burrowing of animals, or by roots of trees which have decayed leaving a free passage for water. While a large profile will tend to render failure from these causes less likely, increased size of profile within reasonable limits cannot secure against it, and the precautions to be taken are :—

(1) to construct the bund of material not liable to crack under the conditions to which it will be subjected ;

(2) to cover the up-stream slope of the bund (or preferably both slopes) with a layer of material through which burrowing animals will not penetrate ; and

(3) to keep the up-stream, if possible both slopes and also the top of the bund, clear of trees ; or in alternative, whenever a tree dies or is cut down, to open the bund and cut out the roots carefully filling in the excavations so made with good earth well consolidated.

354. Slipping of earthen banks —A slip in earth is due to the earth standing at a slope steeper than that at which it can be retained by the combined friction and cohesion between the particles composing it.

355. Qualities of plastic and grit soil —Both friction and cohesion are greatly affected in plastic soil by the amount of moisture it has absorbed in its pores ; in dry plastic soil both are considerable and a small quantity of absorption if anything increases cohesion but excessive absorption greatly reduces both friction and cohesion ; and a saturated pure plastic soil is likely to become slush. Plastic soils are also liable to crack when dry and this is specially so in the case of soils dried after admixture with such an excess of water as to form slush.

Grit soils, on the contrary, are little affected by moisture ; they have generally little cohesion but friction is considerable and increases with the size of the particles of grit ; these soils are not

liable to crack when dry. Pure grit is freely permeable and the larger the particles the greater the permeability.

The recital of these properties at once suggests a mixture of plastic and grit soils as giving a suitable material for an earth dam and as a fact both soils are used in the construction of bunds of type B, the plastic soil forming the "hearting" and a grit soil generally mixed with some adhering plastic soil is used for the "casing" that is the covering material.

Natural mixture of plastic and grit soils are frequently used for earth bunds of all classes.

356. Face slopes of earthen bunds.—To guard against slipping the face slopes of a bund should be fixed after consideration of angle of repose of the soil forming the bund under the conditions to which it will be subjected. It is necessary to design the bank so that the outer (down-stream) slope will not become saturated with water and this face of the bank should be given a slope not steeper than the angle of repose of the material composing it when in a dry state.

On the up-stream face it is impossible to exclude moisture but the slope receives much support from the water pressure on it, and from any revetment and backing with which it may be faced and this should be considered in settling the slope of this face.

357. Failure by overflow or topping of the bund.—If the water level in the tank rises to such a height that it flows over the top of the bund failure will speedily occur by erosion. This must be guarded against by surplus works of such capacity as to prevent the water level ever rising to such a height as can lead to failure from this cause. This assumes the top level of the bund will be kept up to standard, by occasional repairs

358. Failure by erosion.—This may take place by the water face of the bund being cut into and eaten away by the action of waves or erosion by rain and wind so that the slope becomes so steep that slipping of the upper layers occurs and the width of top of the dam is gradually eaten through when topping and failure ensure. This is generally prevented by revetment or pitching on the water face of the bund; in small works turfing or growth of shrubs sometimes affords adequate protection. In the absence of such protection constant petty repairs must be resorted to.

Failure from wave action may occur through waves washing over the top of the bund and gradually eroding the top and down-stream slope thus cutting a channel through the bank to a depth below the tank water level which speedily results in a breach. To guard against this, the top of the bund is made up to a certain height, called the "*freeboard*," above the maximum water level. This height depends on the "fetch" of the water-spread of the tank and the strength and direction of prevailing winds. The "freeboard" varies between, about 4 to 10 feet in tanks of different magnitudes.

359. Type of bunds of tanks in South India.—The tank bunds of South India are with few exceptions constructed of type A, and of soil dug from pits in the immediate vicinity of the section and carried by head loads to the bund Under such conditions there are throughout the Madras Presidency great varieties

of soil used for tank bunds, and breaches due to each and all the causes above enumerated are by no means infrequent.

360. Common dimensions of tank bunds for restoration work.—The following are the dimensions commonly taken as standard for repairing and restoring the bunds of small and medium size tanks in South India :—

Depth of deep bed below full tank level	Height of top of bund above maximum water level	Width of top of bund.
5 to 10 feet.	3 feet	4 feet.
10 to 15 „	4 „	5 „
15 to 20 „	5 „	6 „
Over 20 „	6 „	9 „

The above dimensions are suitable under normal conditions of wind and "fetch" but in special cases larger ones are given.

In favourable soil, such as red and white gravel, red and black loams, etc, the slopes of the bunds vary from 1½ to 1 for the smaller tanks with water depths not exceeding 8 feet, to 2 to 1 in larger ones up to about 16 feet.

In light sandy or black cotton or clay soils slopes vary from 2 to 1 to 2½ to 1

Where, as is usually the case, the water face is revetted this slope is generally 1½ to 1 up to 20 feet depth. For inferior soils or greater depths the revetted slope may be as flat as 2 to 1.

It is usual in many countries in spite of the revetment to make the water face flatter than the down-stream face because the soil is saturated, there are however thousands of tanks in Madras with water slopes of 1½ to 1, and failure by slipping of this slope is rare, the local practice in this respect should therefore be followed. In very small tanks or where there are specially heavy revetments water slopes of one to one and even steeper slopes are frequently met with; such steep slopes however are not recommended.

The effect of rainfall in the unprotected down-stream slopes of a tank bund is to erode soil from the slopes which is deposited at the down-stream toe of the slope which becomes widened and flattened, the upper part of the slope being steeper than as originally formed. When repairs are carried out the upper slopes are made up to their proper standard and so by degrees the down-stream slopes of tank bunds will generally be found to be flatter than the slopes above stated indicate.

The above slopes are stated with general reference to what is found to give reasonable security against slipping, having reference to the nature of the soil as regards angle of repose, but in addition it is necessary to see that the down-stream slope is flat enough to prevent dangerous percolation having regard to the qualities of the soil in respect to permeability. This is treated of in paragraph 370 below.

Figures 62 and 62-*a* show tank bunds of the general dimensions above indicated, the dotted lines show the flattening out of the toe of the down-stream (outer) slope and the rounding and lowering of the top by erosion and this is the usual condition of tank bunds unless covered by protective coating of broken stone, gravel, etc In such cases repair estimates would provide for bringing the upper part of the section up to standard profile dipicted.

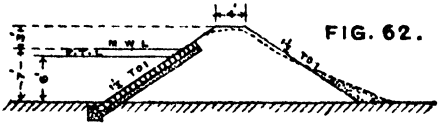


FIG. 62.

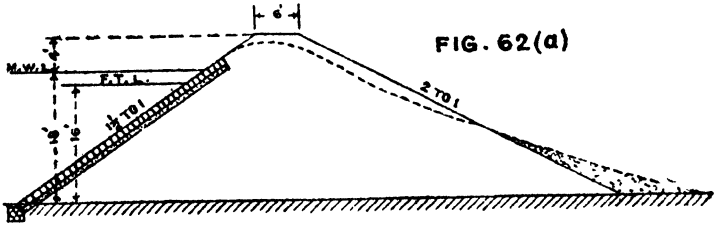
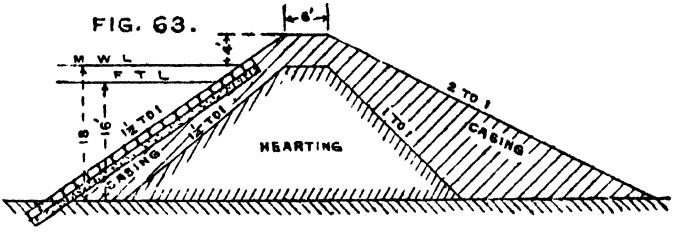


FIG. 62(a)

361. **Earthen bunds, type B.**—Bunds with different classes of material in the profile disposed as hearting and casing have been a good deal used in large earthen dams in India especially in the Bombay Presidency, and the practice of so disposing of material is coming more into use in Madras.

The advantages to be derived from such a construction will be indicated with reference to a profile (figure 63), which is similar in dimensions to that in figure 62-*a*.

FIG. 63.



A profile of type B may be advantageously employed for forming an earthen bund where, as frequently occurs, the soil available for constructing the bank is on the surface 2 to 4 feet of black cotton soil overlying decomposed rock forming soil chiefly grit with a small quantity of plastic material mixed with it.

The surface of a bund made all of black cotton will crack when it becomes dry and when the tank is empty these cracks are likely to extend into the heart of the bund and become a source of danger in the event of the rapid filling of the tank; also the water face is exposed to the danger of slipping in when saturated. A bund

composed throughout of the grit sub-soil would leak freely and much water would be lost by percolation, which also might attain a velocity endangering the safety of the bank. If, however, the plastic soil is used for 'hearting' and grit soil for 'casing' the former will stop percolation, whereas the latter will protect the hearting from cracking and give it support against slipping, while being itself to only a slight degree liable to these dangers; in addition the down-stream casing will owing to its permeability derive the advantage of being safe against saturation. Thus each material is disposed in a position where its peculiar qualities are most required.

It is true that permeability in the up-stream casing is a defect, and on this ground it is desirable to have a larger proportion of plastic soil in this than in the down-stream casing and if the casing material varies in quality that with most plastic soil should be placed up-stream, and for the rest of the section the more impervious casing should be close to the hearting and the most permeable on the down-stream face composing the toe of the dam.

362. Earthen bunds with core walls, Type C.—The core wall of a bund of type C is generally clay puddle as shown in figure 64.

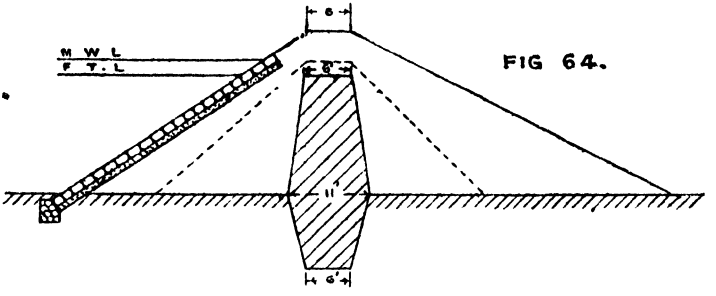


FIG 64.

Such a type only differs in degree from type B the comparatively thin puddle wall taking the place of the 'hearting'. The bulk of the material of the bank on either side of the puddle wall, especially on the water side, should be of the most impermeable material conveniently obtainable, and the wall used as a water tight bulkhead to intercept any percolation which passes through the up-stream thickness of the dam. High class puddle clay is rarely obtainable in South India and the use of puddle in dams is generally restricted to filling trenches excavated below the level of the seating of the dam down to an impervious stratum, or to a depth sufficient to lengthen to a safe degree the line of percolation below the foundation.

The dotted lines figure 64 show the position of a clay hearting of a type B bund and this with the puddle *trench* is a combination which is generally more suitable in South India than the puddle wall.

In some earthen dams, especially in America, concrete or rubble masonry core walls have been used. In India the cost of such a method generally precludes the use of masonry but occasionally it may be adopted to form a stop wall below the foundation level of a dam especially at the stream crossing. Where this is done the

top of the wall must be carried to a sufficient height into the bund to give security against dangerous creep.

363. 'Saturation gradient' in banks.—All earthen banks against which water stands gradually become saturated up to a surface sloping downward through the bank starting at the water line on the upstream face of the bank. The actual plane of saturation through a profile is generally somewhat irregular and it changes with the changes in the level of water standing against the bank. The plane of saturation also varies with the nature of the soil composing the bank and in a perfectly homogeneous bank standing on a similar foundation soil the saturation gradient due to a steadily maintained water level, would be a straight line and the steeper the inclination to the horizontal the greater the resisting power to percolation of the bank material and vice versa (vide figures 65 and 65-*a*)

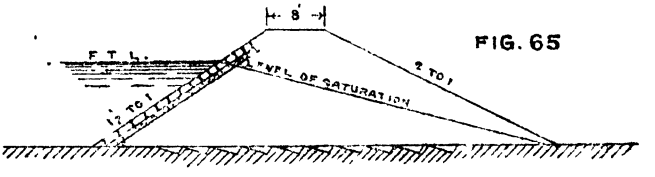


FIG. 65

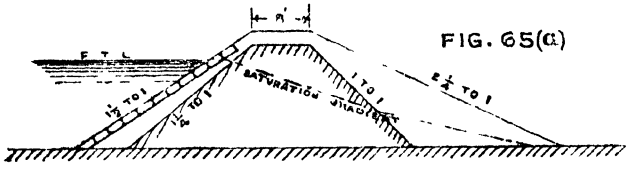


FIG. 65(a)

The slope of the line (or plane) of saturation is called the 'saturation gradient' or sometimes the 'hydraulic gradient' of the bank.

A number of observations of the level of the line of saturation in earthen banks have been made in many places and particularly in the Bombay Presidency by observing the level at which water stands in perforated pipes sunk in various places across profiles of the banks subjected to observation. The line of saturation is taken as the level of the water surface in such pipes.

From the Bombay observations it was found that the slope of the line (or plane) of saturation measured from the water surface in the tank to the sub-soil water level at the down-stream toe of the bund for seven tanks with type A embankments was 0·32 vertical to one horizontal; the slopes varied greatly for individual banks and in one case was as flat as 0·12. In earthen dams of type B the slope through the 'casing' soil averaged about 0·15 while that through the hearting was about 0·33.

It is usual in Madras to design a bank of fair average material so that a saturation gradient of 0·25, that is a slope of one in four, will fall within the profile and in most cases it is desirable to have a margin of not less than 1½ to 3 feet, in accordance with the importance of the work, between the gradient and the surface of the bund. This soil will never be saturated and will reduce the tendency to slip.

364. Drains under the outer slope of a tank bund.—When the soil at the site of a bund, or of the outer slope is of a nature which becomes soft when saturated involving danger of settlement or slip the case is frequently dealt with by constructing rough stone drains under the outer slope of the bund thus providing a path through which percolation water can find ready exit and preventing dangerous saturation of the slope and of the ground on which it stands.

These drains are merely trenches filled with rough stone and protected from being chocked with soil washed into them from the bund, by being enclosed in casings of broken stone and pebbles or gravel or chips from the quarries so as to form a strainer which prevents the soil being carried with the water into the drains. The water from drains must be led right away from the toe of the bund; whether there are drains in the bund or not it is objectionable to have water or sloppy saturated soil at the outer toe of a bund and this ground should whenever practicable be drained and kept free from stagnant water.

Figures 66-*a* and 66 show a plan and section of a system of drains inserted in a bound foundation where crossing a marshy soft place. The drain *a a* parallel to the length of the bund is given a bed fall towards the outfalls generally following the fall of the

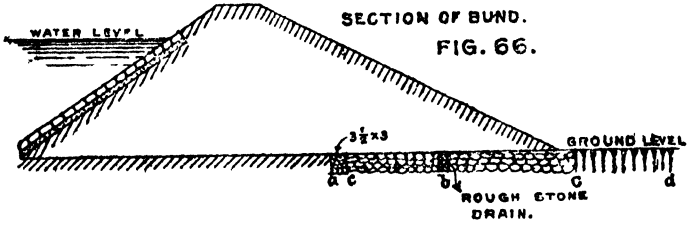


FIG. 66(a)

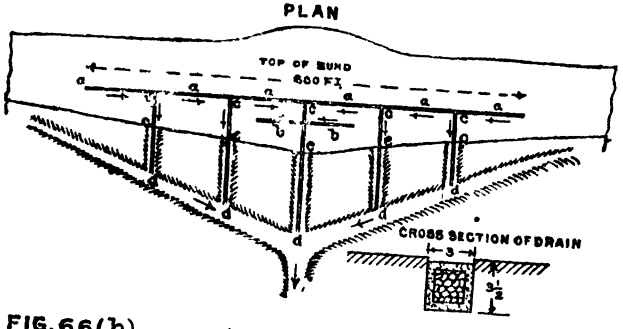
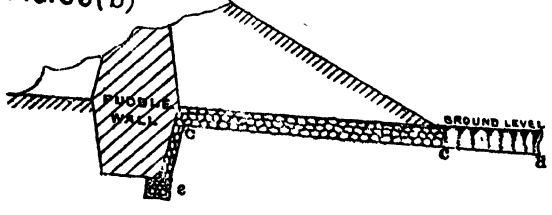


FIG. 66(b)



ground, drains *c c* at right angles to the line of bund also have a suitable bed fall; these drains when clear of the toe of the bund are led away in open channels graded so as to keep the ground in rear of the bund well drained and to have a free run-off. When the subsoil is permeable and a puddle trench is resorted to, a drain *e*, figure 66-*b*, along the down-stream lower edge of the trench is desirable if the subsoil is of a nature which gets soft when saturated. Such a drain is connected at intervals with the outfall drains *c c* by drain shafts *e c* (figure 66-*b*).

365. Top width and free board of tank bunds.—The top width and free board of bunds of small and medium sized tanks are usually kept the same throughout, but this practice may frequently, with advantage, be departed from. The freeboard and top width require to be greatest opposite those places where either the fetch is greatest, or on which the prevailing winds act with most force. Towards the flanks the freeboard may frequently be reduced, with advantage, both to economy and safety and the same remark is applicable to the top widths also. While it is necessary that banks should be sufficiently high and wide as to give reasonable security against breaching it is desirable that if a breach by overflow should occur, it should occur at a place which will do the least damage and allowing a lower freeboard at such places conduces to such a result. On the other hand it is desirable to give small local increases in top level at places where a breach would be specially disastrous, such as at and near sluices, and at stream crossings, etc.

366. Breaching sections.—In many tanks economic considerations do not allow construction of surplus works on a scale which will dispose of all possible floods, such as perhaps occur once in thirty years, and in such cases 'breaching sections' should be provided in the flanks or other parts of the bund where a breach will do the *minimum* damage.

A 'breaching section' is a length kept lower and weaker than the remainder of the bund so as to localize a breach in that length.

367. Revetment to tank bunds.—The water faces of tank bunds are usually protected from erosion by roughstone revetment which is frequently carried from end to end of bund. In many cases, however, revetment is only provided in the lengths of face where the waterspread is widest or the bund most exposed to wave action.

Revetment not only forms a most effective protection against erosion but acts to some extent as a retaining wall so that a revetted slope will stand at a steeper slope than unrevetted one.

The thickness and length of revetment must be decided in each case on its merits.

It is common to revet with 1½ feet thickness of roughstone which is packed over a backing 6 inches thick of broken stone, quarry rubbish or gravel. In the case of small tanks and low revetments not exposed to heavy wave action one foot thickness is frequently sufficient, on the other hand on slopes exposed to heavy wave action or the lower layers of high revetted slopes the thickness may be increased to 2 feet or occasionally more.

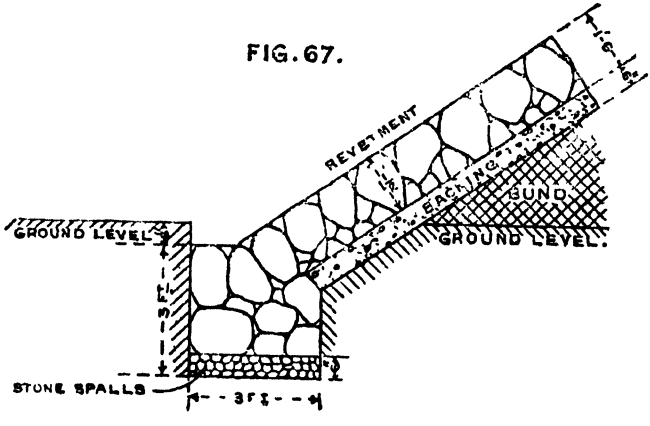
The backing must be of material which does not get soft when saturated with water and which burrowing animals will not bore through ; the particles should be of considerable size and weight so that they will not be drawn out from behind the revetment stones by the action of receding waves. Stone metal or pebbles, clean quarry spalls, broken laterite or clean gravel all make good backing.

It is common to provide for gravel backing, but the material frequently supplied as gravel contains but a small proportion of stone pebbles to a large one of plastic red soil; such material does not make suitable backing and should not be used for the purpose. Gritty soil suitable for casing such as disintegrated rock, shale kunkur, etc., is also suitable for backing and when the water slope of the bund is faced with such soil no special backing is necessary.

Generally the top of the revetment is carried to one foot above M.W.L. of a tank but in important tanks or in very exposed situations revetment should be carried up to the top of the slope.

The foot of a revetment should generally be carried two to three feet below the foot of the protected slope in order to give it a footing below saturated and soft top soil of the tank bed. It should be widened out at the toe so as to distribute the pressure over a wider area (vide figure 67).

FIG. 67.



The revetment should be built with as large a proportion of through stones as possible and all the work should be carefully built with due regard to bond and the interstices between stones should be filled with smaller stones and spalls and wedged up tight with spalls driven in with a hammer between the larger stones.

DESIGN OF EARTHEN BUNDS.

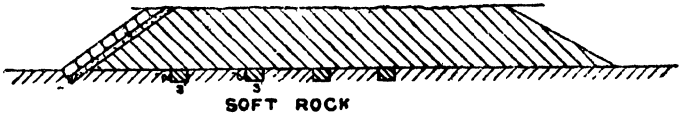
368. Cross section of bund to suit the earth available.—The design of a suitable profile for an earthen dam to retain a known depth of water requires a detailed consideration of the nature of the soil or soils available in the immediate vicinity for the construction of the bund. The angle of repose of these soils dry, damp, and

saturated must be considered and the nature and permeability of the foundation soil on which the bund must be founded must also be taken into account. If the head of water to be retained is not great (say not exceeding 30 feet) it is generally necessary on the score of economy to make use of soil, for each length of bund, which is not too far removed to be carried by head-load direct from the pits to the bund Thus generally it is necessary to design the bund to suit whatever soil is available at site and where the soil or foundation is unsuitable for an earthen bund the alternatives are, to abandon the site, or to make a safe bund by designing to suit the special circumstances of the site and soils available

In the case of a new tank an otherwise suitable site may have to be abandoned on account of the unfavourable nature of the earth and foundations available, but in the case of earthen bunds required for river and channel banks, and for closing breaches in existing tanks the site is fixed and the work is frequently imperatively necessary, and there it is necessary to make use of whatever soil is available In extreme cases it may be necessary, while using for the bulk of the work the soil excavated from adjacent pits, to convey superior material from a distance by cart or tramline to form a core wall

369. **Foundation of the bund.**—If the site of the bund is on the surface of impervious soil of sufficient thickness to stop percolation under the work and capable of carrying the weight of earth to be placed on it, the bund should be built on the surface which however should be dug up or ploughed along the length of the bund so as to form a good joint with the new earth to be laid over it. When the depth of water exceeds 12 feet, trenches as in figure 65 should be dug. Surface vegetation and roots should be removed. Where the foundation is soft rock this should be trenched as in figure 68 and if of hard rock which cannot be so treated a serrated joint may be made between it and the earth by a series of dwarf walls built on it of similar dimensions and spacing to the trenches shown in figure 68.

FIG 68.



When the surface is permeable soil overlying an impermeable stratum the former may either be altogether removed and the bund seated on the latter, or one or more trenches dug down to a foot or two into the impermeable strata and filled with puddle, of the best procurable material, to form stop walls In exceptional cases a masonry wall may be used as a stop wall. When the permeable soil goes down to a great depth the stop wall (whether of puddle or masonry) must be of such a depth as—

(1) to prevent percolation attaining such velocity as to endanger the solidity of the foundation soil by piping, and

(2) to prevent excessive loss of impounded water.

In connexion with the seating of the bund it should also be considered if it is necessary to secure the foundation or outerslope of the bund from settlement or slipping by introduction of drains under the outer slope.

370. Profile of the bund.—The best type of bund must be decided with reference to the nature and distribution of material available and the bund designed with reference to the probable slope of the saturation gradient in the material to be used and the probable angle of repose under the conditions of the case. A plastic clay such as black cotton or black silt (tank-bed silt) is so liable to cracks when dry and to slip when saturated that it should be used in combination with casing of some other soil whenever this is economically practicable.

When a little casing material only can be obtained it may be used on the water slope this being the part of the profile in which cracks are the greatest source of danger. If a gravel or other gritty material with large particles is available it only requires sufficient plastic material to fill the interstices to make an excellent material for a bund and if naturally deficient in plastic material this may be rectified by mixing with the grit a suitable quantity of plastic soil and using this for the upstream two-thirds of the bund, or in alternative only for a hearting.

Such a mixture of materials is however costly and it may be economically more suitable to use the material available without going to the cost of mixing and to make a wider bund allowing for a flat saturation gradient. Light sandy loams of deficient power of resisting percolation may be used as type A bunds by allowing for a flat saturation gradient. A puddle wall is useful in combination with such soils.

Very light loamy soils only are generally available for flood banks and banks of canals, etc., in the Punjab and it is there a common experience that banks of irrigation works are only safe when a saturation gradient of 1/7 falls within the toe of the bank. In Madras it is seldom necessary to make banks of such light soil and a saturation gradient of 1 in 4 is the ordinary basis of design.

In settling a suitable up-stream slope for a bund the nature and thickness of the revetment or other slope protective work must be considered as well as the nature of the earth and the slope which it can stand at when saturated with water.

371. Important points in regard to actual execution of embankments.—(1) The majority of canal and flood banks are of type A, and when supervising the construction of an earthen dam of any type, the dangers to which such a bank is liable should at all times be borne in mind and the soil available disposed of in the way best calculated to promote safety and this will generally be secured by throwing the most clayey soil in the centre of the bund the tendency being rather towards the water face.

(2) The position and limits of depth of pits with respect to the bank must be decided before commencing work and the positions of all pits required for completing the work roughly located. It is generally desirable to make most of the pits in the tank bed as these will silt up much more rapidly than pits on the outer side. If, however, the surface soil is an impervious layer and excavation

of pits will expose a pervious and otherwise open a path for percolation, excavation of pits in the tank bed should be avoided. In any case no pits should be excavated nearer to the toe of either slope than thirty feet or four to six times the ultimate height of the bund whichever is greater, and an additional four to six feet in accordance with the nature of the soil should be added to this for every foot the depth of the pit exceeds three feet.

(3) When commencing work it is desirable to take earth first from the more distant pits gradually lessening the lead as the work rises.

(4) The positions of pits from which earth required for the final closing of the bund will be taken should be those most convenient for the rapid execution of this work, and these positions should be reserved for this purpose. Failing this there is likely to be a long lead for the earth for a work the success of which greatly depends on rapidity of execution.

(5) Arrangements should be made to prevent interruption of work by the pits being filled with surface drainage after rain.

(6) The seat of the bund should be prepared to receive new earth by removal of all vegetation and roots and roughing the surface by ploughing or digging all over. Any special work, such as trenching, which is necessary to make a good joint between the earth and the base of the bund should be shown on the design.

(7) The bund should be formed in layers of earth stretching right across the whole section; such layers should be of thickness from four inches to ten inches. The surface of the layers should be formed so as to slope inwards at about one in twelve towards the centre of the bund.

(8) All clods should be broken up small before being brought on to the bund.

(9) Consolidation of earthwork is most important in the formation of a water-tight bund and must be done in accordance with specification It may be effected by ramming, by driving animals such as buffaloes and cattle over the layers, or by rollers either moved by power or drawn by animals or men. A power roller is the best means of consolidation but is seldom available except on specially important works. In very small bunds retaining only three or four feet of water these methods may sometimes be dispensed with and in such cases the layers of earthwork should not be more than four inches thick.

(10) On no account must a bund be originally made of less than the full width with a view to widening subsequently; this leads to the formation of a joint which is likely to start a slip.

(11) It is an advantage to apply plastic soil in a slightly damp condition to a bund; in such condition the soil under consolidation flattens into a homogeneous mass devoid of visible interstices. If the surface of a layer is quite dry it should be lightly watered before adding the next layer, this causes the two layers to adhere and there is less danger of forming a joint plane. As such soil becomes sloppy and expands when mixed with much water the least quantity of water which makes the soil plastic should be used. When pits are dry, it is a good plan to water them in the evening as this facilitates excavation and results if the watering is judiciously done, in the soil being brought to bank with sufficient but not too much moisture.

(12) When puddle is used it must be thoroughly worked up into a plastic mass with as *little water as suffices to bring it to this state* and when placed in the work it must be well incorporated with the layer below. Special precautions must be taken to prevent the puddle in a trench (or wall) ever becoming dry, as this will cause cracks; any crack which is formed must be cut out and fresh material laid in place of that removed.

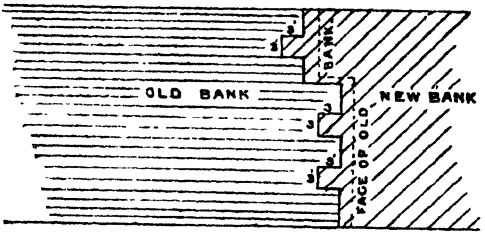
(13) Where practicable the bank should be given time to settle before the revetment is built; the settlement of a new bank sometimes causes the revetment to bulge.

372. Junctions in embankments.—Junctions in earthwork should be as few as possible and should be levelled in section and joggled in plan as shown in figures 69 and 69-*a*

FIG. 69. SECTION.



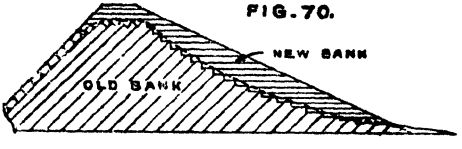
FIG. 69(a) PLAN.



When avoidable, junctions to a depth of more than 20 feet should not be made in one season. Deep junctions of this kind will normally only be necessary where closing the gap left for passing the flow when constructing a new bund or closing a breach in an old one

Where an old bund has to be widened by the addition of earth the old slope should be cut into steps with the treads sloping slightly towards the centre of the bund and when throwing on the new soil the surface of the old work should be wetted so that the new earth may adhere to the old (vide figure 70).

FIG. 70.



373. Draining or filling of scour holes.—The importance of draining the site on which a bund stands has already been alluded to and this also applies to the ground immediately down-stream of a bund. If water stagnates here it will prevent the drainage of the rear slope and seat of the bund.

A deep scour hole is frequently formed at the site of a breach, and water lies in this and may form a deep pool at the foot of the outer slope of the embankment closing the breach.

If as frequently happens, the fall of the country does not permit of natural drainage of such a pool, it must be filled up, if possible, with soil which will not slush, or if too large to fill altogether it must be filled for some distance, say twice the height of the bund, from its rear toe.

374. Testing soil for embankments.—In the case of important works the qualities of any soil or mixtures of soil may be tested by marking with it a test tank, say 10 feet square at the foot of the water faces and of depth 3 or 3½ feet. The water face may be 1½ to 1 and a similar outer slope with top width 3 feet and free board 1½ feet. The tank should be kept for about a week filled to within 1½ feet of the top, any percolation losses being made good at intervals. The actual slope of the percolation plane in the banks should then be observed and will be a guide to the qualities of the soil. By gradually cutting back the outer slope the qualities of the soil as regards slipping when saturated may be observed.

The quality of a plastic soil, or of a mixture of plastic and grit soils, to resist saturation by water may be tested by making up balls of the soil 4 or 5 inches in diameter; the soil should be damped and kneaded up in the hands until it becomes a stiff plastic mass before making into balls. The balls may then be immersed in *still* water 1½ to 2 feet deep and left there. A good clay soil will resist saturation for days, and even for weeks, while an inferior soil will become saturated and fall to pieces in a few hours.

It is useless testing a dry lump of plastic soil by immersion in water as it will speedily absorb excess of water in its interstices and fall to pieces; this would not however happen if the soil is pulverized so as to have but little interstitial content and consolidated and confined in an enclosed space where it cannot expand when water reaches it; under these conditions it can only absorb water gradually, and in doing so attains the state of plasticity in which its properties of resisting saturation and percolation are fully developed. Clay soil in such a condition is not only highly impermeable but is also very retentive of the water absorbed, and pure clay soil in the heart of a big bund when once in a suitable plastic condition will retain moisture and remain plastic in a very dry climate for one or two years, even if no water has been standing against the bank for such a period.

375. Testing the permeability of a tank bed.—If the surface soil in the bed of the proposed tank is not more pervious than the subsoil, pits should be dug in various places in the bed to a depth of about 3 feet, but not more than half the thickness of the retentive surface soil; if this surface layer is not more than 4 feet thick small tanks on the surface may be made in place of pits. The pits, or tanks, should be filled with water and the rate per hour at which the water surface in each pit (or tank) falls should be observed and from this the loss from combined percolation and evaporation per unit of area can be computed. If the surface soil is pervious and the subsoil is relied on to retain the water in the

tank, the pits should be excavated down to the subsoil level and their sides lined with clay or other impervious soil so that all percolation loss will take place through the bottom of the pits. The rate of percolation should then be observed as above.

In all cases the pits should be kept full of water for not less than 24 hours before commencing to observe the rate of loss; this is necessary to give the surrounding soil a suitable amount of moisture.

RESERVOIR SUPPLY SLUICES.

376. **Tank sluices.**—A sluice consists of a culvert or pipe running through or under the bund to convey water from the tank

FIG. 71.

TANK SLUICE. ARCHED TUNNEL WITH PLUG.

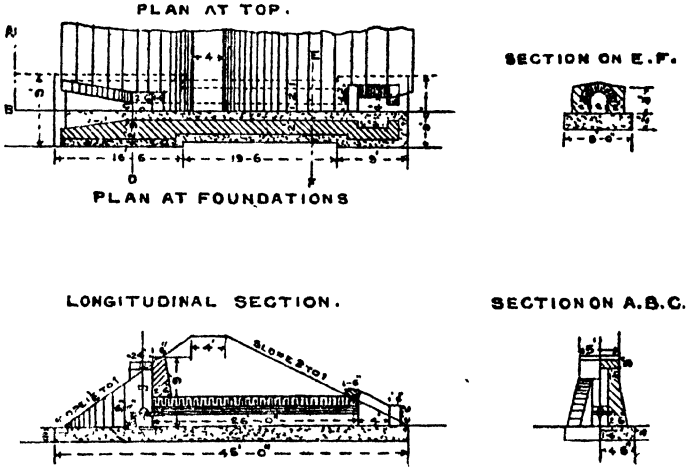
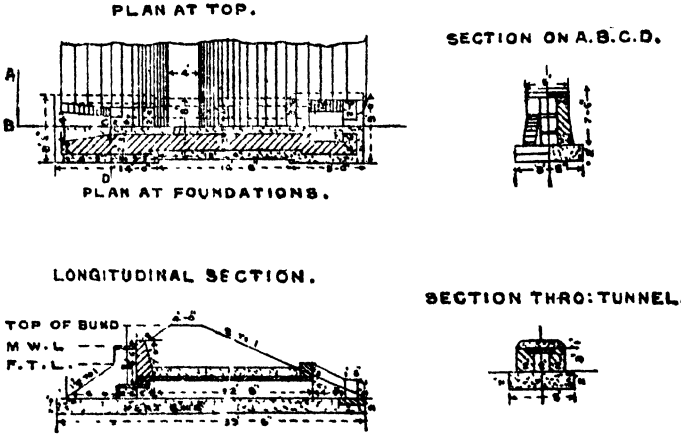


FIG. 72.

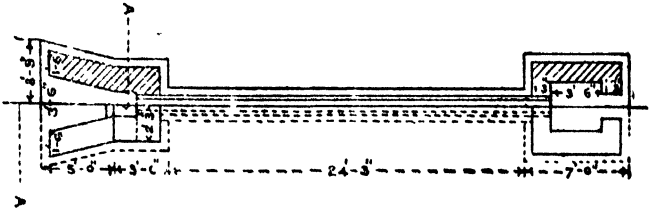
TANK SLUICE. STONE SLABS.

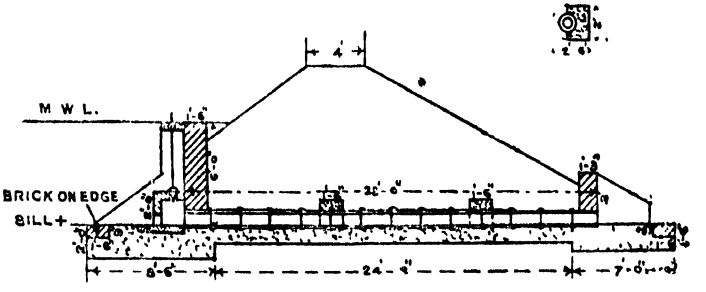


to the distribution canal or channel with a wall, or tower head, at or near the up-stream end on which are installed the regulating arrangements and from which these can be operated. Suitable wing walls or other bank connexions are required at the head and tail end of the culvert (vide figures 71 to 74).

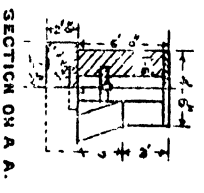
FIG. 73.

EARTHENWARE PIPE SLUICE.





LONGITUDINAL SECTION.



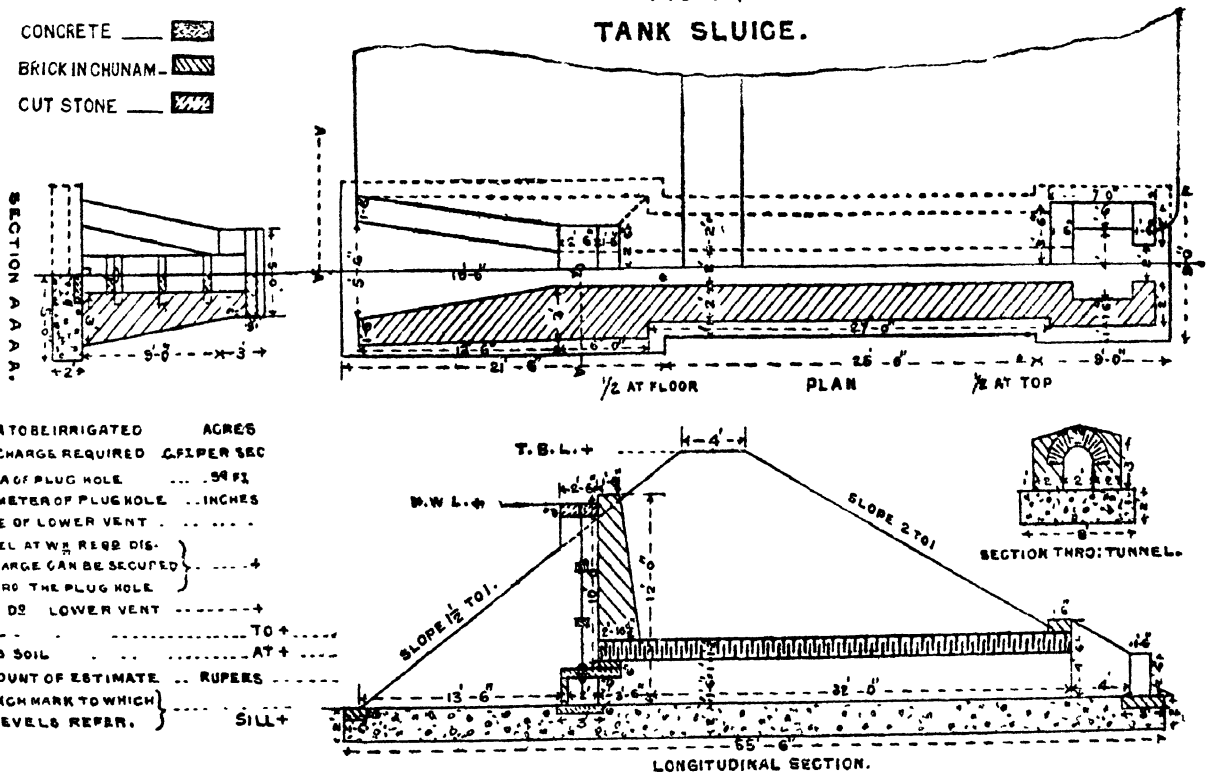
NOTE —

IF THE SOIL IS HARD THE CONCRETE UNDER THE PIPES MAY BE OMITTED AND THE PIPES LAID IN 9 INCHES PUDDLE AS PER SECTION

AREA TO BE IRRIGATED ..... ACRES
DISCHARGE REQUIRED . C.FT. PER SEC
AREA OF PLUG HOLE .. .. .. SQ. FT.
DIAMETER OF PLUG HOLE . . INCHES
SIZE OF LOWER VENT ..... —
LEVEL AT WHICH DISCHARGE CAN BE SECURED THRO THE PLUG HOLE } ... +
DO— DO— LOWER VENT .. +
SOIL .. ..TO +
SUB SOIL .. .. AT +
AMOUNT OF ESTIMATE .... RUPEES

FIG. 74.

TANK SLUICE.



377. The culvert or barrel of the sluice.—The size of this culvert will, of course, be proportioned to the maximum quantity of water it is required to convey but in no case should be less than 2 feet wide and 2½ feet high so as to allow a man to enter it for examination and repairs of removal of obstructions. The size should also be such as to limit the velocity through the sluice barrel to 15 feet per sec. under the condition of the plug hole being fully open with the water at full tank level (F.T.L.) For very small sluices, pipes either of earthenware, cement, or cast iron may be used to take the place of masonry culverts. Sluices in which pipes are thus used are termed 'pipe sluices' (figure 73). It is not desirable to use earthenware or cement pipes in parts of bunds at depths exceeding 8 feet below full tank level as the fracture of a pipe or leakage of the joints may result in a breach while the examination of the condition of the pipes or repairs cannot be carried out without cutting open the bund. It is rarely economical to use cast iron pipes in place of masonry for tank sluices.

378. Foundations of sluice culverts.—The foundations of masonry culverts are almost invariably a slab of concrete, which should be of suitable width and thickness having in view the nature of the foundation soil, and the weight and nature of earth above the culvert. It is important to secure as far as practicable a uniform foundation soil throughout the length of the culvert so that the settlement may be uniform. If there is a puddle trench the crossing of this is a point of weakness and if practicable the sluice should be placed in a site where no such crossing is necessary. Where a crossing is unavoidable the puddle wall may be stopped on either side of the site of the sluice and a masonry stop wall substituted. As an alternative the part of the barrel which passes over the puddle may be constructed as a separate piece of work jointing into the lengths on either side with a plain butt joint filled with mortar without bond; this is termed a 'slip joint' The former method is generally to be preferred

379. The superstructure of a sluice culvert.—The sluice culvert is generally covered with masonry archwork either of stone, brick, or concrete. When the bank above is not very high and when economy is served, stone or ferro concrete slabs may be substituted for archwork. When this is done a layer of concrete 6 inches to 1 foot thick should be laid over the slabs to prevent leakage between them. The two types are shown in figures 71 and 72.

The length of the barrel should be designed to suit the cross section of the tank bund. In the case of a new bund it is generally desirable to prolong the barrel under the outer slope to suit the flatter slope which this will eventually assume and the earthwork over a sluice may be made slightly higher than the normal section of the bund; in the case of an old tank the length of the barrel should conform to the slopes of the bund on either side of the sluice site This is a point frequently overlooked in design and in consequence the lengthening of barrels of tank sluices is frequently found necessary in carrying out restoration works; it is also very common to find the narrowest profile of the bunds of tanks at sluices which thus are below average at these works instead of being given, as they should be, extra strength at such places. The

cost of adding to the length of a sluice barrel at the time of construction is always far less than that of subsequently building an equal additional length.

Figure 71 gives an ordinary type of tank sluice to suit an outer slope of two to one. If this is the normal outer slope of the tank bund after settlement on either side of the work the length of barrel is suitable but otherwise it would be desirable to extend the barrel from 4 to 6 feet in the original design. An alternative to this would be raising the tail cistern walls to take a flatter outer slope; this although not so efficient is much cheaper than extending the barrel after construction and may consequently be frequently found suitable.

380. Stop walls across the barrel of a sluice.—A sluice is always regarded as a danger point in an earthen dam chiefly on account of the liability of water under pressure to creep along the joint between the masonry of the barrel and the earth. It is therefore desirable to keep the surface of the masonry along this joint as rough as possible so as to offer every facility for making a tight joint This is a point often overlooked and the joints and surfaces are flushed smooth so as to present a good appearance. It is of the utmost importance to see that the best kind of soil procurable is used for filling in round the sluice barrel and behind the wings and that this is put in, in thin layers, and thoroughly consolidated by ramming. Such precautions are sufficient for ordinary heads of 16 to 20 feet but with heads of 20 feet and upwards it is advisable to build two or more stop walls along the up-stream two-thirds of the barrel length to form a further obstruction to creep. Two such walls are shown in figure 77 below

381. Head wall and wings of tank sluices.—Heads should be built up to a sufficient level to carry a platform at or above maximum water level (M.W.L.) so as to allow convenience for regulating the sluice at all times. Where a head wall is provided it must be connected with wings of suitable length and slope to retain the earth slopes of the bund; the types shown in figures 71 to 74 show a common and suitable arrangement.

382. Sluice with a tower head.—Figure 75 gives a drawing of a tank sluice with a tower (or well) head. This form is sometimes more economical than the type previously referred to, owing to the saving by avoiding the heavy wing walls of the previous type; there is also less danger of failure by cracking and bulging which not infrequently occurs in the wings of the other type; on the other hand with the tower head an additional length of barrel is required. Tower heads are generally placed in the water slope of the bund as thereby the expense of a bridge or causeway leading from the bank to the regulation platform is saved. This has the disadvantage of making it impossible, while the tank has water, to have access to the portion of the barrel up-stream of the well.

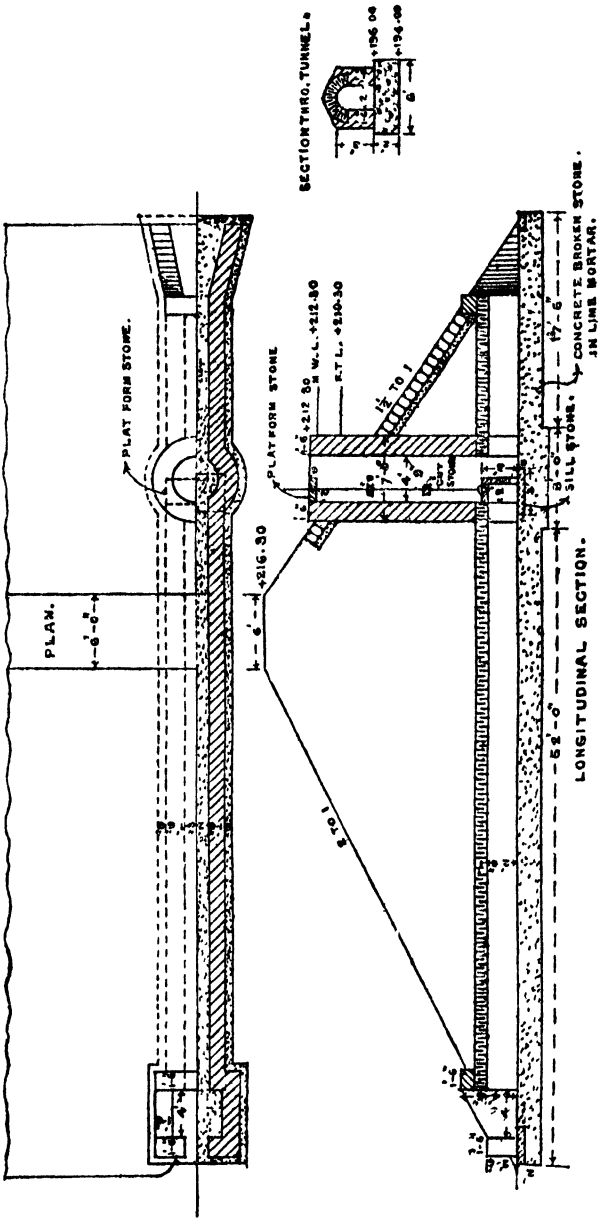


FIG. 75.
TANK SLUICE WITH TOWER HEAD.

383. Method of regulation of tank sluices.—The vent generally used for sluices of moderate size consist of one or more circular orifices of suitable size in a horizontal stone and the area of opening is regulated by a long coned plug the wider end of which is of the same diameter (allowing for clearance) as the vent

which is termed the 'plug-hole' (vide figures 71, 72, etc.). As the plug is raised the orifice is opened more and more and when the plug is raised altogether clear of the plug hole by an amount not less than the diameter of the hole the full discharge of the orifice is obtained. The arrangement is both simple and economical and affords an easily adjusted means of regulation to suit the very varying conditions of water-level which are inseparable from reservoirs and is more suitable for tank sluices than the flat shutters which are in general use for channel sluices.

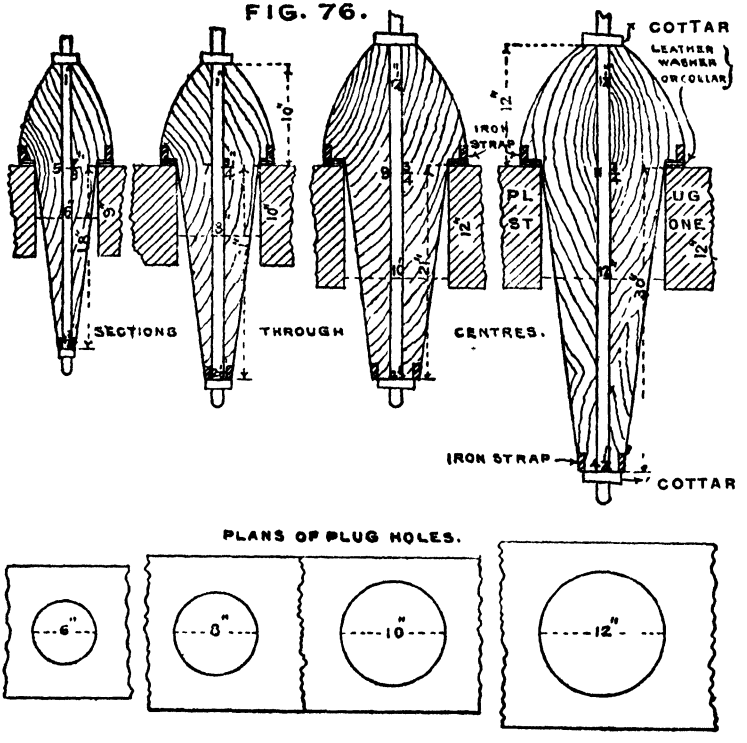
Where plug holes would exceed 8 to 10 inches in diameter, it is generally advisable to provide two or more holes and plugs in the plug platform stone, while for deep sluices, say over 25 feet depth, two or more plug stones at different levels each with separate plug holes give great facilities for easy regulation, the upper platform being used when the water is high and the lower one when it falls to such a level that all the water required cannot be passed through the holes in the upper plug stone. The size of the orifices in the plug stone are generally calculated so as to enable full supply to be passed with vents fully open and with one foot of water standing over the plug stone platform. For regulation below this level a rectangular vertical vent is provided which is closed by a shutter, which may be wood but is generally a slab of stone; this vent is made of sufficient size to pass full supply with ½ foot head. This shutter is brought into use only when water in the tank is less than 4 feet over the floor of the sluice and at other times the shutter is kept completely covering the orifice and leakage past it should be stopped by a filling of silt against its up-stream face. Regulation of this vent will be done by men standing on the plug stone. In the case of large tanks where the quantity of water to be issued is great and where adequate establishment is maintained, flat shutters working in grooves and regulated by screw spears may be installed This type of shutter and regulating gear is shown in figure 77 and is more fully described in Chapter XV. Flat shutters should never be used, for tank sluices except in the low water vents unless they are fitted with screw gear, and even when so fitted they are undesirable unless large quantities of water have to be passed. For heads of over 30 feet special balanced valves or shutters moving on rollers are usually provided.

384. Details of plugs for tank sluices.—Details of plugs for tank sluices are shown in figure 76 and the standard sizes are 4", 6", 8", 10" and 12" diameter, this being the size of the hole in the plug stone, the largest diameter of the cone part of the plug being about ¼ inch less; the width of the seating should be from 1 to 1¼ inches. The cones should have a taper of one to four, and be made of hard wood; the smaller end of the cone and the greatest diameter of the plug should be cut cylindrical to take countersunk iron straps which should be shrunk on to prevent the plug from splitting. The lifting rod should be round iron and secured by a split cottar pin at each end of the plug. When the rod is long it should be passed through holes in guide beams fixed at intervals. The top end of the rod passes through a hole in the regulation platform; it should be flat iron with holes at about 3 inches interval through which an iron pin is passed to keep the plug suspended at whatever height it gives suitable elevation.

385. Cistern or wings at tail end of sluice barrel.—In the case of a sluice of normal capacity, up to about 400 acres, the tail end of the culvert usually delivers into a cistern of the form shown in figures 71 to 74. This serves to retain the slope of the banks and of the irrigation channel. Where several small channels take off from the tail of the culvert at different levels the cistern form of tail is very convenient as notches or vents in the cistern can be built to dimensions and at levels which will effect delivery of water in suitable proportions and at suitable levels. The rear wings and returns of sluices supplying larger channels should be

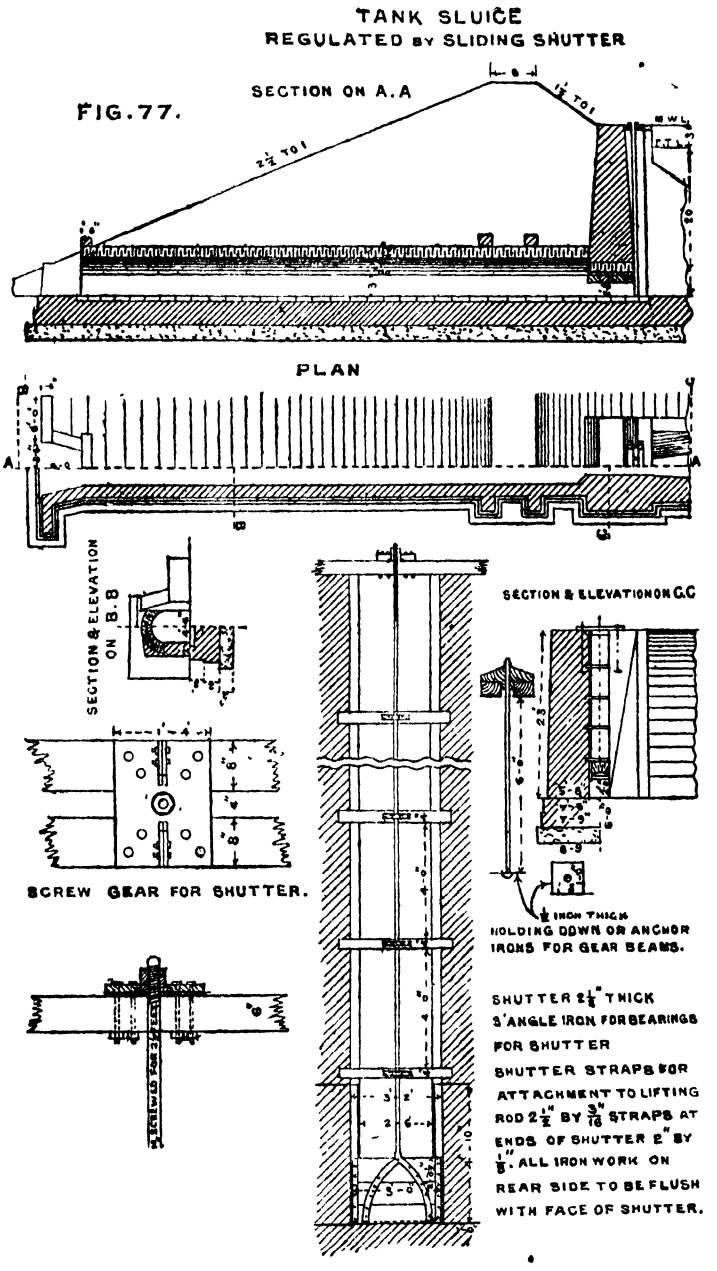
PLUGS FOR TANK SLUICES.

FIG. 76.



similar to those of sluices of similar size used in canal distribution; such a sluice is shown in figure 77.

FIG. 77.



386. Selection of site and sill level of a sluice.—None of the water impounded in a tank below the level of the sill of a sluice

can be passed by gravitation through it, and in fixing the level of a sluice sill it should be remembered that it is useless to draw off water below a level which secures command of the lands to be irrigated, and also that the deepest part of a tank bed impounds a relatively small content and a material part of this content will be gradually filled with silt.

The sill should thus be at such a level as will only draw water which commands the area to be irrigated and subject to this should be so deep as to be able to draw from the major portion of the impoundable content.

If an irrigation channel already exists, the bed level of this channel fixes the lowest effective level of sluice sill, but otherwise in order to fix the most suitable level of the sill of a tank sluice it is necessary to know the level of the highest and lowest fields and the general mean level of the lands to be irrigated, and the level of the sill should generally be between the mean and highest field levels but may be lower when much extra capacity is thereby made available; the approximate capacity of the tank within the range of levels between the highest and lowest fields should therefore be ascertained.

If the foundation soil is favourable at the place where natural ground is at sill level this would generally be the most suitable site. Where the soil is unfavourable the sluice site should be shifted to the nearest suitable site and generally this would be towards the nearest flank of the bund so that the sluice barrel would be somewhat in cutting and in such case approach and run-off cutting to and from the sluice would have to be made. Where the head of water is great it is by no means unusual to get greater security by shifting the site of a sluice to a flank, but in shallow tanks, say up to 20 feet depth, this is unusual on account of the cost and inconvenience of making and maintaining the approach cutting

387. Information which should be recorded on the design of a sluice.—The heading of information which is generally recorded on the design of a tank sluice is shown on the designs figures 73 and 74

TANK SURPLUS WORKS.

388. Surplus works of tanks.—The surplus works of tanks are usually either weirs or flush escapes. In ordinary cases the lengths must be such that the quantity of water estimated as the maximum flood discharge likely to enter the reservoir can be disposed of with a depth over the work equal to the difference between the full tank level (F.T.L.) and the maximum water level (M.W.L).

The effective storage capacity of the tank is limited by the F.T.L. but the area submerged by the tank water-spread and the height of the bund and revetment are all dependent on the M.W.L. It follows that in order to restrict the dimensions of these it is desirable that the difference between F.T.L. and M.W.L.

should be small. On the other hand the smaller this difference the longer must be the surplus escape in order to enable it to pass a given discharge. The difference is fixed in each particular case with reference to the estimated cost of the items mentioned, the most efficient arrangement which is also economical being adopted. In small and medium sized tanks the usual difference between M.W.L. and F T.L. is from 1 to 2 feet and it is rare for the difference to exceed 3 feet

389. Types of tank weirs.—Tank weirs may be classified in he same types as river weirs (vide para. 278) as below :—

Type A. -In which the water is dropped on to a horizontal masonry apron, or on to rock, immediately below the crest wall.

Type B —In which the water is delivered from the crest on to a sloping masonry apron.

Type C —Similar to type B, but with a sloping apron made up of a mass of rough stones

Type D —Weir with stepped apron

To the types A to C which have been already described in relation to anicuts must be added type D which is a combination of types A and B and may be called a weir with stepped apron. In this type the water is dropped in a series of vertical falls or steps on to successive horizontal or almost horizontal aprons.

Type A weir is of this type, but with one step only, whereas if the number of steps is large, and the height of each small, the weir approximates to type B.

390. Principles applicable to anicuts also applicable to tank weirs.—The general considerations and principles which guide the design of river weirs whether on rock or on loose pervious or impervious soils also apply to tank surplus weirs, but in the case of the tanks the conditions as regards the nature of foundation soil and also the working conditions are more varied than in the case of river weirs.

391. Working conditions of anicuts and tank weirs contrasted.—As regards foundation soil for tank weirs almost every class of soil has occasionally to be built on, but only rarely is the foundation sand.

This last is the most common foundation soil for anicuts and the details of anicut design on soft soils have been worked out with special reference to this class of soil.

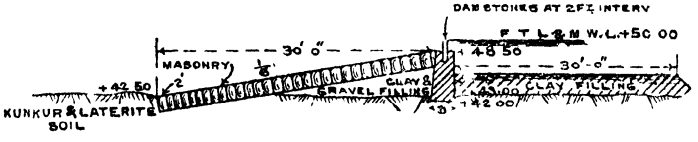
The conditions generally applicable to anicuts as regards tail water backing up during floods and affording some extent of protection to the talus and to the soil at the down-stream toe of the apron, are absent in the majority of tank surplus weirs, and therefore the weirs of tanks built on soft soil would, other things being equal, be more exposed to damage by scour at the toe of the apron and retrogression of levels than river weirs.

On the other hand the depth of water passing over the crests of tank weirs is generally far less than over anicuts, and in the case of very many tanks it is only on an average for a few days each year that any flow at all passes the weir, and while the duration of continuous heavy flow is frequently limited to a maximum of two or three days.

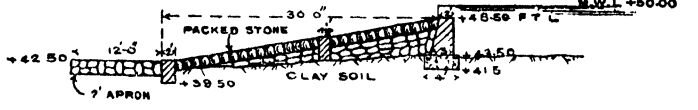
Tank weirs are thus usually worked far less heavily and far less continuously than river weirs and this should be considered in designing

392 Positions suitable for weirs of types B and C —Every type of weir mentioned above is to be met with in the existing tanks of South India. Types B and C, figures 78 and 79, although common are not generally to be recommended for tank weirs on account of the special tendency to scour at the toe of the apron to which these types are liable.

WEIR TYPE B. FIG. 78.



WEIR TYPE C. FIG. 79.



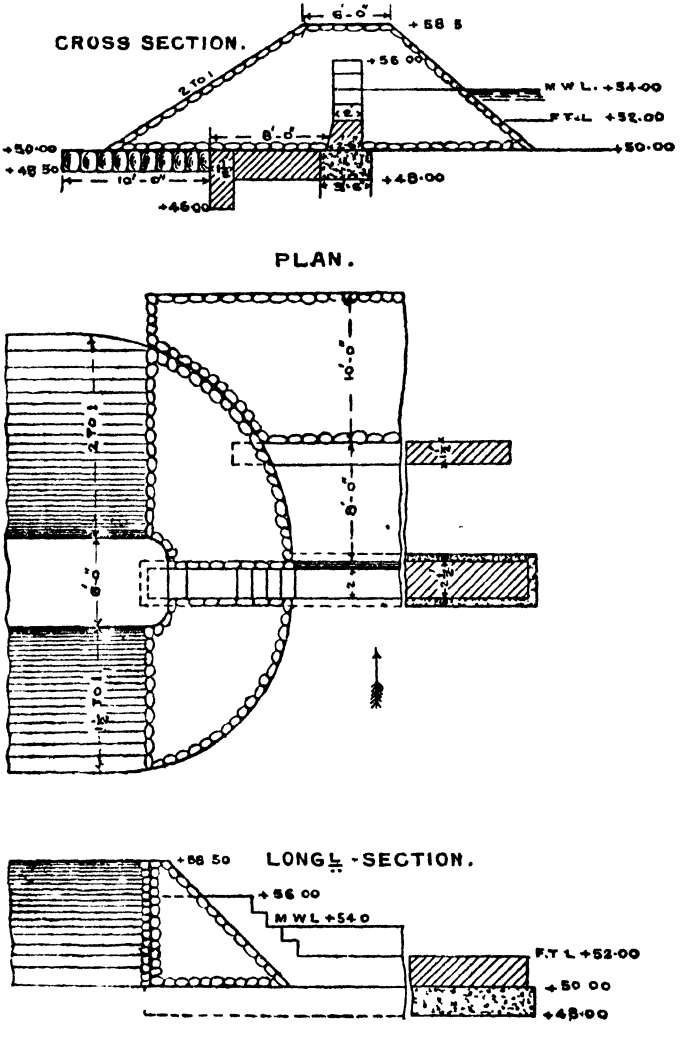
Where the soil is hard enough to resist scour, for instance soils of soft rock, or laterite, or kunkur, one of these types may suitably be employed. In an exceptional case where rough stone is so cheap that an extra heavy down-stream talus can be economically provided, these types may be found suitable even in friable soil

Generally however the weirs most suitable for tanks are of type A or D

393. Positions suitable for type A weir.—Where the soil is rock, or where a water cushion is brought into use, or where the vertical drop does not exceed 2 to 2½ feet, type A is most suitable;

where the quality of the masonry is really good the vertical drop may be increased to 3 feet maximum. The above restriction assumes that the apron is not protected from impact as in the case of anicuts by the backing up of tail water. Where this occurs a higher vertical drop may be given. Figure 80 shows a weir of this type.

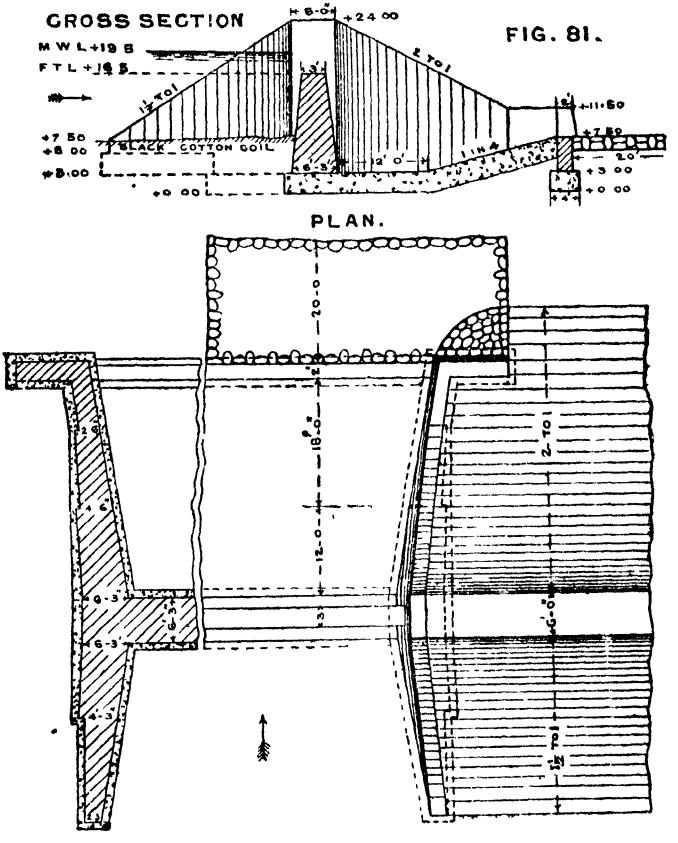
WEIR TYPE A FIG 80



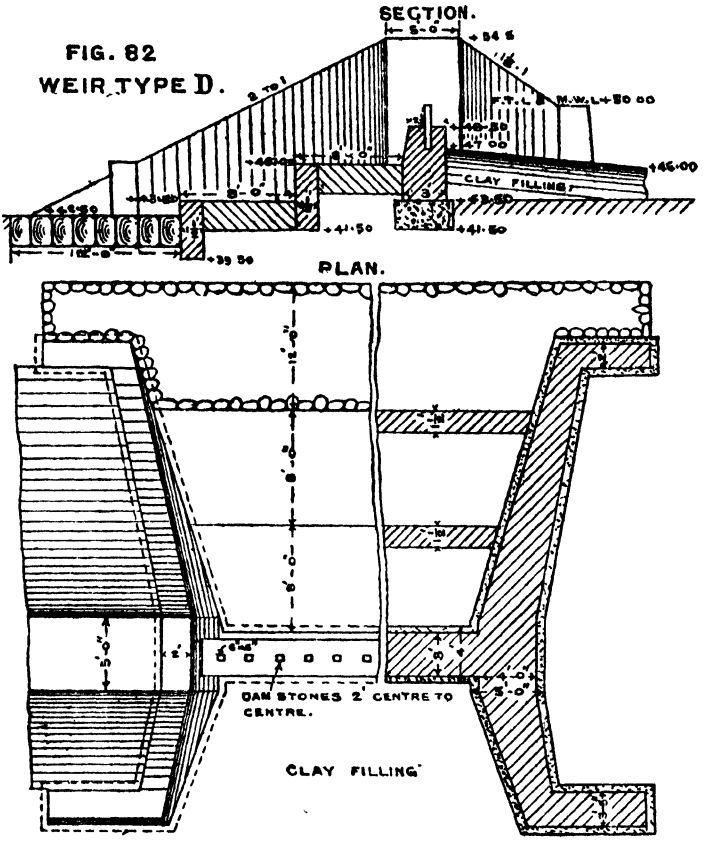
When the fall is great, say 9 feet or upwards, and the soil soft but only slightly pervious, type A weir with water cushion may, in certain cases, prove more economical than type D.

Figure 81 gives a design of a type A weir with water cushion. In certain places water cushion may be made in soft rock by building only a cushion wall leaving the soft rock as the floor of the cushion This type of water cushion weir is very economical.

WEIR TYPE A WITH WATER CUSHION.



394. Weirs (type D) with stepped aprons.—This type of weir is usually most suitable for sites on soft soil when the drop exceeds 3 feet. Figure 82 is a weir of this type.



395. Material for aprons of tank weirs.—It is advisable to make any apron on to which the drop exceeds 1½ feet, of not less than 1½ feet thickness of solid masonry (rough stone or concrete in mortar). Aprons receiving overfalls up to and not exceeding 1½ feet with a depth of water not exceeding 2 feet over the crest may be of 1½ to 2 feet rough stone grouted with concrete; the individual stones should be 1½ feet in length and packed with the length vertical. For overfalls of one foot, grouting is generally unnecessary.

Rough stone aprons should be laid on a layer of coarse grit or quarry rubbish of about 6 inches thickness.

All masonry aprons should be heavy enough to be secure against uplift This proviso, however, rarely affects the design of tank weirs of this type.

396. Width of floors of tank weirs.—The widths of the horizontal floors of type A or D weirs, from the foot of the drop wall to the down-stream edge of the floor should never be less than

2(H+d) where H is the depth of the drop and d the maximum depth of water to pass over the weir crest. In important works the width may be increased to 3 (H+d). The rough stone apron forming a talus below the last curtain wall should be of varying width according to the nature of the soil and the velocity and the probable annual quantity and intensity of run-off; it would generally vary from 2½ (H + d) to 5 (H + d) according to the conditions.

Width of floor = 2 (H + d) . . . . . (49)

397. Bank connexions of weirs.—Bank connexions are designed to secure a good joint between the masonry weir and the bank of the tank and the length of the line of creep must be sufficient to guard against dangerous percolation round the flanks of the work.

There are two general types of bank connexions:—

(a) abutments and wings (figures 81 and 82),

(b) core walls (figures 80 and 84-a).

(a) is generally the more expensive type but it securely protects the end of the bank from the rush of water while the wings serve as guide walls to the current on approach and run-off from the weir. In high weirs this type of bank connexion would generally be adopted and in low weirs the core wall type where cheaper will be found generally suitable.

The nature of the soil and the safe percolation gradient is of importance in designing bank connexions which should generally be made so as to give a percolation gradient of 1 in 5 round the joint between the masonry and the earth.

When surplus escapes are on the extreme flank of a tank it is often unnecessary to have any bank or bank connexion on the outer end, the weir wall or flush escape floor being stopped where the natural ground level is at F.T.L. and the continuation protected from scour by a little flush pitching.

This is shown in figure 83 on the outer flank of a flush escape. Even where these conditions do not obtain there is no reason why the bank connexions should be similar at both ends of a weir which connects at one end with a heavy bank where a breach would be disastrous and at the other with a light one where a breach would cause little injury.

It is usual to see two wings of similar design but these frequently comprise cases where uniformity is only secured by making the outer flank wing of an unnecessarily expensive type.

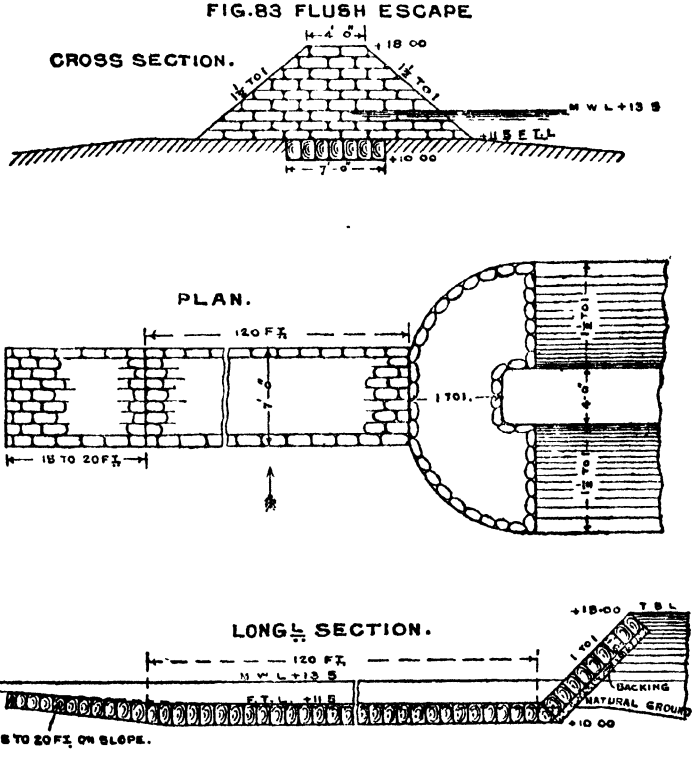
398. Flush escapes.—A flush escape may be constructed where, for a sufficient width at either flank of the tank or elsewhere along the perimeter of the waterspread contour, the natural ground surface is at F.T.L. and the ground falls away from this giving a suitable surplus escape channel. If the ground at the site is sufficiently hard not to be eroded by the surplus water passing over it and down the escape channel no construction work is necessary other than clearing the site and the approach and run-off channels of any obstructions. The tank bund should be carried up to one (or both) end of the length of the escape, there stopped

and the end revetted as protection against erosion by the water passing over the flush escape.

A flush escape is thus merely the open head of a surplus channel.

Where the soil is not sufficiently hard to withstand the erosive action of the water it is protected by drystone pitching. Figure 83 shows the usual form of flush escape. If any cutting back action from below is apprehended a stop wall should be built in a trench along the line of surplus, the flush apron being packed on its down-stream side and the foundation of the wall being either on hard soil or carried to such depth as will give the necessary security against cutting back. Flush escapes very commonly serve as surplus works for the smaller tanks in South India. Where these escapes require no pavement or other protective works they are termed 'natural escapes'.

FIG. 83 FLUSH ESCAPE



399. **Dam stones on tank surplus escapes.**—A common practice which has been resorted to for centuries in South India is to fix dam stones in the crest wall of weirs or along the floors of flush escapes These stones are generally about six inches square in section and 2½ to 3 feet long and are built into the masonry wall (or floor) at about 1½ to 2 feet interval in the clear and project above the weir crest to a height of one to two feet, their tops being generally at M.W.L. Figure 78 shows dam stones fixed in the crest of a type B weir.

The space between these stones are blocked up with clay and turf by the cultivators with the result that water can be held up in the tank to M.W.L. before any surplusing takes place, and in effect so long as these turfs remain intact the F.T.L. and M.W.L. are the same When the tank receives additional water which causes the level to rise above M.W.L. the water overtops the turf bund which is rapidly washed away and the surplus weir comes into operation without the M.W.L. being exceeded to any appreciable degree. Whenever this occurs all the water stored in the tank above the weir crest level must be passed off before the turf dam can be rebuilt.

The dam stones cause obstruction to the discharge and when these are installed the length of weir must be correspondingly increased. Sometimes iron uprights are fixed in place of dam stones as these offer less obstruction to flow but the object is the same. Weirs or flush escapes fitted with dam stones are termed 'calingulahs' a term derived from the Tamil word for such works. Planks or brushwood, etc., should never be used in blocking calingulahs as such obstructions would not be rapidly cleared by a flood and the tank would be liable to be breached.

Crest shutters, such as are used on anicuts, are sometimes used on surplus weirs of tanks, the object aimed at being the same ; so with the dam stones. Where there is a special resident establishment for working such shutters they may be satisfactory, but failing this, their use is to be deprecated as they will sometimes not be lowered when necessity arises. Automatic falling crest shutters, unless carefully looked after, are likely to jam when used on tank weirs owing to obstructions used with a view of diminishing leakage past the shutters and for this reason they should only be used where in charge of an adequate establishment.

400. Sluices as surplus works of tanks.—It is dangerous to depend on sluices for disposing of the surplus of reservoirs unless an efficient and trustworthy establishment living at the work site is maintained for working them. This is not practicable for any but large works. Tank surplus sluices do not materially differ from the regulators and sluices described elsewhere. This system of disposing of surplus is in force in a few large reservoirs in Madras (e.g., Periyar and Kanigiri reservoirs, Veeranam tank, etc.). A great advantage derivable from fully controlled sluices or weir shutters is that F.T.L. and M.W.L are the same and retain water at M.W.L. all the time surplusing is going on, while whenever a weir with dam stones commences to surplus, the impounding bank is washed away and cannot be reconstructed till water falls to weir crest, and all water impounded between weir crest level and the level of the top of the dam stones is lost.

401. Surplus vents or notches in weir walls.—Occasionally sluice vents or notches fitted with shutters form a part of weirs. Such works have been used—

(*a*) For irrigation in place of separate irrigation sluices.

There is no objection to such a sluice being used in place of one of the ordinary type where economy or convenience is thereby attained.

(*b*) To control water level on weirs fitted with dam stones and thus when the quantity of surplus water is moderate to prevent the breaching of the turf bund.

When the conditions of site allow, such sluices are likely to prove most useful, and surplus sluices for this object, whether in the weir or elsewhere, might with advantage be constructed more frequently than is now usual.

(*c*) To supplement the otherwise inadequate discharging power of the surplus works

This is a dangerous practice, and unless under the conditions as regards establishment stated in last paragraph a surplus weir should always be provided with adequate capacity for discharging the full quantity of surplus without any dependence being placed on weir sluices

402. Approach to surplus works.—It is most important to provide and maintain a free and unobstructed approach to all surplus works. The width of the approach should not be less than the width of the weir or flush escape and the bed level in no place higher, and preferably half a foot or more lower, than the crest of the weir Where a weir is several feet in height access to it is generally adequate without the necessity for carrying out any work to provide for this, but in the case of low weirs or flush escapes on the extreme flank of a tank bund, or on an isolated saddle, it is frequently necessary to enlarge the natural approach by excavation so that the water may have free access to every part of the work. If this is neglected there will be a loss of head on the approach to the surplus work and the full depth of water will only pass over it when the level of open water in the tank is higher than the M.W.L. proposed Figure 84 gives an example of cutting out access to a surplus weir, the area of the cutting being cross-lined in plan The area within which all obstructions must be cut down is the quadrant of a circle struck with the inner flank of the weir as centre and the length of the weir as radius.

403. Surplus channel—The disposal of the surplus water after it passes the weir requires careful consideration. The natural line of run-off is a line at right angles to the existing contours of the natural surface of the ground below the weir, and if these give a suitable surplus course without damage to existing or prospective cultivation and without running in close proximity to the down-stream toe of the tank bund all that would be necessary would be to remove any local obstructions, and possibly in some cases to erect small guide banks so as to prevent the water spreading out to an undue width before it reached a valley in which the natural contours of the ground would retain it. As soon as a surplus work on soft soil comes into use the water itself will quickly form a surplus water course and if the water is at the first guided into a suitable line much subsequent trouble may be avoided. With this object in view it is necessary where the line of weir is at right angles to the ground contours as in figure 84, to make guide banks and cuttings which have for their object the leading of the surplus water across side-long ground in a direction at right angles to the weir for some distance away from the weir and tank bunds before allowing it to take a course at right angles to the contours of the natural ground surface. This involves a bank on the lower side of

the cross slope and also a cutting to level the cross slope, from the spoil of which cutting the stop bank and possibly also some of the tank bund may be made. Such cutting should be graded into a channel with a suitable fall and its alignment laid out with a view to its gradual widening by erosion of surplus water to form a suitable and safe channel clear of the toe of the bund (vide figure 84).

404. Disposal of surplus water from tanks.—Most of the surplus works built in South India are in connexion with existing tanks and from these surplus water channels already exist which would in most cases be utilized, but where there is no existing channel the disposal of surplus flow requires careful consideration and suitable provision.

In the case of a new weir for an existing tank it is even more necessary to fully consider this question than in the case of a new tank as more numerous well-established local interests are likely to be affected and all these have to be considered in addition to the engineering aspects of the case

Wherever the construction of a new work has the effect of diverting the yield of a catchment from an existing into a new line of run-off the effect on the irrigation supply and on the flood discharge of existing works lower down must be carefully considered and provided for.

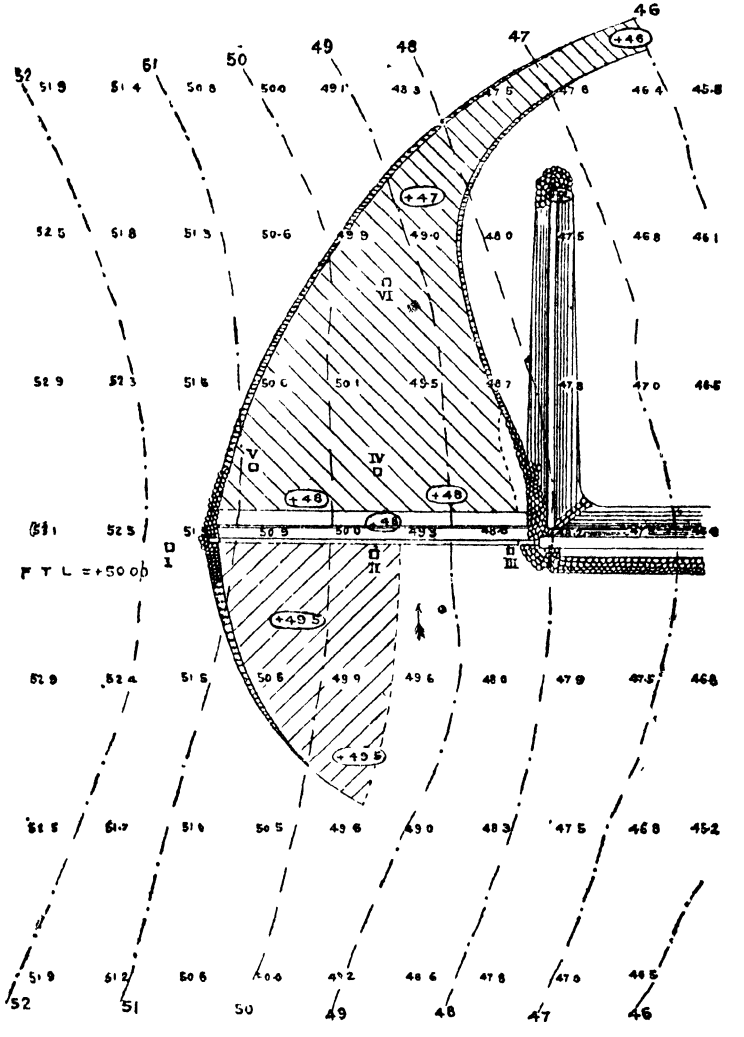
If the surplus channel from a tank has too rapid a fall, having in consideration the nature of the soil and the frequency of surplusing, the erosion caused may cut so deep as to affect the stability of the weir by retrogression of levels. The risk of this, if it exists, must be provided for, as in the case of an anicut, by adequate depth of foundation of the curtain walls of the weir and by a suitable talus of loose stone or by protective works such as grade walls or stone flooring placed at intervals in the bed of the surplus channel or by diverting the surplus water to a course with a more gentle gradient. In every case the surplus water must be prevented from flowing close along the outer toe of the tank bund as a deep water course in such a situation would be a standing menace to the safety of the bund.

Where works have to be carried out to guide the surplus flow into a suitable channel, the width of surplus channel which immediately below the weir would be very wide and shallow, should be brought to suitable dimensions by gradually converging banks or cuttings so as to establish a suitable relation between depth and bed width.

405. Detail plans of sites of surplus escapes.—For the fixing of the exact site of a surplus weir or flush escape in the best place and for the detailed design of the work and of its approach and surplus channel it is necessary to have very detailed information regarding the levels and slopes of the ground on the flank of the tank, or other site, selected for such a work. A large scale detailed plan must be made of the ground along the line of tank bund where it is proposed to place the weir extending from twice to three times the probable length of weir; the plan should extend in a direction perpendicular to the length of the weir for about the length of the weir on the tank side and from 200 feet to 500 feet or more on the down-stream side of the weir.

All over this area levels of the ground surface at suitable distances should be plotted so as to give fully detailed information as regards the ground surface slopes. Trial pits along the centre line of weir and also down-stream of this should be dug to afford full information as to the nature and depths of surface and subsoils at the site of the work and on the line of surplus channel.

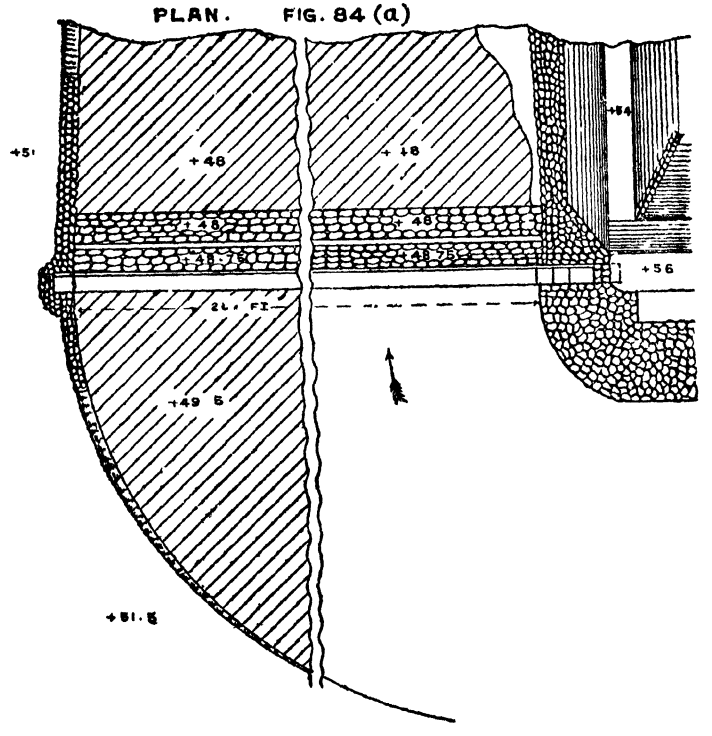
FIG. 84.

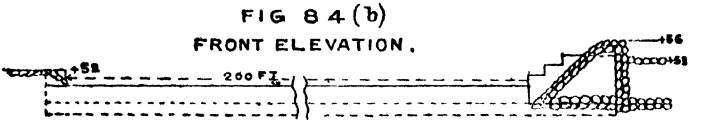


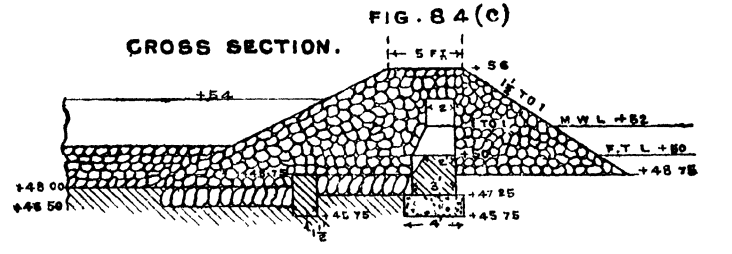
THE FIGURES ON PLAN SHEW NATURAL GROUND LEVELS.
THE FIGURES SURROUNDED BY LOOPS SHEW LEVELS TO WHICH EXCAVATION IS CARRIED.

406. Example of site plan and connected weir design.—Figure 84 gives an example of a plan of the kind referred to with levels plotted at intervals of 50 feet parallel to the weir and at

100 feet interval in a perpendicular direction and contours interpolated from the ground levels; six trial pits are also shown numbered in Roman numerals. From the site survey a design has been made for a suitable weir, approach and surplus channel for a tank of about 15 square miles catchment—figures 84-*a*, *b* and *c*. It is assumed that the soil and subsoil has been shown by the trial pits to be red loam







Study of these plans will make it clear how necessary a detailed site survey showing levels is for the proper design of

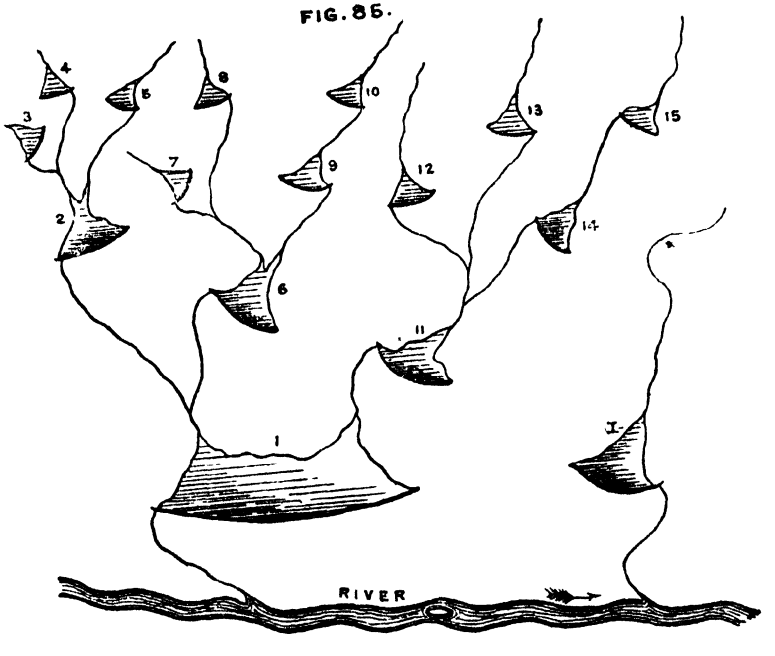
weir and connected works; in this case suitable detailed designs could not be made without it.

407. Choice of sites for surplus works.—When a saddle disconnected with the tank bund affords a suitable site for a surplus work it should generally be used in preference to a work directly connected with the bund but it is generally necessary to place the surplus works at or near one flank of the bank and connected with it. If all the cultivation is on one side of the stream which is dammed by the bund it is desirable to have the surplus on the other flank; otherwise it would be necessary in many cases to carry the distributary channels across the surplus channel and there may also be danger of the cultivation being injured by surplus water when the tank surpluses freely. Apart from these considerations the most favourable sites for surplus weirs are (1) where the natural ground surface along the line of weir is at a level approximately F.T L. (2) where the soil or subsoil close to the surface is hard both on the weir site and along the run-off channel and the contour of the ground gives clear approach to the weir site and a suitable course for the surplus water.

408 Groups of tanks.—Most of the tanks in Southern India form part of groups of tanks and either receive the surplus of upper tanks or surplus into lower tanks or do both. A tank which neither receives water from an upper nor discharges surplus into a lower tank is termed an isolated tank.

Figure 85 shows a typical group of 15 tanks numbered 1 to 15 and an isolated tank (I). It will be evident that considerable economy of water is obtainable from the system of grouping because the

FIG. 85.



surplus water of each tank and also the drainage of its wet cultivation are caught up by the next lower tank. Each tank of the group takes a share in the yield of the whole catchment above it which area is classified as follows:—

(1) the 'free catchment' which is the area which only drains into the tank under consideration, and

(2) the combined catchment which is the area of the whole catchment above the tank

The difference between the combined and the free catchment thus gives the area of catchment intercepted by upper tanks; the whole catchment of the highest tank on each drainage is free. Each tank receives all the run-off from its free catchment but from the remainder of its catchment only the balance which remains after the upper tanks have been filled.

The great objection to grouping tanks in this way is that a breach in an upper tank exposes all the tanks in the series below it to risk of similar failure. It is thus important in taking up repairs and restorations to consider groups of tanks collectively as a whole and to carry out repairs from the upper end of each group. The danger to which lower tanks are subjected by breaching of upper ones is in proportion to their relative capacities; there is little danger to a tank of large capacity from a very small one breaching into it although the breach may completely empty the small tank.

The provision of suitable breaching sections in tanks forming part of a group is a very desirable precaution as a breach at a well selected place does not empty a tank and indeed frequently does little more than supplement its surplusing power to a reasonable degree to the greater security both of the tank itself and of those below it in the group.

409. Computation of capacity of surplus works of tanks.—The surplus escape of a tank should be capable of passing the estimated maximum run-off of the catchment. The method of estimating maximum run-off has already been dealt with in Chapter III.

In South India for small and moderate catchments the maximum run-off is generally estimated by Ryves' formula, the co-efficient suitable being fixed, as already described by reference to that successfully applied to similar works and catchments in the locality.

In making such an estimate it is necessary to have in view the consequences of error and a more liberal estimate should be made for application to a large work where the failure would entail a serious disaster than to a small one where a breach would merely cause a measure of local loss and inconvenience. The estimate of maximum flood discharge to be provided for must thus be decided in each case on its merits, the chief considerations being the importance of the work and the nature and extent of the damage which failure would entail

410. Flood absorbing capacity of a tank.—In cases where the F.T.L. and M.W.L. of a tank are different the impounding capacity between these two levels must be filled up before the surplus escape can discharge at its full capacity. Where the capacity of the tank between F.T.L. and M.W.L. is considerable as compared with the flood discharge of the catchment, this absorbing capacity

may be so great as to permit of materially reducing the discharging capacity of the surplus works. It is unusual in Madras practice to make any allowance for this in the case of small or medium sized tanks and the absorbing capacity serves as a factor of safety in the case of such works.

In the case of large tanks, say exceeding 100 m.c.ft. capacity, or where the difference of level between F.T.L. and M.W.L. is considerable, say more than 2½ feet, allowance should be made for absorbing capacity when fixing the length of surplus weirs. A method of computing the allowances which may be made for the absorbing capacity of tanks has been worked out by Capt. Garrett, R.E., and is described in his paper on the "General Theory of the Storage Capacity and Flood Regulation of Reservoirs" published in 1912 by the Government of India.

This method is not here described as its application without the tables published with the paper involves much labour; if it is required to design weirs for new tanks such as specified above the original paper should be referred to.

411. Computation of flood discharge from combined catchments.—Although it is unusual in Madras in the case of isolated tanks or free catchments to make allowance for absorbing capacity, it is usual in the case of combined catchments to make a deduction from the maximum run-off of the combined catchment calculated by Ryves' formula for the area of that catchment intercepted by upper tanks.

The maximum run-off (D) from Ryves' formula is $D = C.M.^{2/3}$

The allowance for the run-off of a combined catchment is computed from the modified formula:—

$$D = C.M.^{2/3} - c.m.^{2/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (50)$$

where C is the co-efficient in Ryves' formula.

M is the area of the combined catchment in square miles.

c is a co-efficient from 1/5th to 1/3rd of C and frequently fixed at 100.

m is the area in square miles of the catchment intercepted by upper tanks

The formula is purely empirical and should only be applied when the upper tanks are provided with adequate surplus works and kept in good state of repair.

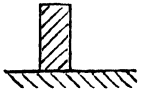
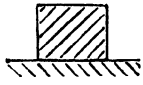
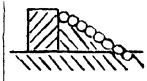
412. Computation of the capacity of a tank.—The gross capacity of a tank is the cubic content of water impounded up to F.T.L. The effective capacity is the cubic content between the level of the sill of the lowest sluice and F.T.L. Capacity is computed from a contour plan of the waterspread, the total capacity being the sum of the capacities between successive contours; the smaller the vertical interval between the contours the more accurate is the computation of capacity. The cubic content between two successive contours which are at a vertical interval apart which is small with reference to the total effective depth of the tank is generally taken as the mean of the areas of the contours multiplied by the vertical interval between them, that is $\frac{1}{2}(A_1 + A_2) \times I$, where A_1 and A_2 are the areas of successive contours and I the vertical interval between them.

Where a contour plan is not available and only the area of the tank at F.T L is known, the effective content of the tank in cubic

feet may be roughly computed as this area multiplied by one-third of the depth from this level to the deep bed of the tank, or to the level of the sill of the lowest sluice, whichever is higher of the two.

If the area (A_1) of the tank bed at thelevel of the sill of the lowest sluice and the area (A_2) at F.T.L. are known even roughly the effective content may be computed as $\frac{1}{3}$ ($A_1 + A_2 + \sqrt{A_1 A_2}$) × I, the assumption being that the content is that of a frustum of a cone.

413. Co-efficients of discharge of tank weirs and flush escapes.—The formula for the discharge (D) of a free weir, without any velocity of approach, per foot of length is $D = \frac{2}{3} C d \sqrt{2gd} = 5{\cdot}35\ C\ d^{\frac{3}{2}}$. The under-noted statement gives the co-efficients usually applied in South India to different forms of tank weirs and flush escapes.

Depth of flow over escape in feet=d.	Type of escape.			
	C = ·625  Weirs with crests up to 3 ft. wide.	C = ·562  Weirs with crests over 3 ft wide or with dam stones.	C = ·5  Rough stone sloping escapes	C = ·437  Flush escapes.
d	$D = \frac{1}{10} d^{\frac{3}{2}}$	$D = \frac{9}{3} d^{\frac{3}{2}}$	$D = \frac{8}{3} d^{\frac{5}{2}}$	$D = \frac{1}{3} d^{\frac{8}{2}}$
0·25	0 417	0 375	0 333	0·292
0 50	1·178	1·060	0 943	0 824
0 75	2·157	1·941	1 725	1·510
1 00	3·333	3·000	2·666	2 333
1 25	4·658	4·192	3·727	3·260
1·50	6·124	5·512	4 899	4·288
1 75	7 718	6·946	6 174	5·402
2·00	9 428	8·485	7·542	6 599
2·25	11·250	10 125	9 000	7 875
2·50	13·176	11 859	10·542	9 225
2 75	15·201	13 681	12·161	10 641
3·00	17 320	15·588	13 856	12 124

414. Capacity of supply channels for tanks.—Tanks are frequently supplied wholly or in part from rivers and streams by means of diversion works and supply channels. A supply channel is similar to a main canal for direct flow irrigation and its flow capacity must be fixed from consideration of the total quantity of water to be passed to the storage work, and incidence of freshes in the river.

The flow of rivers which supply tanks is generally very fluctuating and the channel must be made of sufficient capacity to carry the required supply during the duration of the freshes of an average bad year or season. For instance if the capacity of a tank were 100 mills. c. ft. and the supply was solely dependent on a diversion channel from a river in which freshes in a bad season could be

counted on for 10 days the discharge capacity of the supply channel should be about 10 m.c.ft. per day or 110 cusecs.

415. Working tables.—The most suitable area to be irrigated and the most suitable impounding capacity for a storage system are best ascertained by means of working tables. These tables are based on the available data of periodic run-off of past years and show what the periodic receipts, issues, balances, and surpluses of water would have been if the proposed irrigation system had been in operation throughout the above years. The intervals of time for which the tables are drawn may be monthly, weekly, daily or other suitable intervals.

If the number of years or seasons dealt with is sufficient and the data of run-off and duties of water are accurate such tables will give a reliable basis for a forecast of the prospects of the proposed system From consideration of the details of the supplies and surpluses as displayed in the working tables the suitability or otherwise of the proposed irrigated area and impounding capacity can be judged and any necessary readjustment of proposals made. For instance if the tables showed that there was invariably, even in bad seasons, a considerable unused balance of water in the reservoir, or if, in such seasons, the reservoir did not fill the inference is that, the capacity is unnecessarily large; if on the other hand during the irrigation season the reservoir is not infrequently empty although it occasionally surpluses the inference is that the storage capacity is inadequate. Intervals of time of from ¼ to ½ a month are usually suitable for working tables although in special cases tables showing daily working are desirable.

The following are suitable headings for the columns of working tables:—

(1) Intervals of time	
(2) Impounded balance from previous period (m.c ft.)	Receipts.
(3) Run-off of period (m c. ft.)	
(4) Total (2) + (3)	
(5) Acres irrigated	
(6) Duty of the period m c. ft. per acre	
(7) Issues to irrigation (5) × (6)	Issues.
(8) Evaporation and absorption m.c. ft.	
(9) Total issues (7) + (8)	

(10) Balance impounded (4) — (9) subject to maximum (S) = the full storage capacity

(11) Quantity surplused [(4) − (9) − S].

CHAPTER XI.

DISTRIBUTION WORKS—CANALS AND CHANNELS.

416. Distribution works.—The distribution works of a system are the canals and channels by which the irrigation water is conveyed from the head works to the fields, with the subsidiary works pertaining to them. The canals, or channels, which take their supply directly from the head works are main canals, or channels, while canals and channels taking off from main canals are termed 'branch canals' or 'distributaries.'

417. Classification of distribution channels.—The canals and channels of a large canal system may be classified as follows:—

(1) A main canal (or channel)—This has been defined above.

(2) A branch canal—A canal taking off from a main, or branch canal and having a head capacity of not less than 300 cusecs.

(3) Distributary channel—A channel taking off from a main, or branch canal or from another distributary having a head capacity as below:—

(*a*) A major distributary under 300 and above 25 cusecs.

(*b*) A minor distributary 25 cusecs and under.

The above channels constitute the Government distribution works of an irrigation system.

(4) Field channels (also termed 'water courses')—These are small channels constructed by cultivators and are not the property of Government; they run from outlets in the Government channels through the cultivators' own lands and convey and distribute water to individual fields.

In modern systems these channels seldom carry more water at the head than 3 or 4 cusecs and generally considerably less. On an average field channels would carry less than one cusec.

418. Different methods of alignment of irrigation channels.—Irrigation channels are generally aligned with reference to the contour of the country in one of the following different ways:—

(1) as contour channels,

(2) as watershed channels,

(3) as side-slope channels.

(1) *A contour channel* is carried on an alignment which conforms generally to that of the contours of the country traversed being given however such slope along its length as is necessary to produce the required velocity of flow. As the line of flow of surface drainage is at right angles to the ground contours such a channel cuts across the natural drainage lines of the country traversed. Such an alignment does not imply an exact conformation with the contours of the natural ground, and it is usual, when traversing undulating country which displays any marked natural

features to shorten the line by crossing spurs in deeper cutting and the valleys in higher bank than the general average. Thus in such country a contour channel would be of shorter length than the corresponding falling contour laid out along the ground surface without taking any short cuts.

(2) *A watershed channel* is one aligned along any natural watershed There will clearly be no drainage crossing such a line

(3) *A side-slope channel* is one aligned at right angles to the contours of the country traversed and not on a watershed or valley line. Such a line would be paralled to natural run-off of drainage and thus such an alignment avoids intercepting cross drainage.

419. Points to be aimed at in the "lay-out" of a distribution system.—The main aim of the arrangement of the "lay-out" of a distribution system is to secure in the most economical way effective water distribution combined with adequate command of the area to be irrigated with as littte interference as possible with natural drainage The above statement of aims at once suggests an alignment along any watershed within the irrigable area as securing command of all the ground up to the next valleys on either side of the alignment without any interference with drainage; and it may be taken as axiomatic that the watersheds lying in any irrigated tract should be occupied by distribution channels unless there are strong reasons against such an arrangement.

Side-slope channels have the advantage of not intercepting cross drainage, but their course must follow the shortest route to the nearest valley and such channel will lie along a line of steepest possible slope; except in very flat areas only the smaller of the distributary channels should be so located.

Contour channels are carried on slopes, the main watersheds of which are not commanded. Such channels with minor exceptions, only irrigate on one side and themselves form the upper boundaries of the irrigated area.

420. Main canals as contour channels.—As a rule head works are situated in a valley and the upper parts of main canals are aligned as contour canals In the case of direct-flow systems, the first reach of main canals is generally so deep in cutting that no land can be commanded, and the trace of this reach is located with a view to carry the canal along the most economical line to the point where it commences to command the land, and from which it can be continued as a contour canal proper.

A contour canal in its course cuts across all the spurs and valleys intersecting the slopes along which it is aligned, and a main canal is carried on a contour alignment, until either it commands the full area to be irrigated, or until it attains to the top of the ridge, that is to the watershed line, along the side of which it has been so far aligned in contour From such a point it would be aligned down the watershed and cease to be a contour canal.

421. Offtakes from contour canals.—Branch canals and distributary channels should take off from a contour canal from, or near, the places where the canal crosses the watershed of spurs; they should be aligned as watershed channels along these spurs

and will irrigate the area up to the adjacent drainages on either side of the spur crests. From these branches, subsidiary channels will, in turn, offtake so as to cover the whole area to be irrigated.

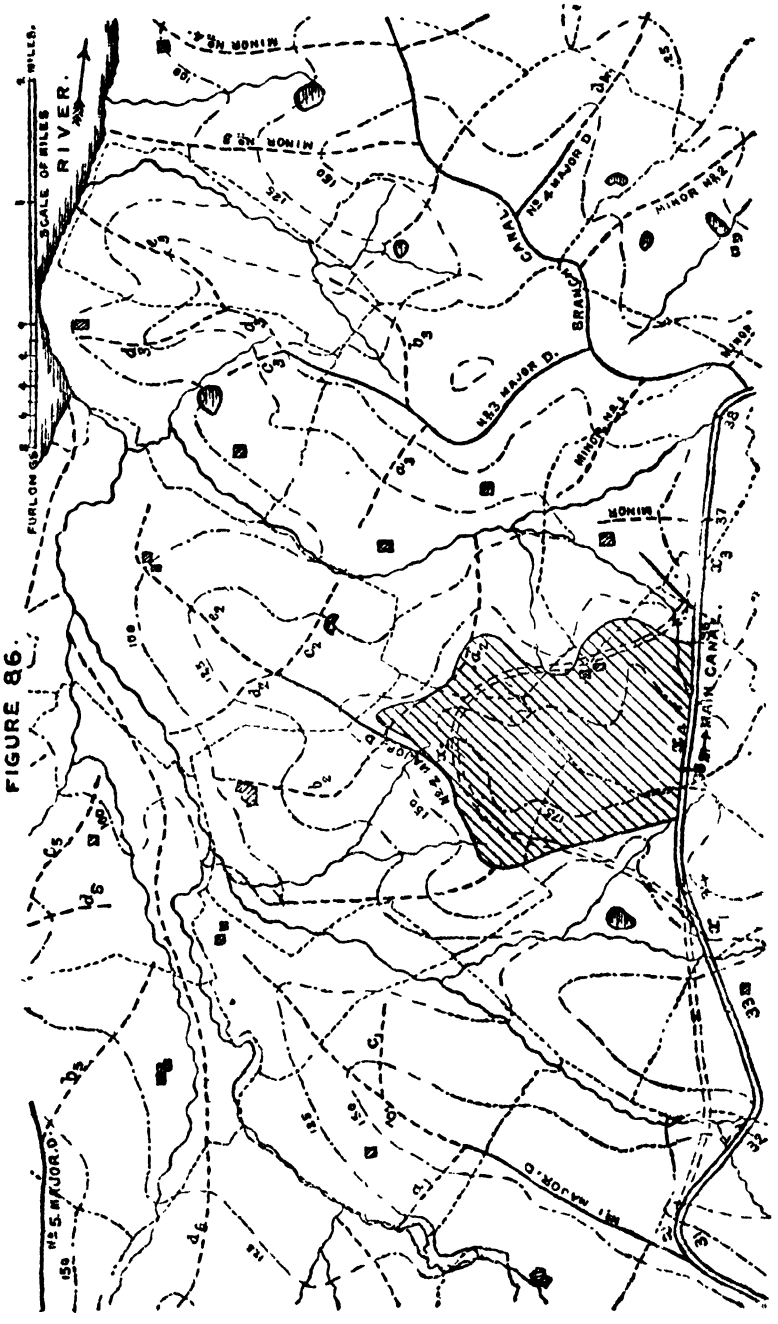


FIGURE 86.

It is frequently possible to restrict contour channels almost entirely to main canals, the major part of all the other distribution canals of the system being carried at right angles to the contours.

422 Typical distribution system.—The typical canal system may thus be represented as a main canal aligned as a contour canal and branches and distributaries aligned as watershed or side-slope channels so as to carry water throughout the area to be irrigated at levels giving adequate command without obstructing the drainage of the area.

423. Plan showing part of a typical distribution system.—Figure 86 shows a part of the lay-out of a distribution system proposed for a project prepared a few years ago in the Madras Presidency. The main canal is shown from the 31st to the 39th mile and is a contour canal. It will be noticed that instead of following the contours, as shown in dotted lines along the alignment x_1 x_2 x_3, the line has been taken in deep cutting through the saddle at x_4, thus saving some 2 miles in length of canal

424. Alternative alignments of contour canals—The considerations which would guide the selection of the best of the alternative routes in this case would be:—

(*a*) Cost of earthwork and land acquisition by each route.

(*b*) Cost of the No 2 major distributary from its head to the vicinity of x_2 and also part of the cost of the minor distributary a_2 both of which would be saved if the main canal followed the contour line.

(*c*) Additional cost of cross drainage works necessary if the main canal followed the contour line.

(*d*) The value of the head saved by taking the shorter route.

It would be necessary in making alternative estimates of cost to carefully ascertain by digging trial pits or making borings at intervals the nature of soil to be excavated on both alignments.

The head saved, item (*d*), would be equal to the surface fall in 2 miles; what the value of additional head may be depends on the circumstances of each individual case Such a saving might have the effect of bringing an additional area under command and it might have the effect of reducing the cost of other works, especially depth of cutting at any saddle further down the canal. Command of additional area might under some conditions be valuable while under others it might not be of any material advantage, and the value to be ascribed to gain of head must be separately estimated in each case

The questions of alternative alignments of contour canals, especially in the upper reaches, call for most careful consideration and frequently involve much investigation.

Apart from such questions the exact location of a contour channel across all valleys and spurs lines is a matter requiring careful consideration. The alignment will not follow all the sinuosities of the ground contours, and will generally cross valleys in higher bank and spurs in deeper cutting than the normal. The carrying a canal in high bank involves danger of breaches from percolation and careful judgment has to be exercised in fixing the points of crossing of each drainage and in adjusting the alignment of canal to suit the requirements of the cross drainage work without involving excessive cost of risk in approach banks. For instance

between mile 31 and mile 34 in figure 86 a shorter and more direct line for the main canal is shown in dotted lines. This would involve much heavier bank across the valley but would allow more head-way for the syphon aqueduct carrying the drainage under the canal.

Figure 86 shows, that the whole of the land to the left of the main canal, except the shaded plot x_1, x_2, is commanded and where such is the case the distribution canals should be kept as far as possible on watersheds Where this results in distributaries being too far apart, subsidiary smaller distributaries should be run at right angles to the contours.

The branch canals and major distributaries are shown in heavy continuous lines, minor distributaries in heavy dotted lines, village boundaries in fine dotted lines and contours in chain dotted lines. Field channels have not been shown on the plan as the scale is too small, but the distributaries are designed to so cover the ground that these need not exceed 1½ miles in length. Further so far as economically practicable, different distributaries are proposed for different villages and all distributaries, except No. 2 major distributary and minor distributary a_2, are aligned at right angles to the contours. In no case does any channel cross any drainage in the irrigated tract while suitable cross drainage works are provided for passing drainage intercepted by contour distributaries.

425 Lay-out of field channels.—For paddy cultivation the fields are surrounded with low turf bunds, the slopes of the ground being terraced It is unnecessary and unusual to provide a separate field channel for each field and all that is required is to group together fields which drain from one to the other, the highest field of each group being supplied from the field channel and the others by field to field flow. This is called 'field to field irrigation.'

The field channels are so small as to suffer but little from the effects of cross drainage when laid out as contour channels, while the heading up caused by the low banks of these channels is insufficient to submerge and damage the wet crops, and drainage can be conveniently passed through temporary breaches made in field channel for this purpose. There is therefore no serious objection to aligning these channels as contour channels where by doing so they can be conveniently given a suitable slope with adequate command throughout their length. It is therefore not unusual to lay out field channels as contour channels without making special provision for cross drainage.

The points to be chiefly considered in laying out field channels are—

(*a*) They should, if possible, be laid along field boundaries

(*b*) They should conveniently command all their ayacut.

(*c*) As far as practicable separate channels should be made for low and high lying lands.

(*d*) Separate outlets and field channels should be made for separate villages.

(*e*) As a rule outlets along distributaries spaced at about 1 to 2 furlongs apart are found to be adequate and suitable. A multiplicity of outlets to field channels results in great waste of water.

Although field channels are made by ryots through their own lands it is desirable to supervise their location so as to ensure suitable alignment and proper command of all the irrigated land.

Field channels are worked on internal turns and their capacity must be fixed accordingly. Very large quantities of water are required for individual fields for transplantation and it is necessary that capacity to provide for this should be given to the tail reaches of these channels.

426. Standards of canals.—The following statements give widths and heights of banks, widths of berms and width of land which represent the general standard at present accepted as suitable in Madras. It is to be clearly understood that, while these standards are generally suitable, they are not to be looked up as necessarily always so the nature of the soil especially that forming banks must always be considered in designing canal sections and suitable saturation gradients allowed for.

Standard for canal cross sections.

Item	Main and branch canals		Major distributaries		Minor distributaries.	
	5,000 to 1,000 cusecs	1,000 to 300 cusecs	300 to 150 cusecs	150 to 25 cusecs.	25 to 10 cusecs.	Below 10 cusecs.
Maximum crest width of widest bank.	15	6	6	6	4	3
Free board	3	3	3	2	1½	1½
Width of berm ...	$4 + \frac{Q}{1000} + \frac{d}{2}$	$4 + \frac{d}{2}$	$4 + \frac{d}{2}$	$2 + \frac{d}{2}$	$2 + \frac{d}{2}$	$2 + \frac{d}{2}$ Occasionally no berms are provided.
Depth of earth over saturation gradient	4	3	1	1	1	1
Width of land to be acquired clear of banks when canal is in cutting deeper than balancing depth.	15	Half the height of the bank above ground subject to a minimum of 5 feet				
Width of land to be acquired clear of banks when canal is in less than balancing depth of cutting.	Full height of bank *plus* 15 feet	Full height of bank above ground *plus* 5 feet.				

Note.—(1) Q = discharge in cusecs.
(2) d = depth of excavation in feet.
(3) Land widths are to nearest whole foot of width.
(4) The side slopes of banks are assumed as 1½ to 1 and of cuttings as 1 to 1 and the widths of berms are computed on this assumption.
(5) A saturation gradient of 1 in 4 is usually suitable
(6) Where the tops of banks are used for wheeled traffic they must not be less than 15 feet wide and must be metalled or gravelled.

427. 'Balancing depth' of cutting.—Where the spoil from the excavation of a canal is equal to that required to form the banks of minimum sanctioned standard free board, top width, and slopes, the canal is said to be in 'balancing depth' of cutting.

Where the cutting is deeper than balancing depth the extra spoil should be used in widening rather than raising the banks. In all cases even when the water level in the canal is below ground surface the spoil from the excavation must be formed into regular bank sections, the top should be given a slight slope or have a small ridge formed along the inner edge so that it should drain outwards, that is, away from the canal.

The equation for balancing depth of cutting (D) for a channel with sides of cutting sloped at 1 to 1 and with 2 banks of equal cross sections and of 1½ to 1 slopes is:—

$$D^2 - (3h + B + \tfrac{1}{2}b)D = -h(B + \tfrac{3}{2}h) \quad \ldots \quad (51)$$

where h = vertical height of top of bank above bed of the channel,
" b = bed width of channel,
" B = top width of Bank,
" D = balancing depth of cutting in feet.

428. Rule for width of berms.—The expression $\frac{d}{2}$ forming part of the standard width of berms is inserted to ensure that the centres of banks shall be parallel to the centres of the canals. The standard slopes of cutting being 1 to 1, and of bank being 1½ to 1, if the berms were made of equal width irrespective of the depth of cutting the distance of the centre of the bank from the centre of the channel would increase and diminish in accordance as d (the depth of cutting) became smaller or greater. In a canal carrying silty water when the berm is below full supply level it is gradually raised to that level by silt deposit.

The widths of berms given in paragraph 426 are only a guide and should not be considered as fixed, except in so far as regards the item $\frac{d}{2}$, which should always be allowed, when setting out the toe of the bank, even in cases where it is decided that no berms are required. This is necessary in order that the centre lines of the banks and the channel shall be parallel.

Berms are frequently omitted in small distributary channels running through valuable lands, and may also be omitted from lengths of channels running wholly, or almost wholly, in bank unless the irrigation water is so heavily charged with silt as to give prospect of rapid formation of silt berms.

Figures 87, 87 (a) and (b) show the section of a distributary of 200 cusecs capacity, on the right side without berms, and on the left with standard dimension berms.

In figure 87 the canal is wholly in bank.
" figure 87-a " is one-half in bank.
" figure 87-b " is wholly in cutting.

The bed width bb' would be set out on the ground 6 feet on either side of the centre line.

Then $b\,c$, $b'\,c'$ would be set out equal to the depth of cutting (d).

For the standard berm, $cf = 4 + \frac{d}{2}$ would be set out as shown on the left-hand side of the figure.

FIG. 87.

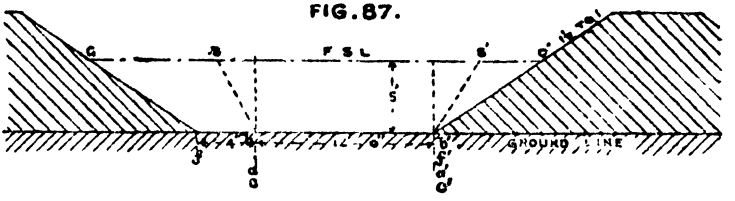


FIG 87(a)

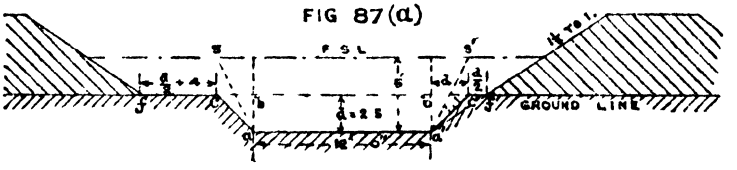
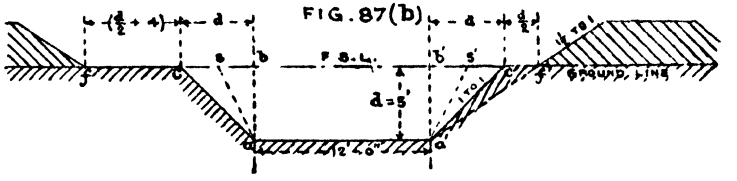


FIG. 87(b)



If no berm is to be left and the slope of banks and cuttings are respectively 1½ to 1 and 1 to 1, a space $c'\ f'$ equal to $\left(\frac{d}{2}\right)$ between the cutting line and the inner toe of the bank must nevertheless be allowed in setting out the work in order that the centre lines of banks and of the canal bed may be parallel.

The dotted lines $a\ s$ and $a'\ s'$ show slopes of ½ to 1 to which the edges of the canal will probably eventually silt while the level to which the berms in figures 87 and 87-*a* should be formed up by silt deposit to the level of F.S L. is shown chain dotted. The areas on either side of the figures between the bank and the sloping dotted lines show the areas to be silted up and from these it will be seen how great a quantity of silting is necessary for the section "with berms" of standard dimensions when the canal is wholly or nearly altogether in bank.

Once berms are up to full supply level (F.S.L.) they undoubtedly strengthen the banks and form a desirable feature in the section of a channel but where berms are much below F.S.L. the extra water section interferes with the *regime* of flow, and may make the maintenance of designed water levels, impossible without resorting to temporary expedients When space is left for berms to be formed by silting, the deposit of silt in the right places must be induced by fences and bushing of thorns or brushwood, etc., and careful supervision is necessary until the channel attains its proper section.

The general formula for the width of berms is stated below:—

$$\text{Width of berm} = C + (r_2 - r_1)\, d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (52)$$

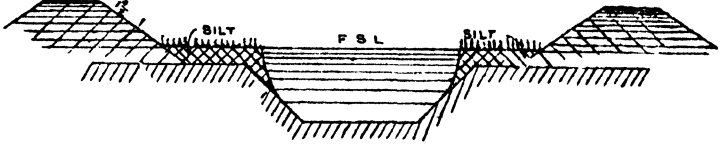
where C is a constant, d the actual depth of cutting at each place, and r_2 and r_1 the horizontal components of the slopes of bank and cutting respectively in 1 foot vertical.

Where S_2 (Slope of bank) is equal to S_1 (Slope of cutting) the width of the berm is C and where no berms are to be provided C is *nil*. In small channels without berms it is desirable to make r_2 equal to r_1.

429. Side slopes of canals.—Although side slopes are generally in ordinary soil excavated at one to one the silt deposit on the edges and berms in most cases gradually converts the slopes to about one half to one and although the actual estimates for excavation and the designs must be made for 1 to 1 slopes it is usual to compute discharges as if side slopes had been cut at ½ to 1

Figure 88 shows a typical case of alteration of side slopes of a distributary and silting up of berms by silt deposit

FIG. 88.



430. Curves in canals.—Bends in canals and channels should be set out as circular curves of as great radius as can conveniently be arranged; in general, the minimum allowable radius is 20 times the bed width of the canal

431. Location of a contour canal.—The area commanded by a contour canal varies with its longitudinal slope, the flatter the slope the larger the area. The slope required to give a suitable velocity of flow is dependent on the capacity of the canal which is in turn dependent on the area to be irrigated; thus these factors are interdependent In laying out the trace of such a canal the capacity, depth, and slope of various reaches from the head must be computed from a rough estimate of areas commanded, and of the approximate positions of the offtakes. From the trial alignment laid out on the above data, corrected figures of areas and quantities of water will be obtained, from which the originally assumed slopes of the canal must be revised, and the location of the trial line adjusted to suit.

432. Velocities of flow and limit of depth of canals.—The velocities of each canal should not be less than that given by Kennedy's formula (paragraph 164 above) modified as may be necessary to suit the conditions of each case and the maximum velocity must be limited to that which the nature of the margins and bed soil of the canal are capable of withstanding without erosion.

In the case of a large canal the latter will frequently be the consideration which fixes the limit of depth. For instance, suppose

the critical velocity of a system at the head was to be 1·1 times Kennedy's v_0 and the maximum mean velocity allowable to ensure freedom from marginal scour is 2·9 f.s.; then (vide paragraph 164 above) the greatest depth of canal allowable is six feet, which requires a non-silting velocity of 1·1 × 2·64 = 2·9 f.s. Although a deeper and narrower canal might be more economical of excavation and in other respects suitable, it would nevertheless in such a case be necessary to limit the depth to 6 feet.

433. Proportion of bed-width to depth in the best discharging channel.—The relative dimensions of the best discharging channel will now be considered. The best discharging channel, that is the channel which, for the same cross section area and slope, passes water with the greatest velocity will be the channel of greatest hydraulic mean depth. Practical considerations necessitate making a channel of trapezoidal section, the slope of the sides being dependent on the nature of the soil, but generally about 1 to 1. This slope as already stated in paragraph 429 above, after some period of use gradually becomes about ½ to 1 and it is usual to allow this as the side slopes in computation of discharges. The problem thus is what is the proportion of bed-width to depth which gives the greatest hydraulic mean depths for channels. Sir Thomas Higham examined this question (Punjab Irrigation Paper No 7) and stated the result in the following form:—

For any trapezoidal channel of fixed sectional area (A), if r is the ratio, horizontal to vertical of the side slopes, and s is the length of the two side slopes per unit of depth which equals $(2\sqrt{r^2+1})$, and M is the ratio of bed-width to depth,

the Hydraulic mean depth $(R) = \frac{\sqrt{M+r}}{M+s}\sqrt{A}$ and it is a maximum when $M = (s - 2r)$.

From this, for a channel of side slopes ½ to 1 the best discharging section works out with $M = 1{\cdot}236$ or say $M = 1\frac{1}{4}$.

434. Cross section of least absorption.—It is necessary also to consider the value of M which will entail least loss by absorption. If absorption per unit of time is considered as varying with the area of wetted surface and depth overlying this surface then the absorption on the bottom of a channel $= k d \times M d = k M d^2$ and absorption of side slopes $= k \times \frac{2}{3}d \times s d = \frac{2}{3} k s d^2$.

∴ Total absorption $(M + \frac{2}{3} s) k d^2$.

The area of the section $A = d(Md + rd) = d^2 (M + r)$.

$$\therefore d^2 = \frac{A}{M+r}.$$

Substituting,

$$\text{total absorption} = \frac{M + \frac{2}{3} s}{M + r} Ak.$$

Thus the absorption per unit of time in any channel of uniform cross sectional area varies as $\frac{M + \frac{2}{3} s}{M + r}$ and this becomes smaller as M increases

To get the proportionate absorption *per unit of flow* it is necessary to divide the above proportion by the velocity of flow.

For ordinary small channels having the same surface fall and cross section area, with the rugosity co-efficient $N = 0{\cdot}025$ and

depths varying between 1 and 5 feet, the velocity varies approximately as $R^{0\cdot82}$—vide Buckley's Irrigation Pocket Book, second edition, page 161. From this, columns 3 and 4 of the table below are obtained. Column 7 gives the proportionate absorption per unit of flow, and shows that for channels of equal cross sectional areas the cross section giving least absorption has a proportion of bed-width to depth of four.

This table refers to channels with side slopes ½ to 1 and of equal area of waterway. (In this case $r = 0\cdot5$ and $s = 2\cdot236$.)

Proportion of bed-width to depth (M)	Hydraulic mean depth (R) $\frac{\sqrt{M^2 + r}}{M + s}$	Proportionate mean velocity.		Proportionate absorption losses per unit of time		Proportionate absorption per unit quantity of water flowing $100\frac{Pa}{Pv}$.
		$R^{0\cdot82}$	Equivalent percentage (Pv).	$\frac{M + \frac{2}{3}s}{M \cdot r}$	Equivalent percentage (Pa).	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0·33	0·355	0 428	95	2·194	139	146
0·50	0 366	0·439	97	1 991	126	130
1 00	0 378	0·450	99	1 660	106	107
1·236	0 379	0 451	100*	1 570	100	100
1·50	0 378	0 450	99	1·495	95	96
2·0	0·373	0 445	98	1·396	89	91
2 5	0·366	0·438	97	1 330	85	88
3 0	0·357	0·428	95	1 284	82	86
3·5	0·348	0 421	93	1 248	80	86
4·0	0·340	0 413	92	1 220	78	84 7†
5·0	0·324	0 397	88	1 180	75	85
6·0	0 306	0 379	84	1 153	73	87
8 0	0·285	0 357	79	1·116	71	90
10 0	0 265	0·336	74	1 094	70	95
12 0	0·248	0·319	71	1 079	69	97
15·0	0 228	0 298	66	1 069	68	103
20·0	0·204	0·271	60	1 048	67	112
25 0	0 185	0·251	56	1 039	66	118

NOTE.—* Best discharging proportion $M = 1\frac{1}{4}$.
† Smallest absorbing proportion $M = 4$.

435. Proportioning bed-width to depth for canal of fixed discharge—In practice it is the quantity of discharge in each length of channel, and not the channel slope and cross section, which is fixed and the variables are the channel surface slope and the proportion (M) of bed-width to depth, and the problem is to design a channel which will carry the fixed discharge with a non-silting velocity and with the smallest absorption losses commensurate with economy of construction

436. Tables of dimensions of canals and distributaries—The following table shows the proportion of bed-width to depth and the corresponding proportionate absorption and cross sectional areas of excavation, for a number of Kennedy (non-silting) canals and major and minor distributaries, and reference to the table will facilitate selection of sections of channels to suit the various conditions which occur in practical design.

These computations are for channels of side slopes ½ to 1, it being assumed that the side slopes originally cut at 1 to 1 will silt up to this slope.

The balancing depth of cutting and the corresponding quantity of excavation, are also computed on the assumption that the banks are of the standard dimensions given in paragraph 426, and the side slopes of the channel 1 to 1.

Table of Main and Branch Canals.

Depth of flow.	Velocity.	Bed-width	Proportion (M) of bed width to depth	Percentage of absorption of quantity per unit of discharge	Water-way, sq ft.	To balancing depth of cutting.			Surface fall from Kutter's formula with values of N.		
						Depth of cutting	Equivalent quantity of excavation	Water surface above (+) and below (−) ground.	N = 0·020.	N = 0·0225	N = 0·025.
Feet	f. s	Feet	M	Per cent	Sq. ft	Feet	C ft.	Feet.	Slope.	one	in
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
500 cusec canal—Bank top width, 6 feet, free board 3 feet											
4·0	2·04	59	14·7	119	244	2·13	130	+ 1·87	7,500	5,920	.
5·0	2·35	40	8·0	108	215	3·07	132	+ 1·93	6,800	5,400	...
6·0	2·64	28	4·7	103	189	4·09	131	+ 1·91	6,000	4,760	...
7·0	2·92	21	3·0	100	171·5	5·0	132	+ 1·92	5,100	4,080	
7·5	3·05	18	2·4	100	163	5·54	131	+ 1·96	4,700	3,780	..
8·0	3·18	15·5	1·9	101	156	6·08	131	+ 1·92	4,[illegible]	[illegible]	
9·0	3·43	11·5	1·3	103	144	7·08	131	+ 1·92	3	15/20	also 25sorr
1,000 cusec canal—Bank top width, 6 feet, freeboard 3 feet											W
4·0	2·04	120	30·0	125	488	1·36	164	+ 2·64	7,920	6,240	. .
5·0	2·35	82	16·4	123	422·5	2·10	176	+ 2·90	[illegible],500	6,080	
6·0	2·64	60	10·0	112	378	2·92	184	+ 3·08	7,280	5,760	.
7·0	2·92	44	6·1	106	332·5	3·88	186	+ 3·12	6,560	5,280	.
8·0	3·18	35	4·4	102	312	4·78	189	+ 3·22	6,000	4,680	...
9·0	3·43	29	3·2	100	301·5	5·67	196	+ 3·33	5,360	4,280	...
10·0	3·67	22	2·2	104	270	6·73	193	+ 3·27	4,620	3,700	...
11·0	3·89	18	1·6	105	258·5	7·67	196	+ 3·33	4,100	3,260	
2,000 cusec canal—Bank top width, 15 feet, freeboard 3 feet.											
5·0	2·35	168	48·4	140	852·5	1·79	303	+ 3·21	8,240	6,480	.
6·0	2·64	124	20·7	125	762	2·53	320	+ 3·47	7,920	6,320	...
7·0	2·92	92	13·1	120	668·5	3·43	327	+ 3·54	7,680	6,080	...
8·0	3·18	74	9·2	112	624	4·30	336	+ 3·70	7,120	5,680	.
9·0	3·43	60	6·6	107	580·5	5·23	341	+ 3·77	6,720	5,280	..
10·0	3·67	50	5·0	104	550	6·16	344	+ 3·84	6,240	4,960	.
12·0	4·12	34	2·8	103	480	8·17	345	+ 3·83	5,120	4,080	
14·0	4·54	24	2·0	100	434	10·14	346	+ 3·86	4,100	3,340	.
15·0	4·75	20	1·3	105	412·5	11·14	347	+ 3·86	3,700	3,000	

Table of Major and Minor Distributaries.

Depth of flow.	Velocity	Bed-width	Proportion bed-width to depth	Percentage of absorption per unit quantity of discharge.	Water-way, sq. ft.	To balancing depth of cutting. Depth of cutting.	Equivalent quantity excavation.	Water surface above ground (+) and below (−)	Surface fall from Kutter's formulae with values of N. N = 0 020.	N = 0 0225	N = 0 025.
Feet	f s	Feet	M.	Percent	Sq ft.	Feet	C ft	Feet	Slope	one	in
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
5 cusec cnannel—Bank top width 3 feet, freeboard 1½ feet.											
0·75	0·54	12·0	16 0	148	9·28	0·97	12·6	− 0·22	..	7,000	5,280
1·0	6·84	5·5	5 5	103	6·0	1 42	9 8	− 0·42	.	3,000	2,550
1 5	1·07	2 5	1·7	100	4 9	2·0	9 0	− 0·50	...	2,500	2,150
1·6	1·13	2 0	1·25	102	4 5	2·12	8·7	− 0 52	..	2,450	2, 000
2 0	1 30	1·0	0 5	113	4 0	2·52	8·8	− 0·52		980	850
10 cusec channel—Bank top width 4 feet, free board 1½ feet.											
0·75	0·54	24·	32 0	170	18·28	0 75	18 6	0		7,000	6,080
1 0	0 84	11·5	11 5	115	12·0	1·2	15·2	− 0·20		4,100	3,500
1·5	1·07	5·5	3·7	103	9·38	1·84	13·5	− 0·34		3,500	2,900
1·8	1·22	4·0	2 2	100	8·82	2·33	10 6	− 0·53		3,000	2,440
2·0	1 3	3 0	1 5	103	8·0	2·40	13 0	− 0 40	.	2,500	2,000
2·5	1·5	1 5	0·6	111	7·0	2 90	12·8	− 0·30		1,700	1,400
25 cusec channel—Bank top width 4 feet, free board 1½ feet											
1 0	0 84	29·	29 0	134	29·5	0·76	22·6	+ 0 24		4,900	3,800
1 5	1 07	15	10 0	113	23 63	1·33	21·7	+ 0·17	.	4,699	3,700
2 0	1 30	8·5	4·2	103	19·0	1·92	20·0	+ 0 08		3,600	3,· 60
2·5	1 50	5 5	2 2	100	16·9	2 46	19 5	+ 0 04	.	3,200	2,600
2 75	1·60	4 5	1 6	101 5	16·2	2 71	19 4	+ 0 04		2,800	2,200
3 0	1·70	3 5	1·2	103	15·0	2 97	19 2	+ 0 03		2,400	1,900
50 cusec channel—Bank top width 6 feet, free board 2 feet.											
2·0	1 30	18	9 0	118	38 0	1 92	38	+ 0·08	.	4,800	3,900
2 5	1·50	12	4 4	111	33 12	2 50	36	0		4,400	3,500
3 0	1 70	8 5	2·8	105	30·0	3 04	35	− 0 06		3,600	3,000
3 5	1 87	6·	1 7	100	27 1	3·60	34	− 0 10		3,000	2,400
4 0	2·04	4	1 0	112	24·0	4 10	34	− 0·10		2,300	1,900
100 cusec channel—Bank top width 6 feet, free board 2 feet.											
2 0	1 30	38	14 0	127	77 0	1 25	53	+ 0 65	7,000	5,200	.
2 5	1·50	25 5	10·0	112	65 6	1 92	51	+ 0 58	6,560	5,200	
3 0	1 70	18·	6 0	103	58·5	2 45	50	+ 0 55	6,000	4,700	.
4·0	2 04	10·	2 5	100	48 0	3 54	48	+ 0 46	4,650	3,500	
4·5	2·19	8·	1 8	100	46·0	4·02	48	+ 0·48	4,000	3,180	...
5·0	2·35	6·	1·2	103	42·5	4·54	48	+ 0·46	3,200	2,700	..
200 cusec channel—Bank top width 6 feet, free board 3 feet.											
3 0	1 70	38	12 9	117	118 5	2·20	88	+ 0·80	6,800	5,440	..
4·0	2 04	22·5	5 6	104	98 0	3 32	85	+ 0 68	6,000	4,700	
5·0	2·35	14·5	2·9	100	85·0	4·38	83	+ 0·62	5,000	3,900	.
5·5	2·50	12·	2 2	100	81 0	4·89	83	+ 0 61	4,500	3,500	
6 0	2 64	9·5	1 6	103	75 0	5·42	81	+ 0 58	3,000	3,050	..
6·5	2·78	8	1 2	104	73 1	5 89	82	+ 0 61	3,500	2,750	...
300 cusec channel—Bank top width 6 feet, free board 3 feet.											
3 0	1·70	58	19·3	124	178·5	1·75	105	+ 1·25	7,200	5,840	
4·0	2 04	35·	8 7	109	148	2·78	104	+ 1·22	6,720	5,440	..
5 0	2 35	23·	4·6	102	127 5	3 84	103	+ 1·15	5,840	4,500	.
6·0	2·64	16·	2·7	100	114·0	4·87	102	+ 1 13	4,940	3,840	
6 5	2 78	13·	2 0	101	105·5	5·40	99	+ 1·10	4,400	3,400	.
7·0	2 9·	11	1·6	101·1	101·5	5·9	100	+ 1 10	3,840	3,100	
7·5	3 05	9·5	1·3	102	99·4	6 37	101	+ 1·13	3,600	2,850	.

437. Characteristics of channels of non-silting velocities with varying proportions of bed width to depth.—Analysis to the above table shows—

(*a*) That, in the case of every size canal and channel in which the relation of velocity to depth is that given by Kennedy's equation, the absorption is a minimum for values of M (Bed width/depth) between 1·5 and 3·2, and for values of M varying from 1·2 and 5 the loss of absorption is never more than 5 per cent in excess of the minimum.

So far as absorption is concerned any value of M between 1 and 5 may be given. To minimise evaporation losses a low value of M is advantageous, but where absorption is considerable these losses are generally relatively small.

(*b*) That when such canals or channels are carried in balancing depth of cutting the quantity of excavation varies but little for values of M between the above limits. The area of land occupied would be greater, the greater the value of M.

(*c*) That the level of the full supply water surface above or below ground level, when running in balancing depth of cutting, varies but little for channels of the same capacity for any ordinary value of M.

For Kennedy channels in balancing depth and width banks of standard dimensions, the level of full supply with reference to the ground surface is approximately as below:—

5	cusec channel about	½ foot in soil.
10	,,	⅓ ,,
·25 and 50	,,	flush with ground.
100	,,	½ foot out of soil.
200	,,	⅔ ,,
300	,,	1⅛ feet out of soil
500	,,	1 1⁶⁄₁₀ ,,
1,000	,,	3 ,,
2,000	,,	3⅔ ,,

438. Balancing depth too shallow for large channels and too deep for very small ones.—It would frequently be undesirable to carry water so much as 3 feet out of soil and for the larger canals deeper excavation than balancing depth would generally be aimed at.

Similarly sacrifice of command would frequently be necessary if 5 and 10 cusec channels were carried in cuttings so deep as balancing depth, and for these channels it will generally be necessary to carry the water at a higher level and dig side pits in order to get earth for the banks.

439. Kutter's co-efficients for channels of different sizes.—The co-efficient of discharge of a channel varies with its condition, a channel with flat regular curves, smooth bed and side slopes and free from weeds would have a low co-efficient and a channel with rough surfaces or obstructed by weeds, a high one. The table demonstrates how much the accuracy of the calculated discharges is affected by the selection of suitable co-efficients.

Small channels of 50 cusecs and under may have a co-efficient as low as N = 0·0225 but when designing it is advisable to assume a co-efficient N = 0·025 for such channels. For larger channels in good condition N = 0·020 may be realized but it is advisable when

designing to allow N = 0·0225. For canals above 1,000 cusecs capacity, N = 0·020 may be allowed.

440. The proportion of bed width to depth to be fixed having in view the slope of the country traversed.—It will be seen from columns 10 to 12 of the above table that, *with non-silting velocities*, the shallower the channel the flatter the slope, and vice versa. Thus when aligning a channel in very flat country it may be necessary to make it wider and shallower than would, otherwise, be economical or desirable.

For instance to carry a 5-cusec channel through ground falling at 1 foot a mile (1/5280) with a velocity equal to Kennedy's V_0 it is necessary to make it 0·75 foot deep, and 12 feet bed width. A channel of the same capacity carried over ground falling 2½ feet a mile (1/2112) might suitably have bed width 2½ feet, and depth 1½ feet.

In carrying a channel at a slope less than the ground slope it is desirable to have a small value of M so as to reduce the height of 'drops.'

Where reservoirs are the source of supply there is no danger of silting, and canals and channels may be given velocities subject only to non-scouring limits. In such cases it will seldom be desirable to make very shallow wide channels and in very flat country it will be desirable to make channels deep in proportion to their width that is the practice is the reverse of that necessary in the case of a 'Kennedy (non-silting) channel.'

441. Other considerations for fixing the proportion of bed width to depth.—There are practical considerations in addition to the above which must be considered in fixing the depth of a channel and depth of cutting and these have special reference to the conditions of the locality through which the channel is to be carried. It is desirable as far as practicable to avoid cutting into very hard, or very porous strata, and a shallow cutting may avoid the necessity for this. Again if the soil is unsuitable for making good banks the channel water would be carried as much within ground as is possible without sacrificing command. If there are few bridges it is desirable where practicable to keep the smaller distributaries at depths not greater than 2 to 2½ feet in order to facilitate fording the channel. It is inconvenient to be constantly changing the depth, and for this reason channels are frequently designed with larger values of M at the head of a reach than would otherwise be desirable.

The depths of large canals are generally limited by practical considerations, and in these the values of M are frequently much larger than would be allowable or desirable for smaller ones.

Where there is no good reason to the contrary the values of M should be kept within limits between 1¼ and 5.

442. Depth of cutting of a channel.—When a canal is running as a contour canal, or on any alignment where the fall of the ground surface is approximately the same as the canal fall, it is desirable, where other considerations do not interfere, to carry it in cutting at, or a little deeper than, balancing depth.

It is generally impracticable to do this for any considerable continuous length in the case of a contour channel, running through a freely undulating country such as that shown in figure 86, as the

frequency with which spurs and valleys have to be crossed, makes it necessary to carry the canal through constantly varying depths of cutting. Where the country traversed has even slopes and little diversity of contour this is not the case, and suitable location will result in keeping the canal in an even depth of excavation.

In very flat country the requirements of 'command' will frequently, *especially in the case of small distributary channels*, make it desirable to carry the channels at a higher level than the above would indicate, and this must never be overlooked, as sufficient 'command' is of the *first* importance and must override all other considerations [the above instruction should only be followed when doing so does not entail loss of effective command.]

443. Depth of a channel bed above a 'drop'—When a canal is running on an alignment where the fall of the ground surface is greater than the canal slope, the longitudinal section will be divided up into reaches by a series of drops (falls), and the depth of the canal bed below the natural surface will where the ground slope is uniform, vary from a maximum just below the drops, to a minimum just above them.

In such cases command must be obtained immediately above each drop, and for some distance up-stream of this, and the outlets will in most cases be located in the lower part of each reach or length between two drops.

The spacing, and therefore the heights, of the drops is generally fixed by the requirements of irrigation convenience in the matter of location and spacing of outlets, and the requirements of command, and it is unnecessary to compare the relative cost of building a large number of low drops with a smaller number of high ones.

Given the height and spacing of the drops, and the capacity, bed-fall, and standard section of a channel, it is desirable to ascertain what is approximately the most economical depth in each reach for the bed of the channel with reference to the ground levels. This depth will be fixed by the depth, with reference to the natural ground level, of the bed of the channel just above each drop.

For example in figures 89, 89-*a* and *b* the ground surface is shown as $g\ g$ the intersection of the ground surface with the drop wall being at g'. The channel bed in each reach is $b\ b'$ the intersection of the bed above the drop with the drop wall being at b'.

The positions and heights of the drops being fixed by other considerations it is clear that the positions and levels of the points g' are fixed and what is required is to fix the level of b' with reference to g' in each case.

When b' and g' coincide (as in figure 89-*a*) the channel bed is level with the surface and the channel is just above the fall wholly in bank and it is wholly in cutting for a short distance below the fall.

Figure 89 shows b' as above g' that is the bed level out of soil to the extent $g'\ b'$.

Let $g'\ b'$ be called x and let it be positive when the bed is out of soil, then the problem is to ascertain for each section of canal the most economical value of x.

Figure 89-*b* shows *b'* below *g'* that is *x* is in this case of negative value.

FIG. 89.

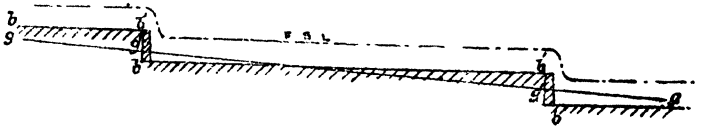


FIG. 89(a)

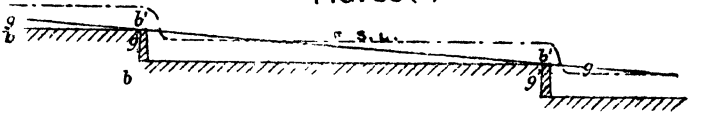
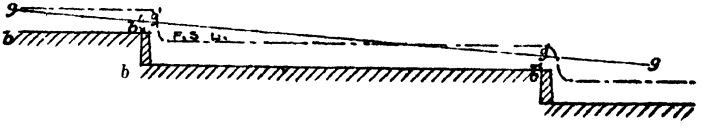


FIG. 89(b)



Where the slope of the ground surface or the rates for excavation, or for the land to be acquired vary greatly in different parts of a length of channel it would be necessary to make trial computations to fix the economic value of *x* for each case

Calculations have been made for homogeneous soil and for ground uniformly sloping and for lengths of channel between balancing depth of cutting in the reach above the drop and balancing depth in the reach below and the results, which are based on land valuation at eighty rupees per acre, and earthwork excavation at an initial rate of Rs. 3-12-0 per 1,000 cubic feet for falls of 4′, 6′, 8′, and 10′, vertical are summarised below.

For channels of standard types with the proportion of bed width to depth from 2 to 4½, the following gives approximately the economic value of *x*:—

(1) Minors of 25 cusecs and under (Standard banks of 3 or 4 feet top width).	For 4 feet drops $x = -\frac{5}{8}$ foot and add $\frac{3}{8}$ of a foot for every foot increase in height of fall.

Thus for a channel of 10 cusecs and a fall of 10 feet, $x = (6 \times \frac{3}{8} - \frac{5}{8}) = +1\frac{5}{8}$ feet and the bed above the drop wall would be $1\frac{5}{8}$ feet out of soil.

(2) Major distributaries of 200 cusecs down to 25 cusecs (Standard banks of 6 feet top width).	For 4 feet drop $x = -1\frac{1}{8}$ and add $+\frac{3}{8}$ of a foot for every foot increase in height of fall.

For larger canals the algebraic value of x for a 4 feet drop becomes slightly smaller as the canal capacity increases. It is, however, undesirable to carry channels in high banks on account of the danger of breaching and it will generally be desirable to keep the channel bed above a fall from one-quarter to one-half a foot lower than economic level.

Deviations up to half a foot on either side of the economic level do not increase the cost of a length of channel by more than 2 to 3 per cent.

It is to be remembered that the economic value of x should be considered only where this does not injuriously affect the *command* required which should always be the first consideration.

444. Capacity of canals.—The duty of water, the allowances for absorption, and the system of distribution are the *data* from which the carrying capacity of canals and distributaries must be computed. These must be fixed before commencing detailed design and the data on which these are arrived at, have been discussed in chapters II and IV above

For paddy cultivation, in new systems in Madras the continuous flow system of distribution is prescribed for ordinary use but there are many existing systems in which regulation is effected by turns and in such cases the carrying capacity of canals or distributaries must be increased with reference to the full supply duty to provide for the non-supply during the periods of complete closure, or of restricted supply under the distribution system in force.

The above preliminaries having been fixed the carrying capacity of each length of each distributary, branch and main canal must be computed *working from the tail end of each canal and channel upwards to the head.* The lay-out of the whole proposed system and the position of, and area irrigated from, each off-take in each distributary should be plotted on an irrigation map. From this, tables must be drawn up for each distribution channel commencing from the tail end showing the discharge required after allowing for absorption in each length of distributary and canal to keep the system fully supplied at the time of greatest demand.

From the above tables the carrying capacity required of each part of each canal and channel throughout the system will be ascertained and the canals must be designed to carry the maximum supply and distribute the whole in the required proportion throughout the whole system.

445. Command.—Each canal and distributary must be designed not only to carry the quantity required at times of maximum demand but to carry the normal supplies at all times at such a level that the water can be run by gravitation on to the land to be irrigated. This applies not only to periods when the canals are running full but to times when only very limited supplies are required for the crops.

It is generally desirable to design canals and channels so that full command of all land is attained when half of full supplies are carried. The matter should be considered with reference to the special circumstances of each system and such consideration is a preliminary to design of canals. To secure fully effective command it is desirable, wherever it is practicable, to carry water in a

distributary at such a level at the outlet to the field channel as to be half a foot above the level of the highest land to be irrigated and this condition should obtain during periods of low supply; where such land is any considerable distance from the outlet further allowance must be made for loss of level by flow through the field channels. The tabular statement referred to in paragraph 444 should give, in addition to the area under each outlet, the level and distance from each outlet of the highest land to be irrigated.

From these the designer can compute the level of water required in the channel at the outlet in order to fully command the land.

It will frequently happen as mentioned in paragraph 431 above that the level at which a canal or distributory can be carried is the factor which fixes the area of land to be irrigated by such channel; in such cases it is most undesirable to include in the *ayacut* (assigned irrigable area) of a channel any land which can only be commanded with difficulty. Such action leads to many irregularities and difficulties in distribution in times of scarcity.

446. Considerations influencing the proportions of cross sections of channels.—It will be gathered from what has been said above that the sections of channels of the same carrying capacities may properly be designed to differ very greatly in the proportion of bed width to depth in accordance with the special conditions which attach to each case. For instance—

(1) In most direct flow irrigation systems it is necessary, wherever practicable, to design channels of velocity, not less than the non-silting velocity computed from Kennedy's rule, and not greater than is safe in view of the liability to scour of the side slopes, or the bed of the channel.

In the case of storage systems the disposal of silt entering at the head need not be considered but the velocity must be limited so as to be non-scouring.

In most soils marginal scour will not occur with mean velocities of less than 2·5 f.s., and considerable higher velocities up to 3½ feet are generally safe in clay, while in mooram, kunkur, soft rock, etc., velocities even higher than these are safe.

(2) Where other considerations permit, channels should be designed with a proportion of bed width to depth between 1¼ and 5

(3) Where the soil consists of an impervious surface with a pervious subsoil the design should aim at a depth of channel which will as far as possible not uncover the pervious strata.

(4) Where the material excavated is bad for making banks the channel should be kept more in soil than would be desirable with good material.

(5) Where there are crossings for which bridges are not provided the full supply depth of smaller channels should not exceed 2½ feet in order to give greater facilities for fording.

Bridges or other masonry works across channels are more cheaply constructed in deep narrow channels than in broad shallow ones and where there are many such works a deep channel is an advantage.

(6) With a view to making fewer changes of depth of a channel necessary it is frequently desirable to have a higher

proportion of bed width to depth at the head of a reach than would otherwise be given.

447. Change of carrying capacity of a canal.—Change of capacity is effected—

(1) By reduction of bed width.
(2) By reduction of depth.
(3) By reduction of velocity of flow.

Grading of the surface fall should generally be arranged to give approximately the velocity of flow due to the depth under Kennedy's theory and therefore (2) and (3) re-act on each other, and where depth is altered grading is also altered to give the critical velocity due to the new depth.

The most common method of reducing capacity is to change the bed width, such change being made just below the offtake to which the required reduction of capacity is due

As the bed width of a canal becomes narrower the proportion (M) bed width to depth becomes smaller and after a time a change of depth is necessary to give a more suitable value to M.

448. Changes of depth of a channel.—Changes of full supply depth must be made between the head and tail of a canal of any considerable length or magnitude; a depth which is suitable for the upper end of the canal becomes unsuitable as the full supply capacity is reduced. In the case of a canal with drops at intervals along its length change of depths may most conveniently be made at such works. In other canals changes may be made at regulators or at regulating notches which are built across the canal. Where there are no drops or regulators near a place where reduction of depth is called for, this must be effected by allowing a bed fall of less steep slope than the surface fall of the channel In extreme cases the channel bed may be kept level for the length necessary to effect a change. This method is not so convenient as changing depth at regulators or falls or notches nor does it prove so satisfactory in operation.

449. Mile and bench mark stones.—Mile and furlong stones set in concrete foundations are planted along the length of all canals and channels in Madras. The mileage is measured along the axis of the canal.

In some systems all mileage is measured from the headworks of the system while in others the mileage of each canal and channel is measured from the canal or channel head sluice. It appears generally to be desirable to measure mileage of a main or branch canal from the headworks of the system and that of a distributary from the sluice in the main or branch canal or tank through which the channel draws its supply.

Mile and furlong stones should generally be set up clear of the outer toe of the bank of a canal.

Bench mark stones should be set up along the line of a canal or channel, at intervals not exceeding one mile, before the execution of the work is commenced, and the levels of the bench marks must be ascertained by carefully checked levelling. The stones must be placed where they will not be disturbed during the execution of the work.

It will frequently be found convenient to set up mile stones before the commencement of any work and use these as bench marks during execution. As deviations in sanctioned alignments are frequently made during execution it will sometimes be necessary to shift some of the mile stones when the work is complete and for this reason furlong stones should not be planted during the earlier stages of the work.

Mile or bench mark stones may be 3 feet long and about 8 inches square in section, the top foot should be dressed true and square and have the mileage or number of the stone cut on one face. The lower two feet should be set in concrete foundation

450. Contour channels in single bank—When traversing country with a steep cross slope, channels are sometimes made throughout the whole, or part of their length with a bank only on the lower side of the slope. When this is so, all drainage intercepted in the single banked length is taken into the canal and must be disposed of by outlets at suitable sites. A single banked channel will have a maximum water level (M.W.L.) higher than F.S.L. and the bank must be given a suitable free-board above M.W.L. Where the channel crosses a valley in bank the water will ordinarily fill the depression up to F.S.L. forming a tank, the extent of which depends on the height of banking and the general slope and contour of the depression, and full supply can be passed to the canal below only so long as this tank is filled up to the F.S.L. of the channel issuing from it. If the united capacity of the tanks along the length of channel is great in comparison to the channel carrying capacity, it may take a considerable time at the commencement of an irrigation season before full supply can be passed down to the tail end of the channel; this is one objection to crossing depressions in single bank. Further objections are that the submerged area is not available for cultivation, losses by the percolation and evaporation are increased, and the silt carrying *regime* of the channel is interfered with, most of the silt in suspension being deposited in the tanks, on the other hand the disposal of the drainage entering the tank can frequently be effected by an ordinary tank surplus work at much less cost than by other forms of cross drainage works and the bed silt of the drainage will be deposited close to the waterspread boundary and is unlikely to affect the canal

In South India a number of channels of considerable size are carried in single bank for a great part of their length. The largest of these is the Kurnool-Cuddapah canal, and many other examples of this class are afforded by the channels taking off from the upper Cauvery and the Tungabhadra The *regime* of such channels is generally much interfered with by the silt brought into the canal with surface drainage and the maintenance charges are high.

On the whole the carrying of a canal in single bank is only suitable to special and somewhat exceptional conditions and is ordinarily not to be recommended.

451. Tanks formed at drainage crossings used as storage works.—It may be here pointed out that tanks such as have been described may serve a useful purpose of storage works wherever a drop of F.S.L. takes place just below the tank. Under such

circumstances the yield of the local drainage and the capacity of the tank between the F.T.L. and the F.S.L. of the channel become available and the tank will be an auxiliary storage work and may be most useful in steadying the flow in the portion of the channel below it.

452. Masonry works of distribution systems.—The masonry works of distribution systems consist of—

(*a*) Works required to control velocity of flow, to maintain the required water levels, and effect distribution throughout a canal system.

These works comprise drops, regulators, regulating notches and dividing dams, irrigation sluices and pipe outlets.

(*b*) Communication works.

These comprise bridges, road syphons, fords and cattle crossings.

(*c*) Cross drainage, surplus, and scouring works.

These comprise aqueducts and syphon aqueducts, superpassages and syphons, level crossings, inlets and outlets, surplus and scouring sluices, and tail escapes.

Of the above, (*a*) and (*b*) are described in chapter XII and (*c*) in chapter XIII

CHAPTER XII.

DISTRIBUTION SYSTEM—MASONRY WORKS.

CANAL 'DROPS' OR 'FALLS.'

453. Canal 'drops.'—Where the natural slope of the ground over which a canal is carried is greater than the slope of the canal the difference is adjusted by constructing vertical 'drops' or 'falls' at suitable intervals The terms 'drop' and 'fall' are interchangeable, the former is generally used in America and South India, and the latter in Northern India.

454. Positions of 'drops.'—The best positions of offtakes from a canal must be fixed with reference to the contours of the ground to be irrigated and plotted on a longitudinal section of the canal alignment showing the ground surface levels; from this section the approximate positions and the required heights of drops for the slope and full supply depth of the canal must be fixed so that the canal water levels at the points of offtake will be at levels giving suitable command.

The approximate position of a drop being fixed as above, the exact position should be decided with reference to local considerations such as the suitability of soil and subsoil, communications, etc. The economical level of the canal bed just above a fall with reference to the ground level has been dealt with in paragraph 443 above.

455. Essential features of a drop.—A drop is merely a vertical step in a channel bed, the drop wall being a retaining wall forming the riser, and the aprons and wings forming protective works to prevent the drop wall being under-scoured or outflanked, or the channel eroded by the turmoil or velocity due to the drop.

The conditions of a drop are similar to those of a river weir in cases where the stream bed level above the work has been filled to about the level of the weir crest.

456. Control of the velocity and of the water level in the approach to a drop—The mean velocity of water passing over the crest of a weir built across a channel is greater than that of the channel above; it is therefore desirable to contract the waterway over the drop wall in such a way that the surface slope, and velocity of the canal water up-stream of the work, may not be affected by it. If the canal were always carrying the same quantity of water this could be effected by raising the sill (or crest) of the drop wall above the bed level of the canal, and this device was generally used for many years and is still occasionally employed. It is however fully effective for only one stage of flow and an increase of flow above this must result in abnormal surface fall and velocity in the water for some distance above the drop, with liability to scour, while diminished flow causes 'pooling' above the work with liability to silt deposit.

Another alternative is to contract the width of the sill making a single rectangular notch. This localizes and concentrates

discharge in such a way as to induce local scours and erosions in the canal while, as in the case of the raised sill, it will only keep the approach flow at the normal canal surface slope and level for one stage of discharge.

457. Control by 'Trapezoidal notches.'—What is required is to so restrict the area of flow that the approach water shall be maintained at the normal velocities and surface slope of the canal for the varying discharges which may be passed down. This may be effected by raising the drop wall to the full supply level of the canal above the work leaving along the length of the raising one or more trapezoidal notches with sloping sides, the sills of the notches being at the level of the canal bed. This method of automatic regulation was originally introduced in the Punjab and has been most successful in attaining the object sought, and all drops should be designed with trapezoidal notches, except where the crest of a drop wall is fitted with movable shutters forming a combined regulator and drop. In such a case the level and velocity of the approach to the drop is controlled by the regulator and trapezoidal notches are unnecessary.

458. Theoretical and practical form of trapezoidal notch.—The theoretical correct form of the notch, or set of notches, capable of passing every discharge of a canal with the normal depth in the canal due to that discharge is a regular egg-shaped curve. Such a curve is difficult to set out and build, and a notch with a horizontal sill and straight sides of suitable slope gives the required conditions without such errors as to cause practical inconvenience. The section of flow through such notch is thus a trapezium and as the side slopes are straight instead of curved, there will be two levels of discharge and only two which will *exactly* meet the theoretic requirements and in order to find the slope required for the sides, the width of the notch for two suitable water levels must be calculated

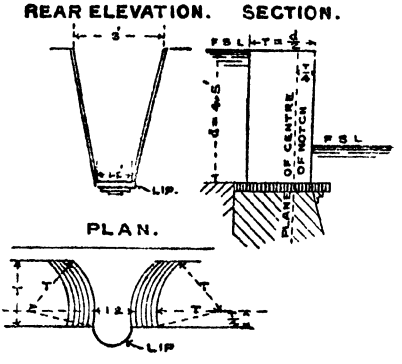
459. Water levels for which notches should be computed — It is important to select suitable water levels for which to compute the widths of notches, and these cannot be fixed by any invariable rule but must be selected to suit the conditions of regulation and variations of flow in the canal. For ordinary purposes the upper level may be taken at, or a little below, the ordinary full supply level (F.S.L.) of the canal, and the lower one a little below the level of the lowest working supply, but not higher than the level of half supply.

460. Details of plan of a trapezoidal notch.—The plan of notch originally used in the Punjab involved a complicated construction of curves with a very wide splay on the up-stream side, which in turn required a width of piers between notches which is inconveniently great in many cases; while such a shape gives a high co-efficient of discharge, it is an unnecessary complication and a simple and suitable construction is shown in figure 90.

If T be the thickness of the notch wall, the plan of sides of the notch may be a series of circular arcs struck with radius T and centres on a line ¼ T from the down-stream face of the drop wall. If the drop wall is curved in the plan the radius may be somewhat reduced. A spreader stone, called the 'lip,' supported

if necessary by corbelling, projects beyond the down-stream face of the drop, the amount of projection being limited to one foot or half the width of the notch sill at its down-stream edge whichever is the smaller, and the circular arc forming the edge of the lip cuts the down-stream extremities of the sill of the notch. The lip serves to spread out and distribute the water falling from each notch over the surface of the apron

FIG. 90.



461. Formulæ for discharge of a trapezoidal notch.—Let D be the discharge in cusecs, divided into two parts, the discharge (R) through the rectangular portion of the notch, and the discharge (S) of the sloping triangular portions.

Let E = the submersion, that is the depth of tail water over the sill of the notch.

l = the width of the horizontal sill of the notch in feet.

$n = 2 \tan \alpha$, where α is the angle made by each of the sides of the notch with the vertical.

C = the co-efficient of discharge of the notch.

d = the depth in feet over the sill of the notch.

Then, excluding velocity of approach, the general equation for the discharge of a submerged notch is—

$$D = C\sqrt{2g}\,(d-E)^{\frac{1}{2}}\,(l + \tfrac{1}{2}\,En)\,E + \tfrac{2}{5}\,C\sqrt{2g}\,(d-E)^{\frac{5}{2}} \times \left\{ l + En + 0{\cdot}4n\,(d-E) \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (53)$$

(Vide equation (*e*), Punjab Irrigation Branch Paper No. 2).

The first item of the expression is the discharge due to the submerged (or drowned) portion of whole discharge and the remainder that due to the free portion. If the notch is wholly free, without any submergence, E becomes *zero* and also the first item disappears and the equation becomes

$$D = 5{\cdot}35\,C\,d^{\frac{3}{2}}\,(l + 0{\cdot}4\,dn) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (54)$$

which is the general equation for the discharge of a trapezoidal notch with free overfall.

Equation (53) above may be expressed as follows :—

$$D = 5{\cdot}35\,C\sqrt{d-E}\left\{ (\tfrac{1}{2}\,E + d)\,l + [\tfrac{8}{15}\,E^2 + (d-E)\,E + 0{\cdot}4\,(d-E)^2]n \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (55)$$

In equations (54) and (55) above the numerical value 5·35 is substituted for $\frac{2}{3}\sqrt{2g}$.

462. Nethersole's Discharge Tables of trapezoidal notches.—The first expression within the main bracket of equation (55) represents (R) the discharge through the rectangular portion of the notch, and the second expression (S) the discharge through the two sloping portions.

Thus:—

$$R = 5{\cdot}35\ C\sqrt{d - E} \times (\tfrac{1}{2}E + d) \times l \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (a)$$

$$S = 5{\cdot}35\ C\sqrt{d - E} \times [\tfrac{3}{4}E^2 + (d - E)E + {\cdot}4\,(d - E)^2]\ n \quad . \quad (b)$$

Nethersole's "Discharge Tables for Trapezoidal Notches for Distributary Falls" are derived from equations (a) and (b) and give:—

(1) Values of R for each $\frac{1}{10}$th foot width of sill, for values of 'E' from 0·1 up to 4·5 feet, and values of d from 1·0 to 5 feet.

(2) Values of S for values of E and d as above, and for notches of side slopes $\frac{1}{2}$, $\frac{1}{3}$, $\frac{1}{4}$, $\frac{1}{5}$, $\frac{1}{6}$.

The value of C in the tables is 0·70.

The use of these tables if available greatly facilitates design of suitable notches.

The above formulæ do not include separate computation for velocity of approach and it has been found in practice that the error due to this can be eliminated within reasonable limits by applying slightly higher values to the co-efficient C than would otherwise be suitable. A co-efficient of 0·7 has been found suitable for channels up to a depth of 5 feet. For large canals of higher velocities the co-efficient may be increased and 0·78 has been generally found suitable. If tables are not available the dimensions of notches should be worked out from equations (54) or (55) above

463. Example of calculation for a trapezoidal notch.—The following example of computations for the notches of a drop in which both the equations (54) and (55) are brought into use gives an illustration of the method of applying them:—

A full supply of 200 cusecs is to be passed in a channel of bed width 20 feet and depth 4½ feet above the drop. The channel carries half supply with a depth of 3 feet.

At full supply the tail water submerges the sill by 1·2 feet, and at half supply the sill is unsubmerged and the fall a free one.

The whole discharge will be passed through 3 notches.

In this case we have therefore

$$D_1 = \frac{200}{3} = 66{\cdot}7,\ d_1 = 4\tfrac{1}{2},\ E_1 = 1{\cdot}2.$$

$$D_2 = \frac{100}{3} = 33{\cdot}3,\ d_2 = 3,\ E_2 = 0.$$

It is required to find values of l and n of a suitable notch. The co-efficient 0·70 is used throughout.

From equation (55) above

$$3{\cdot}74 \times \sqrt{3{\cdot}3}\left\{5{\cdot}1\ l + \left(1{\cdot}08 + 3{\cdot}96 + 4{\cdot}36\right) n\right\} = 66{\cdot}7$$

therefore $5{\cdot}1\ l + 9{\cdot}4\ n = 9{\cdot}96$ . . . . . (a)

again for the half supply applying equation (54)

$$3{\cdot}74 \times 3\tfrac{2}{3}\left(l + 1{\cdot}2\,n\right) = 33{\cdot}3.$$

$$\therefore l + 1{\cdot}2\,n = 1{\cdot}71.$$

or $5{\cdot}1\,l + 6{\cdot}12\,n = 8{\cdot}72$ . . . . . (*b*)

Combining equations (*a*) and (*b*) to eliminate '*l*' we get

$$3{\cdot}28\,n = 1{\cdot}24.$$

$$n = \frac{1{\cdot}24}{3{\cdot}28} = 0\,38.$$

From this tan $a = 0{\cdot}19$ and the slope of the sides of the notch would be $\frac{19}{100}$.

Substituting this in equation (*b*) above

$$l = 1{\cdot}71 - 1{\cdot}2 \times {\cdot}38 = 1{\cdot}25 \text{ ft.}$$

In practice a slope of $\frac{1}{5}$ would be taken and the notch would have a sill $1\frac{1}{2}$ feet wide.

Thus each notch would have a sill of 1·2 feet and side slopes of one in five.

The top width of the notch at F.S.L would thus be

$$1{\cdot}2 + 2 \times \frac{4\frac{1}{2}}{5} = 3 \text{ feet.}$$

This notch is depicted in figure 90 above

464. General features of design of drops.—A canal drop is essentially a weir over which flow is controlled and frequently continuous for weeks and months together. The apron below is generally protected by a depth of tail water overlying it not greatly less than the depth of water passing over it. As it is important to limit velocities below the work so that but little protective pitching below the fall may be required it is desirable to dissipate the turmoil due to the fall as quickly as possible. Under these conditions drops are almost invariably designed as type A weirs, in which the water is dropped vertically immediately below the weir wall on to a masonry apron or a water cushion (vide paragraph 278).

465. Profile and length of drop wall and number of notches.—When the soil is impervious the drop wall is of the nature of a retaining wall surcharged with the weight due to the depth of the water up-stream of the wall, and need not be designed as subject to the full hydrostatic pressure due to the level of water above it; the extent to which it is so subject is a matter for judgment having in view the permeability of the soil. Generally the base width of the drop wall may be about that of the elementary triangle that is about $\frac{(H+d)}{\sqrt{\rho}}$, where H is the vertical height from the sill to the apron, and *d* the maximum depth of water passing over the drop; the formula is similar to that for the base width of the elementary triangle for a dam subject to full hydrostatic pressure *without uplift* (vide paragraph 203 above). Where *H* is large and the soil is but moderately pervious the formula gives a base width somewhat in excess of requirements while for very pervious soils or for large values of *d* a some what wider base may be required. In every case the filling between the up-stream face of the drop wall and the face of the excavation should be puddled

clay as shown in figure 91. If the natural soil is very pervious a puddle apron (as in the case of a weir) may be laid in the bed of the channel for some distance up-stream of the drop and this apron should be carried well under the water face of the banks on either side of the bed

The length of the drop wall between abutments should not be less than seven-eighths, or more than the full bed width of the channel up-stream of the fall.

Number and sill level of notches.—The sills of the notches should be at the bed level of the canal and the notches should be of suitable number to distribute the discharge fairly throughout the length of the drop wall.

The width of the notch piers at F.S.L should be not less than half the full supply depth (d) of the channel above, and the length of the piers should be subject to the same minimum unless in either case a lower dimension can be proved by stability computations to be suitable.

The tops of the piers should be at F.S.L. unless they are utilized for carrying a foot bridge or other superstructure. Three-quarters width piers should be provided contiguous to each abutment. The full supply level above referred to is the greatest depth at which the canal is to be run *for supply purposes* and not necessarily the ordinary full supply.

The top width of a notch will generally not exceed 0·75 d where the notch is 'free,' and d where it is submerged; the top widths generally will be between 0·7 d and d.

Notches which give greater depth on the sill for any discharge than the normal depth of the canal above are termed 'tight,' and those giving less depth are 'loose.' In cases of doubt it is preferable to build a notch a little 'loose,' as this can more easily be rectified than a 'tight' notch.

The thickness of the drop wall at sill level may be from ½ to 1 foot wider than the length of the notch piers, that is between ($\frac{1}{2}\,d + \frac{1}{2}$) and ($\frac{1}{2}\,d + 1$) feet.

466. Width of the main apron of a drop.—The width (A) of the main apron down-stream of the drop wall should be equal to the length (L) of the drop wall *plus* half the depth (d) of the canal up-stream, subject to a minimum of the bed width of canal below the drop.

$$A = L + \tfrac{1}{2}\,d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (56)$$

If the width is greater than the bed width down-stream this width should be contracted gradually in the revetted slopes down-stream of the wings, the contraction being not more rapid than one foot in bed width to five feet in the length of the canal.

467. Depth and limits on use of water cushions.—The nature of the apron below the drop wall, viz, whether a solid or a cushioned apron, depends on the depth of the drop (h) which is the difference between the water levels up and down-stream of the fall, and on the depth (d_c) of the water passing over the crest (or sill) of the drop wall and the depth (d_1) of the water in the channel below. With a notched fall, (d_c) equals the depth (d) in the canal up-stream

of the drop, but in cases where the sill is raised above the canal bed this is not the case.

If a solid apron is built this will be cushioned to a depth d_1 by water in the channel below.

If a water cushion cistern is built of depth (X) below the bed of the channel down-stream of the drop, the *effective* depth of water acting as a cushion is $(d_1 + X)$.

There is much difference of opinion among experts as to the extent to which it is desirable to build water cushion aprons, and also regarding the depth and dimensions to be given to the cisterns of such aprons From recent experience of successful application of solid aprons to deep falls it appears probable that a more extended application of solid aprons to canal falls is desirable.

The following formula which was introduced by Dyas is frequently used to give the depth of a water cushion :—

$$(X + d_1) = d_1 + (d_c^{\frac{1}{3}} h^{\frac{1}{2}})$$

$$\text{or } X = d_c^{\frac{1}{3}} h^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (57)$$

This formula is in general use in the Punjab.

The following formula is considered more suitable to ordinary conditions in Madras and in places where the soils give better foundations than is usual in the Punjab works :—

$$(X + d_1) = \tfrac{1}{2} d_c \sqrt{h} \quad . \quad . \quad . \quad . \quad . \quad (58)$$

Where $d_1 = d_c$ this formula gives a *nil* or a negative value to 'X' for all values of 'h' up to 4 feet, which implies that, under these conditions, a solid apron would be built.

468. Lengths of main aprons of drops.—The down-stream exit of the cistern of a water cushion must be built with a slope so that stones and other debris carried into the cistern may be washed out. The counter slope may be either masonry dry pitching at a slope of 1/5, or solid masonry at a slope of 1/3 As a rule the former is most suitable and economical.

The length of the horizontal floor of the cushion from the toe of the drop wall should be sufficient to contain the horizontal projection of the overfall ($2\sqrt{d_c h}$) due to the values of d_c and h with some additional margin of length, which margin should increase with d_c.

A suitable length is given by the expression

$$2d_c + 2\sqrt{d_c h} \quad . \quad . \quad . \quad . \quad . \quad (59)$$

subject to a minimum of $2 + 2\sqrt{d_c h}$

For a solid apron the same considerations apply but a greater length is required as there is no counter slope and a suitable expression is

$$2d_c + 2\sqrt{d_c h} \quad . \quad . \quad . \quad . \quad . \quad . \quad (60)$$

subject to a minimum of $4 + 2\sqrt{d_c h}$

The thickness of the masonry apron depends on the nature of its foundations. It must be sufficient to resist uplift and in ordinary soils should not generally be less than $\sqrt{d_c + h}$ . . . (61)

469. Length of revetment and bed pitching.—The length of side revetment and pitching in the bed of the channel above or below a drop must vary with the nature of the soil. In ordinary

loamy soils the following lengths of revetment are suitable, *the lengths of bed pitching being one-half the length of revetment.*

Measured from the face of the drop wall:—

the length of the up-stream revetment = $3d$ . . . . . (62)
(subject to a minimum of 10 feet).

Measured from the down-stream edge of the main apron or of the counter slope of the water cushion:—

the length of the down-stream revetment = $4\,(d_c + h)$. (63)
(subject to a minimum of 20 feet).

470. Bank connexions.—Masonry bank connexions are either of the 'wing type' or of the 'core-wall type.' These have been briefly described in connexion with tank weirs (vide paragraph 397) and the requirements do not differ greatly in the various types of irrigation works, such as regulators, sluices, weirs and drops, which are subject to a difference of water level on either side.

As the bank connexions of the same type of drops having the same values of d, d_1, and D, will be practically the same for works of greatly varying crest length it follows that for works of small discharging capacity the connexions are likely to bear a relatively large proportion to the whole cost of the work and vice versa.

The wing type of bank connexion is the most satisfactory as giving best protection to the work, and forming the best guides to the water on the approach and run off and containing the earth slopes in the most effective way, they are also cheaper to maintain.

The core wall type of bank connexion is generally cheaper to construct than the wing type and where the earth slopes to be retained are of small vertical height, this type is almost as effective as the wing type.

From the above it will be gathered that for the larger works, or works involving high banks or cuttings, wing bank connexions should generally be used, and core walls for small works with low cuttings and banks.

471. Example of design of a drop.—An example of working out the design of a drop on stated data is now given:—

A six feet drop is to be built across a 200 cusec channel of depth $4\frac{1}{2}$ feet and bed slope $\frac{1}{4600}$ up-stream, and depth 4 feet down-stream. The bed width up-stream is 20 feet and down-stream is 23 feet.

Notch calculations.—

The crest length would be between 20 and $(7/8 \times 20) = 17\frac{1}{2}$ feet.

Let there be three notches. These will be worked out for full and half supply levels the latter being 3 feet depth.

Then in equation (54), page 253 above,

$D_1 = 66{\cdot}7$, $d_1 = 4{\cdot}5$

$D_2 = 33{\cdot}3$, $d_2 = 3{\cdot}0$

$C = 0{\cdot}70$

then $3{\cdot}745 \times (4{\cdot}5)^{\frac{3}{2}}\,(l + 1{\cdot}8n) = 66{\cdot}7$ . . . . . (*a*)

and $3{\cdot}745 \times 3^{\frac{3}{2}}\,(l + 1{\cdot}2n) = 33{\cdot}3$ . . . . . (*b*)

hence $35{\cdot}75\,(l + 1{\cdot}8n) = 66{\cdot}7$

and $19{\cdot}5\,(l + 1{\cdot}2n) = 33{\cdot}3$

$\therefore\ l + 1{\cdot}8n = 1{\cdot}86$

$l + 1{\cdot}2n = 1{\cdot}79$

Subtracting $\cdot 6n = 0\cdot 16$

Therefore $\tan a = \frac{16}{120} = \frac{2}{15}$

Make the side slope of the notch 1 in 8.

From this $n = 0\cdot 25$

$$l = 1\cdot 7 - 0\cdot 25 \times 1\cdot 2 = 1\cdot 40$$

The sill of the notch will be made 1′ 5″ wide.

The top width will be 1′ 5″ + ¼ (4′ 6″) = 2′ 6½″.

The top width of the piers may be made 2′ 9″ and the abutment piers may have tops 2′ 2¼″.

This gives a total top width of the drop wall between abutments of 17½ feet.

The width of the main apron, from paragraph 466, would be 17½ + ½ of 4½ = 19¾ feet; but as it must not be less than the down-stream bed width it will be 23 feet.

The depth X of cushion, if any, should be found from the equation $X + d_1 = \frac{1}{2} d_c \sqrt{h}$ (vide equation 58, paragraph 467).

Here $h = 6$, $d_1 = 4$, $d = 4\frac{1}{2} = d_c$.

$X = 2\cdot 25 \times 2\cdot 45 - 4 = 1\cdot 5$ feet.

Length of floor of cistern of cushion $= d_c + 2\sqrt{d_c h} = 4\cdot 5 + 2 \times 5\ 2 = 14\ 9$, say, 15 feet.

The soil in which the work is to be situated is good loam and a percolation gradient of 1 in 5 may be taken as safe.

The length of creep must not be less than $5 \times 6 = 30$ feet.

The thickness of the floor will be $\sqrt{d_c + h} = 3\frac{1}{4}$ feet.

The bed level of the channel just above the drop will be kept at economical depth in soil, viz., $\left(1\frac{1}{8} - \frac{6}{8}\right) = \frac{3}{8}$ foot in cutting. (Vide paragraph 443 above).

The length of revetment up-stream will be $3d$, say, 14 feet.

The floor will be pitched for 7 feet up-stream of the drop wall.

The revetment down-stream of the lower end of the reverse slope of the cistern will be of length $4(d_c + h) = 42$ feet and the pitching on the bed 21 feet.

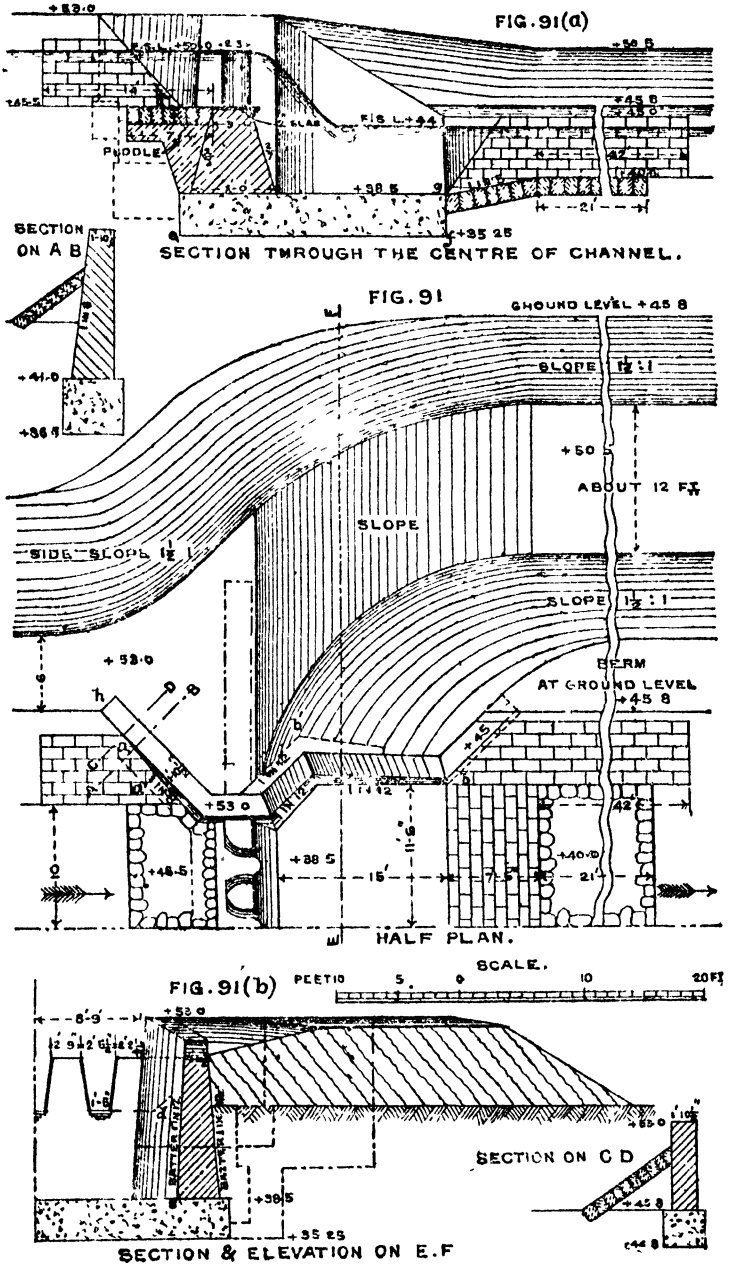
The thickness of the drop wall at sill level of notches may be $\left(\frac{3}{4} + \frac{d}{2}\right) = 3$ feet.

The drop wall should have a bottom width somewhat less than that given by the formula $\frac{h + d_1 + X}{\sqrt{\rho}} = \frac{11\frac{1}{2}}{\sqrt{2}} = 8$ feet (paragraph 465).

A base width of 7 feet and a batter of 2 in 7 will be given to the up-stream and down-stream faces. The bank connexions will be of the wing type.

Figure 91 shows the design of this work with 'wing' bank connexions, the masonry being brick and the tops of retaining walls 1′ 10½″ that is 2½ bricks thick; the dimensions of the channel banks follow the ordinary *standards* with the exception that *in the immediate vicinity of the fall*, berms are omitted in order to shorten the length of the wings or returns and this is generally recommended. The height and width of the bank down-stream and abreast of the work is made suitable to the amount of spoil to be disposed of.

The up-stream wing is splayed at an angle of 45 degrees in plan, its batter 1 in 8 is all on the outer face this being the face which gives greatest stability. It is assumed that the brickwork is to be plastered and therefore this can be done without inconvenience by



offsets at each course of brickwork. The outer end of this wing is of reduced section as it has little thrust to support and its chief *rôle* is that of a stop wall. The foundation of this part of the wing is stepped up to level + 42·8 from the point where the line of percolation under such stepping will be not less than 30 feet. The abutment and down-stream wings and splayed returns are given a face batter of $\frac{1}{12}$ while additional back batters to secure stability are given to the higher lengths of the down-stream falling wings. The splayed return has its top at 1 foot above F.S.L. down-stream.

472. Testing a design for percolation and uplift.—It is necessary to test the design :—

(*a*) For safe length of percolation under the apron.
(*b*) Do do round and under the wings.
(*c*) For security of the floor against uplift.

(*a*) The shortest line measured on the section is *d e f g* and measures about 35 feet

(*b*) One percolation line is that under the foundations of the splayed returns from *a* to *b* on plan, a distance of 27 feet In addition the vertical lengths of earth through which percolation must pass under the stepped-up wing gives about 8 feet more. The shortest line of percolation might be *a' b' b* under the wing, or *a h b* round the end of it. The lengths of all the 3 lines named are about the same (35 feet).

(*c*) The greatest net uplift head on the apron is at the down-stream toe of the drop wall and is (18/35 × 6), say, 3 feet As the apron is affected by flotation, its effective downward pressure is $(\rho - 1) \times 3\frac{1}{4}$ and if $\rho = 2$ this is $3\frac{1}{4}$ feet which is sufficient to counterbalance the uplift with 8 per cent to spare. On the whole it might be better to make the apron $3\frac{1}{2}$ feet thick as $3\frac{1}{4}$ feet leaves so little margin.

473. Splayed Wings.—The up-stream wings have to extend to the top of the slope of the bank and wings splayed at 45 degrees are 40 per cent longer than if made parallel to the crest of the drop wall.

On the other hand the splay gives:—
(1) a better approach to the work,
(2) a wider top of bank on each flank, and
(3) a longer length for the path of percolation.

The question whether to employ splayed or square wings is one on which no definite instruction can be given, the matter rather is one of opinion, it being recognized that the splayed wings are slightly to be preferred but are generally more expensive. The reasons for and against the splay of the down-stream wing returns are similar to those about the up-stream wings, except that the splay does not increase the length of percolation, as the point of exit of percolation for both splayed and square wing returns is at the angle *b* (figure 91).

The advantages of splayed wing returns on the down-stream side are small and many engineers generally employ square returns. It is best to decide the question by drawing earth slopes,

and if there is plenty of room for these to provide the cheaper, that is square returns.

474. Example of drop with solid apron and core wall bank connexions.—Another example figure 92 is now given of a drop of a distributary of discharge 25 cusecs.

Bed width above $4\frac{1}{2}$ feet.
„ below $5\frac{1}{2}$ feet.
Depth above 2·75 feet.
„ below 2·5 feet.
Bed slope above $\frac{1}{2,800}$

Drop in F.S.L. of water surfaces 3 feet = h.

There will be only a single notch in this case.

The depth of water cushion $X = \frac{1}{2} d \sqrt{h - d_1}$

$= 1{\cdot}375 \sqrt{3} - 2{\cdot}5.$
$= (2{\cdot}38 - 2{\cdot}5).$
$= -0\ 12$ foot.

There will therefore be a solid apron.
Width of apron = $5\frac{1}{2}$ feet.
Length of apron = $2d + 2\sqrt{d\,h} = 11{\cdot}24$ say 11 feet.
Length of revetment up-stream 10 feet.
„ „ down-stream 4 $(h + d) = 23$ feet.
Bed pitching to be half of the above, viz., $11\frac{1}{2}$ feet.

The normal free board of such a channel is $1\frac{1}{2}$ feet but is raised just above the fall to 2 feet. Top width of bank is 4 feet.

The bed of the channel just above the drop will be about $\frac{3}{4}$ foot below natural ground level.

Notch calculations will be made for—
Full supply, $d_1 = 2{\cdot}75$, $D_1 = 25$ cusecs.
and for $\frac{1}{2}$ depth, $d_2 = 1{\cdot}40$, $D_2 = 8$ cusecs.
Co-efficient $C = 0\ 70$

Then $3{\cdot}745 \times (2{\cdot}75)^{\frac{3}{2}} (l + 1{\cdot}1\ n) = 25$

$3{\cdot}745 \times (1{\cdot}4)^{\frac{3}{2}} (l + 0{\cdot}56\ n) = 8$

Reducing, $l + 1\ 1\ n = 1{\cdot}46$

and $l + 0{\cdot}56\ n = 1{\cdot}29.$

$$\cdot 54\ n = 0{\cdot}17 \text{ or } n = \frac{17}{54}$$

$$\therefore \tan \alpha = \frac{17}{108} = \frac{1}{6} \text{ nearly.}$$

Make the side slopes $\frac{1}{6}$
then $n = {\cdot}33$
$l = 1{\cdot}46 - {\cdot}33 \times 1\ 1 = 1{\cdot}1$
Make $l = 1$ foot 1 inch.
Top width of notch at level $2\frac{3}{4}$ feet above the sill

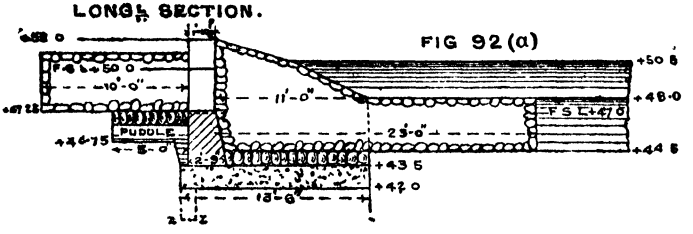
$$= \left(1\tfrac{1}{12} + \tfrac{11}{12}\right) = 2 \text{ feet.}$$

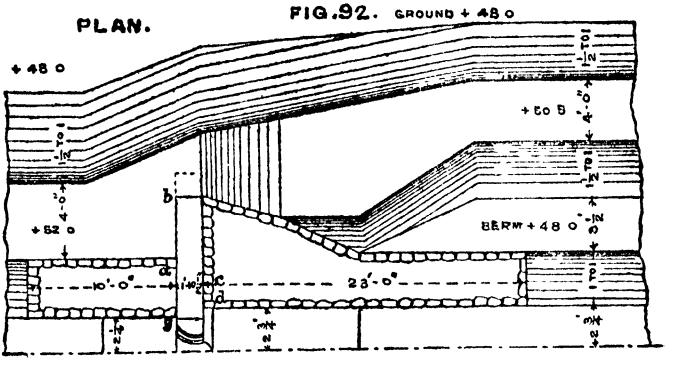
The length of the drop wall will be $4\frac{1}{4}$ feet, 2 feet for the top of the notch and $1' 1\frac{1}{2}''$ on either side. The length of the notch piers may be $1' 10\frac{1}{2}''$.

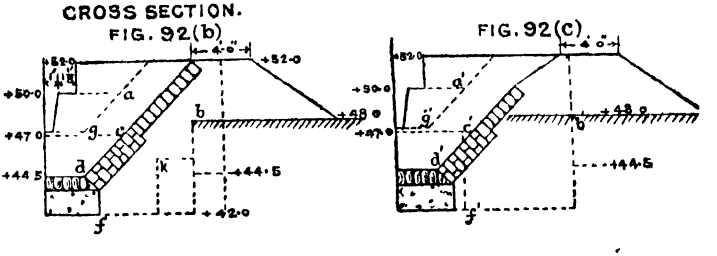
From below the sill level the wall is given a down-stream batter to give stability to resist the combined down-stream thrust of earth

and water. It is, however, not necessary to carry this batter throughout the length of the wall as the outer ends of the walls are subjected to little or no thrust.

The minimum legth of the wall is fixed by the top of the down-stream revetted slope. The plan and longitudinal and cross sections (figures 92, 92-*a* and *b*) show the general arrangements.







The design must now be tested for uplift and percolation.

(*a*) As regards length and uplift of apron:—

The minimum line of percolation under the apron scaled from the section, figure 92-*a*, is 21 feet, giving a gradient of $\frac{3}{21}$ or $\frac{1}{7}$. Net uplift head on apron at the toe of the drop wall is $3\left(1-\frac{9}{21}\right)=1\frac{5}{7}$ feet. The effective resisting weight of the apron (sp. gr. 2) is $2\frac{1}{2}$ feet which is ample.

(*b*) As regards the percolation gradient through the soil pas the core wall:—

(1) Round the end of the wall—

The length *a b c* (figure 92-*b*) *plus* the thickness (1′ 10½″) of the end of the wall is the length of the line of percolation and is just over 12 feet, giving a gradient of 1 in 4.

(2) Under the foundation of the wall—

The line of percolation runs from *g* on the upper side to *d* on the lower side (figures 92 and 92-*b*) This distance scaled from the section is 8 feet *plus* the width of foundation (4¼ feet) and is also just over 12 feet, giving a gradient of 1 in 4.

If the soil is of an impervious nature this gradient would suffice; if this is accepted it will be practicable to step up the foundation of the ends of the core wall, the limit of such stepping being that which would give a percolation gradient of 1 in 4 under the step. The width of the foundation under the stepping being 3¾ feet, the length under the foundation stepped at *k* [figure 92-*b*] is $(a\,k + k\,c + 3\frac{3}{4}) = 12\frac{1}{4}$ feet, and it would thus be admissible to step up the foundation to this extent from *k* to the end of the wall, a length of 2½ feet. This is too short a stepping to make such a course desirable in practice, and the foundation of the wall would not be stepped up

This matter is treated in detail to show to what extent the stepping-up of a core wall bank connexion is permissible having in view the percolation gradient

475 Modifications necessary to get a percolation gradient of 1 in 5 —While a percolation gradient of 1 in 4 may be allowed for small works in retentive soil it is generally desirable to design for a gradient to 1 in 5, and the modifications of the design necessary to secure such a gradient will now be detailed.

Obviously the wall must be lengthened to give a gradient of 1 in 5 round the end This prolongation is shown in dotted lines on the plan (figure 92)

To get a similar gradient underneath the foundation a stop wall might be made as shown dotted at *ss* (figure 92-*a*). In this case a wall 2 feet deep carried for 3 feet under the toe of the slope on each side would suffice. An impervious apron up-stream of the drop wall about 5 feet wide similarly carried under the slopes of the channel would also serve.

Such an apron is shown in figure 92-*a*

Figure 92-*c* is a revised cross section to suit a percolation gradient of 1 in 5, showing a prolonged wall; the flatter slopes thereby rendered practicable, allow of revetment of the upper part of the slopes being omitted.

As before, the length of permissible stepping-up of the foundations of the ends of the wall is insufficient to make stepping desirable.

Where a core wall is employed with a high drop the difficulty of securing a suitable percolation gradient is greater than with a low one. In figures 91 and 91-*b* chain dotted lines shown the position of the end of a core wall bank connexion applied to such a case, and dotted lines in the latter figure also show the sections of channel up-stream and down-stream under such conditions. If the width of the concrete foundation of the wall is 8 feet the length of the line of percolation passing under the wall from *d* to *s* would be

21 feet; with a 6-feet head a length of 30 feet is required and this would have to be secured either by—

(1) an up-stream impervious apron of length about 18 feet, or
(2) a stop wall about 6 feet deep below the foundation level, or
(3) widening the main apron below the drop wall by carrying it under the side slopes

The alternative of wing bank connexions is a more satisfactory method of treating such a case

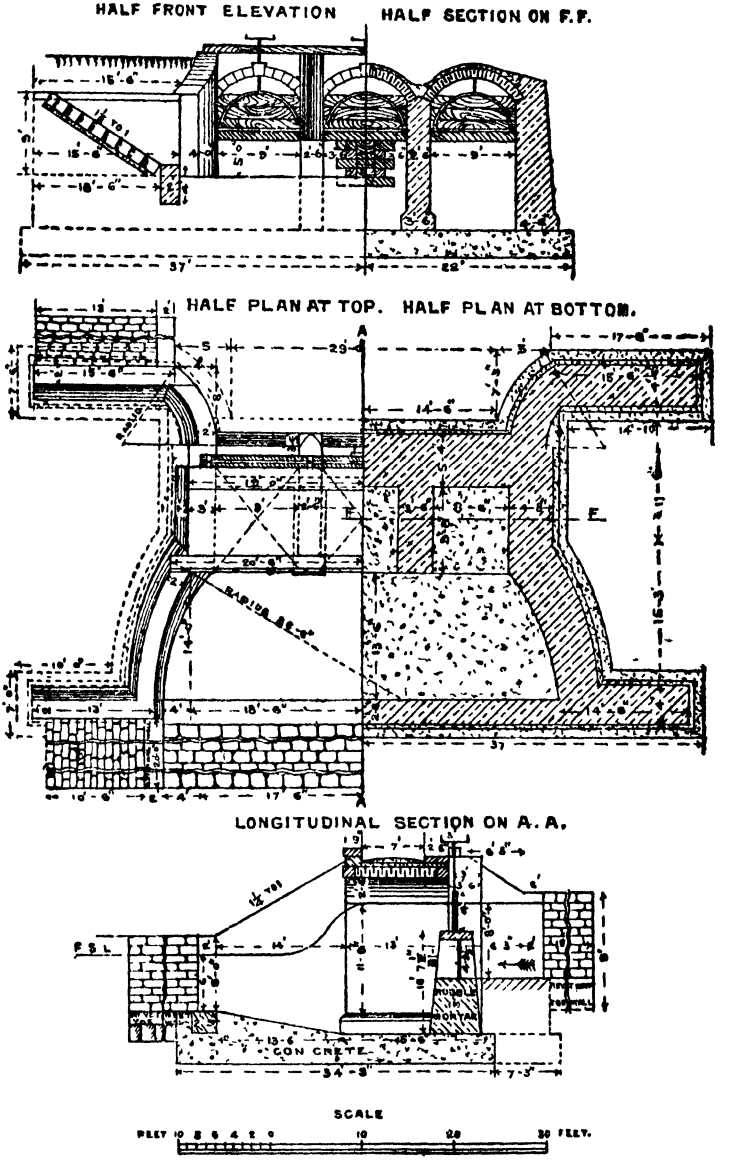
476. Types of bank connexions contrasted.—The advantage of the core wall type of bank connexion is cheapness of construction, which is specially accentuated in the case where revetment is cheap.

The wing type contains the banks in a more solid style, gives a better guide to the water over the work and requires fewer repairs. Generally the wing type is preferable for high falls, or low falls of considerable bed width, in which cases the cost of the bank connexions is small in comparison to that of the drop wall and apron. High revetted slopes below a work are a decided disadvantage as such slopes are apt to give trouble, but low revetted slopes require but little more repair than retaining walls. Generally it appears desirable to design wing connexions for falls exceeding 3 to 4 feet height or exceeding 100 cusecs discharge.

Full reliance cannot be placed on puddle to form a water-tight apron in channels which run much above subsoil water level and which are closed for long periods, as the puddle will crack if it becomes dry. The condition is quite different from that of puddle laid in a river bed at about lowest water level.

477. Designs of existing drops.—Figure 93 shows Lolla weir in the Gōdāvari Central Delta. This is a combined bridge, regulator, and drop with water cushion, in which the crest wall is raised above the bed of the canal. The flow levels are regulated by working the shutters; the deep vent in the central span is used for increasing discharge when the canal water is so low that

FIG. 93. LOLLA LOCK WEIR.



necessary proportionate supply cannot be passed down with the weir shutters raised clear; it is also required for draining the canal at the time of the annual closure

It frequently happens that a drop is located in the vicinity of a road crossing and that economy as well as greater facility of inspection is secured by constructing a combined bridge and drop.

Figure 94 shows a design for a combined road bridge and 3 feet drop with a solid floor for a four-cusec distributary.

FIG. 94 3 FEET DROP AND BRIDGE.
TO DISCHARGE 4 05 CUSECS

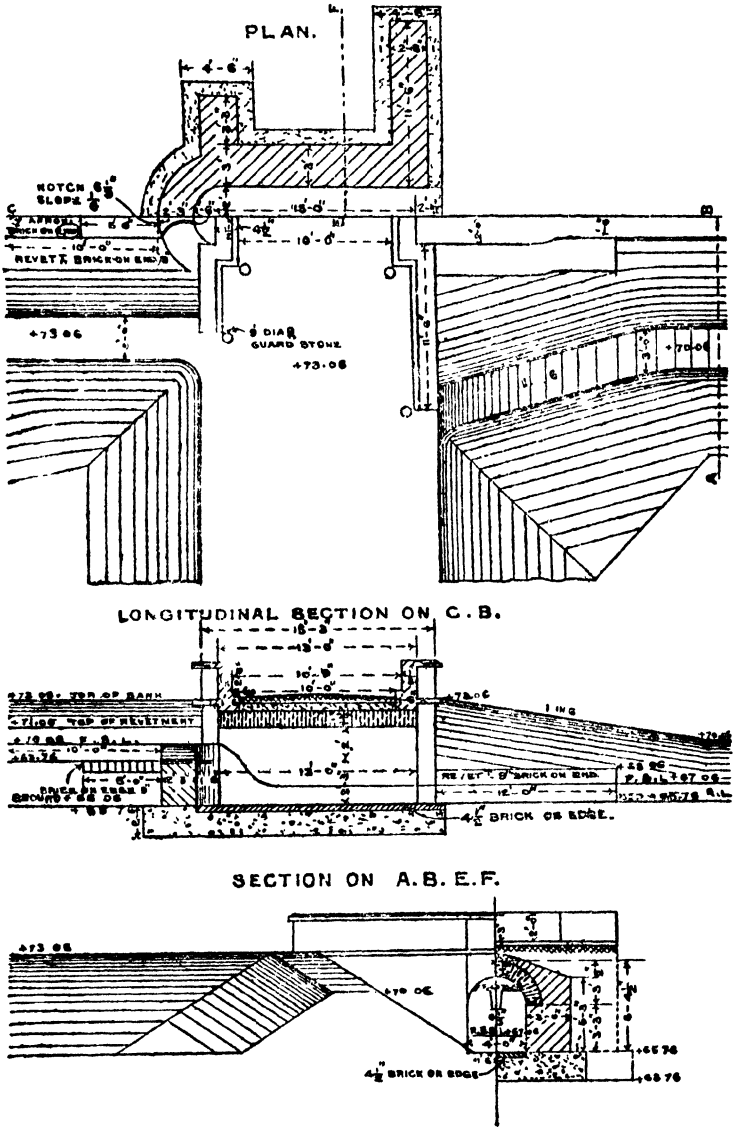


Figure 106, page 280 below, shows a distributary head sluice and drop combined.

478. Syphon well-drops.—In small channels with deep drops the coast of bank connexions is great in comparison to the cost of the floor and bed pitching. With a view to meeting this difficulty a type of work termed a 'syphon well drop' was designed in connexion with the Madras Cauvery Project for use in minor distributaries for drops of 6 feet and upwards, or for drops combined with bridges of 4 feet and upwards.

Figure 95 shows a design for a 4-feet syphon drop and bridge combined for a two-cusec distributary, and figure 96 a similar work for a capacity of ten cusecs.

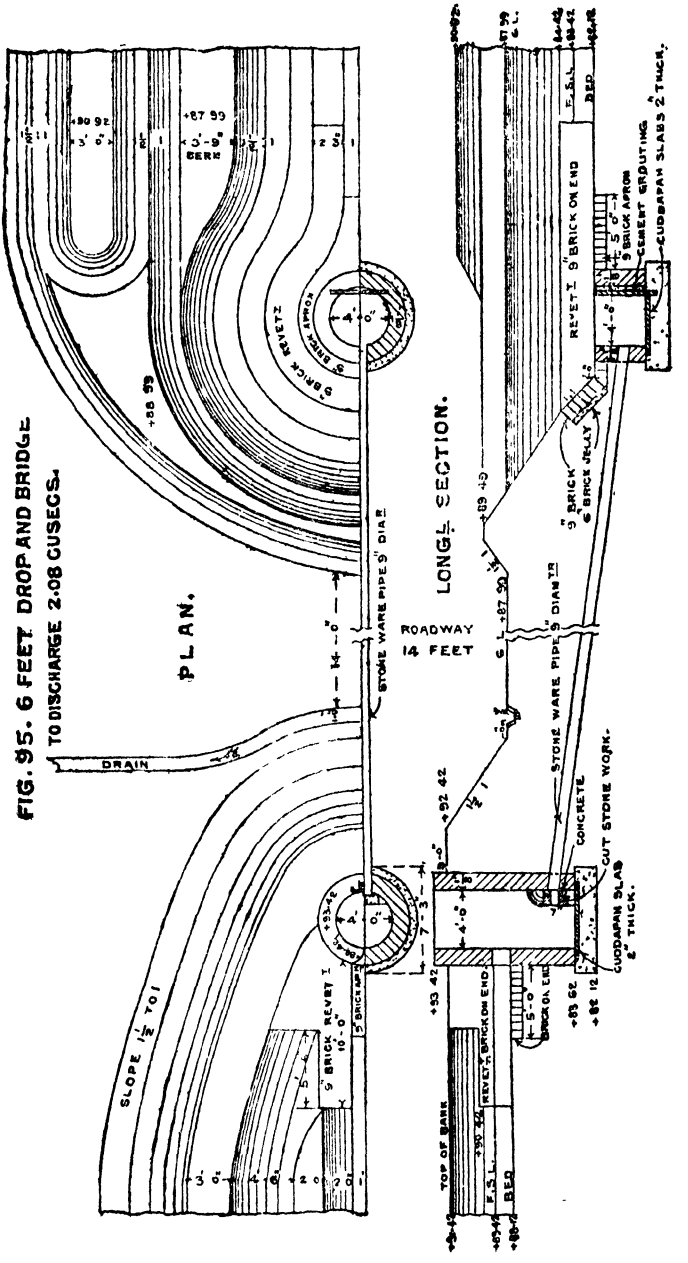
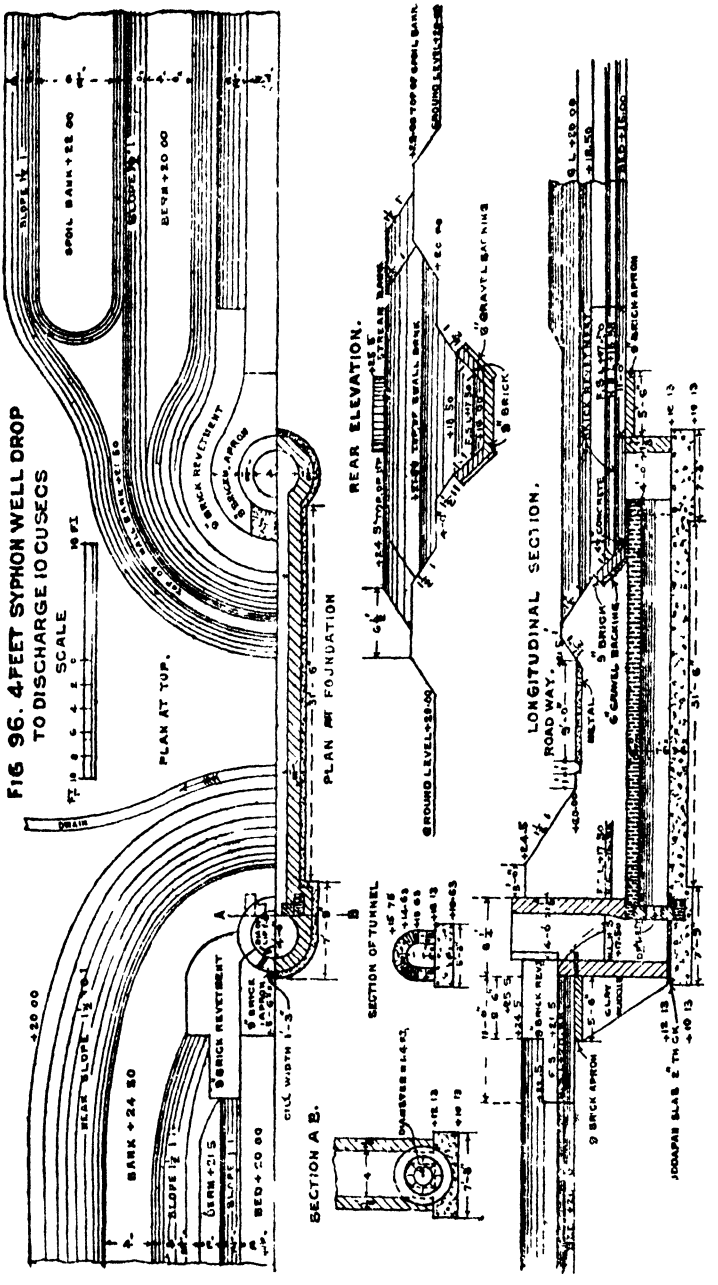


FIG. 95. 6 FEET DROP AND BRIDGE TO DISCHARGE 2.08 CUSECS.

As the occasions for building high drops are of infrequent occurrence but few works of this type have been constructed and although the type promises well, its success cannot, without further experience, be considered as established.



CANAL REGULATORS AND DIVIDING DAMS.

479. Canal regulators.—A canal or channel regulator consists of a floor at bed level, with piers at intervals across the canal provided with grooves in which are installed movable shutters or planks, by means of which the level of water above the work can be regulated and the total discharge distributed in varying proportions to the canal below the work and any canals or channels off-taking immediately above it Regulators are generally built just below the off-take of a branch canal, distributary, or surplus sluice.

Canal regulators of large size are somewhat similar to river regulators already described and, so far as regards resistance to percolation and uplift the designs should be on similar lines As the velocities and water levels in a canal are under control the protective aprons and revetments necessary for river regulators may be much reduced and would be similar to the aprons of head sluices.

Small channel regulators frequently consist of a wall with one or more rectangular notches regulated by planks inserted in grooves in the sides of the notches. Such regulators are in Madras frequently referred to as 'regulating dams'

480 Regulating notches.—Where changes in depth of channels are made without any drop in the bed level trapezoidal notches built on a floor across the channel are frequently constructed to regulate supply Such notches are not fitted with planks or shutters but automatically regulate the discharge above and below the work. They are termed 'regulating notches' or shortly 'notches'.

481. Designs of regulators—Canal regulators are very frequently combined with bridges or with drops Figure 93 above shows a canal regulator combined with a bridge and a drop.

Figure 97 shows a canal regulator of the ordinary type used on large canals in Madras In this case a road bridge is combined with the regulator.

FIG. 97. CANAL REGULATOR.

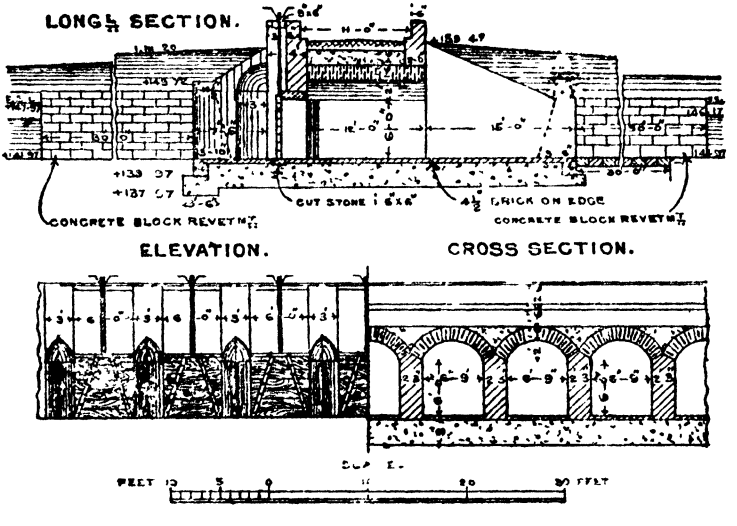


Figure 98 shows a regulating notch.

FIG. 98. REGULATING NOTCH.

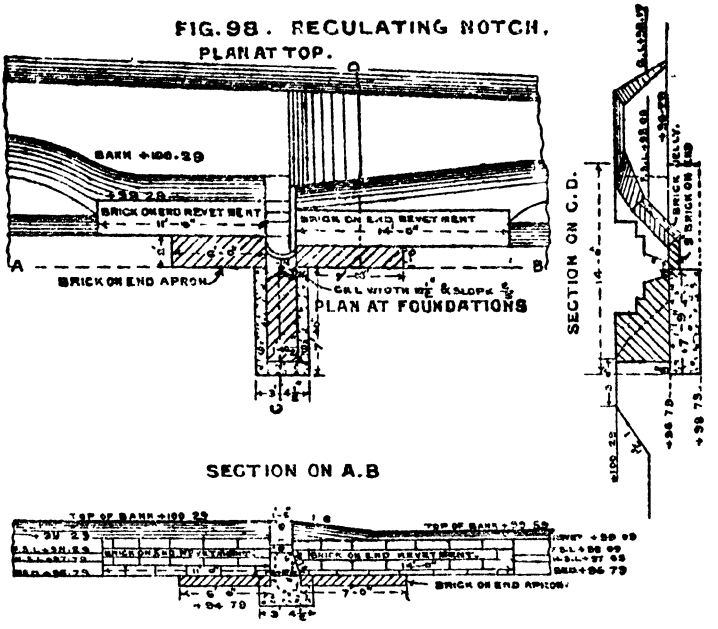
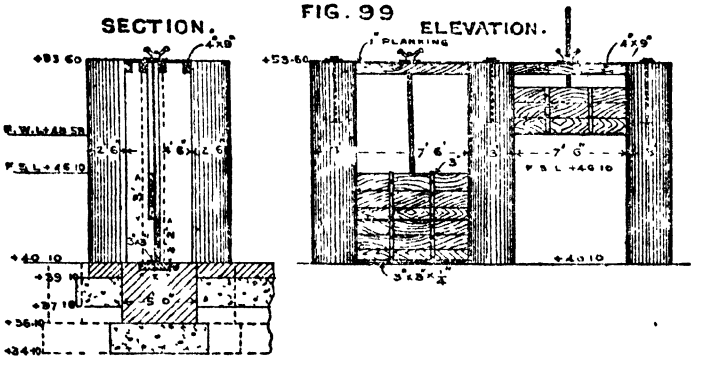


Figure 99 shows the section and elevation of a regulator of the Cauvery delta with two tiers of shutters working in separate grooves but worked only by a single screw, the lower shutter as it rises picking up the upper one.

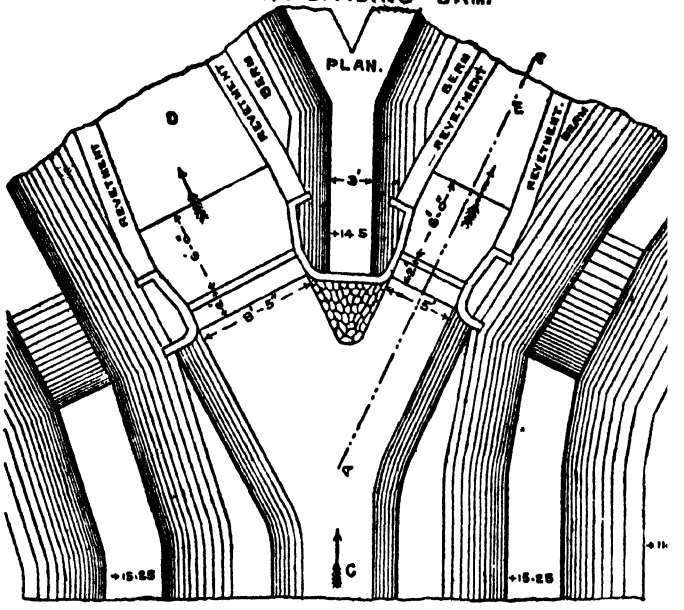
FIG. 99

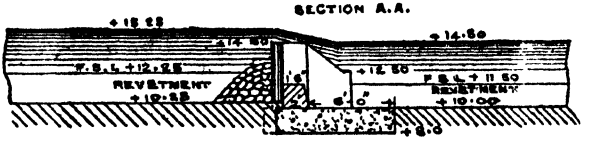


The object of this is to save headway and diminish the resistance to motion of the shutters, and not, as in the case of head regulators, to draw surface water. In the case of this work there is a considerable difference of level between F.S.L. and M.W.L. in the channel and it is necessary that the shutters be capable of being raised clear of M.W L.

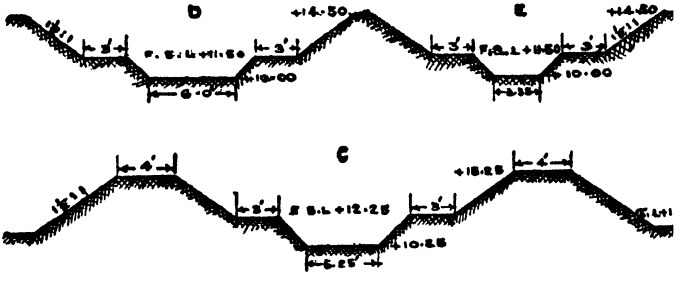
482. Dividing dams.—When a channel divides up into two or more parts a dividing dam is frequently built across the channel at the point of bifurcation. Such works are of the nature of regulating notches, and are designed to automatically subdivide the whole flow of the channel in the required proportion.

FIG. 160. DIVIDING DAM.





SECTIONS OF CHANNELS.



HYDRAULIC DETAILS

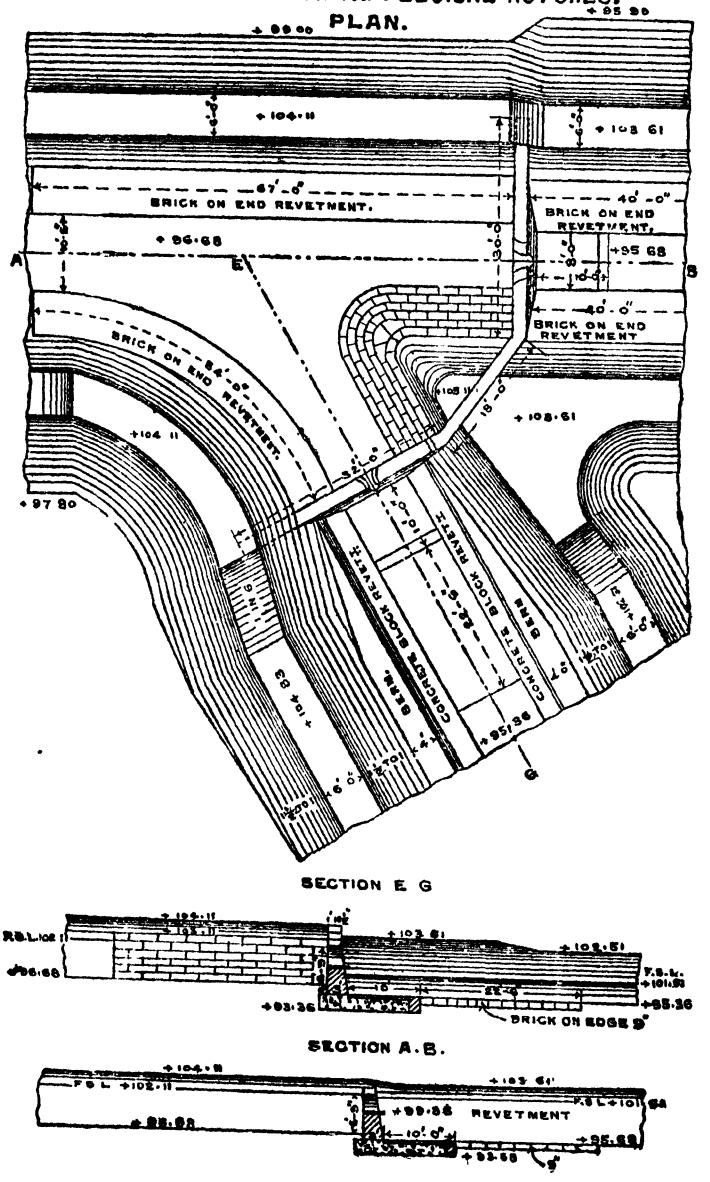
C CHANNEL	924 ACRES	15.44 C.FT.		WEIR CREST	D CH.	8.5 FT.
D DO.	586	DO.	9.83 C.FT.		DO.	E CH.	5.0 FT.
E DO.	336	DO.	5.61 C.FT.	DEPTH ON CREST		7.5 FT.
FALL 1.50 FT. PER MILE				C = .606			

When there are no offtake sluices immediately above the bifurcation the best forms for dividing dams where the necessary heads are available are rectangular notches (weirs) with clear overfalls.

Figure 100 shows a design of such a work.

FIG. 101.

DIVIDING DAM WITH TRAPEZOIDAL NOTCHES.



When the overfall is clear the discharge of each notch is in proportion to its crest length. This simple method of division is easily understood by ryots and is thus to be recommended. Where one or both of the notches is submerged this simple proportion is destroyed, a point which has in many cases been overlooked. In such a case, or where there are offtake sluices immediately above the dividing dam, the division should be made by trapezoidal, in place of rectangular notches and these should be calculated for the actual conditions of submergence of each notch.

Figure 101 shows a dividing dam of this type.

Alteration by deepening or widening the channel below any *submerged* notch will increase its discharge relative to the adjacent notch.

For this reason when a submerged dividing dam is made it is desirable (unless this already exists) to construct a grade or section wall across the channel 3 or 4 furlongs below each submerged dam so as to minimize any alteration in proportionate flow caused by deepening the channel.

IRRIGATION SLUICES.

483. Irrigation sluices.—Irrigation sluices form the class of masonry works most numerous in a distribution system. These sluices differ from the head sluices which control the passage of water from a river in the fact that it is unnecessary to make special provision for drawing surface water with a view to the exclusion of bed silt and there is thus no object in arranging the regulating shutters of distribution sluices in tiers unless this is convenient on account of the limited headway available for lifting the shutters. So far from it being objectionable to draw bed silt through the sluices it is desirable that each sluice should pass its fair share of the bed silt which enters the canal in which the work is situated and for this reason the sills of irrigation sluices should generally be built at the level of the bed of the parent channel

A sluice provides for carrying the water of the parent channel through one or more culverts or pipes under one of the canal banks into a branch canal, distributary or field channel (watercourse). At each end of the culvert suitable wings are provided to retain the rth slopes and so form effective bank connexions and also to serve as stop walls to percolation. At the upper end of each culvert is a 'head wall' fitted with a shutter, moving in grooves built in the masonry, by means of which each vent can be closed or opened or the amount of opening regulated.

The considerations as regards percolation and uplift which operate in designing the floors, aprons and wings of head sluices, drops, etc., are applicable to design of all kinds of irrigation distribution sluices.

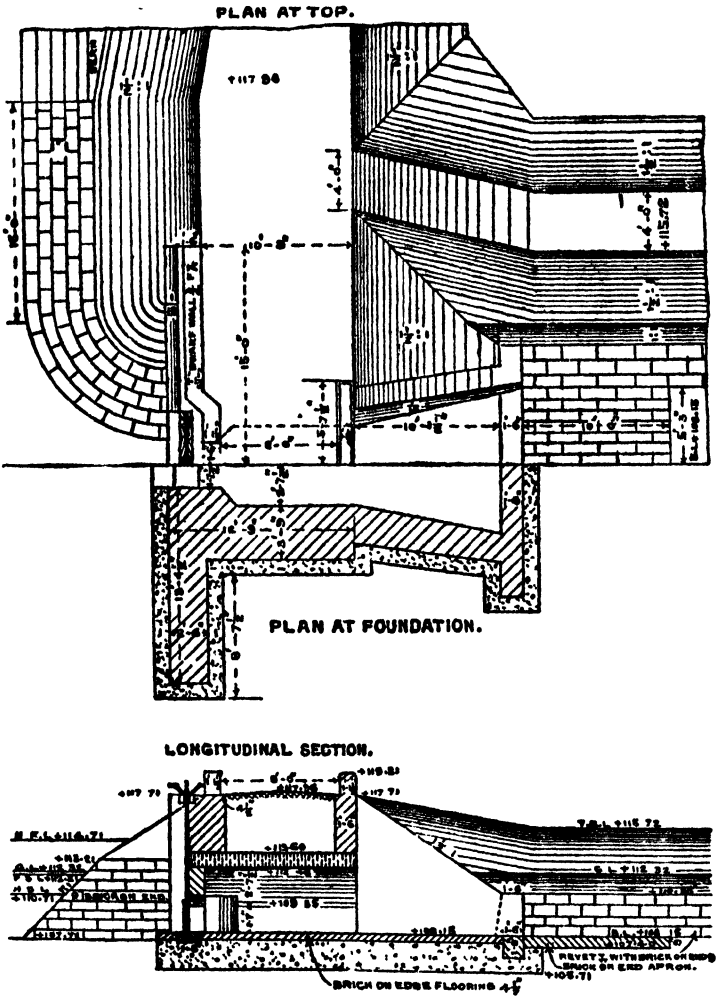
The lowest water level in the parent canal from which full supply should be capable of being passed into the distributary varies with the circumstances and methods of supply and regulation of each system and this must be known before the size of sluice vents can be fixed.

The size of sluice vents must be sufficient when fully open to pass the water required for the full supply of the channel of which it forms the head under the available head or difference of level

between the water surface in the parent canal and the level of the distribution channel when at F.S.L.

Generally the pipe outlet sluices which supply field channels are designed so that full supply can only be passed into the field channel when there is full supply in the parent channel. The sluices at the head of the larger distribution channels are if practicable designed to be capable of passing full supply into the distributary with three-quarters to half supply in the parent canal.

FIG. 102. CHANNEL HEAD SLUICE.

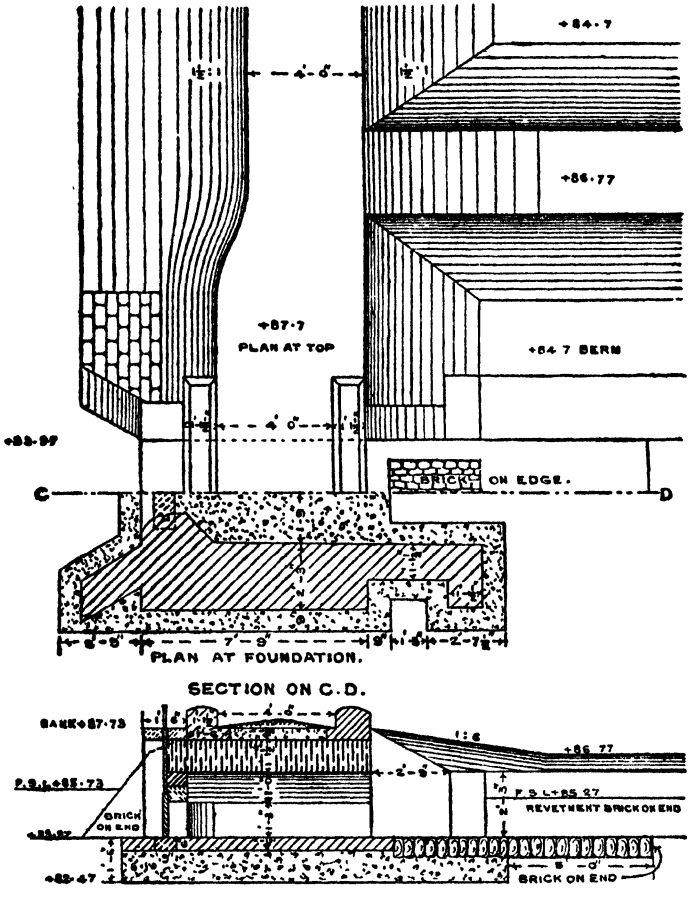


The width of the sluice culverts should be sufficient to pass th full supply at a velocity not exceeding 5 feet per second. Excep where the head is small the width of the culvert may witl advantage be greater than the width of the vent.

In pipe outlets the sectional area of the pipes is generally the same as that of the vents.

In no case should a masonry culvert be built of smaller section than 2 feet wide and 2½ feet high.

FIG. 103. CHANNEL HEAD SLUICE.



484. Shutters of canal sluices.—The heads on distribution sluices being comparatively small and not subject to very great variation the plug type of shutter, so convenient for regulating tank sluices, is not used and flat shutters are universally employed. These are raised and lowered by a stem, frequently referred to as the shutter 'spear,' which is attached to the centre of the shutter and is long enough to project a little above the regulating platform of the sluice when the shutter is resting on the sill. In the Madras Presidency the spear is usually an iron bar and the shutter is raised and lowered by screw gear. The width of shutter worked by a single screw spear is limited to 10 feet and 6 to 8 feet spans are more suitable for this type of gear.

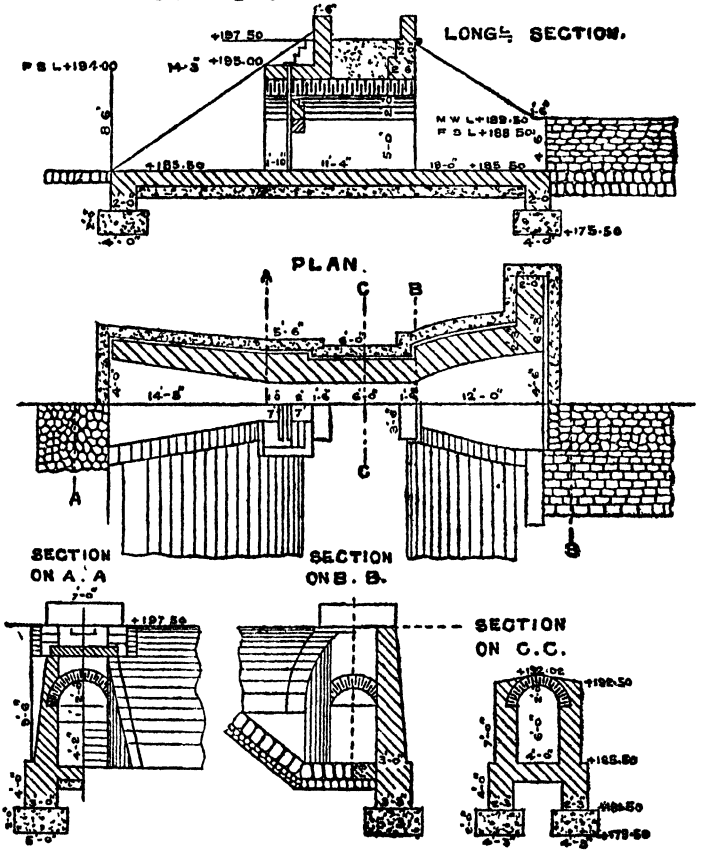
For small sluices or for sluices in places where regulating establishment is not maintained wood spears without gear may be fitted to shutters, which in such cases should generally not exceed 3 to 4 feet span. It is unusual to provide any shutters for the pipe outlet sluices supplying field channels where the size of pipe does not exceed 6 inches diameter. Regulation of pipe outlet sluices is generally effected by raising or lowering the water levels in the parent channel but individual sluices may easily be closed by an improvised plug or a clod of earth. Sluice shutter and gear are dealt with in further detail in Chapter XV below.

485. Types of irrigation sluices.—Figures 102 to 106 give designs of distribution sluices of various types.

Figures 102 and 103 show face walls without and with splayed wings. Where the extended face wall is not required as a stop wall against percolation the latter type is generally more economical.

FIG. 104.

HEAD SLUICE OF DISTRIBUTARY.



gures 104 and 105 are drawings of existing sluices in the Gōdāvari Western Delta.

FIG. 105.

HEAD SLUICE FOR A SIDE CHANNEL.

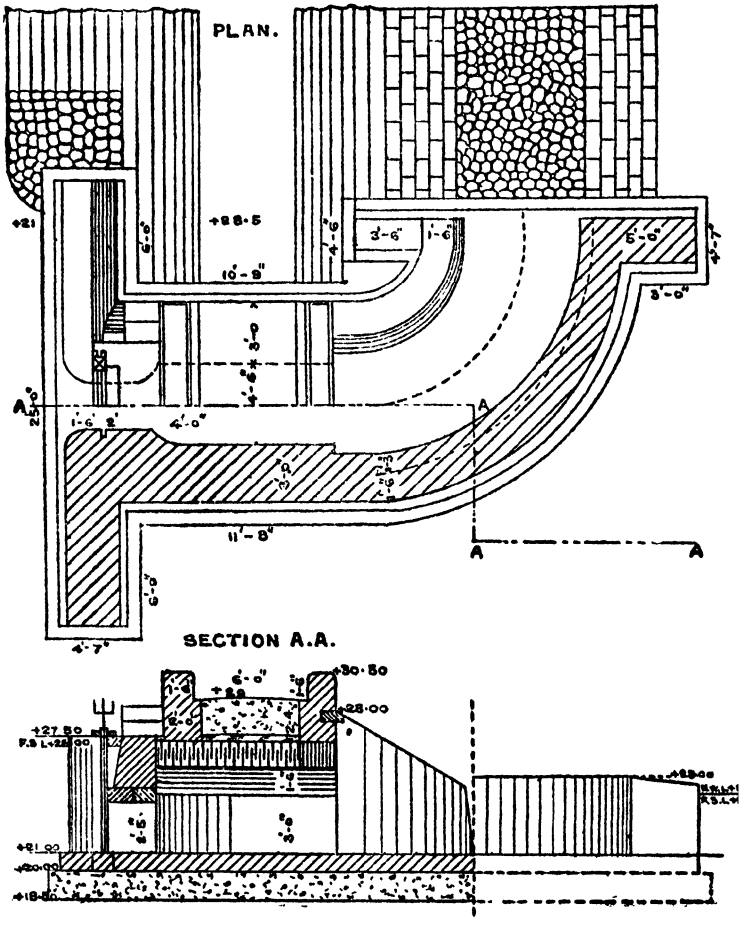
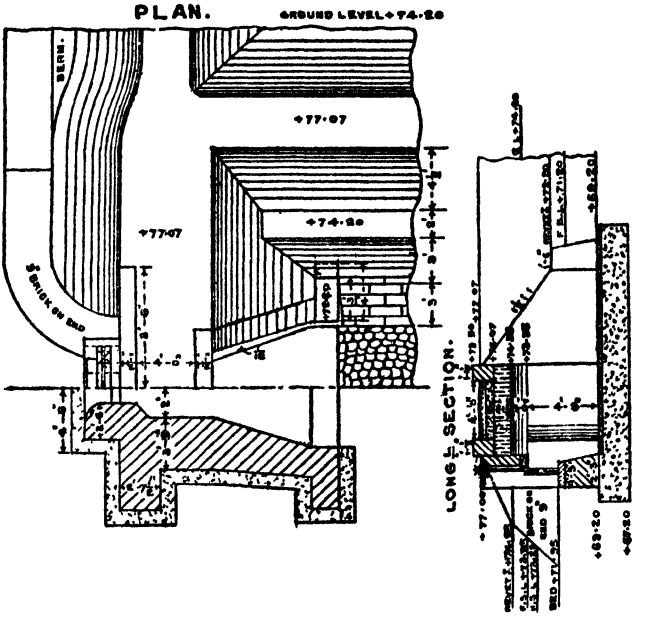


Figure 106 is an irrigation sluice combined with a drop of 2¾ feet.

FIG. 106. CHANNEL HEAD SLUICE AND DROP COMBINED.
FULL SUPPLY 17 CUSECS.



486. Pipe outlets.—The barrels of the small sluices passing supplies from distributary channels to field channels are generally made of cement or glazed earthenware pipes of suitable sizes, with masonry head and tail walls of sufficient length to retain the earth slopes. These small sluices are termed 'pipe outlets' or frequently merely 'pipes,' they vary in size from 4″ to 12″ diameter in accordance with the areas supplied. The pipes are frequently laid on a light concrete foundation to prevent uneven settlement and consequent leakage. If the soil is good the pipes may be laid through the bank without any special foundation, and on being first placed they are generally so laid, head and tail walls, and foundation if required being subsequently built.

Figures 107 and 108 are pipe outlets with head and tail walls. The upper face of the head wall should be placed at a distance equal to half the full supply depth from the intersection of the channel bed and side slope, and the top of the head wall should be at F.S.L. Two types of head wall are shown, viz., with and without wings; in the first type the length of the wall is $(\frac{3}{4} + 2d)$ feet and in the second a clear space of 1¾ feet between the inner faces of the wings is allowed. The tops of the wings are stepped to suit a canal side slope of ½ to 1, the steppings facilitate examination or regulation of flow; on the whole the latter type of head wall although slightly more expensive is considered the better.

The tail walls of pipe outlets should be built up to F.S.L. in the field channel and the length should conform to the cross section

FIG.107. 4" PIPE OUTLET.

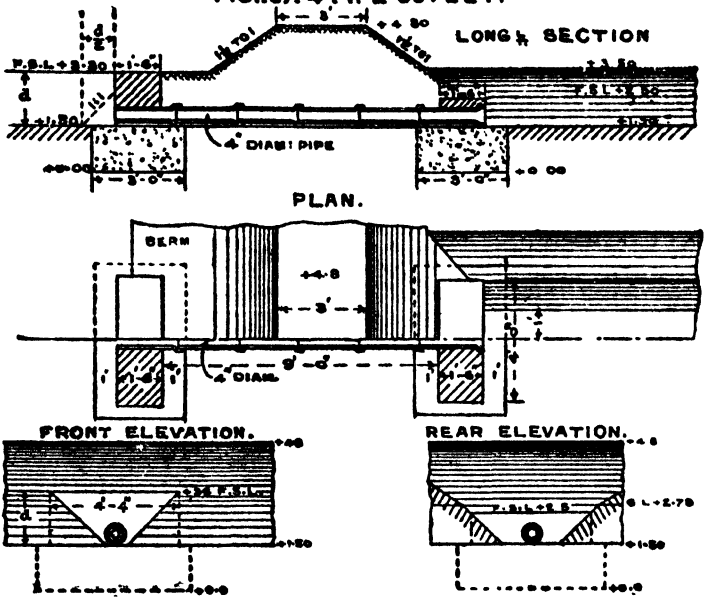
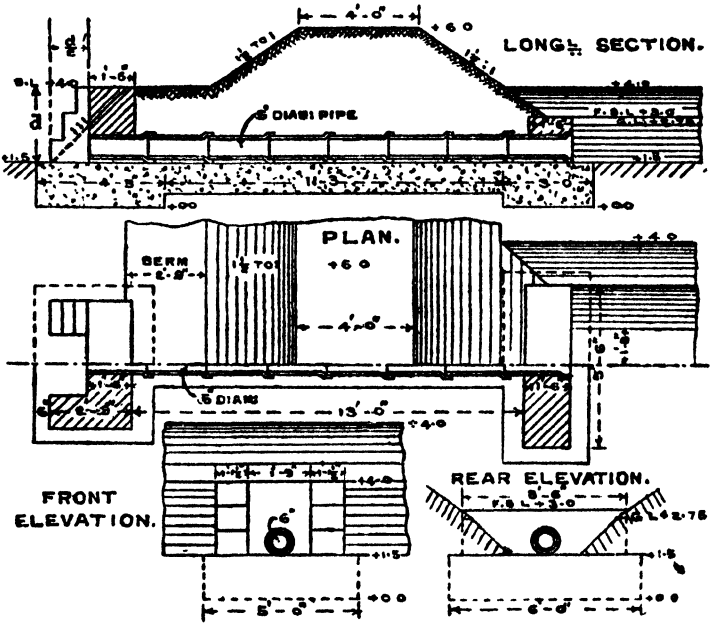


FIG.108. 6" PIPE OUTLET.



dimensions of that channel. In one of the types a concrete foundation for the pipes one foot thick is shown, but this is frequently unnecessary, and entails considerable additional cost.

Although each individual outlet is a very small work it has to be built in such numbers that very careful consideration of the details of design is desirable so as to secure efficiency with the least possible cost.

487. Fixing size of pipes in outlets —The sizes of the pipes of outlets are proportioned to suit the area to be irrigated under the conditions of maximum supply and the head available under these conditions.

The water level in the field channel at the outfall of a pipe is likely to vary considerably for the same discharge owing to alterations in the bed levels caused by silt clearance and by the cross bunding of these channels to suit the different internal turns under which they distribute their supply.

For the above reason the 'head available' under conditions of full supply at pipe outlets constantly varies, and some reasonable assumption as regards head has to be made in computing the size of a pipe. The usual assumption is, that the water level in rear of the pipe is ¼ foot above the mean level of the highest field irrigated by the sluice; where the ground to be irrigated is at a considerable distance from the outlet it is necessary to make a further allowance for surface fall in the field channel. Where a small fraction of the whole area is at a high level and the rest at a low one, the average level of the area irrigated by the pipe should be taken in place of the highest.

The smallest size of pipe outlet usually used in Madras is 4″ diameter. On account of this limitation many of the plots irrigated by 4″ pipes necessarily receive disproportionately large quantities of water, and in laying out a new distribution system every effort should be made to group the fields together so as to form areas requiring pipes of 6″ diameter and upwards. Multiplicity of outlets lead to extra expense, extra difficulties in distribution, and to waste of water, and the smaller the proportion of the whole area of a system irrigated through pipe outlets of the minimum size the better.

488. Calculation of discharge of pipe outlets.—Pipe outlets are short pipes with cylindrical entry.

Let h = the whole head required to force water through the pipe at a velocity v.

„ h_e = the head required to produce the velocity (v) in the pipe and to overcome the resistance at entry.

„ h_r = the head expended in overcoming the fluid friction in the length (L) of pipe of diameter d.

„ μ = the coefficient of fluid friction of the pipe

then $h = h_e + h_r$

and $h_e = 1{\cdot}5 \times \frac{v^2}{2g}$, $\quad h_r = \frac{4\mu L}{d} \times \frac{v^2}{2g}$

(Vide Love's Hydraulics, Chapter VI.)

Let l_1 be the length of pipe which gives a frictional resistance equal to h,

$$\text{then } \frac{4\mu l_1}{d} = 1{\cdot}5 \text{ and } l_1 = \frac{1{\cdot}5d}{4\mu}$$

$$\text{and } h = \frac{4\mu}{d}(L + l_1) \times \frac{v^2}{2g}$$

The value of μ is ·005 and ·01 for clean and slightly encrusted iron pipes. Pipe outlets of glazed earthenware pipes do not become encrusted but as they are in short lengths of 2 feet, the normal friction will be greater than a clean iron pipe but less than an encrusted one.

Taking $\mu = ·0075$ for earthenware pipe outlets

$$l_1 = \frac{1{\cdot}5\,d}{4 \times 0075} = 50\,d$$

$$\text{Thus } h = \frac{0075 \times 4 \times (L + 50\,d)}{d} \times \frac{v^2}{2g}$$

$$\text{From this } h = \frac{·03}{d} \times (L + 50\,d) \times \frac{v^2}{2g}$$

$$\text{and } v = \sqrt{\frac{d}{·03\,(L + 50\,d)}} \times \sqrt{2gh}$$

If d be expressed in inches instead of feet, this formula becomes

$$v = \sqrt{2gh}\ \sqrt{\frac{d}{·36\,(4\ 17\ d + L)}}$$

and the discharge D in cusecs through a pipe outlet of diameter d inches and length L and head h in feet becomes

$$\sqrt{h} \times \left[8{\cdot}025 \times \frac{\pi d^2}{4 \times 144} \sqrt{\frac{d}{·36\,(4\ 17\ d + L)}}\right]$$

$$D = \left[·0438\,d^2 \times \sqrt{\frac{d}{0\ 36\,(4\ 17\ d + L)}}\right] \times \sqrt{h} \quad . \quad . \quad . \quad (64)$$

489. Table of discharge of pipe outlets.—The following table is computed from equation (64) and gives, for pipes of various sizes and lengths, the discharges in cusecs for a head of 1 foot. The discharges for any other head can readily be computed by multiplying the figure in the table by $\sqrt{h}$

Length of pipe in feet.	Diameter in inches.				
	4	6	8	10	12
8	·470	1 12	2 05	3·27	4·78
12	436	1·06	1 96	3·15	4·62
16	·408	1 00	1·88	3·09	4·48
20	385	·958	1 81	2·91	4·35
24	·366	·918	1·74	2·85	4·23
30	·342	·867	1·66	2 72	4·07
40	·310	798	1·54	2·55	3·84
50	·286	·743	1·45	2 41	3·64
60	·267	·699	1·37	2 29	3·47

$\sqrt{h}$ for values of h from ·1 to 1 foot.

$h =$	·1	·2	·25	·3	·4	·5	·6	·7	·75	·8	·9	1·0
$\sqrt{h} =$	·316	·447	·453	·548	·652	·707	·775	·837	·849	·894	·949	1·0

490. Defects in distribution by pipe outlets.—If the discharge of all the pipe outlets of a distributary channel varied in exact accordance with the discharge of the channel the quantity of water available in the distributary would be delivered into heads of the field channel in proportion to the areas under each and distribution would attain the ideal aimed at. This however is far from being the case.

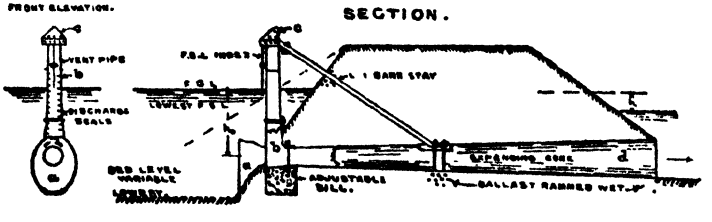
Even when a steady water level is maintained throughout the length of the distributary the supplies passed through the pipe outlets, except those with free outfalls, vary with the variations in the water levels in the field channels, and these levels are likely to constantly vary in accordance with whether any field channel is irrigating the higher or lower lands supplied by it, and also in accordance with the condition of its bed as regards silting.

If the outlet is of proper size an unduly large discharge cannot be taken when the higher fields are receiving supply, but by lowering the bed of the field channel excessive supply can be drawn when the lower fields are being irrigated, and on the whole only a rough approximation to the distribution aimed at can be realized. The inaccuracies are reduced in the case of pipes with free outfalls as in these the discharge is independent of the level of the water in the field channel.

491. The 'Kennedy gauge outlet.'—Several forms of outlet have been devised to give a discharge unaffected by the level of the tail water with a view to getting automatic discharge through outlets proportionate to the discharge in the parent channel. The most widely used device is the 'Kennedy gauge outlet' which is described below, and through which, as in the case of a pipe with a free outfall, the discharge varies as $\sqrt{h_0}$, where h_0 is the difference of level of the centre of the orifice and the water surface of the parent channel; it is independent of the water level in the tail channel.

Figure 109 shows the latest form of this outlet. It consists of a cast iron bell mouth (*a*) from which at its narrowest diameter a jet shoots across the air in the base of a vertical pipe (*b*) (termed the 'vent pipe'), which pipe reaches above the water level and is covered with a cowl (*c*) pierced with air holes. The base of the vent pipe is conical and allows a free circulation of air round the jet *so that the latter is in exactly the same condition as if it were discharging into open air.* After crossing the vent pipe the jet enters the small end of a long expanding cone (*a*) made of sheet steel. The water passes through the cone and forces its way into

FIG. 109. KENNEDY GAUGE OUTLET.



the channel against the back pressure of any water at the outfall. The discharge of the jet, being the same as that of free discharge into air, varies as the square root of h_0 the difference of level of the centre of the jet and the surface of the water immediately above the bell mouth—h_0 is referred to as 'the depression head'. It is found by experiment that the losses of head by friction, eddies, etc., are about 0·21 h_0 and that the available working head (h^1), that is difference of water level between the parent and offtake channel, must not be less than $\frac{1}{5}$th of h_0, the depression. To allow a working margin the rule limiting the application of Kennedy gauge outlets is that the *depression must not exceed four and a half times the working head.* So long as this condition is complied with, the discharge of the outlet varies as $\sqrt{h_0}$ whether the outfall end is submerged or not and it is independent of the level of water at the outfall.

492. Limitations on usefulness of Kennedy gauge outlets.—The Kennedy gauge outlet secures a supply varying as $\sqrt{h_0}$ while the supply in the parent channel does not vary at this rate. In spite of this, where the variations of level of channel supply is small in comparison to h_0 the results to be attained by the use of Kennedy's outlets are close enough to the proportionate distribution to suit practical requirements. These outlets are therefore very suitable for use in distributaries which are worked on a system providing that they shall be either run full or wholly closed. Such a system is that generally adopted in Northern India where the bulk of irrigation is for dry crops. In the Madras systems in which distribution is carried out by continuous flow, the Kennedy gauge outlet is by no means so effective as the differences in level of flow of the distributaries will vary as much as $\frac{1}{3}$rd the full supply depth of the channel. In a 3 feet channel the flow of the channel at 2 feet depth would be reduced to about one half, that is, by 50 per cent but the discharges of Kennedy outlets with the centres of the outlets fixed $\frac{1}{2}$ foot above bed level, would be in proportions of $\sqrt{2\frac{1}{2}}$ to $\sqrt{1\frac{1}{2}}$, that is, 1·41 to 1·22, a reduction of only 14 per cent; thus the upper reach outlets would in such a case pass an altogether disproportionate quantity of the whole supply. This outlet is thus not suited for application to channels worked by continuous flow, and what is required is an outlet the discharges of which are proportionate to the discharge of the parent channel and which are not affected by the water levels in the field channels. Such outlets have not yet been established anywhere, but a promising device aiming at this result is now under trial in Northern India.

COMMUNICATION WORKS.

493. Bridges.—Canal bridges are only distinguished from bridges over rivers and streams in the fact that flow to be passed, the section of the stream, and the velocity and levels of water are controlled and defined. Unless required for navigation purposes there is seldom any advantage in making large span canal bridges and the width of spans should be fixed by considerations of cost. It is generally undesirable to head up water at a canal bridge and the waterway through the bridge should therefore be approximately

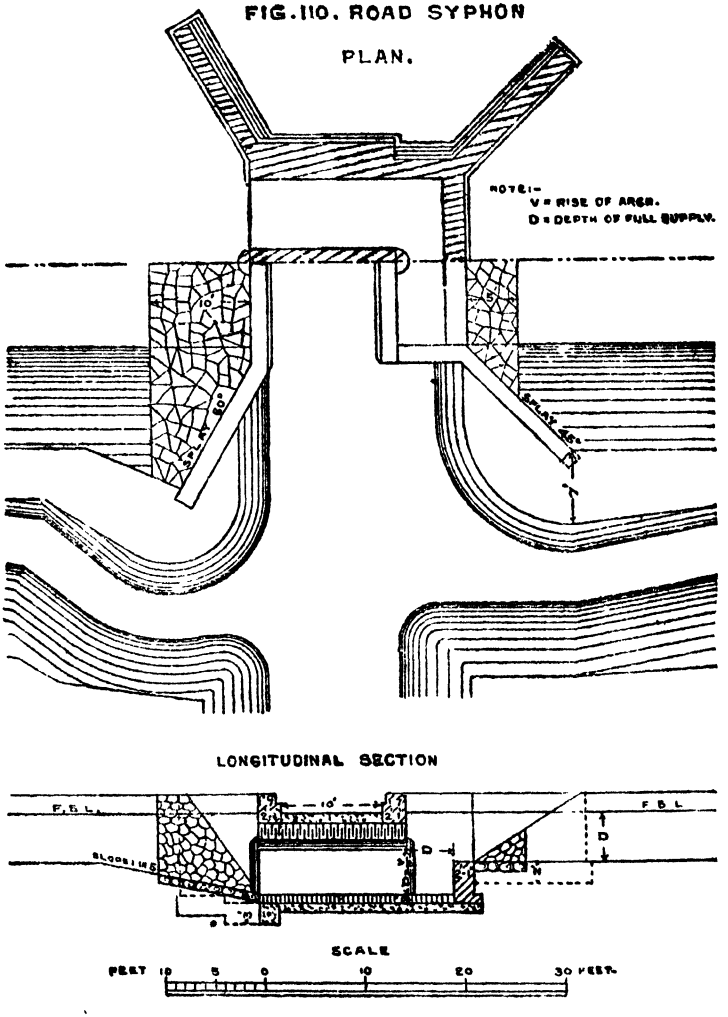
equal to that of the canal. Deep foundations are practically never required and where there is no contraction of waterway, flooring is unnecessary except in the immediate vicinity of piers and abutments round which bed pitching for a width of about 5 feet is desirable. In small span bridges this involves provision of a pitched floor throughout the span. Cutwaters should always be provided both up and down-stream of piers and half cutwaters at abutments. Semi-circular cutwaters are practically as efficient as the usual pointed shape and are in some cases easier to build and are better suited for carrying superstructure. The springing of arches will generally be not below F.S.L. of the canal. Masonry bridges with brick or concrete arches are generally most suitable for irrigation works being little more expensive in original cost and much cheaper in maintenance than bridges carried on steel, or even ferro-concrete beams. Spans six feet and under may be sometimes economically carried by ferro-concrete slabs.

Road and foot bridges are frequently combined with structures required for irrigation purposes such as weirs, regulators, falls, regulating notches, and sluices, and designs showing such combinations have been given when describing the various kinds of irrigation works mentioned (vide figures 93, 94, 97, 102 to 106).

494. Road syphons.—As the F.S.L. in irrigation channels is generally above the general level of the country traversed, the construction of arched bridges with springing at F.S L. frequently entails comparatively heavy approaches to a bridge which, while involving considerable expense, entail inconvenience to traffic; this can be avoided by syphoning the channel under the road way; such a structure is called a 'road syphon'

Road syphons have not hitherto been generally employed in Madras, but they are, it is understood, frequently constructed in Northern India, and their employment for bridging distributary channels in place of the ordinary type of bridge appears desirable in many cases.

Figure 110 shows a type design of a road syphon received from he Central Provinces.



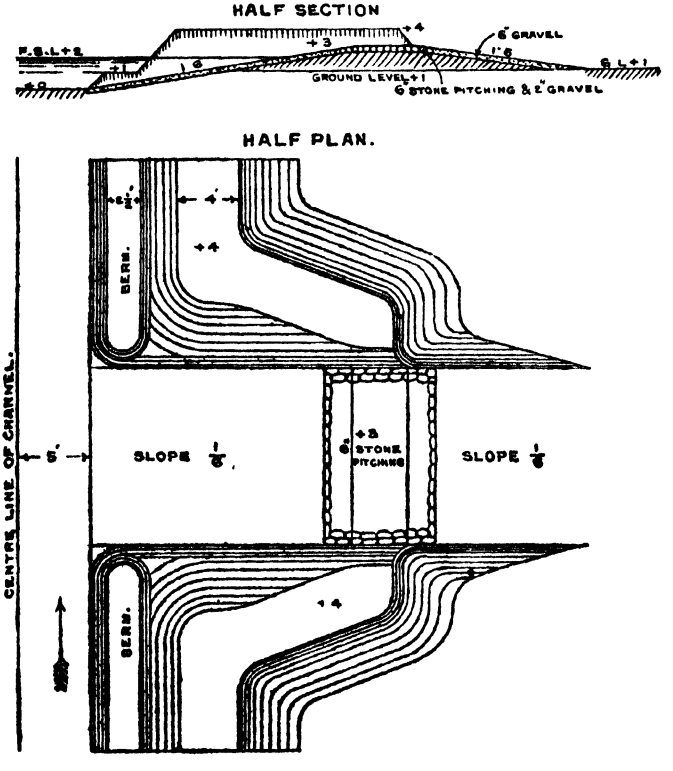
495. Cart and cattle crossings.—A ford suitable for wheeled or foot traffic may be constructed across channels where the full supply depth does not exceed 2½ feet; cattle crossings are not restricted in depth as cattle can cross a canal by swimming without inconvenience if suitable taking off and landing places are provided on each side of the canal. The approaches to fords and cattle crossings consist of ramps on each side of the canal, the ramps commence at bed level where the side slopes of the canal intersect the bed and are carried up at a uniform slope from this to above F.S.L. of the canal.

In the case of a cart crossing, the slope of the ramps should not be steeper than 1/12 and may, where convenient, be flatter. Crossings for cattle and foot traffic may have slopes from ¼ to ⅛.

Unless the soil is sand or gravel, etc., which does not slush, the crossing should be paved and gravelled, or metalled.

It is desirable where practicable to make crossings at places where the water up to F.S.L. is wholly in soil. In such places the spoil banks of the canal are stopped on either side of the crossing giving an open approach. Where the canal runs partially in bank, the bank must be retired sufficiently to give the required slope for the ramps as shown in figure 111. The top of the bank at the crossing may be as low as one foot above F.S.L. in the canal, but if cut down to this extent must be protected from wear by stone or brick paving.

FIG. 111. CHANNEL CROSSING.

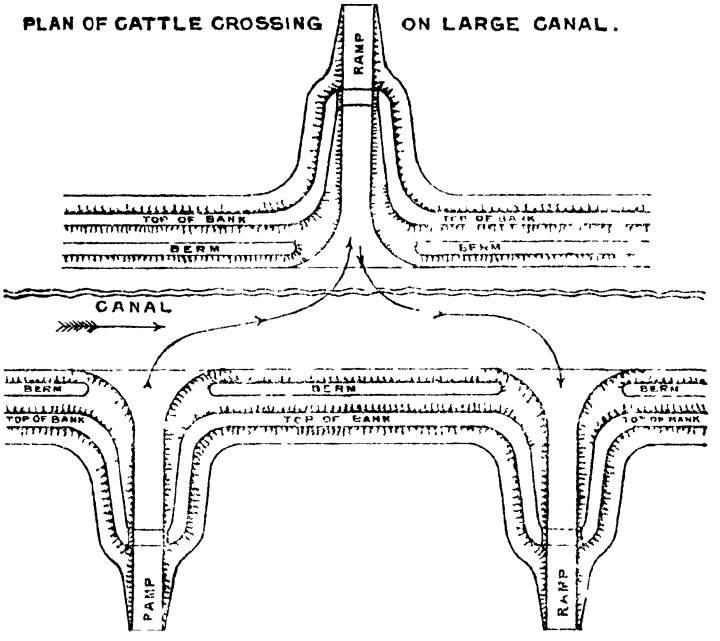


496. Cattle crossings in deep canals.—Where the full supply depth of a canal exceeds four feet, cattle have to swim and where the canal is wide say 50 feet or over, they get carried down with the current while swimming across. A landing place is therefore required some distance below the point of start from the opposite bank. This is arranged for by provision of three ramps in place of

two, the single ramp on one side being spaced opposite the middle distance between the two ramps on the other. Figure 112 shows the arrangement, the arrows indicating the route of cattle swimming the canal in either direction.

FIG. 112.

PLAN OF CATTLE CROSSING ON LARGE CANAL.



Crossings get much worn by traffic and require a good deal of repairs while the banks of the canal in the vicinity are much trampled and worn by cattle so that apart from the greater convenience to the public, it is generally desirable to make a bridge in place of a crossing when the cost of the bridge would not exceed that of the crossing by more than 50 per cent.

RETAINING WALLS FOR IRRIGATION WORKS.

497 Remarks on retaining walls.—Examination of the stability of retaining walls under various disposition of earth thrust is outside the scope of this text book, but, as retaining walls are among the most important parts of irrigation works, a few words on the economical disposition of material in such structures will not be out of place ; this is more especially necessary as such dispositon of material is seldom shown in designs of Indian irrigation works, and judging from works in South India it appears to be becoming rarer than it used to be 20 to 30 years ago.

In the case of a retaining wall, as in the case of a dam, the disposition of material which gives the greatest stability is one in which the batter is on the side of the wall remote from the thrust, that is in the case of a dam on the down-stream side, and in the

case of a retaining wall on the outer side, which is here termed its 'face', while the earthside of the wall is termed its 'back.'

In spite of this fact it is customary to give all the batter required for the retaining walls of irrigation works on the backs keeping the faces vertical. It is to be recognized that there are often practical reasons for this disposition of material in works which are built of brickwork the faces of which are pointed, or of squared stone in courses, because in such cases, if the joints of the masonry are horizontal the batter must be given by offsets It is generally objectionable to allow offsets on the face of a wall because the earth which is retained on them gives lodging to seeds of shrubs which take root and if not promptly removed will damage the masonry, and further because the exposed edges are subject to excessive weathering. Where, as is often the case in South India, retaining walls are built of random rubble, or of brickwork which is rendered with plaster, the above practical objection does not operate, and in such cases retaining walls with face batters are to be recommended and should find place in designs very much more frequently than is now the case.

498. The importance of this matter has been specially emphasised by Bligh who has worked out a number of examples and deduced the following proportions for the base width of retaining walls of height H with 2 feet top width and specific gravity 2·1 to retain earth of specific gravity 1·8 and angle of repose 1½ to 1.

Outer face batter	Formula for base width in feet.	Safe limit of height of wall (according to formula col 2) with the back vertical
(1)	(2)	(3)
Vertical.	0 4 H	5 feet.
1 in 12	0.4 H− ¼	8½ ,, *
1 in 10	0 4 H—1	10 ,,
1 in 8	0 4 H−1½	13 ,,
1 in 6	0 4 H—2½ for 20 ft and below 0 4 H—2½ above 20 feet	18 ,,

* This item has been added to those given by Bligh.

The base widths of column 2 are suitable guides for designing rubble stone walls in ordinary soils. For brick masonry ⅓ to ½ a foot may be added to these base widths. For walls of top width smaller or greater than 2 feet, add or subtract from the base one half of this deficiency or excess.

The economy to be attained where face batters can conveniently and suitably be substituted for back batters is shown in the statement below in the form of percentage of the masonry in each

type section of wall as compared with that in a wall of equal stability with a vertical face taken as 100.

Batter of front face	Height of wall in feet			
	10.	15	20.	25.
(1)	(2)	(3)	(4)	(5)
Vertical	100	100	100	100
1 in 12	95	91	92	94
1 in 10	91	86	90	92
1 in 8	81	82	85	87
1 in 6	73	72	75	79

Face slopes between 1 in 12, and 1 in 8, are those which would ordinarily be most frequently suitable and the designs of retaining walls with these face batters result in reduction of quantities of masonry of from 10 to 15 per cent as compared with walls of equal stability with vertical faces.

499 Retaining walls with sloping tops.—These walls are very common in irrigation works for wing walls retaining earth slopes and the tops are designed with uniform slopes of 1½ to 1 or 2 to 1 or whatever may be the slope of the earth bank to be contained If such walls were made of equal stability the batter would have to be constantly changed with the change of height of the retaining wall, and the face or back slopes could not be uniform but would gradually change from a flatter slope at the highest part of the wall to a steeper one at the lowest part Such a slope is called a 'winding slope' Although there is no great practical difficulty in building a winding slope it is not usually done, and masons are not accustomed to such work so that, except for important works when good supervision is likely to be constantly available, it is not desirable to design falling wings with winding slopes. This considerably restricts the employment of face batters on falling wings, and where these are employed tends to keep them at steeper slopes than would otherwise be the most suitable.

In the absence of a winding slope the alternatives for building a falling wing are—

(*a*) To provide a uniform slope on the face or the back of the wall allowing a sufficient batter to secure the stability of the wall at its highest part.

In a wall with much difference of level between either end this involves uneconomical use of material.

(*b*) To divide the wall into lengths giving a different batter to each separate length.

This will produce unsightly offsets unsuitable for the face of the wing, and such varying slopes must generally be at the back of the wall in spite of the extra quantity of material required.

(*c*) Where the wing is of such a minimum height that some batter is required throughout its whole length then the least (steepest) batter required throughout the length of the falling wing

can be given as a uniform slope to the face, the back of the lowest part of the wing being vertical ; additional thickness in the higher lengths of wall will then be given on the back, offsets in plan being given where the batter is changed. Figure 61, page 185 and figure 91, page 260 show this arrangement.

500. Building a winding slope.—Although building a winding slope is unusual there is no practical difficulty in doing this. To set out such a work, mark on the concrete foundation the base thickness of the sloping wing throughout its length, and from each extremity of the winding slope set up a batten of wood at the face slope of the wall at that place, and divide the batten into *vertical* heights of, say, 1 foot interval. A string tightly stretched between corresponding heights on the 2 battens will give the true face of the winding slope all along that level.

In building, the masons have only to keep a string stretched at the level of the top of the course being built and bring the masonry face up to this.

CHAPTER XIII.

CROSS-DRAINAGE WORKS AND DRAINAGE OF IRRIGATED TRACTS

501. Methods of disposing of cross drainage.—The methods of disposal of drainage intercepted by canals and channels are—

(*a*) To pass the irrigation canal over the drainage.

This is done through (1) an aqueduct or (2) a syphon aqueduct

(*b*) To pass the drainage over the irrigation canal.

This is done through (1) a superpassage or (2) a syphon.

(*c*) To pass the drainage through the irrigation canal so that the drainage and irrigation waters are intermixed.

This is effected by 'a level crossing', or 'an inlet and outlet', or 'an inlet' only.

502. Aqueducts and syphon aqueducts.—These are the most common type of cross-drainage works, and in the case of a contour canal, these works frequently involve a large proportion of the whole cost of the masonry works on the canal.

When a canal is carried over a drainage without having to drop the bed level of the lower waterway, the work is termed an aqueduct (see figure 116, page 311). A similar work is called a 'Superpassage' if the drainage is passed over the canal.

When the bed level of the drainage is dropped where it passes under the canal, so that the drainage passes through an inverted syphon, the work is termed a 'syphon aqueduct', which distinguishes it from a similar description of work, termed a 'syphon', in which the irrigation water is passed through an inverted syphon under the drainage

There is a type of work intermediate between the aqueduct and the syphon-aqueduct where the drainage is dropped just above the crossing, and its bed cut out from this point at a gentle slope, there being no rise in the bed on the down-stream side of the work.

There is no distinctive term for such a work, which is a combined aqueduct and drop, and is generally termed an aqueduct. (Figure 117, page 312 shows a work of this type)

503. Types of aqueducts and syphon aqueducts.—These works may be classified in accordance with the nature of the sides of the aqueduct, in three types as follows:—

Type I.—Where the sides of the aqueduct are earthen banks with complete earthen slopes.

In a work of this description the length of the culverts through which the drainage water is passed under the canal must be sufficient not only to take the water section but also to carry the earthen banks of the canal with their slopes.

Type II.—This is similar to type I but a retaining wall takes the place of the outer slopes of the earthen banks through the aqueduct. (Figure 115, page 309).

Type III.—Where the earthen banks are discontinued through the aqueduct, the sides of which are made generally of masonry walls, but occasionally of wood or steel forming a trough through which the canal is carried (figure 113, page 297).

504. Selection of a suitable type of aqueduct.—In all types the cost of the abutments and bank connexions is independent of the length of the aqueduct. In type I the cost of the bank connexions is a minimum while on the other hand the width of the work from face to face of the drainage culverts is a maximum.

Type I is thus only suitable where the length of the aqueduct is small and the bank connexions large in comparison to the cost of the remainder of the work.

In type III the conditions are exactly reversed, the cost of bank connexions being a maximum and the width from face to face of the drainage culverts a minimum. The type is thus suitable for aqueducts of considerable length between abutments and where the cost of the aqueduct proper is large in proportion to that of the earth connexions.

Type II is a type intermediate between types I and III, both as regards length from face to face of the drainage culverts, and cost of the earth connexions.

The selection of type of aqueduct thus depends chiefly on the width and magnitude of the drainage to be passed. A very small drainage will obviously best be passed by type I work, which, in an extreme case, is merely a pipe or a small culvert, passing under the bed of the canal.

Such works, although in essence aqueducts or syphon aqueducts of type I, are generally called 'drainage pipes' or 'drainage culverts' or 'drainage syphons' as the case may be, or sometimes 'undertunnels'.

Over a stream or river of considerable size an aqueduct of type III is clearly more economical than any other and type II is intermediate.

The limits in regard to width of drainage dividing the types will vary somewhat with local conditions and rates; the following are the results of the investigations made in this matter with regard to aqueducts and syphon aqueducts for the Madras Cauvery Reservoir Project and will be found generally applicable to the normal conditions of South India.

Type I, most suitable up to a lineal width of culvert waterway of 8 feet (2 spans of 4 feet).

Type II, for a lineal width of culvert waterway above 8 feet and not more than 54 feet (9 spans of 6 feet).

(4 feet spans were proposed up to a limit of 4 spans, 16 feet lineal, and above this 6 feet spans).

Type III, for lineal waterways above 54 feet (12 feet spans were universally proposed for this type).

The above may serve to give some idea of the magnitude of the drainages suitable for the various types of works but should not be taken as defining any hard and fast limits either as regards spans or types.

505. Headway required at the point of crossing of a drainage by an aqueduct—The level of the bed of a contour canal at

the point where it passes over a drainage is fixed within fairly narrow limits, depending on the mileage at the point of crossing, and the difference of level between the natural bed of the drainage and that of the canal can be increased by taking the canal across the drainage lower down, or diminished by crossing it higher up.

In the case of an aqueduct the bed of the canal must be at sufficient height above the bed of the drainage to allow of sufficient waterway being given in the culverts to pass the flood discharge of the drainage over which the canal is carried. If the river has cut out a bed deep below the level of the ground through which it takes its course, and its waters in flood are confined within these margins with some freeboard to spare, there will generally be little difficulty as regards sufficiency of headway, and the point of crossing of the irrigation channel may then be selected from other considerations than those of headway, such as the most convenient and economical alignment of canal or the most suitable point of crossing the river having in view nature of foundations, etc.

In the plains of South India the above conditions are not common, and generally the drainages are wide shallow streams which in floods submerge their margins.

It is desirable on the ground of both efficiency and economy to limit the length of the aqueduct to the width of the stream and this fixes approximately the maximum lineal width of waterway, it is usually inadvisable to contract the width of the work as compared with the width of the drainage by more than 25 per cent; from these considerations the most suitable length of the work between abutments is fixed.

The total area of the waterway is fixed by the magnitude of discharge of drainage to be provided for divided by the maximum allowable velocity through the openings. The mean effective height of the openings can be computed from these and the approximate difference of level between the canal bed and river bed at the point of crossing will thus be known This is frequently so great as to necessitate placing the work so low down the drainage course that the canal bed at the crossing must be several feet above the level of the stream margins, and it follows, that the canal must be carried in heavy banks for some distance on either side of the aqueduct. This entails both expense and danger of breaching and is objectionable. The alternatives are either to lower the bed of the drainage where the canal crosses it forming a syphon aqueduct, or to drop the bed at the drainage just above the crossing so as to leave sufficient headway. Below the drop the bed of the drainage would, in the latter case, have to be cut out with a slope flatter than that of the drainage bed till it intersects the existing bed below the crossing.

506 Lowering bed of drainage below a syphon aqueduct.— In the case of a syphon aqueduct similar lowering of the drainage bed is also occasionally necessary, as it is desirable, if possible, to leave some clear headway between the bed level of the drainage downstream of the crossing and the underside of the culvert covering; this headway should, if practicable, be not less than 3 feet or half the height of the culvert, whichever is the less.

The headway is desirable in order to minimise the risk of the blocking of the culverts by silt rolled along the bed of the

drainage, and this is a real danger and constitutes the greatest objection to the syphon aqueduct type of cross drainage works.

Apart from considerations of cost, the more nearly the syphon aqueduct approaches the combined aqueduct and drop, that is the less the rise in level of bed on the downstream side of the drainage culverts, the better

507. Considerations which determine the site of aqueduct or syphon aqueduct, and which type of work is most suitable.—The relative size of the canal and of the drainage to be crossed is a primary consideration in determining whether the work shall be an aqueduct or syphon aqueduct.

In the case of a large river the lowering of the bed by a drop would be very costly, and where the amount of bed silt is great, the risk of serious failure by silting of the syphon increases with the size of the drainage; an aqueduct is thus generally necessary where a canal is carried over a large river in spite of the cost of heavily banked approaches

On the other hand when the drainage is small compared to the canal the syphoning of the drainage will be preferable to carrying the canal in heavy banks and a syphon aqueduct is indicated.

Additional considerations are the most convenient lines of canal alignment, the nature of the foundations, the nature of the soil available for making banks, the discharge of springs likely to be met with when putting in foundations, and the uplift pressures to be provided for; all the above must be considered when deciding on the site of the crossing and whether the work will be an aqueduct or syphon aqueduct.

508. Crossing a drainage by an irrigation syphon.—An alternative to an aqueduct or syphon aqueduct is an irrigation syphon where the irrigation canal is syphoned under the drainage. For such a work the conditions of approach to the crossing are the reverse of those for an aqueduct and the canal approach to the drainage should be in cutting rather than in bank. This method of crossing is frequently the best and especially in the case when drainages are large in comparison to the size of the canal. The syphoning of the canal as an alternative to syphoning the drainage has the great advantage that no trouble from a sudden accumulation of bed silt is to be feared and the precaution with regard to headway of paragraph 506 above is not in this case necessary.

509. Example of type III syphon aqueduct—(Figure 113).—This is the type on which the largest syphon aqueducts are built and the details of design of a work of this type will now be described. The data are as follows:—

The drainage is to be syphoned under the canal through culverts 12 feet clear span covered by brick arches, with 2 feet rise.

The canal at F.S.L. is $4\frac{1}{2}$ feet deep, but the side walls are to be $1\frac{1}{2}$ feet above this, so that water can be carried as much as 6 feet deep, the weights on the structure are therefore calculated for a depth of 6 feet.

Thickness of culvert arches.—Formula 42, paragraph 321 above, gives the thickness of the arch of an ordinary bridge and regulator as '45 V Radius. In the case of an aqueduct, as the load on the arch

is generally greater than in the case of an ordinary bridge, the co-efficient may be increased from ·45 to ·5

TYPE III
SYPHON AQUEDUCT.
FIG 113.
PLAN

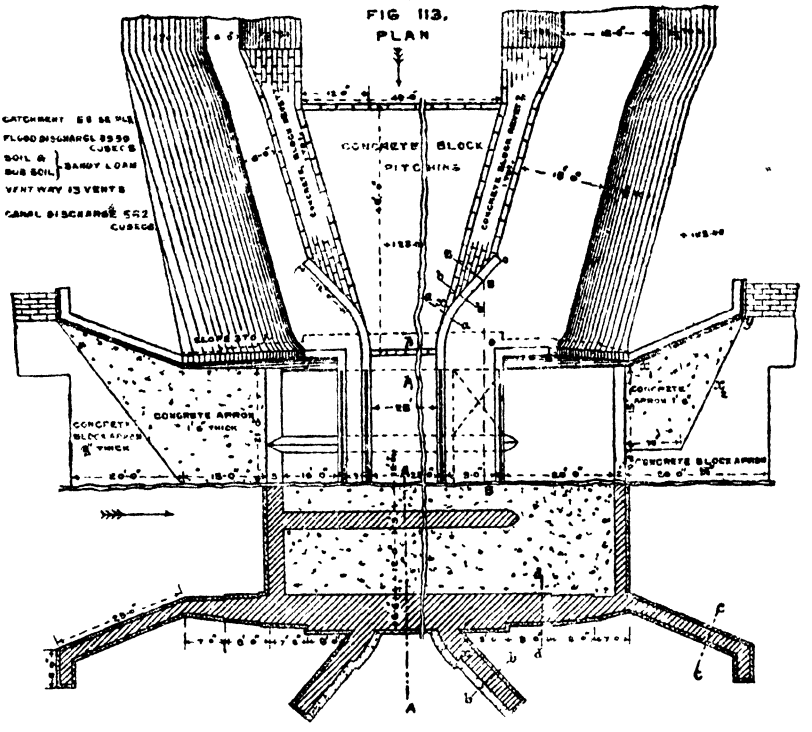


FIG 113 (a) LONGL. SECTION ALONG DRAINAGE

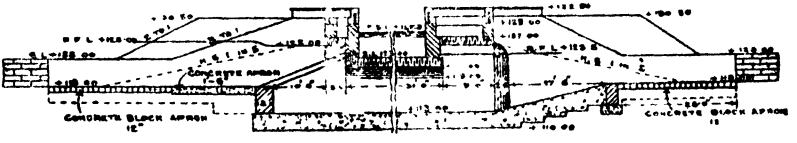
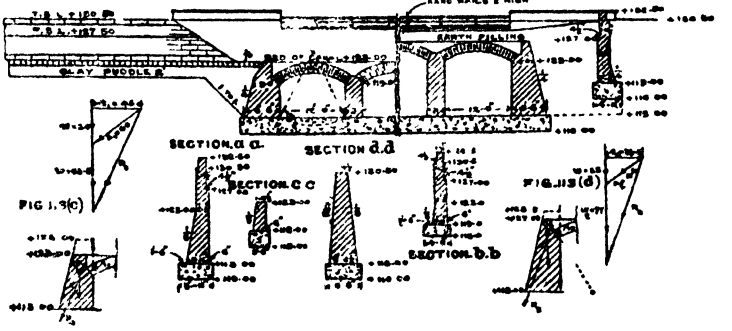


FIG 113 (b) SECTION A A B B.



The radius of the intrados of the arch in this case is 10 feet and (A) the angle at the centre is 74°.

The thickness of the arch from the above rule would be 1·60 feet and the arch may be made $2\frac{1}{2}$, bricks $=1'\ 10\frac{1}{2}''$ thick and the horizontal thrust and pressure on this will now be computed

The spandril filling over the arch will be concrete carried up to the floor of the aqueduct to a level 2 feet above the underside of the crown of the arch. The specific gravity of the masonry is 2 and the water load on the arch is therefore equivalent to a weight of solid masonry to a level of 3 feet above the aqueduct floor level. The weight (W) of half the arch *plus* the above load $=(6\times 7)-8=34$ units

The horizontal thrust at the crown of the arch $=W\cot\frac{A}{2}$

$=34\cot 37°=45$ units.

$=(1\frac{1}{8}\times 45)$ tons $=2\frac{1}{2}$ tons

The mean load per square foot on the cross section of the arch is $(2\frac{1}{2}-1\frac{7}{8})=1\frac{1}{4}$ tons and the maximum stress $2\frac{1}{4}$ which leaves a large margin of safety.

The thickness of piers.—For trial the same width at the springing as for a regulator may be taken, viz., from $\frac{1}{8}$th to $\frac{1}{3}$rd of (S) the span (vide formula 41, paragraph 321 above). In this case $\frac{1}{4}$S may be taken $=3$ ft. It is necessary to verify this for crushing stress and to ascertain the limit of height of pier for which this width is safe without battering the sides of the pier.

The weight (due to 6 feet depth of water) resting on each foot of length of each pier at the level of springing of arch is $(12\times 7-16+3\times 7)\frac{A}{\pi 8}$ tons $=5$ tons. The mean pressure is $\frac{1}{4}$ of $5=1\cdot 7$ tons per square foot

If the mean pressure be limited to 2·2 tons, the piers may be carried to a depth of 18 $(2\cdot 2-1\ 7)=9$ feet below springing before any widening is necessary.

Number of culvert vents—The limiting velocity through the culverts has to be decided in order to compute the amount of waterway required. This is fixed at 8 feet per second. The flood discharge to be disposed of is 9,000 cusecs being that due to a catchment area of 58 square miles with a co-efficient of 600 (Ryves' formula). The drainage has a bed width of 215 feet. The canal bed width is 49 feet and bed-level is + 123. The thickness of the masonry being 2 feet at the crown, the intrados of the arch cannot be higher than + 121, and if 3 feet clearance, without any excavation of the bed of the drainage be allowed the bed of the latter at the site of crossing should be not higher than + 118.

The drainage is a large one and this is unsuitable for cutting down the level of the bed below the crossing. The site of crossing is therefore chosen where the bed level of the drainage is + 118.

The limit of 8 feet per second placed on the velocity of flow through the culverts gives a flow area of $9,000/8=1,125$ sq. ft. The width of the drainage between margins is 215 feet. The width of each span and pier is 15 feet. The number of spans which would fit between the stream margins is $215/15=14\frac{1}{3}$. It is however economical to slightly contract the waterway between abutments and 13 spans will be suitable giving an area of $1,125/13=86$ sq ft.

for each span The segmental area over each vent is 16 sq. ft. and the height of piers to springing should therefore be about 70/12 = 6 feet.

510. Profile of abutments of aqueducts.— Owing to the great weights carried by the arches of an aqueduct an addition to the width of abutments at springing may be given to that derived from Trautwine's rule which is—

$$\left.\begin{matrix}\text{Width at springing}\\ \text{(vide formula 43)}\end{matrix}\right\} = 2+2/10 \times (\text{Radius}) + 1/10 \times (\text{Rise}).$$

Bligh recommends an addition of 1/5 of the depth of the water to be carried in the canal, and this may be adopted.

The approximate width of the abutment of an aqueduct at springing will therefore be

$$2+2/10\ (\text{Radius of arch} + \text{depth of canal}) + 1/10\ (\text{Rise of arch}) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (65)$$

511. In the example under consideration this formula gives a thickness of about 5 feet.

The batter of the abutment by Trautwine's rule (formula 44, paragraph 321) will be

$$1/25 \times \frac{\text{span of arch}}{\text{Rise of arch}} = 12/50 = \tfrac{1}{4}\ (\text{approximately}).$$

It is desirable to test the stability of the abutments of so important a work by a diagram and this is done in figure 113-*c*.

The thrust (t_1) of the arch at the crown = 45 units.

The weight (w) carried on the half arch = 34 units.

From this the direction and magnitude (A_1) of the thrust of the arch at the abutment is found by diagram and equals 60 units = 3·3 tons.

The mean pressure on the skew back at the springing of arch is thus 3·3 − $1\frac{7}{8}$ = 1·7 tons and the maximum pressure 3·4 tons

Looking at the diagram figure 113-*c* the resultant cuts the base one-fourth of its length from the outer toe and this is suitable. In the diagram it is assumed that the resultant arch thrust acts through the centre of the skew back The pressure of the earth at the back of the wall is not allowed for as this is set off against the thrust due to pressure of the drainage water in the culverts or of any arch action of the floors in resisting uplift as explained in paragraph 515 below.

The abutments under the side walls of the aqueduct have not so heavy a load and the width at springing level may be the same as for an ordinary bridge abutment on Trautwine's rule and this would be 4 feet with batter ¼.

A stability diagram figure 113-*d* confirms the suitability of this profile.

The aqueduct as designed is shown in figures 113, 113 (*a*) and (*b*) and various connected points of design not hitherto dicussed will now be commented on.

512. Bank connexions —These must retain the earth slopes and act as stop walls to percolation so as to allow no line of percolation steeper than the allowable percolation gradient, which in this case is 1 in 5.

A double set of wings are required in such a work, viz., the canal wings making a staunch connexion between masonry sides of the aqueduct and the canal banks, and the drainage wings, retaining and protecting the earth slopes, forming guide walls to the water entering and leaving the work, and also to some extent acting as stop walls to lengthen the path of percolation of water from the canal above.

In order to design wings properly it is absolutely necessary to make a plan showing the canal banks, stream margins and earth slopes as is done in figure 113.

513. In the example under consideration the length of the wings are clearly seen from the plan to be sufficient to retain the slopes. The canal wings have no additional length but to the lower wings an additional length $x_1\ y$ is added which, but for the necessity of checking percolation would not be required. The percolation head is from the F.S.L. in the canal to the drainage bed level and, allowing an extra half foot as working margin, equals 10 feet, involving a minimum percolation length of 50 feet. The shortest line for percolation is under the foundation of the wing along the line $x\ x_1$ from x, where this is stepped up from +110 to + 116. If a section is drawn on this line the length of percolation between x and x_1 under the wall foundations will be found to be about 42 feet. This length is increased by a concrete apron carried out to x_2 not less than 10 feet wide in all directions from x_1. The width of this apron is reduced towards the end of the wings at y where it becomes *nil*; the least distance to the canal water from y along a line of possible percolation is nowhere less than 50 feet

The prolongation of the drainage wings from v to y and the impervious concrete floor $p^2\ y$ are introduced solely to lengthen the line of percolation. The same purpose might be served by lengthening the canal wings and keeping the depth of their foundations sufficient to secure the required length of percolation round the ends of and under the wings.

This would in this case be more expensive than the design adopted but the alternative should always be considered.

If the conditions are such that full reliance can at all times be placed on the puddle in the canal bed being kept damp and free from cracks there could be no percolation under the canal wing wall if a good joint were made between the puddle and the face of the canal wings and under such conditions the wing foundation might be stepped up throughout the whole of its length. Wherever a canal is periodically closed for a prolonged period and the puddle is laid at a level considerably above that of subsoil water it is likely to become dry and crack and full reliance cannot be placed on it ; it is nevertheless usual to lay it in the canal bed as is shown in this design.

In the drainage bed a puddle apron overlaid by protective pitching might take the place of impervious concrete apron in any site where the dry season subsoil water is at or near the level of the puddle, so that it is protected against becoming dry.

514. Uplift pressures on syphon flooring.—The floor of the syphon culverts are subjected to uplift—

(*a*) from the subsoil water in the drainage bed, the maximum net uplift being when there is no drainage flow and when there is no water or silt standing on the syphon floor, but when the subsoil water is at the level of the bed of the drainage, and

(*b*) from the canal ; it is a maximum when the canal is full and no water is flowing in the drain.

515. The maximum uplift head due to (*a*) which can be exerted on the under side of the floor is when subsoil water is just at bed level and is equal to the vertical distance from the subsoil water level to the under side of the apron

The head is a statical one due to uplift acting from both up and down-stream In this case, figure 113, the uplift bed would be (118 −110) = 8 feet

The head due to (*b*) is more difficult to compute. The difference of level between the canal F.S.L. and the bed of the drain is 10 ft. allowing a working margin of ½ foot, and the greatest uplift due to this on the culvert floor is developed in the centre of the length of the flank culverts. Supposing the puddle in the canal bed to be ineffective, and only to be equal to the ordinary soil in resisting percolation the minimum line of percolation under the middle of flank culvert is $p\ p_1\ p_2$, 70 feet in length (figure 113) of which $p\ p_1$ is of length about 21 feet. The uplift at p_1 will be (*a*) the statical uplift due to subsoil water (8 feet) as already stated plus (*b*) which is $\left(10 \times \frac{70-21}{70}\right) = 7$ ft The total is 15 feet and if the specific gravity of the floor is 2 a thickness of floor of over 7½ feet would be required to resist this by dead weight whereas the design only provides a floor of 3 feet thickness

It may be here pointed out that 7½ feet thickness of floor below level + 113 would still be quite insufficient to resist uplift by the dead weight of the floor because the level of the underside of the floor would be lowered by 4½ feet which adds this head to the statical uplift pressure, and a floor 12 feet thick with an uplift head of 17 feet due to (*a*) and about 7 feet due to (*b*) thus making a total of 24 feet would be required to get equilibrium from the dead weight of flooring. This would involve great extra cost and difficulty in construction—

There are two ordinary methods of getting out of this difficulty.—

(1) Providing a puddle apron in the canal bed up and down-stream of the aqueduct In this case a puddle apron has been provided increasing the line of percolation as far as p_1 to about 50 feet. This reduces (*b*) the uplift due to the canal water at this point to about half of 10 or 5 feet but leaves unaltered (*a*) the uplift due to the subsoil water (8 feet) and therefore the total is still 13 feet, even under the supposition that the apron in the canal bed is quite water tight.

(2) Although the uplift due to canal water may be reduced it is impossible to reduce that due to subsoil water and in practice the difficulty is frequently met by springing inverted arches between the footings of the culvert piers so that the uplift load is carried by arch action and the whole weight of the superstructure carried by the piers is thus employed in resisting uplift. Inverted arches or inverts also serve to distribute the weight of the superstructure

over the whole of the flooring and thus are doubly useful Where *small* spans are used the normal concrete slab floor can be considered as acting as inverted concrete arches of such rise and thickness as to be entirely enclosed in the section of the slab.

In the present case an arch of 1½ feet thickness and 1½ feet *versed sine* is shown in dotted lines in figure 113 inscribed within the 3 feet slab floor, the radius of the intrados of such an arch is 12·7 feet and the centre angle 56 . If u be the maximum uplift head such an arch can take with a limit of mean pressure at the crown of the arch at 2·5 tons per square foot and if P be the horizontal pressure at the crown, $P = \frac{u}{36} \times 6 \times \cot 28°$.

In this case with P a maximum

$$1\tfrac{1}{2} \times 2{\cdot}5 = \tfrac{u}{6} \cot 28°$$

$$u = 12 \text{ feet.}$$

Thus the floor by arch action alone can resist an uplift head of 12 feet while by its dead weight it can resist a further 6 feet or a total of 18 feet and is thus secure against blowing up.

There would in this and similar cases be no objection to actually building a brick invert as shown in the dotted lines on the flank culvert (figure 113) and thereby increasing the waterway of the culverts, but in the case of small spans the greater complexity and cost of construction would not make this desirable In spans above 12 to 14 feet inverts should be used for flooring unless (1) the foundation soil is impervious or (2) the foundation so deep or the soil so compact that no flooring is required or (3) a pitched flooring which is previous is provided. This latter is obviously not a desirable feature in the floor of a syphon in which there is any considerable drop unless the soil is safe against piping.

In the case of aqueducts where the drainage bed level is not dropped there will be no net uplift on the floor due to (*a*) of paragraph 514 above other than that due to the thickness of the floor and in addition to this (*b*) must be provided for.

516. Uplift pressures on overhead covering of syphon culverts --When the tail water level below a syphon is higher than the under surface of the coverings of the culverts uplift is exerted on these, and a number of cases of blowing up of syphons from this cause have actually occurred.

If there is water in the aqueduct when the uplift is applied, this exerts a counter pressure proportionate to its depth; it is, however, generally desirable to design a syphon aqueduct on the assumption of a maximum flood when the aqueduct trough is empty. In addition to the uplift due to the level of the tail water there is, at each point of the barrel of a submerged culvert, an uplift due to the head necessary to force the water through the remaining length of culvert, which must be added to the level of the tail water to arrive at the uplift at any point; in short the pressure on the roof of the culvert is defined by the hydraulic gradient through the culvert barrel.

The internal surface of culvert barrels should invariably be finished of quite smooth so as to give them maximum efficiency of discharge, and for smooth barrels the loss of head due to friction is the culvert is always small; for any ordinary smooth culvert with discharging velocity under 10 feet per second, an allowance of half a

foot may be assumed as sufficient to cover this which does not include loss of head on entry. Thus we may take uplift head as due to a level of one half foot higher than maximum tailwater level.

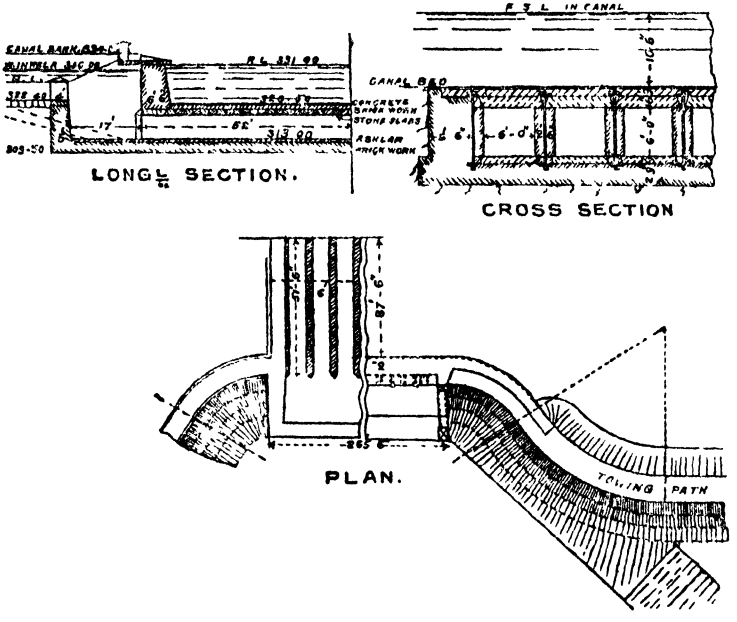
517. In the case figure 113, tailwater level is computed to be +123·5 and uplift at the crown of the culvert arches is thus (123·5 +0·5−121·0=3·0 feet.

The minimum thickness of masonry over the culverts is 2 feet which is capable of resisting 4 feet uplift head and this is safe.

518. General remarks on uplift on the coverings of syphons.— When the bed of the tail channel is 3 feet or more below the underside of the culvert covering it is seldom necessary to make special provision for uplift of the culvert covering, but where the tail channel is at high level considerable cost has to be incurred in providing for uplift, and the covering of culverts has frequently to be increased in thickness solely for this cause.

This is undesirable, as every increase in thickness of covering entails a lower level of the under surface of both the covering and floor of the syphon with corresponding increase of uplift on both

FIG. 114. KAO NADI SYPHON AQUEDUCT, SON CANAL.



The uplift may be provided for by using a covering for the culverts capable of taking bending stresses, and anchoring this covering through the piers of the culverts, thus employing the weight of the piers (and also if necessary the floor on which they stand) to withstand the uplift on the roof of the culverts.

Figure 114 shows the Kao Nadi syphon aqueduct on the Sôn Canal which is an example of this class of work, stone slabs covered with concrete, which are anchored through the piers to the flooring, being used as culvert coverings.

In applying this method of dealing with such cases ferro-concrete culvert coverings are especially suitable as they require very little headway, and they have latterly been much used.

519. Formula for the discharge of an inverted syphon.—Unwin's formula for the discharge of a syphon is as follows:—

$$h = (1 + f_1 + f_2 \frac{L}{R}) \frac{V^2}{2g} \quad \ldots \quad (66)$$

where h = difference of level of water up and down-stream.
L = Length of barrel in feet.
R = Hydraulic mean radius of barrel in feet.
V = Velocity through the barrel in f.s.

f_1 is a co-efficient which provides for the loss of head on entry; it may be taken as 0·505 for an unshaped mouth of the same sectional area as the barrel and 0·08 for a bell mouth.

f_2 is a co-efficient such that the loss of head by friction through the barrel is $f_2 \frac{L}{R} + \frac{V^2}{2g}$.

f_2 is equal to $a\,(1 + \frac{b}{R})$ and the following table gives the values of a and b.—

Nature of surface of pipe or culvert.	a	b
(1) Smooth iron pipe	0 00497	0·084
(2) Encrusted ,,	0 00996	0 084
(3) Smooth cement plaster or planed wood ...	0 00316	0 10
(4) Ashlar or brickwork ...	0 00401	0·23
(5) Rubble masonry or stone pitching	0 00507	0 83

If (V_a) the velocity of approach to the syphon is considered then the formula becomes

$$h = \frac{V^2}{2g}(1 + f_1 + f_2\frac{L}{R}) - (0\,0155\ V_a^2) \quad \ldots \quad (66\text{-}a)$$

Generally the velocity of approach is neglected.

As a rule the interior of culverts should be cement rendered and worked up quite smooth and in no case should the interior be left rough.

520. Head on a syphon.—In most cases a certain limiting velocity (generally 8 to 10 feet a second for a drainage syphon) through the barrels of a syphon is fixed and the water level at the tail must be ascertained from observation or computation, the head (h) required to give the limiting velocity through the culverts is then computed

521 In the example figure 113, from the levels on the plan it is seen that $h = (125{\cdot}0 - 123{\cdot}5) = 1{\cdot}5$ feet. It will be noted that the figure 113-a shows that there is a second rise up to + 126 above the work; this is to get the depth required to give the necessary discharge over the drop wall crest. With velocity of approach this would become very small, but it is prudent to allow it, so that there may be ample free board to the canal banks to prevent any danger of the local drainage breaching into the canal.

522. General remarks on the design of a syphon aqueduct.—The design, figure 113, shows a rounded entry to the culverts which much reduces loss of head on entry but the co-efficient for a square entry has been taken in calculation.

The drop at the up-stream end of the culvert may be either a slope or a drop wall as most convenient but the lift at the tail end should always be sloped to give facilities for the water clearing out all kinds of rolled silt and debris from the culvert barrels.

The slope should be not steeper than ¼.

The width of the cistern between the drop wall and the face of the culverts should be about equal to the difference between the culvert floor level and the M F L.

The raising, as in this design, of the arches outside the side walls of the aqueduct is desirable—

(1) to reduce the pressure on the piers to the same limits as under the floor of the aqueduct,

(2) to give a clearer entry and exit, and

(3) to reduce the total amount of masonry

The arrangement allows the thickness of the side walls of the aqueduct as well as of the arches to be reduced

The design provides for a cart road on one side of the aqueduct and a track for riding and cattle on the other If this were not required, single solid side walls would be suitable and would considerably reduce the cost of the work.

523 A type III aqueduct suitable for a surplus work.—Examination of the design, figure 113, will show how easily and economically surplus sluices can be installed in one of the side walls of a type III aqueduct Vents through the side walls closed by screw geared shutters moving in grooves and worked from the roadway above would constitute an effective surplus sluice the outfall being into a water cushion formed by the counterslope of the syphon while the drain constitutes a ready made surplus channel.

In this case no such sluices have been provided but they could be cheaply inserted if required. It should be noticed that the tops of the side walls of the aqueduct trough are kept 1½ feet lower than at the ends, and thus form surplus weirs which would come into action for any depth of water over 6 feet. A design showing escape sluice in the aqueduct side walls is shown in fig. 118, page 313.

524. Contraction of waterway through an aqueduct.—It will be noticed from the plan, fig. 113, that the canal is considerably contracted on approach to the aqueduct and that the mean width in the latter is only 25 feet as compared with 53½ feet in the full section of canal at F.S.L.

The mean velocity through the aqueduct must therefore be about double that of the canal.

The general question is thus raised as to when, and to what extent, contraction of section through an aqueduct is desirable.

It is to be recognized that contraction involves alterations of velocity which produces extra eddies and disturbance of flow and is therefore to be deprecated where it can be economically avoided.

Contraction of waterway is not desirable in short aqueducts as in such works the saving in cost will be comparatively small while

the disturbance will be the same as in long ones. Generally contraction should not be given in aqueducts of types I or II but in those of type III the question should in each case be considered on its merits. For every new system of canals a general policy as regards contraction should be formulated having in view (1) the economic aspect, and (2) the possibility of future expansion.

For the Madras-Cauvery Project Main Canal (a contour canal) contraction was given in designs for aqueducts where the aggregate floor width of the drainage culverts was not less than 60 lineal feet, and the amount of contraction provided was such that the velocity of flow through the aqueduct would be approximately either twice that in the canal or 5 feet a second, whichever was the smaller.

In the case of trough aqueducts of iron, wood, or ferro-concrete, or in the case of smaller works, relatively greater contraction may occasionally be desirable, but in no case should contraction be given unless substantial reduction in the cost is realized. Where contraction is given the change of the canal cross section above and below the work should be gradual, so as to give a good entry and exit, and the canal sides and bed, where subjected to increased velocity, protected from erosion by pitching.

525. Loss of head or heading up in a canal at a contracted aqueduct —There must be a fall in the water level at the entry to a contracted aqueduct due to the head required to raise the flow from the normal to the higher velocity, while some further head due to loss on " entry " is necessary. This latter varies with the nature of the entry which, as above stated, should not entail abrupt change On the other hand at the exit of the canal there is a reduction of velocity which is accompanied by a rise in water level reduced by losses in exit.

In addition there is the normal surface slope throughout the length of the work required to overcome ordinary frictional resistance to flow.

The co-efficient of resistance to flow ('N' of Kutter's formula) is much smaller in a masonry trough with smooth sides than in an earthen channel, and thus the surface slope would be reduced, but for any ordinary work the length of an aqueduct is so small that the value of the co-efficient has no practical effect in the matter of head required.

This remark does not apply to works of extraordinary length such as for instance the Gunnaram aqueduct (figure 116) which is nearly half a mile in length.

Special stress is placed on this point as very undue prominence is frequently given to the low co-efficient of rugosity in trough aqueducts in justifying proposals for contraction of waterway.

526. Formula and calculations for head required.—The net heading up (h) of a canal at a contracted aqueduct may be considered to be the difference of water levels at points just above and below the sections where contraction commences and ceases *less* the fall due to the normal canal surface slope in this distance; except in very long aqueducts the latter is so small that it may be neglected.

In the case of an *abrupt* enlargement at the tail of an aqueduct if V_1 be the velocity in the canal and V_2 that in the aqueduct, then h_t the theoretic rise in the level of the water surface on exit is approximately given by the equation :—

$$h^t = \frac{V_1(V_2 - V_1)}{g} \quad . \quad . \quad . \quad . \quad (67)$$

(Bellasis hydraulics, Chapter I, paragraph 18.)

In ordinary practice the enlargement at the tail should not be abrupt, on the other hand owing to eddies, etc., theoretic results cannot be realized. Actual measurement of the rise of water levels at the tail of contracted aqueducts with gradual enlargement shows rises of water level at exits somewhat less than h_t, and for purposes of computation it is safe to deduct ¾ h_t from the head of entry (h_e) to get h the net head.

The general formula for entry into a contracted aqueduct of velocity V_2, from a canal of velocity V_1 is the same as for flow through a notch of similar cross-sectional area and is as follows :—

$$D = cl\sqrt{2g}\left[\tfrac{2}{3}\left\{\left(h_e + \frac{V_1^2}{2g}\right)^{\frac{3}{2}} - \left(\frac{V_1^2}{2g}\right)^{\frac{3}{2}}\right\} + d\left(h_e + \frac{V_1^2}{2g}\right)^{\frac{1}{2}}\right] \quad (68)$$

Where D = discharge.

d = depth of water in the aqueduct

h_e = the heading up above the aqueduct trough that is the head of entry.

l = the mean width of the aqueduct (or notch)

With a well shaped and gradual entry c would be from ·95 to·9.

527 In the case of aqueduct (figure 113), D = 562 cusecs
V_1 = 2·5 f.s.
V_2 = 5 0 f.s.
l = 25
c = 0 95.

then $562 = 190\left[\tfrac{2}{3}\left\{(h_e + \cdot097)^{\frac{3}{2}} - \cdot097\right\} + 4\,5\,(h_e + 097)^{\frac{1}{2}}\right]$.

This may be solved by trial

Try h = 0 3 ft.

then discharge = $190\left[\tfrac{2}{3}(\cdot25 - \cdot097) + 4\cdot5 \times 63\right]$
= 190 × 2·932 = 558 cusecs or nearly 562 cusecs

The head of entry is thus 0·3 feet

The approximate head of entry for a *co-efficient unity* may be obtained from the following simple equation :—

$$h_e = \frac{V_2^2 - V_1^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (69)$$

In this case this becomes $\frac{(25 - 6\,25)}{64} = 297$, or about 1 per cent below the result given by the more accurate formula with a co-efficient of ·95.

To find the recovery of level on exit—

$$h_t = \frac{V_1(V_2 - V_1)}{g} = \frac{2\,5\,(5 - 2\,5)}{32}$$

$$= \frac{6\cdot25}{32} = \cdot194$$

The net heading up at the aqueduct = h = h_e — ¾ h_t.
= 0 3 — 0·145.
= 0·155 feet.

528. Normal limit of heading up due to contraction of an aqueduct.—Generally when the velocity through an aqueduct does not exceed about 5 feet per second or double the velocity of the canal it will be found that the net heading up occasioned does not exceed 0·2 feet if a gradual entry and exit be given.

When an aqueduct is of considerable length a bed fall equal to the surface slope of the water in the trough may be given, but for aqueducts of ordinary length this is unnecessary.

If the waterway of the canal is contracted at the aqueduct and a large head (h) is required to give the necessary velocity at entry, the aqueduct bed may be dropped to this amount and this would result in keeping the channel above at exactly correct normal depth at full supply, and prevent any heading up, but, as in the case of ordinary submerged drops, unless the entry were through one or more trapezoidal notches of suitable dimensions, the correct depth would only be obtained at full supply Ordinarily in any channel of considerable size it is quite unnecessary to give any drop at entry, and the canal above a contracted aqueduct may be allowed to head up to the extent required, which would seldom exceed ¼ foot

In a small channel if the contraction is great a drop at the entry may occasionally be desirable.

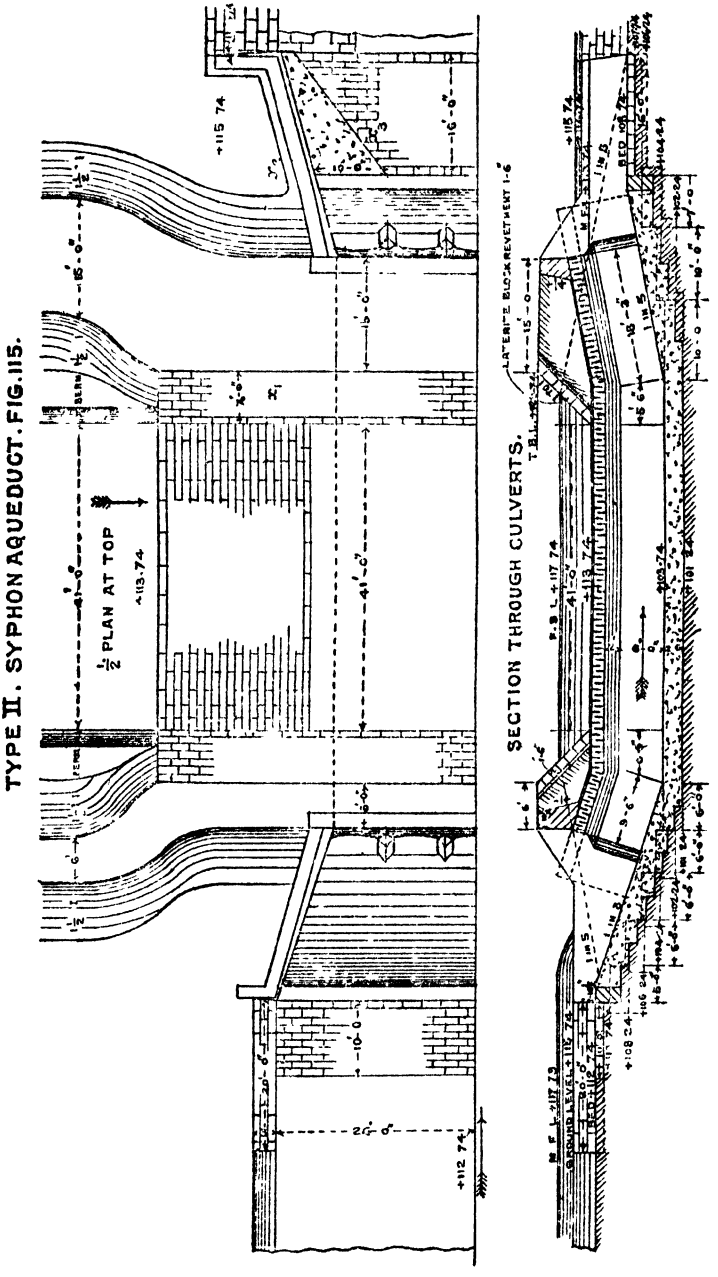
529. Example of aqueduct of type II.—An example is shown in figure 115 of an aqueduct carrying a canal of nearly the same size as in the last example over a smaller drainage of catchment area 5¼ square miles having a maximum flood discharge of 1,800 cusecs.

For crossing such a stream an aqueduct of type II is most suitable. The most important points guiding design such as uplift on floors and covering arches of culverts, percolation gradient, stresses in masonary, etc., are similar to those fully dealt with in relation to the example of a type III aqueduct and only the new points pertaining to this particular type of work will be now discussed. The work provides room for a cart road on one side and for riding path on the other, if a cart road is not required the banks on both sides would be similar to that on the right side and considerable reduction in the length of the culverts and cost of the work could be effected. Both entry and exit of the drainage water are given by slopes which are carried under the banks thus considerably reducing the length of the culverts; this device has also the effect of reducing the heights of the face walls, the quantity of earthwork in banks, and the weight of earth to be carried by the arches and piers

The width of the piers must be computed with reference to the weight of earthwork to be carried this being much greater than the weight of water, and similarly for the arches under the banks. The arch work under the bed of the canal may have a less thickness than under the banks if this results in any material saving in cost; as however the uplift has to be balanced such alteration may not entail material saving and in this design no reduction has been made.

It is to be noted there are no canal wings in this design but the drainage wings and floor are designed to secure a percolation gradient not steeper than 1 in 5

As in the last case (figure 113) the design with minor alterations is one from the Madras Cauvery Project. It will be noticed that the foundations of the wing walls are stepped up as also are the concrete sloping floors, the stepping of the latter being at higher



levels than that of the former. The least length of the line of percolation to secure a gradient not steeper than 1 in 5 is $9 \times 5 = 45$ feet. If a section be drawn along a line of percolation such as $x_1\ x_2\ x_3$ it will be found that this length is secured by the impervious concrete apron, trapezoidal in plan. On the upper side of the culverts no impervious apron above the drop slope is called for.

The difference in bed level (4 feet) of the drainage up and down stream of the culverts is to be noticed. This is a case in which the bed level of the drainage down stream of the culverts is to be excavated to keep it 3 feet below the underside of the culvert arch at its crown.

As a criticism of the design it may be remarked that it might perhaps be improved by flatter culvert arches. The rise is ⅓rd the span ; with arches with ⅙th rise the floor might be raised by nearly a foot.

This would entail heavier abutments and possibly thicker arches but the upstream slope would be shorter, the wings smaller and the excavation less.

The downstream slope would not be shortened as the width of bank calls for this length of slope

530. Drainage culvert or syphon aqueduct of type I.—If the culverts in figure 115 were prolonged (as shown in dotted lines) so as to take the outer slopes of the banks and the wings correspondingly shortened the work would be one of type I.

There would be saving in the wings and face walls but the extra cost of the extension of the culverts would clearly be greater in this case. This would not however be so if the drainage was smaller requiring only 6 to 9 lineal feet of waterway. In such a case type I with reduced spans of culvert would be adopted.

A considerable rise to the arch of type I works is desirable when headway can be spared in order to reduce the necessary thickness of abutments

It is not considered necessary to further discuss this type of work

531. Examples of existing aqueducts and syphon aqueducts.—Drawings of existing aqueducts and syphon aqueducts are given in figures 116 and 117 and noted on as follows :—

Gunnaram aqueduct (figure 116).—This is the largest aqueduct in the Madras Presidency and carries a delta canal over a large branch of the Gōdāvari River. The aqueduct consists of 49 spans of 39 feet and is 2,247 feet between abutments. As the riverbed level has not been interfered the work is essentially a bridge carrying water in place of wheeled traffic and apart from the question of level of canal bed the considerations determining the size of the spans and details of foundations and flooring are similar to those guiding design of an ordinary bridge. There is no uplift tending to blow up the flooring and this consideration does not, as in the case of a syphon, affect the settlement of the best length of span and make it desirable to use a comparatively short one. In a bridge over a large river such as this the use of small spans would unduly block the water way inducing scours and would be altogether undesirable. The flooring and apron up and downstream are those which would be required for a bridge with the shallow pier foundations provided in this work If deep well

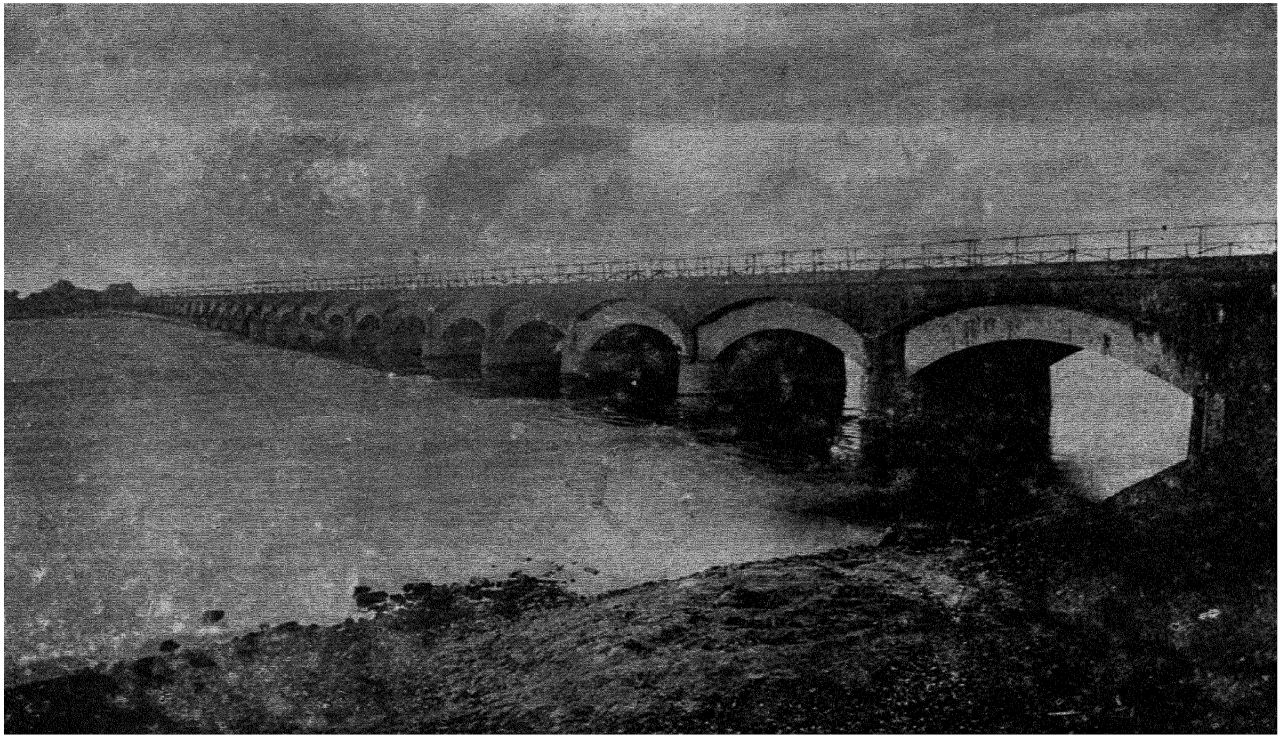


Plate IV.—Gunnaram Aqueduct.

foundations had been put in as is so generally done in railway bridges, it would have been allowable to dispense with protective pitching except round the piers and to allow the riverbed to scour out extra waterway in floods. The difference of level between the canal F S L. and the river ordinary level is 20 feet and the length of percolation round the wings appears to be decidedly short for so

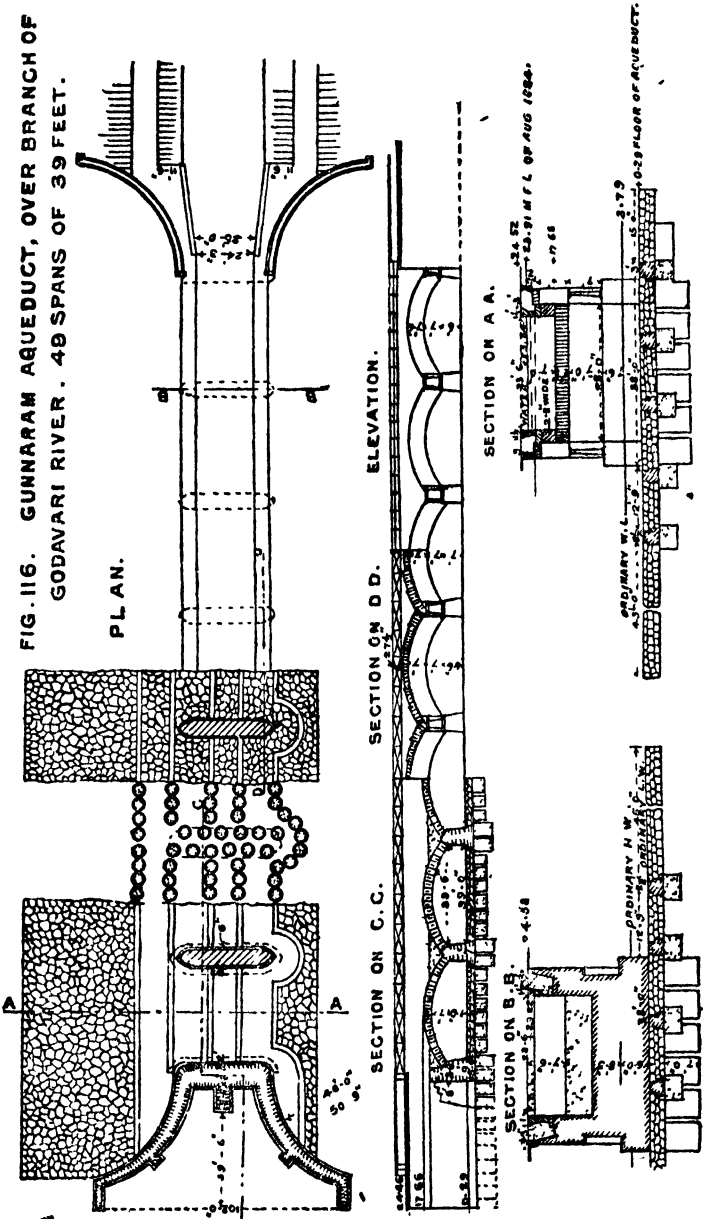
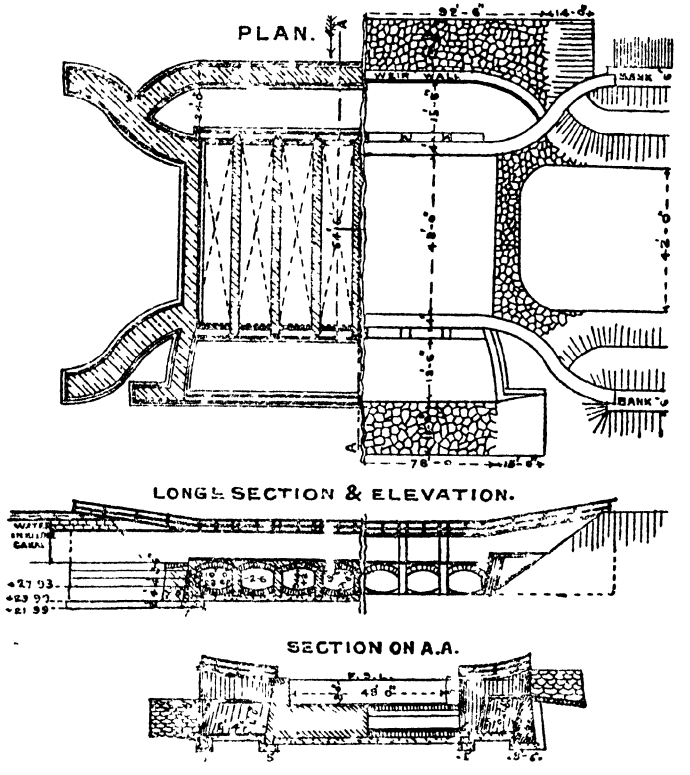


FIG. 116. GUNNARAM AQUEDUCT, OVER BRANCH OF GODAVARI RIVER. 49 SPANS OF 39 FEET.

great a difference of level; the work however has stood for many years and one can conclude that the bank and canal bed are highly impervious to percolation

Kesarapalli syphon aqueduct (figure 117).—Ellore canal, Kistna Eastern Delta.

FIG 117 KESARAPALLI AQUEDUCT, ELLORE CANAL.



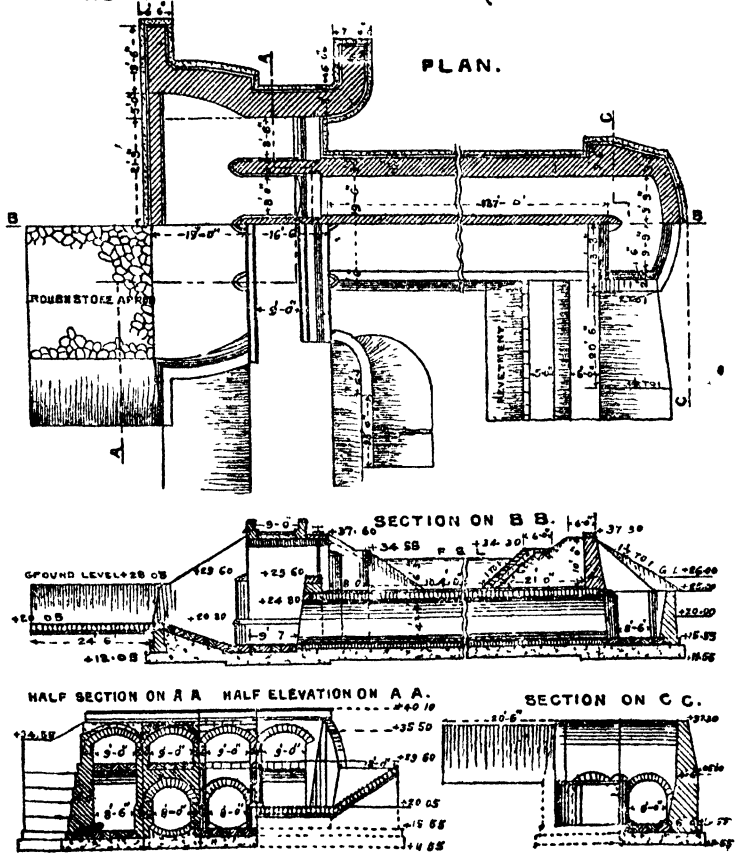
This work is of type III and is specially cited by Bligh as being a sound and economical design. The length of percolation round the wings in this case does not exceed a gradient of 1 in 3. The soil is however retentive black soil.

Combined syphon aqueduct and surplus sluices (figure 118).—This is a type II syphon aqueduct built on the Commamur Canal, Kistna Western Delta; on the left side of the aqueduct surplus sluices take the place of the canal bank.

532. Superpassages and syphons—The only distinction between these works and aqueducts and syphon aqueducts is that in the latter the canal is carried over the cross drainage, and vice versa in the former. The general features and principles of designs of superpassages and syphons closely follow those of aqueducts and syphon aqueducts. In both cases in order to make a satisfactory design of wings and earth connexions a plan of the banks

and other earth slopes must be plotted and the wings designed to suit them; this remark may indeed be taken as applicable to all irrigation works. The irrigation canal will sometimes approach the work in full cutting, and its banks will be merely spoil banks but frequently in the case of syphons this will not be so and the canal will approach the work with part of its water above ground. The drainage on the contrary will generally be unbanked and in such cases there is no advantage in carrying an earthen bank over the syphon culverts and these works should be of type III except where the drainage is embanked The respective level of the canal and drainage beds on the most suitable alignment of canal will generally determine which waterway will be passed underneath the bed of the other; where there is little difference of level it is generally advantageous to syphon the irrigation channel rather than the drainage, because as in an irrigation syphon there is little likelihood of serious obstruction by rolled silt and further the canal water is controlled and can be shut off, if silt clearance is desired. If a driving or riding road along a canal bank is

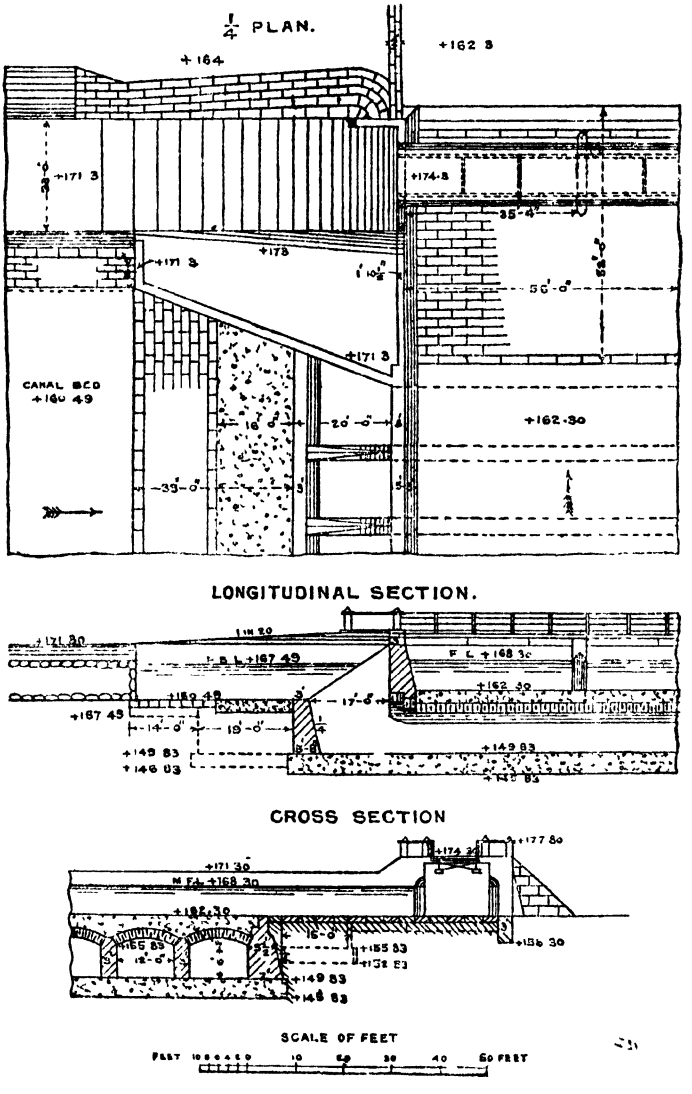
FIG: 118. COMBINED SYPHON AND ESCAPE, COMMAMUR CANAL.



required it will be necessary to construct a bridge at any super-passage or syphon across any drainage which cannot conveniently be forded.

Figure 119 shows a part plan and section of a canal syphon designed in connexion with the Madras Cauvery Reservoir Project to carry the main canal of capacity 3,572 cusecs under a drainage of 16·9 square miles. The level of the bed of the canal is 160·49 and that of the drainage + 162 3 The canal bank carries a carriage road on the left side and a bridge across the drainage is therefore

FIG 119. SUPER PASSAGE.



provided on that side. The canal is forced through the culverts at a velocity of 5·2 feet per second and requires a head of ⅔ of a foot to effect this. It is desirable to pass water through a syphon with considerable velocity to minimise silt troubles. The full supply level in the canal below the work is thus + 156·83 and uplift on the covering of the culverts is considerable being 9·5 feet (allowing half a foot for friction in the barrels) and a minimum thickness of 4½ feet masonry at the crown of the arches is allowed; this appears somewhat small and 5 feet thickness would be better. An alternative would be to use 1½ to 2 feet ferro-concrete slab covering bolted through the piers to the foundation in a similar manner to the Kao Nadi Syphon aqueduct (Fig. 114) above; this would allow the depth of the culvert floor to be reduced by about 4 feet and secure the reduction of the uplift pressures both on culvert covering and flooring by the same amount. The uplift head on the culvert floors is due to (167·49 — 145·83) = 21 66 feet. The design provides for a concrete flooring 4 feet thick to resist this. The dead weight of such a floor would balance 8 feet and the remainder of the pressure would be taken up by arch thrust If the floor were raised 4 feet its thickness might safely be reduced to 3 feet. The piers and abutments of the work could also be reduced in section if a slab covering anchored to the floor were used. It appears probable that the design revised on these lines would prove economical; at any rate it is worth consideration, and the working out of an alternative design on these lines would be an instructive example.

LEVEL CROSSINGS, INLETS AND OUTLETS.

533. Level Crossings—When a canal is taken across and through a drainage at or about the natural level of the drainage bed this is called a 'level crossing' The crossing may be regarded as the crossing of the canal by the river or vice versa and is effected by the construction of a regulator, or weir, or floor with falling shutters across the drainage just below the crossing place; a regulator is generally constructed across the canal immediately below the crossing.

The condition of a level crossing is thus very similar to that of head works, viz., a diversion work across a river with a head regulator on one flank; but in addition there is an infall in the shape of the upper section of the canal entering the river on a level with its bed, on the reverse flank of the diversion work. In the case of a level crossing the canal generally carries the greater part of the irrigation water, which is merely supplemented by the flow of the drainage, and it is thus important that the flow at its infall should not be obstructed by undue deposition of silt above the diversion work across the drainage. When the floods in the drainage are liable to rise to a level much higher than F.S.L in the canal above the infall it is occasionally necessary to provide a regulator across the infall to prevent the flood water and silt from the river flood being carried upstream into the canal. The regulator across the canal below the crossing is similar to the head sluice in ordinary river head works. The diversion work across the drainage may be an open regulator, or a weir with or without crest shutters, or a masonry floor with shutters, the crest in all cases being of the

height required to hold up water to the level necessary for the full supply of the canal below.

There are in South India many level crossings effected by means of solid anicuts or anicuts with dam stones but where this type of work is constructed silt troubles at the crossings are generally experienced, and a river regulator, or shuttered floor, or low weir with relatively high crest shutters, is much to be preferred to a solid weir. If a solid anicut is built it should be provided with efficient scouring sluices, preferably on both flanks.

Level crossings are frequently used in all sizes of irrigation canals but are specially common in crossing very large drainages where the cost of a syphon or syphon aqueduct would be prohibitive. In the case of small channels the only permanent work at a level crossing is frequently a regulator across the channel below the crossing the barrier across the drainage being an earthen bank washed away and replaced after every flood.

The works comprising this type of cross drainage work do not in any way differ from the works already described under "headworks" and no special typical designs are here given.

534. Inlets and outlets—Cross drainage water can be disposed of by an inlet which admits the drainage water on one side of a canal and an outlet which passes out an equal quantity of water on the opposite side. A level crossing is a form of an inlet and outlet with sills at the canal bed level.

535 Inlets.—Inlets are merely openings in canal banks admitting upland drainage water to the canal. If the drainage course is very shallow (say not exceeding 2 feet deep) or if merely surface drainage is to be admitted, the inlet may be a floor or low weir, similar to tank surplus works already described, over which water to a depth of from 1 to 2 feet may pass into the canal. The floor or crest of the low weir would be at M.W.L. in the canal and the canal should be aligned so that at the site of the inlet M W.L. is just about level with the natural ground surface. Where other than shallow drainages are admitted into canals through openings in the margins below F.S L the canal water will back up the drainage and, when drainage is not entering the canal, water will stand stagnant in the drainage channel. Where a road, or inspection path along the bank is necessary, deep inlets must be bridged; if there is any fall from the bed of the inlet to the canal bed a drop wall with apron or a pitched slope must be provided to give adequate protection against scour but if the inlet joins the canal at bed-level but little protection is necessary

Unless the drain entering a canal by an inlet is banked it is necessary to carry the canal at the site of an inlet in such depth of cutting that the F.S L. is at, or below the level of the ground on the margins of the drain, as failing this the canal water would submerge the ground on the upper side of the canal. As an alternative a drainage sluice can be placed across the infall the shutters of which are only opened when the water in the drain is at a higher level than the canal F.S.L.; the shutters may be hand operated or be automatic flap shutters similar to those shown in figures 123 and 124 below. Very small drainages are admitted into canals through pipes or culverts passing through the banks.

In South India during the rainy season the canals are generally running full and thus the outfall from an inlet is nearly always well cushioned and protective flooring or pitching required below an inlet is far less than in the case of a similar tank surplus work; a pitched floor and revetment of the canal slope as shown in figure 120-*a* is frequently all that is necessary, and in many cases even this can be omitted and the inlet then consists of only a rough stone floor at natural ground level with the ends of the canal bank at either end of the inlet protected by revetment. Figure 120 shows a low weir and figure 120-*a* a flush inlet of the types referred to.

FIG. 120. LOW WEIR INLET

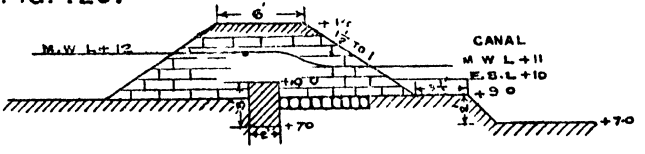
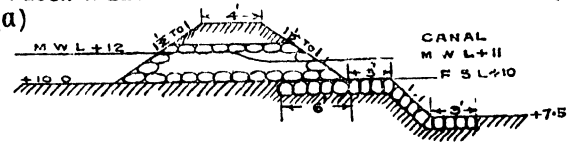


FIG. 120 (a) FLUSH INLET WITH REVETTED CHANNEL SLOPE.



Surface inlets are generally arranged so that the difference of level between the M W L. in the drain above the work, and that in the canal shall not be great, usually this is restricted to between 1 and 1½ feet

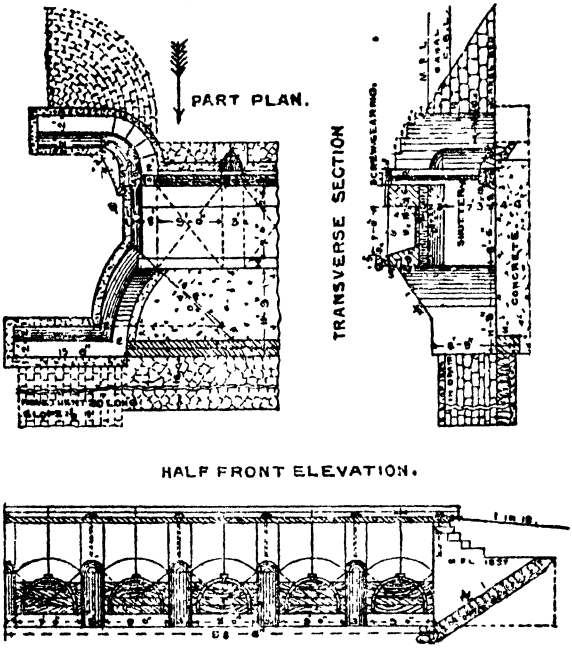
536. Canal outlets or escapes—These are surplus works which may be either flush escapes, weirs, or sluices, and may be built in connexion with and opposite to inlets, or as independent surplus works Outlets may be classified as surface outlets, which are only capable of discharging water from a level above ordinary F.S L. of the canal, and surplus or escape sluices, which can discharge water down to the levels of their sills

537. Surface outlets—Surface outlets may be weirs, or flush escapes, either with or without dam stones or crest shutters, and of design similar to corresponding surplus works of tanks The sites of such works are generally fixed by the facilities which exist for the disposal of the surplus water after leaving the outlet, and the design must be suited to the level and the fall of the bed of the surplus channel. In small irrigation channels, weirs and floors with dam stones are economical and effective forms of surface outlets; they operate in precisely the same way as similar works on tanks, but are not open to the objection, which applies to tank weirs, that their coming into operation entails loss of a considerable quantity of impounded water. Crest shutters should only be used for such works where special resident establishment is maintained to work them.

Surface outlets only permit of surplusing water from levels above their crest, and where this is at F.S.L., the depth of surplus is limited to the difference of level between F.S.L. and M.W.L. in the canal. It is generally desirable to keep this low and in most cases it is limited from 1 to 2 feet.

538. Outlet or surplus sluices.—Sluices, used as surplus escapes, must be designed to suit the level, bed fall, and nature of soil of the outfall channel, and if this channel has to be specially constructed, its bed level and fall must be fixed with reference to that of the drainage into which it outfalls.

FIG. 121. NALLAMADA SURPLUS SLUICES.
IN LEFT BANK OF COMMAMUR CANAL.



It is a great advantage to have the sill of a surplus sluice, and bed of the outfall channel, at (or below) the canal bed level, as this enables the canal to be completely emptied, and also facilitates removal by scour of any accumulation of silt in the canal bed in the vicinity. When practicable, surplus sluices should be constructed where there is an existing drainage, with its bed level at or below the canal bed level, into which it can safely discharge either direct or through a short artificial channel. The conditions governing design of outfall sluices are similar to those of other regulators or sluices, aprons being required to give bed and marginal protection to suit the conditions of volume and velocity of flow, while protection against excessive percolation and uplift must be provided on the usual lines both in respect to impervious aprons, and bank connexions.

Surface outlets and surplus sluices may be combined by making sluice openings in the walls of surplus weirs, or constructing surplus sluices without face walls, and with the level of the top of their shutters at about F.S.L., so that surplus water can pass over their tops of the gates when closed (vide figure 121). Surplus sluices can be installed very economically in the side walls of aqueducts, while, by keeping the top of the walls at F.S.L., these can be made to serve as surface outlets

Figure 118, page 313 shows a syphon aqueduct with surplus sluices, and in figure 117 the side walls of the aqueduct form a surface outlet.

The sluice shown in figure 121 is typical of the usual form of the outlet sluices and canal regulators in Madras. It will be seen that in this sluice there is a sill raised 2 feet above the canal bed although the outfall channel is at canal bed level. This feature of the design is open to criticism and it would have probably been better to have the sill at the canal bed level.

539. Outlets built in connexion with inlets.—Unless the drainage likely to be received into a canal through an inlet is so small, in comparison to the canal capacity, that the water can be absorbed in the canal, it is necessary to construct an outlet to dispose of it. The minimum discharge capacity of such an outlet will be fixed by that of the inlet, but it may occasionally be given greater capacity to enable it to dispose of excess water brought down by the canal. Generally an outlet is built in the canal bank directly opposite to the inlet, but occasionally it is more convenient to build it below the site of the infall and in the latter case the carrying capacity of the canal above the level of the sill of the outlet, must not be less than the maximum discharge from the inlet Surface outlets are automatic in action, but surplus sluices in connexion with inlets should, as in the case of tanks, only be relied on where operating establishment, resident at the sluice site, is maintained.

540. Objection to disposal of cross drainage by inlets and outlets —The great objection to disposal of cross drainage by inlets and outlets is the quantity of silt, especially bed-silt which is thereby carried into the irrigation channel; where this system of crossing drainage is adopted the obstruction by silt deposit is frequently very great, and there are many isolated irrigation channels in South India from which, in order to maintain supply, extensive silt clearance at the site of every inlet has to be undertaken after every considerable fall of rain. Where an outlet sluice with a deep sill is built in connexion with an inlet much of the silt brought into the canal can be passed off through the outlet, but in default of establishments these works cannot be relied on, and surface outlets have frequently to be provided for small channels in spite of the great cost and inconvenience resulting from silt deposit.

541. Outlets not connected with inlets.—Escape sluices having no connexion with cross drainage works are provided in long canals to enable water to be reduced before this could be effected by regulation of the head sluices. If the sills are at, or below, canal bed level these works are also useful for scouring purposes and for quickly draining the canal when closed for

repairs. Escape sluices should be spaced at suitable intervals throughout the lengths of long canals, sites being chosen for preference above reaches in which liability to breaches or dangerous leaks is greatest; the sites however must be located where suitable outfall channels for the surplus water can be arranged for at reasonable cost. No rule can be given for the discharge capacity of each sluice, but it need seldom be greater than half that of the canal, or less than the full supply flow of the canal at the site of the work less its full supply flow at the next lower outlet. Surplus sluices are chiefly required after very heavy falls of rain in the irrigated area, at such times cultivators block the pipe outlets to their field channels when breaches must occur unless the canal water levels can be promptly lowered

542. Outlets for canals carried in single bank.—The unbanked lengths of canals where carried in single bank form practically continuous surface inlets To dispose of the water so received outlets of suitable spacing and capacity must be provided.

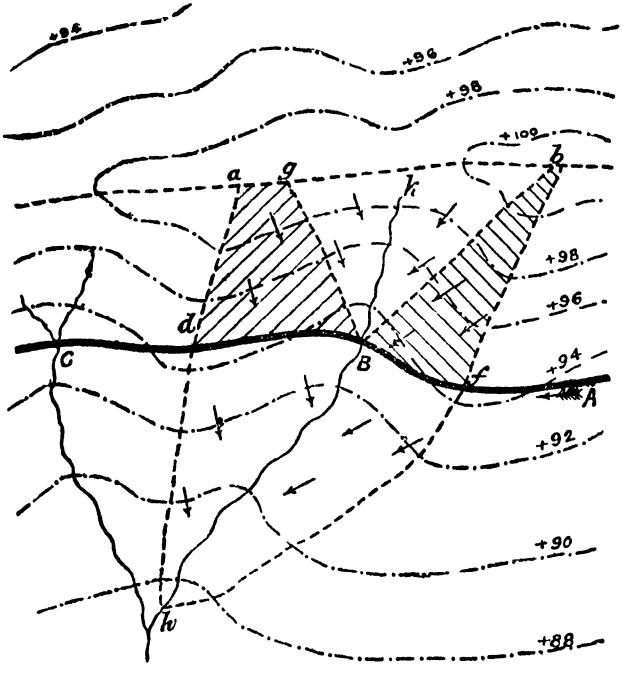
543 Canal scouring sluices —It is very desirable that a scouring sluice should be provided in the vicinity of any inlet through which any considerable quantity of heavy silt is brought into a canal The sill level of such a work should be at or below the bed of the canal, and the bed of outfall channel should also be below the bed level of the canal and have such a slope as to enable it to carry away any silt scoured from the canal bed; if a drop can be given from the canal to the outfall channel so much the better Where the water entering a channel through an inlet is disposed of by a weir or flush outlet, scouring vents in the weir or a special scouring sluice auxiliary to the surplus works should be provided, wherever practicable, to dispose of the silt carried through the inlet into the channel.

544. Scouring sluice in the first reach of main canal.—A certain quantity of heavy bed silt almost invariably finds its way through the head sluices into the first reach of a main canal and it is on this account most desirable, where suitable facilities exist for disposal of the outfall water at a suitable level, to have a scouring sluice between one and three miles from the head of a canal; such a sluice if properly worked will be of great assistance in keeping down the bed level of the first reach of canal. Deep sills, wide spans, and high velocities of discharge, are desirable in scouring sluices, and extensive pitching down-stream of these works is generally necessary.

545 Diversion of natural surface drainage by contour irrigation canals.—Figure 122 shows the nature of the local interference with surface drainage which must result where a canal, railway or road is carried across the lines of drainage. The canal line A B C crosses a drainage at B, and the cross drainage work at B must provide for the run-off of the catchment area (*a d f b*) intercepted, and the whole length of outfall channel from B to *h* must be able to carry all this run-off *plus* the run-off gradually accruing from the area *d h f* below the line of canal. Both before and after the construction of the canal the required capacity of the drain at *h* would be that due to the area *a h b* but before the construction of

this canal the run-off from this area into the natural drain *k h* is that indicated by the arrows and the drain would receive surface drainage throughout its length and would require capacity gradually increasing between *k* and *h* and the capacity at *B* the point of canal crossing would be that due to the area *g B b*.

FIG. 122.



The construction of the canal would have the effect of diverting to the point B the drainage of the cross lined areas, which previously joined the drain in driblets at different places along its course between B and *h*, and the drain at B must in future carry a run-off due to the area *a d f b* in place of that due to the area *g* B *b*, which was all it had previously to carry Thus although previous to construction, the drain may have been of sufficient cross section throughout its length, it *may* be necessary to widen or deepen the drain between B and *h*, the enlargement being a maximum at B below the canal, gradually diminishing to *nil* at *h*

The quantity of surface drainage diverted from its natural course to the site of the canal cross drainage work in the case of large drainages is generally small in comparison to the whole run-off of the drainage, but in the case of small drainages, such as that illustrated in the figure, the reverse may be the case and the matter requires consideration in connexion with cross drainage works.

546. Catch drains.—Where a considerable quantity of surface drainage is intercepted by a contour canal a catch drain graded to lead the intercepted surface drainage to the nearest cross drainage work should be provided clear of the outer toe of the canal bank.

547. Tail escapes.—It is desirable to have some form of automatic surplus escape at the tail end of a canal or channel to enable excess water to be surplused without causing a breach. This end would not be satisfactorily met by leaving an open end to the canal because the required water level at the tail could not be maintained and the tail pipe outlets would not get their proper supply, and also because the arrangement would entail waste of water

The ordinary form of tail escape is a weir across the channel with sill at F.S L of the channel ; such a work holds up water to the required level and only allows surplus water to pass. Where no masonry tail escape is provided the tail of the channel should be cross bunded, the top of the bund being lower than the channel bank so as to form a breaching section.

DRAINAGE WORKS.

548 Drainage of the irrigated area.—In South India the incidence of rainfall is such that the irrigation supply given to any area in a short time, say a week or less, is small in comparison to what is frequently precipitated by heavy falls of rain ; it follows, therefore, so far as it concerns the general drainage of any area, that if natural means of drainage is adequate before the introduction of irrigation and no interference with the drains is caused by the works the natural drainages will, after introducing irrigation, still be adequate

It is essential in laying out an irrigation system to see that natural drainage courses are reserved for drainage purposes and are not obstructed by works more than is absolutely necessary, and that adequate cross drainage works are provided whenever a channel has to be carried across a line of natural drainage. In very flat localities generally, and always in deltas, there exist considerable areas the natural drainage of which is bad and where water stagnates forming marshes Much land of this kind, although unfit for dry cultivation, is even in its undrained state fit for wet cultivation in a year of normal rainfall, while artificial drainage will make such land capable of carrying wet crops even in years of abnormal rainfall and will enable lower lying land to be cultivated in normal years.

The artificial improvement of drainage by enlargement of existing and construction of new drains must therefore form a feature of irrigation projects which include in their scope the irrigation of such low lying lands as are above referred to

549. Grading of drains.—The difference between the level of the water into which a drain discharges and of the land to be drained may be termed the drainage head, and where this is sufficient a drain should be graded to secure a non-silting velocity of flow ; indeed the velocity need only be limited by what would entail inconvenient scour of the bed or margins of the drain.

The maximum surface slope which can be given to a drain is the drainage head divided by the length of flow measured along the course of the drain, and it is where the drainage head is small relatively to the length of the drain that serious difficulties in regard to drainage are met with,

Where a drainage system discharges into a river it is possible to get an increase of slope for a drain by moving the outfall to a site lower down the river, so long as the slope of the former is less than that of the latter

Where the outfall is into the sea, or into a tidal estuary in which the mean daily level of water at the outfall varies but little from mean sea level, the only methods of increasing the slope of a drain are either to shorten its course, or to raise the surface level of the upper reaches of the drain, which entails, excluding from the drainage scheme the lower lying lands in the upper parts of tract to be drained.

The very fluctuating quantities of flow in drains and the restricted fall in flat areas not only lead to the deposit of silt but also to the growth of weeds and grasses, which is most difficult to effectively control and which greatly obstructs discharge; it is on this account desirable wherever the drainage head is small to exclude from a drainage scheme any specially low lying blocks, the inclusion of which would have the effect of unduly restricting the slope of the drain

The ebb and flow of tide in a stream is useful in preventing the deposit of silt and growth of weeds, and it is therefore desirable to keep the tail reaches of drains falling into tidal water open to the run of the tide wherever this does not entail submergence of cultivable lands by salt water

The level of water at the outfall of a drainage system in tidal water may usually for purposes of design be assumed as the mean of the daily water levels. Where the outfall is into a river great fluctuations in level occur at irregular intervals and the level at the outfall may sometimes be taken as the mean level of the month of greatest river discharge; it is however impossible to give a hard and fast rule for this, and the special conditions of each case must be considered on their merits.

The bed fall of a drain will conform generally with its surface slope and the depth will be limited by practical considerations in relation to difficulties of excavation, especially subsoil water levels Where a drainage is open to the run of the tide it is desirable to cut the tail reach as deep as possible, as tidal action is much more powerful in a deep than in a shallow channel of the same cross section.

The difficulty of keeping open an outlet from a drain or backwater to the sea owing to the tendency to the formation of a sand ridge or bar across the outfall has already been referred to in Chapter IV and it is, on this account, better to locate the outfall of a drain in a tidal estuary which keeps permanently open, rather than attempt to open an outfall direct to the sea, although the latter may be the cheapest and shortest line.

550. Banks of drains.—The surface of the water in drains which traverse the lands draining into them must be below the level of these lands and although the earth from the excavation may sometimes be utilized by being spread out in low lying places, it will generally have to be disposed of in spoil banks on one or both sides of the drain; it is necessary to leave numerous openings in such spoil banks to serve as inlets for the drainage of the fields.

The water surface in the tail reaches of drains are frequently carried at levels higher than those of the lands through which they run, and unless the lands are unoccupied waste the water must be prevented from spreading out over the country by continuous banks, and the lands in these limits must either be left undrained, or drained to some lower outfall.

Both continuous and discontinuous banks should be set well back from the edges of the drainage cuts leaving very wide berms; this leaves a clear water way for the disposal of exceptional floods and gives facilities for widening the drains should this be at any time found desirable.

551. Drainage outfall sluices.– Where the level of ordinary floods, or of high tides, is sufficient to submerge cultivated lands by backing up of water through the drainage outfall it is sometimes necessary to build a sluice across the outfall with shutters, either automatic or hand-operated, which are closed, so as to prevent the river water backing up the drain, whenever the water level at the outfall is higher than that in the drain, and opened when these conditions are reversed. The general design of such sluices is similar to outlet sluices from a canal and when fitted with hand-worked shutters these also are similar to outlet sluices or regulators Automatic working is secured by shutters hung on horizontal axes along their upper edges; such shutters are termed *flap shutters* and are closed by water pressure whenever the water below the outfall is at a higher level than the water in the drain and they open when the pressure acts in the reverse direction. The great objection to automatic shutters is that drift is likely to get between the shutter and the sill or sides of the vent and prevent the proper closing of the shutter and when there is much drift in the river this prevents effective working. Tidal outfall sluices must open and close twice every day and for these it is almost essential to have automatic shutters but for outfall sluices which only require closing in time of high floods hand-worked shutters are generally preferable.

Figure 123 shows an open drainage outfall in the Coleroon river flood bank which has been converted into an outfall sluice with wooden flap shutters.

FIG. 123.

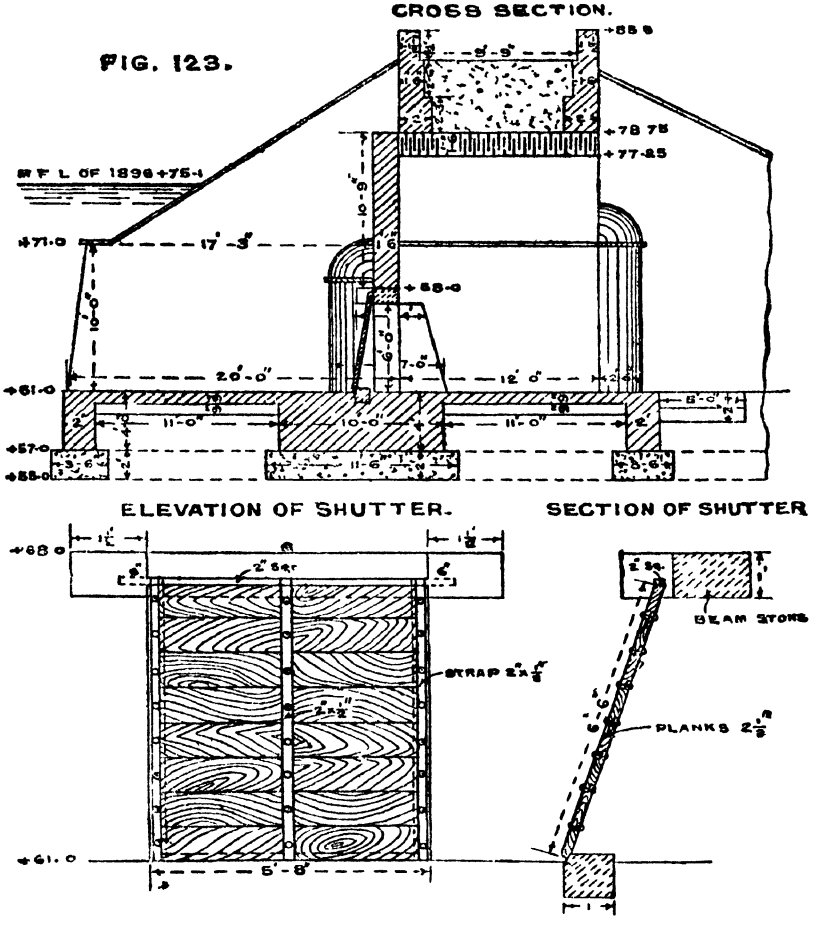
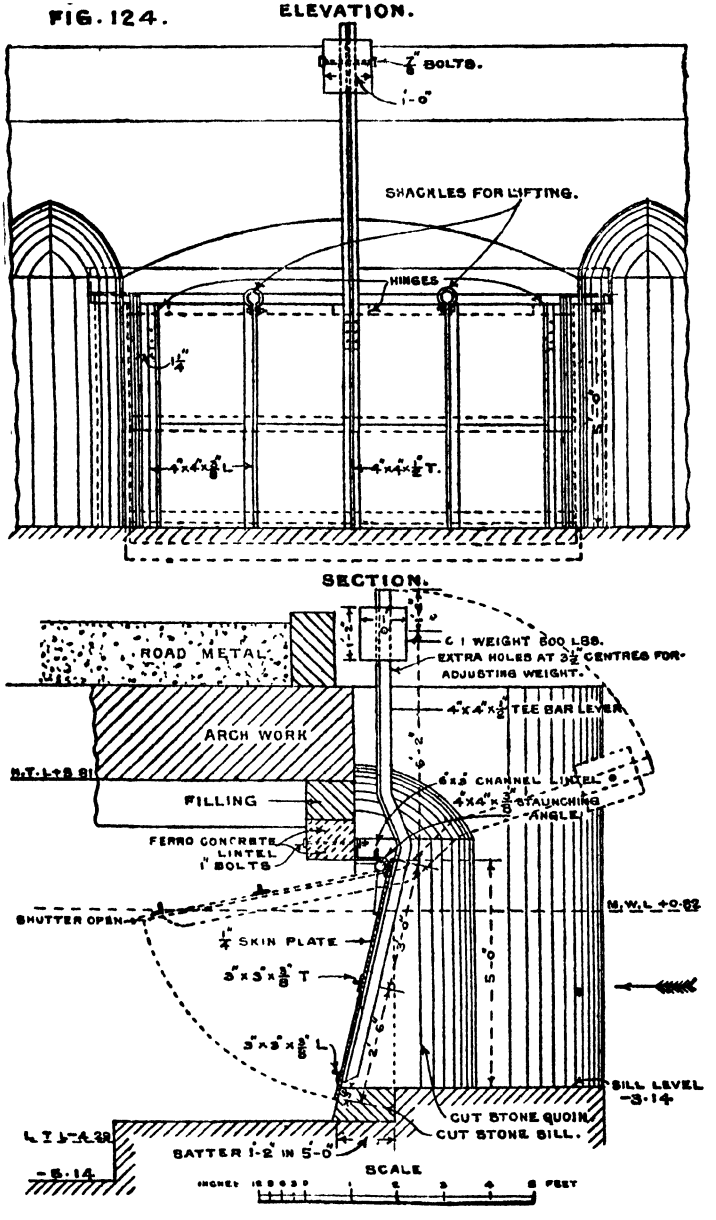

CROSS SECTION.
+85.5
+78.75
+77.25
W F L OF 1896 +75.4
+71.0
17'-3"
+58.0
+61.0
20'-0"
12'-0"
11'-0"
10'-0"
11'-0"
+57.0
+55.0
ELEVATION OF SHUTTER.
SECTION OF SHUTTER
+68.0
STRAP
BEAM STONE
PLANKS
+61.0
5'-8"

Figure 124 shows a balanced iron flap shutter recently designed by Mr. W. P. Roberts of the Madras Public Works Department and fitted to a tidal outfall at Masulipatam.

ROBERTS BALANCED SHUTTER FOR OUTFALL SLUICE

FIG. 124.



The latter design is very suitable for a tidal outfall as being counterbalanced it opens and closes with a very small difference of water level on either side of the shutter.

In designing tidal outfall sluices they must be given extra discharge capacity to allow for the proportion of each day during which the shutters are closed.

552. Capacity of drains.—It is unnecessary to cut a new, or enlarge an existing drain to a capacity equal to the estimated maximum run-off; a drainage channel which can carry within its margins the run-off of the ordinary heavy rainfall meets the requirements of rice cultivation. Rice is of the nature of swamp cultivation and normally grows in water from 2 to 6 inches in depth and an increased depth of water does not affect the plants unless wholly submerged, while even complete submergence for a few days does not cause material injury; also the fields in which rice is cultivated are all levelled and surrounded by banks of one foot or more in height. Thus an area growing rice has the power of impounding several inches of rain, and of spreading out the run-off over a much larger period than that in which it falls. The natural ground fall in deltas and tracts in which drainage stagnates is generally very small and this still further increases the capacity of such areas for impounding rainfall and extends the time of run-off Although therefore rainfall in the irrigated areas of the great delta systems of Madras sometimes amounts to 8 to 10 inches in 24 hours, corresponding to a co-efficient of about 450 in Ryves' formula, the delta drains are designed to give a discharge due to a co-efficient of 92 in the Kistna, and 138 in the Cauvery Deltas; this is equivalent to 2 and 3 inches, respectively, in 24 hours for a standard area of 5 square miles. Although the maximum intensities of rainfall in the above deltas do not differ greatly, the Kistna area is flatter than the Cauvery and thus having greater absorbing capacity a lower co-efficient is applicable to it

It is not from motives of economy alone that the capacities of drains in flat alluvial or deltaic tracts are restricted as above described. If such drains were originally dug out to discharge the run-off of the heaviest rainfall as quickly as it is received they would normally carry water at so shallow a depth and so low a velocity that it would be impossible to keep them clear of weeds and aquatic growths and they would quickly become choked and ineffective.

553. Combined irrigation and drainage channels—The primary functions of irrigation and drainage channels are antagonistic, the former requiring water to be carried at a higher level than the lands irrigated and the latter requiring the water level to be below that of lands drained, yet there are a number of old channels in South India which act in both capacities, sometimes for the same lands, but more frequently for lands differing in level in so slight a degree that when the channel carries water at irrigating level it cannot effectively drain. In such cases both irrigation and drainage must be carried on by turns, the channel acting as a drain when closed for irrigation and vice versa. It is not considered desirable in modern systems to permit a drain to be used for irrigation as it is likely to lead to disputes and irregularities; while undoubted inconvenience is caused by the practice it nevertheless enables drainage water to be utilized for irrigation which would otherwise flow into the sea.

The primitive method of using one channel for both drainage and irrigation is to cross-bund the channel for irrigation purposes and breach the bund when the turn comes for using the channel as a drain. This method is still used in many small channels but it involves excessive labour and constant and recurring injury to the regime of the channel and is moreover open to many abuses. In small channels effective regulation may be secured by masonry regulating notches, the regulation being effected by planks inserted in grooves in the sides of the notches. In South India regulating notches are generally called 'regulating dams.'

In larger drainage channels weirs with dam stones, or with notches or sluices in their body walls are used for the same purpose; these are however open to the objection that after heavy rain they greatly obstruct rapid flow-off and drainage suffers accordingly. There are a great many such works in the older irrigation systems of South India especially in the Cauvery Delta but of late years most of the old dams across these drainages have been converted into open regulators and the drainage of the delta thereby greatly improved

The regulator in figure 99, page 272 above, is a work of this nature, and the great difference between M.W.L. and F.S.L. shows what large quantities of water have to be passed after heavy rain

CHAPTER XIV.

NAVIGATION COMBINED WITH IRRIGATION WORKS.

554. Navigation canals.—It is not proposed in this text-book o deal with canals intended solely for navigation but only with rrigation canals adapted for navigation purposes. The requirements for navigation are waterways with sufficient cross section ınd depth for the passage of boats of the dimensions proposed to ɔe used, supplies of water sufficient to keep the canals deep enough for the transit of boats, and a velocity of flow which does not seriously impede navigation. These conditions to some extent conflict with irrigation requirements which are best met by a cross sectional area of canal gradually diminishing as the area to be rrigated diminishes, and a velocity of flow which will be both ıon-silting and non-scouring.

555. Limits of velocity of navigation canal.—The ideal ıavigation canal is a series of level reaches with vertical steps in he form of locks at intervals to suit the slope of the country traversed A surface fall and current are essential to an irrigation canal, but as the difficulties and cost of navigation are greatly ıncreased as the current increases, in a combined irrigation and ıavigation canal this should be kept at the lowest non-silting velocity suitable for the required depth.

In the Gōdāvari and Kistna Deltas navigation is successfully carried on in velocities as high as 2½ feet and sometimes even as much as 3 feet a second but considerable extra cost in towing ɔoats is experienced when velocity exceeds 2 feet a second.

556. Depths and widths of navigation canals.—The canal ɔarges in Madras when fully laden draw nearly 3 feet of water ınd the minimum depth which permits free navigation is 3 feet, ɔut as haulage is much lighter with a foot of water under the boat ı depth of 4 feet or upwards is an advantage

The minimum bed width of canal which can be navigated by ɔrdinary Madras canal barges may be taken as 20 feet and considerable inconvenience is experienced in such narrow waterways. The necessity for such minimum width and depth for navigation entails cross sectional areas being given to the tail reaches of some canals much in excess of the requirements of irrigation, in addition o extra capital cost such excessive cross section entails deposit of silt and considerable additions to maintenance charges owing to he reduction of velocity of flow.

557. Extra water required for navigation.—A certain bed ınd surface slope has to be given to canals to enable them to pass the water required for irrigation during periods of maximum demand, but the canals must be run at sufficient depth to maintain throughout their lengths the minimum depths of water required for navigation and when the demands for irrigation are small this entails passing water for navigation in excess of irrigation requirements.

558. Special works required for navigation purposes.—Apart from the increased sections which may be required for certain reaches of canals the combination of navigation with irrigation makes the following works necessary:—

(*a*) a lock wherever there is a drop in the canal;

(*b*) combination of regulators with drops across the lines of navigation ;
these combined regulators and drops are called ' lock weirs' ;
(*c*) the provision of fender piles at weirs, surplus and other large sluices,
(*d*) the maintenance of a ' tow path ' throughout the navigable length of canals ;
(*e*) the provision of sufficiently large spans and of sufficient headway in all bridges crossing the canal to enable traffic to pass without inconvenience, and the provision of bridges across lock channels wherever there are bridges across the lock weirs.

559 Canal locks.—A canal lock is a short chamber cut off from a canal where there is a drop, the chamber is closed at each end by water tight gates which can be opened so as to permit boats to enter or leave the lock chamber from either end. One end of the lock chamber opens on to the canal above the weir and the other end opens below it Sluices at either end permit the lock chamber being filled from the upper reach or emptied into the lower, both sets of gates being closed.

560. Lock floors and side walls —A lock is usually built on a heavy flooring or foundation of masonry which is laid all over the whole area of the work forming the floor of the lock and the foundation of the side and drop walls.

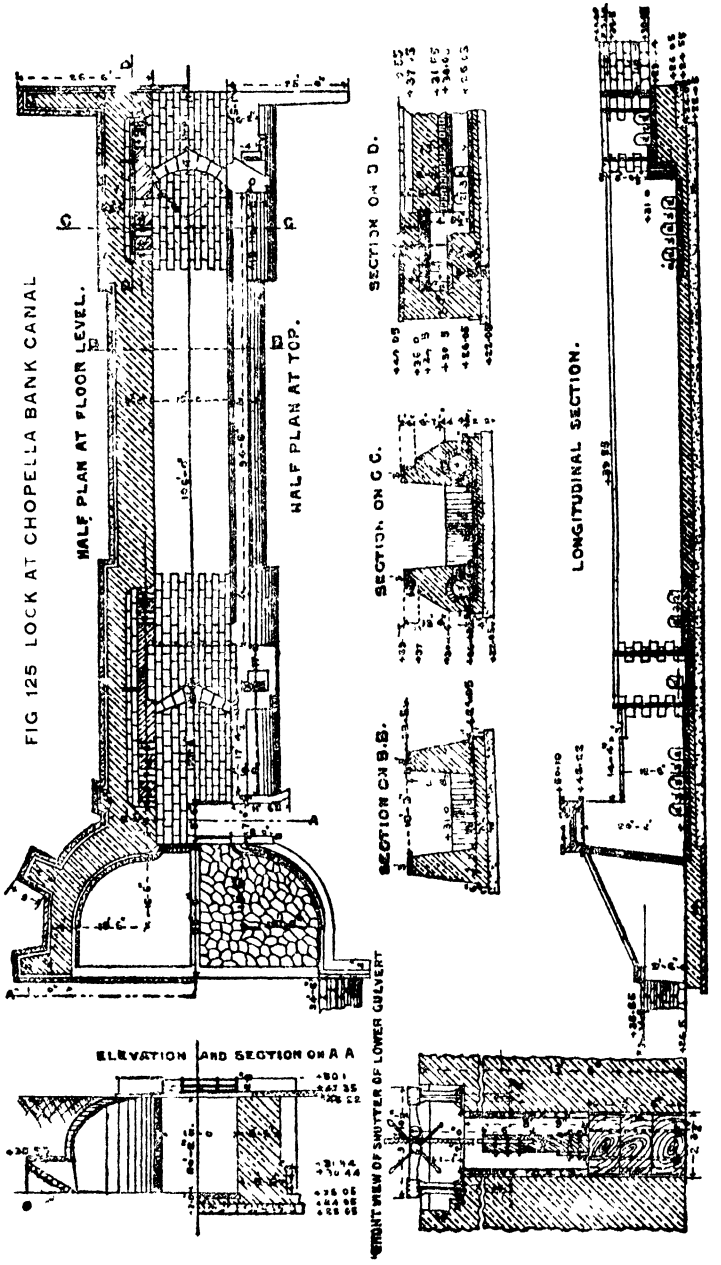
This slab must be of sufficient thickness to carry the weight of the side walls and to resist the uplift from earth and water pressure on the under side of the floor The sides of lock chambers are usually retaining walls with their outer faces vertical ; these walls are recessed at either end to form compartments into which the gates swing so that they do not, when open, project beyond the faces of the lock side walls

There is a tendency for locks to crack longitudinally along the centre of the floor and this is specially likely to happen during construction when there is no weight on the lock floor. This is due to the uneven loading on the foundation and it is desirable as a precaution during construction to fill in the earth behind the side walls as they rise so as to bring the thrust of the earth behind these into operation as this reduces the inequality of loading. The side walls of locks should be given heavier sections than ordinary retaining walls of corresponding height ; ordinarily the pressures may be taken as equal to those on a dam holding up water to the top of the wall but cases have occurred where these have been found insufficient. The tops of lock side walls have to withstand collision from laden barges and should not be less than 2½ feet thick.

561. Dimensions of lock chamber.—The minimum dimensions of a lock chamber are fixed by the dimensions of the largest sized vessels required to pass through the lock Where traffic is considerable these dimensions are increased so as to enable the lock to take at one time two or more of the ordinary size of barges using the canal In the Kistna and Gōdāvari deltas in Madras locks on the lines of navigation are classified according to the size of the lock chamber as follows.

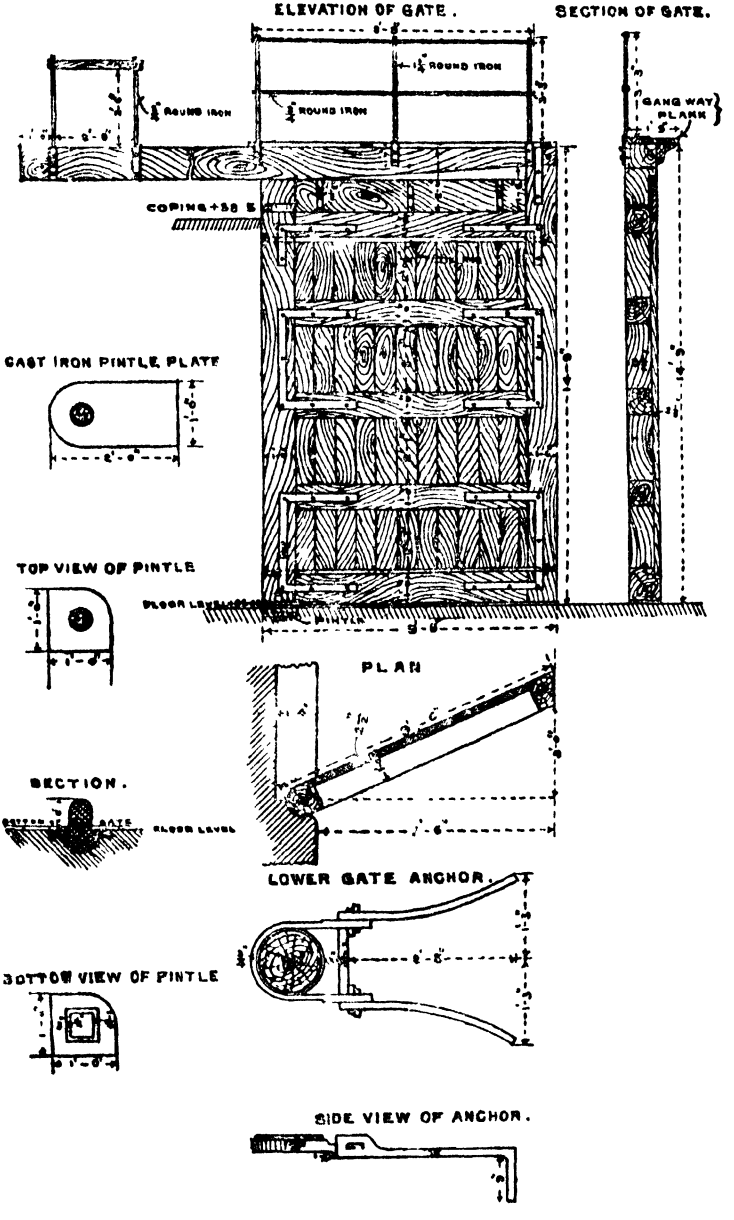
First class locks	...	150′ × 20′
Second class locks	..	105′ × 15′

562. Chopella lock, Godavari Eastern Delta—Figures 12 and 126 show the details of Chopella Lock, Bank Canal, Gōdāvari Eastern Delta



The floor, side walls, and wings are of the normal type, while a road bridge is provided just below the lock chamber which is in line with the bridge over the lock weir The upper and lower lock sills are at the bed levels of the canal above and below the weir and this is the ordinary arrangement.

FIG. 126. DETAILED DRAWING OF GATES FOR LOCK AT CHOPELLA ON THE BANK CANAL.



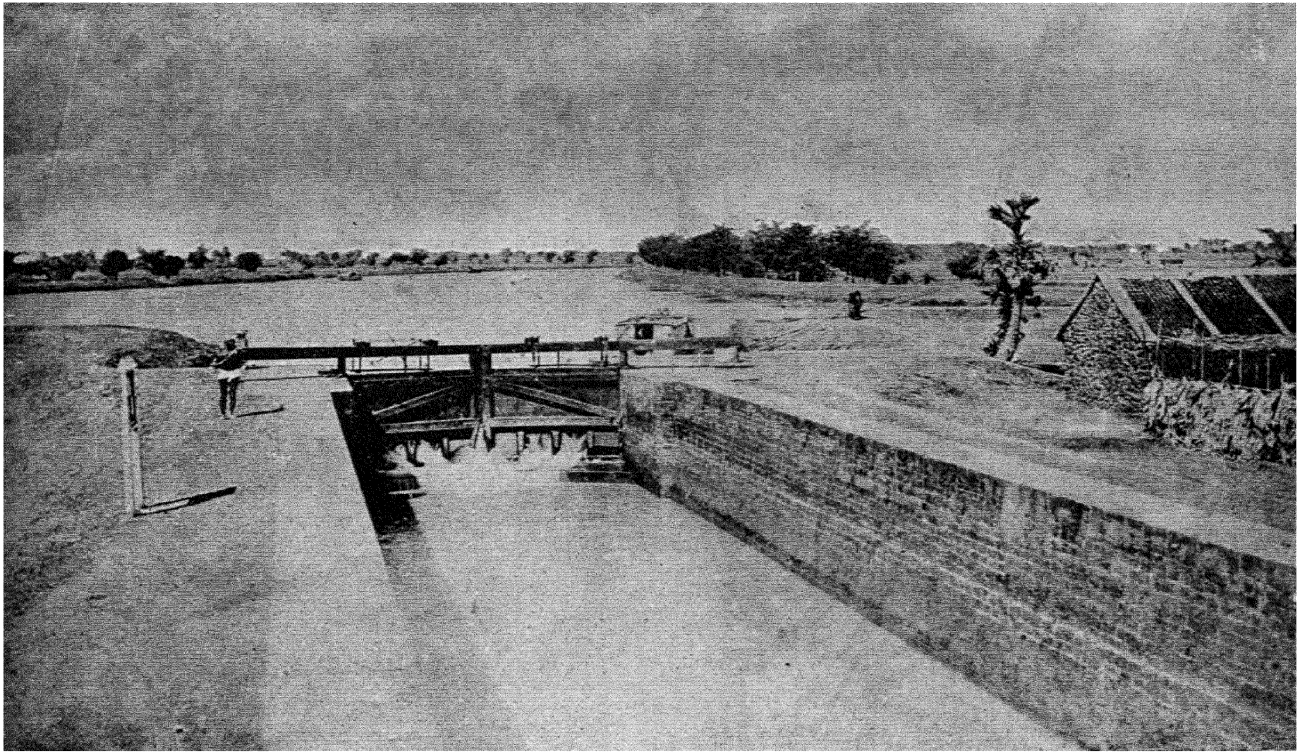


Plate V.—Lock, Godavari Delta System.

563. Lock sluices.—The inlets and outlets for water to fill or empty the lock chamber are either openings in the sheeting of the lock gates just above the sill beams, or culverts passing through the masonry side walls behind the lock gates. Figure 127 shows the former type and figure 125 the latter.

In both cases the operating gear of the sluice shutters, which is usually of the rack and pinion type, is installed either on the upper part of the sluice gates, or on the ground over the culvert as shown in figure 125. Rack and pinion gear is preferred to screw gear for lock sluices because it can be more quickly operated and so saves time in passing traffic through locks. The size of sluice ways should be proportioned to the capacity of the lock chamber so that the filling and emptying of the lock can be accomplished in a reasonable time

No hard and fast rule as to this time can be given, as the amount of traffic to be dealt with has to be considered but the limit of time for filling or emptying generally accepted for the Madras canal locks is about 3 minutes

564. Lock gates.—The usual type of lock gates is made in pairs, pivoted on a vertical axis in a recess in each abutment, the combined width of the pair of gates is greater than the width of the lock entrance so that the outer vertical edges of the gates meet in an angle pointing up-stream and the thrust of the water against the gates is transferred along the horizontal beams of the gates to the masonry abutments

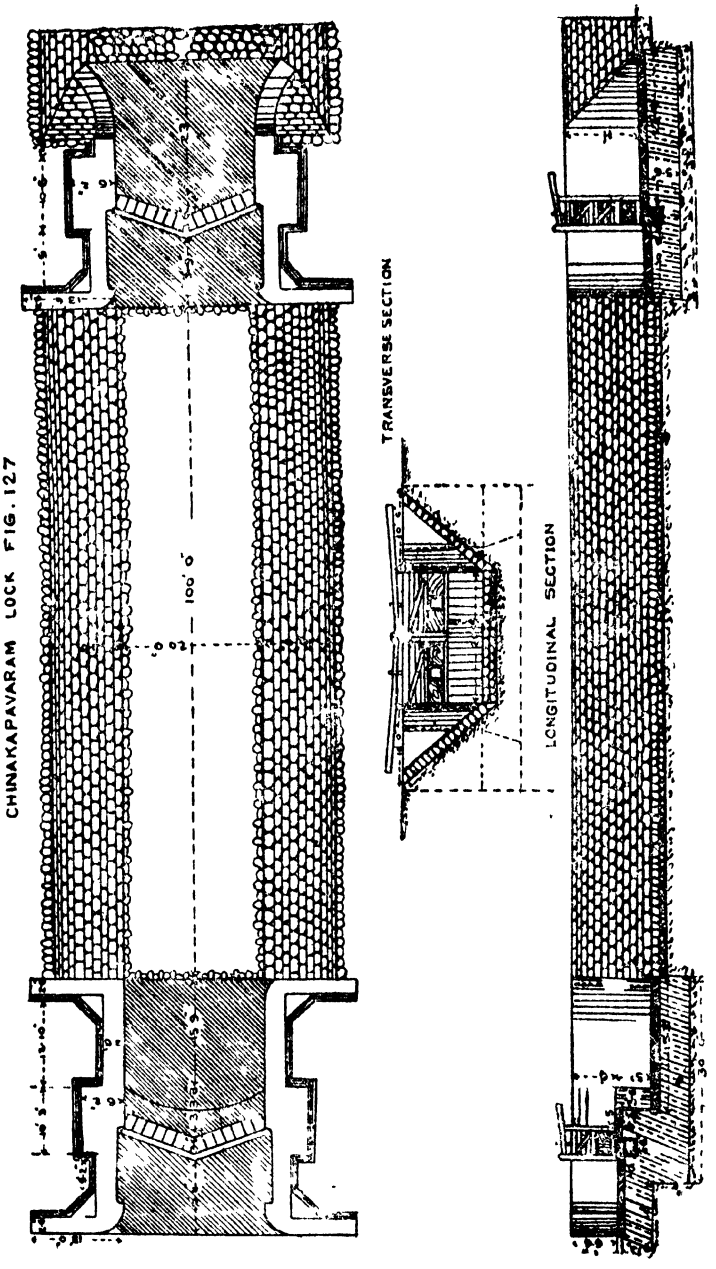
Figure 126 shows a pair of lock gates with the shoes and pintles forming the pivots on which the gates revolve and the collars in which each end of the heel post of each gate revolves; these collars are secured to straps anchored in the masonry by wedges which afford means of accurate adjustment The sills of the lock at each end are raised in a low step against which the gates press when closed, the steps thus form a stop for the gates. The riser of the step is frequently faced with wood in order to reduce shocks to the gates when closing and to make a closer joint between the sill and the gates

The topmost beams of the gates project beyond the heel posts and form levers by which the gates are opened and closed. The large section of the projecting beam is to form a counterbalance to the weight of the gate and for this reason the beam is called the 'balance beam' of the gate. The pivot posts are the 'heel posts' and the outer posts the 'mitre posts' Lock gates are generally opened and closed by means of balance beams but occasionally rack gear is provided for this purpose Large lock gates are generally made of steel but for the ordinary sizes of gates required in South India wood is the most suitable material. A gangway plank as shown in figure 126 is fixed to the top of each gate to enable the lock staff to cross from side to side of the lock when the gates are closed.

565. Tidal locks.—Where tail locks discharge into a river or an estuary in which flood or tidal water rises above the canal F.S.L. it is necessary, in order to exclude such water from the canal, to provide at the upper end of the lock two pairs of lock gates facing in opposite directions and to raise the lock walls and bank connexions to a level above highest flood. Lock gates which

are subjected to the action of salt water should be covered with sheet copper up to one foot above high tide level as a protection against marine borers.

566. Locks with sloping sides.—The cost of locks is considerable and the liability, especially in clay soils, to dangerous



cracks resulting from unequal pressures is considerable. Both the above are greatly reduced by making the lock chamber with earth sides sloping at 1 to 1 and revetted while the floor of the lock is stone pitching. A lock of this economical description is shown in figure 127. The only solid masonry in such locks would be the drop wall, sills, and the abutments for the lock gates with the wings which must be of sufficient length to be an effective defence against failure by percolation. A sufficient length of impervious floor above and below the lock sills must give protection against failure by percolation under the sills. This type of lock is open to the objection that the sloping sides are inconvenient and somewhat dangerous for the boats in the lock when the water is falling, and that the quantity of water required for filling the lock is increased. The former is a serious objection, but the latter is seldom of any importance in an irrigation canal although frequently of great importance in a canal intended solely for navigation. Revetted chamber locks were generally built in the navigable canals of the Gŏdāvari delta when they were first made but have now been almost universally replaced by locks with side walls. It may be noted that the lock sluices in figure 127 are in the lock gates and this is the most economical arrangement.

567. Double locks.—When the fall at a lock exceeds about 10 to 12 feet a double lock is usually employed. This is a lock with 2 chambers with a set of lock gates and sluices between the upper and lower chamber so that there are altogether 3 pairs of lock gates. By this means the total fall is divided into two steps and the head on each set of gates reduced

568 Mooring posts in locks.—All locks should be furnished with a number of mooring posts, or ring bolts fixed in the masonry, to which the vessels passing through the lock may make fast; chains at intervals hanging down from the top of the side walls are also desirable both for the convenience of boats and to afford a support to any one who may accidentally fall into a lock.

569. Site of locks and lock weirs.—The ordinary canal drop extends from side to side of a canal and the approach and run-off are quite straight and on the axis of the canal; where a lock is introduced it is necessary to have an approach for boats separated in some way from the approach to the fall, that is to the lock weir. The general lay-out of the works may be arranged in three ways:—

(*a*) The weir may be directly across the line of canal and the lock on a diversion (figure 128-*a*) or

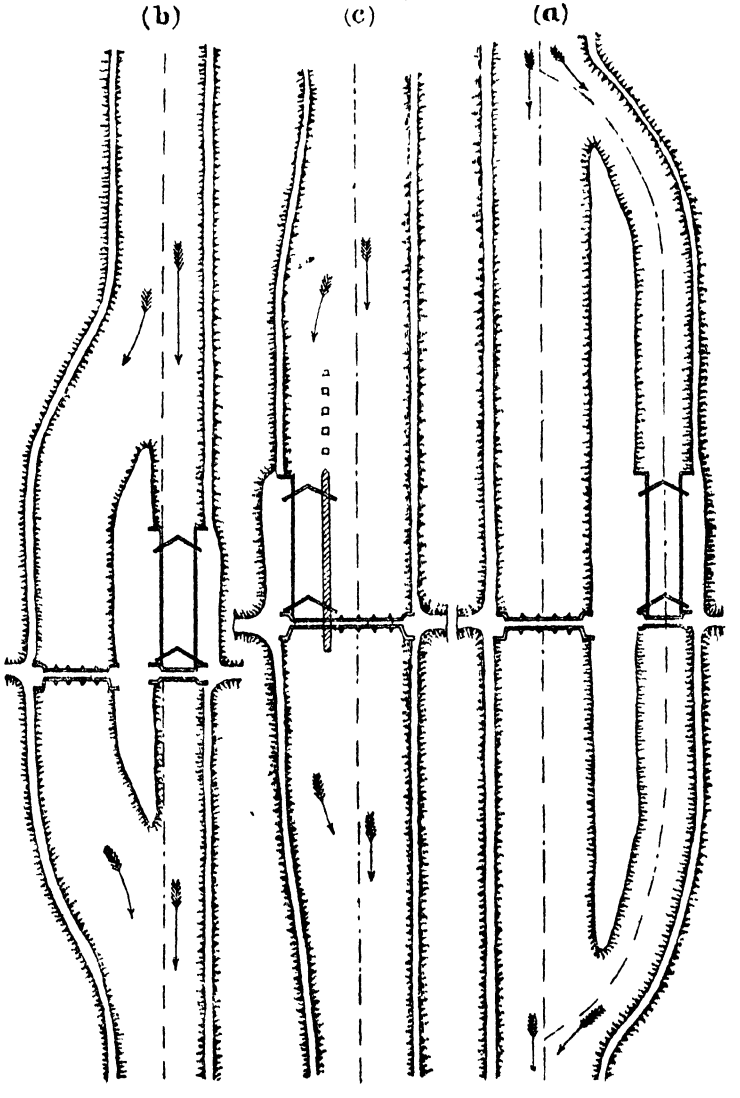
(*b*) the lock may be on the direct line of canal and the weir on a diversion (figure 128-*b*) or

(*c*) both lock and weir may be side by side forming one work across the canal, the approach to the weir being separated from that to the lock by a row of fender piles (figure 128-*c*).

As regards (*a*) and (*b*), the former is preferable on a canal when the weir stream is of very large size in comparison to the lock chamber, but generally (*b*) is preferable. Both the above arrangements especially (*a*) are subject to trouble by the silting of the navigation channel between the point where this leaves the weir

channel to where it again rejoins it. The reason of this is obviously the reduced flow in comparison to the sectional area of this channel, which, while facilitating the handling of the vessels entering the lock, results in silt deposition which, unless relieved by occasional clearance, will obstruct navigation at low water stages The arrangement of figure 128-*c* to a great extent gets over this difficulty at the expense of a more difficult approach to the lock. With a suitable disposition of fender piles and mooring posts all reasonable security can be given and on the whole this arrangement will generally be found most suitable.

FIG 128

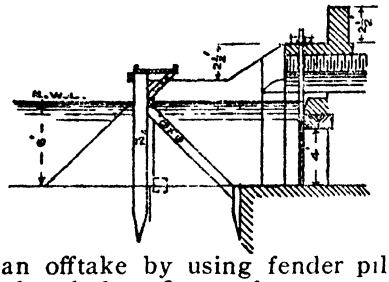


570. Lock weirs.—These are combined drops and regulators which enable the depths of water above the lock to be regulated in accordance with the requirements of navigation while at the same time passing to the reach below, the quantities of water required for irrigation. When the demands of irrigation are small, it is frequently necessary to reduce the surface fall in a reach of navigable canal by holding up water at its tail above ordinary F.S.L., while maintaining water at the minimum navigable depth at its head. This reduces the quantity of water required to maintain navigation but interferes with the *regime* of the canal. Although trapezoidal notches may automatically carry out the necessary irrigation regulation at a drop a regulator is absolutely necessary when navigation is combined with irrigation.

Figure 93, Chapter XII, shows the ordinary form of lock weir built in the Gōdāvari canal system.

571. Fender piles.—In a navigable canal fender piles are necessary across the face of surplus sluices or any offtakes from the canal where the draw is likely to cause difficulty to navigation. Such piles may be either of wood or ferro-concrete and should be well strutted and the heads may be conveniently joined by a waling piece fixed at about F.S.L. It is frequently convenient to construct a light tow-path bridge across the approach channel to an offtake by using fender piles as piers. The piles should be placed about 8 to 12 feet apart on the line which the edge of the water would occupy if there were no offtake. Figure 129 shows a section of fender piles carrying the tow-path in front of a surplus sluice.

FIG. 129. FENDER PILES.

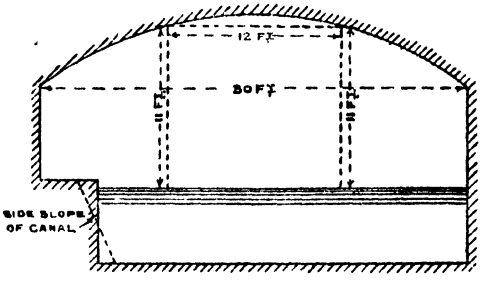


572. The tow-path.—All canals large enough to carry navigation are provided with continuous inspection paths and the extra cost of tow-paths is generally small. Where the canal berms are above F.S.L. the tow-path can most conveniently be along the edge of the berm; in other places where the canal berms are of normal width the tow-path can run along the top of the bank; where there are wide berms below F.S.L. a tow-path 2 feet wide and 1 foot above F.S.L. may be made along the inner edge of the berm. Where there are bridges across the canal a tow-path ½ to 1 foot above F.S.L. and 4 to 5 feet wide should be carried through the side span of the bridge. Light foot-bridges should be provided where necessary to carry the tow-path across cuts leading from inlets or to offtakes.

573. Bridges on navigation canals.—Bridges of navigation canals must give sufficient width and headway above M.W.L. or high tide level to allow of navigation without inconvenience. The actual width and headway on each canal must be decided with reference to the nature and dimensions of the vessels to be accommodated. In Madras the minimum span is 25 feet or 30 feet inclusive of the tow-path and a headway in the centre of the span is allowed of not less than 11 feet for a width of not less than 12 feet,

On account of the requirements of headway girder bridges are generally more convenient than arched bridges on navigation canals. The angles of the tow-path abutment of a bridge should be rounded, rendered with neat cement worked quite smooth within the limits where the towing rope will rub against the angle. This prevents the fraying of the rope and also the damage to the masonry by the continual rubbing of rope. Figure 130 shows the skeleton section of an arch with tow-path and minimum headway from the above rules.

FIG 130.



574. Desirability of combining navigation with irrigation — The question whether navigation should be provided for in irrigation canals is one on which there is much difference of opinion among experts The combination necessitates extra cost both of construction and maintenance of works and entails compromises both in regard to design of works and regulation of flow which affect the efficiency of the canals for irrigation purposes. It is impossible to make an accurate estimate of the extra cost of construction and maintenance of a combined system which is due to navigation, but the separation of the charges and revenue due to irrigation and navigation is made, on a somewhat arbitrary basis, in the accounts of the Government systems in Madras and these display navigation as an unprofitable undertaking. There are, however, many indirect profits from navigation which do not appear in such accounts, and these specially accrue in flat alluvial and deltaic tracts in which stone is not found and in which cart traffic in rainy weather is almost impracticable except on metalled roads. In such tracts the existence of navigable canals enable the produce to be moved at a fraction of the cost of moving it by road, and also affords a cheap means of transport for stone and building material throughout the tract served by the navigable waterways. Considering the great quantity of stone required for construction, maintenance, and extension of irrigation works and for the construction and upkeep of roads it is evident that very large reductions in original and maintenance charges of such works must be realized where a good system of navigable canals provides the cheap conveyance of stone and other building materials. Such savings in construction of public works are not brought to account when considering the returns realized on the navigation works, neither are the far greater aggregate savings realized by private persons from the facilities for cheap transport of produce so afforded.

Where the surface of the country traversed by canals is freely undulating, and stone and other building materials are available throughout the irrigated tract, the indirect credits due to navigation would be much less while the cost of works debitable to navigation is likely to be more than in a delta, or other flat alluvial tract; on the whole it appears probable that in a large delta system the combination of navigation with irrigation should generally be provided for, but in upland systems there will be many cases in which such a combination will be undesirable.

CHAPTER XV.

REGULATING SHUTTERS AND GEAR.

575. The varieties of shutters and gear used for regulating the flow in irrigation works are very numerous and, with one exception, only those types in ordinary use in Madras will be described.

576. Weir crest shutters. – Crest shutters may be classified into two types: –

(*a*) Falling shutters.
These when not in operation lie flat upon the weir crest.

(*b*) Lift shutters.
These are operated from a platform above the weir crest and are precisely similar to the shutters of open regulators.

KISTNA ANICUT FALLING SHUTTERS.
EACH 10 FEET LONG.

FIG. 131.
SECTION THROUGH THE SOLID CAST IRON SHOE (ENLARGED)

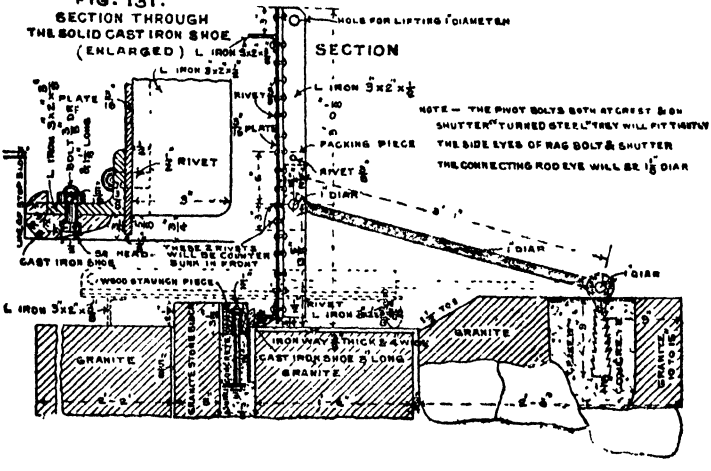
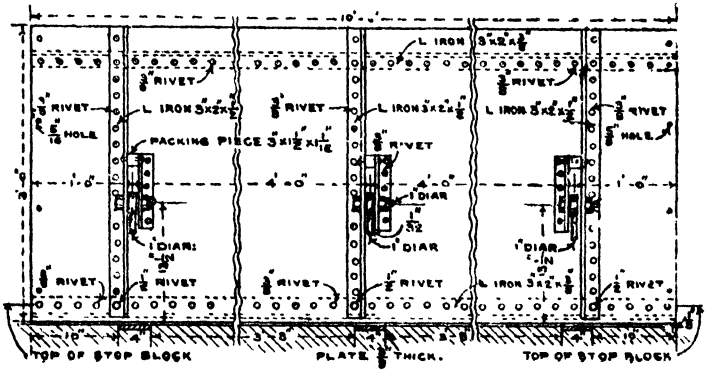


FIG. 131(a) FRONT ELEVATION.



577. Fouracres' fallings shutters.—This type of shutter was first applied to the Sôn weir in Bengal and was subsequently utilized for the Kistna and Gôdâvari anicuts; it is the only type of falling shutter which has hitherto been applied to any of the large works of Madras.

Figures 131 and 131-*a* show the section and front elevation of the Kistna anicut shutters.

Each shutter is 10 feet long and 3 feet high and is maintained in the upright position by 3 tie bars, one end of each bar is attached by a movable pin joint to an eye bolt anchored in the weir crest up-stream of the shutter and the other end is similarly attached to a point on the shutter a little below the centre of pressure of water when the shutter is on the point of overturning.

When water rises to such a level that the moment of the water pressure about the points of attachment of the tie bars becomes sufficient to overturn the shutter it falls into the position shown in the dotted lines in the section and lies flat on the crest, offering no obstruction to flow. The shutters thus fall automatically but they have to be raised, by hand and it is found that this can be done by men walking along the anicut crest when not more than two feet depth of water is passing over it.

One objection to the above type of shutter is that any obstruction or accumulation of silt lying against the up-stream side of the shutter is likely to prevent it from acting and it is unsafe to staunch leakage under or between shutters of this kind by depositing silt up-stream As attempts to staunch leakage are likely to be made by interested ryots the shutters should never be installed except where under the supervision of resident establishment and thus the automatic working which is the chief advantage of this type of shutter is to a great extent nullified. Another drawback to this type is that individual shutters are very likely to be overturned by wind and wave action at times when the general level of water is considerably below the level at which it is desired that the shutters should fall, much inconvenience is experienced from this and in practice a considerable working margin of level must be allowed. Shutters which on a still day will only fall with water passing ½ foot deep over them will sometimes on a stormy day be overturned even when the general level of water is below the shutter crest.

578 Computation of position of pivot of Fouracres' falling shutters—

Let X be the required height (in feet) of the pivot above the bottom of the shutter.

„ B the height of the shutter in feet.

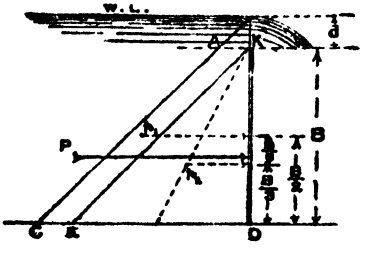
„ *d* the depth in feet of water over the shutter which is to cause it to fall.

„ *f* foot pounds = $62\frac{1}{2}$ F = the moment required per foot run of its length to overturn the shutter.

Then referring to figure 132 the pressure on the shutter with a depth (*d*) of water passing over it is represented by the area of the figure AKDC = AKEC + KDE.

These pressures p_1 and p_2 are respectively of magnitude $d\,B$ applied at a vertical height $\frac{B}{2}$ above the foot of the shutter, and $\frac{1}{2}\,B^2$ applied at $\frac{B}{3}$ above the foot.

FIG. 132.



The total pressure (P) is therefore $(dB + \frac{1}{2}\,B^2)$ and the centre of pressure is at a height above the sill of $\frac{1}{3}\,B + \frac{B}{6}\left(\frac{dB}{dB + \frac{1}{2}\,B^2}\right)$

$$= \frac{1}{3}\left(\frac{B^2 + 3\,dB}{B + 2d}\right)$$

The shutter will fall when the centre of pressure is at such a height (A) above the pivot that $A \times P = F$, the required overturning moment:

$$\therefore F = \left\{ \frac{1}{3}\left(\frac{B^2 + 3\,dB}{B + 2d}\right) - X \right\} \times (dB + \frac{1}{2}\,B^2)$$

Reducing, $X = \frac{B^3 + 3dB^2 - 6}{3B^2 + 6\,dB} \left[= \frac{B^3 + 3dB^2 - 096}{3B^2 + 6dB} \right]$ . . . . . (70)

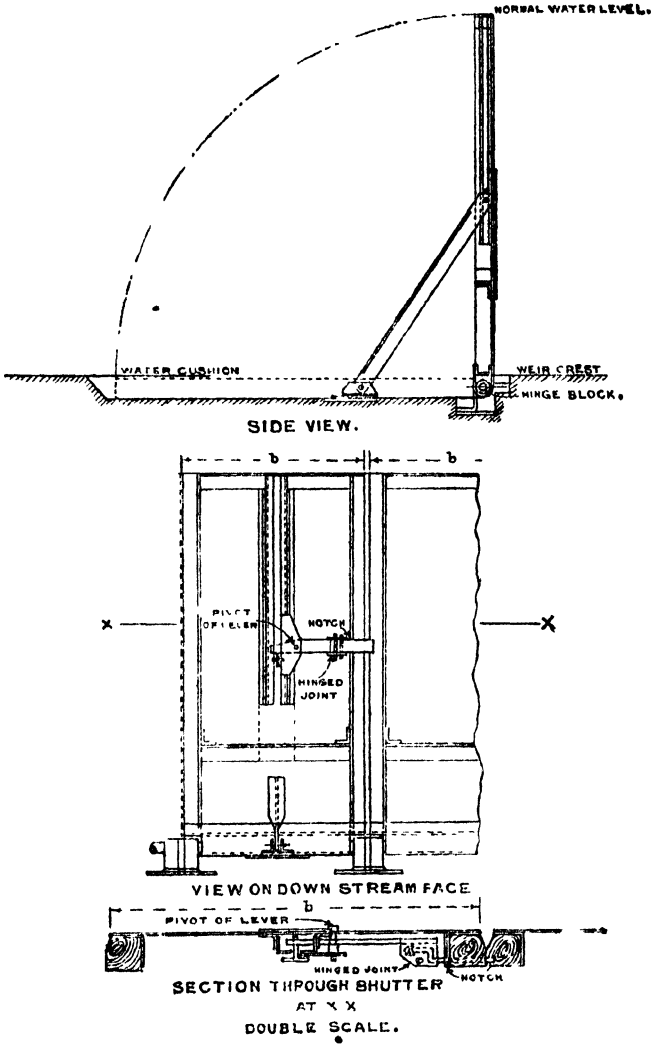
The Kistna anicut shutters are designed to fall with from $\frac{1}{2}$ to 1 foot of water passing over the shutter. The pivot is $1\frac{1}{8}$ foot above the bottom of the shutter which is the height of the centre of pressure when water is standing at $\frac{1}{2}$ foot above the shutter crest level

Applying the formula to this case it will be found that with water at $\frac{1}{4}$ foot above shutter crest level, the overturning moment (f) equals 17·6 foot lbs. per foot run, or 176 foot lbs. per shutter, and this is about the ordinary overturning moment of such a shutter.

579. Ashford's patent drop shutter.—This shutter shown in figure 133 is an example of a falling shutter attached to the weir crest by hinges along its lower edge. Each shutter is held up by a tie rod or by a strut which can be released by the operating staff.

FIG. 133

ASHFORD'S PATENT WEIR "DROP" SHUTTER.



Ashford's drop shutter has been much used in Northern India and it is claimed that on the approach of a flood these shutters can be released with ease and speed by a single man. So far these have not been installed on any weir in Madras.

Each shutter is supported by a single strut one end of which is connected by a pin joint to a plate fixed in the weir masonry, and the other end slides in a groove in the back of the shutter and cannot leave the groove. When the shutter is raised the end of the strut is retained in the corresponding position by the short end of the lever bar shown in the figure. The lever bar is locked in the horizontal position retaining the strut by making a jointed end on the longer limb of the lever and slipping this end into a notch in the shutter frame; the hinged end of the lever projects beyond the frame so that the next shutter when falling may push it out of the notch and thus unlock the lever and release the end of the strut. The shutters are installed in one or more sets working from one end and all the shutters of each set are released in succession by the falling of the next adjacent shutter, only the end shutter being released by hand

The shutters are 3 to 4 feet wide and have been successfully operated up to six feet in height.

They are raised, as in the case of the Fouracres' shutter, either by men walking along the crest or by a travelling crane running on rails laid along the length of the weir; the force required to lift such shutters is greater than for the Fouracres' shutters. The former have the disadvantage that the short length of each shutter necessitates a great number of vertical joints with the consequent leakage, they are however not likely to jam owing to the staunching of the joints and the system of operation appears to be reliable and effective

580. Large span lift shutters.—Large span lift shutters of open weirs, regulators or sluices are usually hung on chains from an overhead platform on which winches for lifting and lowering are installed Occasionally the winches are mounted on travelling carriages so that one set may work several vents in succession but in Madras it is usual to install separate fixed gear for each vent.

Such shutters vary from 15 to 40 feet span and are constructed of a framing of built-up girders or of rolled steel beams with a facing of sheet steel. They are moved on rollers of about six to eight inches diameter either running on axles which are fixed to the shutter, or mounted in a frame or cage entirely separate from the shutter which runs on the surface of the rollers. The latter device was invented by Stoney and is known as 'Stoney rollers' or 'free rollers'.

Rollers running on fixed axles were largely used in Madras when large span shutters were first introduced, in the works in the Cauvery delta and elsewhere but latterly only free rollers have been applied to this purpose.

Figures 134 and 134-*a*, *b*, *c* show the arrangement of a set of shutters recently constructed in the Public Works Workshops, Madras, for a river regulator combined with a road bridge. The shutters are 18½ feet clear span and 6½ feet high, they have frames of rolled steel beams and angle irons are suspended by plate link chains which pass over sprocket wheels, the shutters are hung on one end of the chains and the other ends are attached to counter-weights which run in grooves in the masonry piers. The movement of a body transported on rollers is half that of the

FIG. 134.

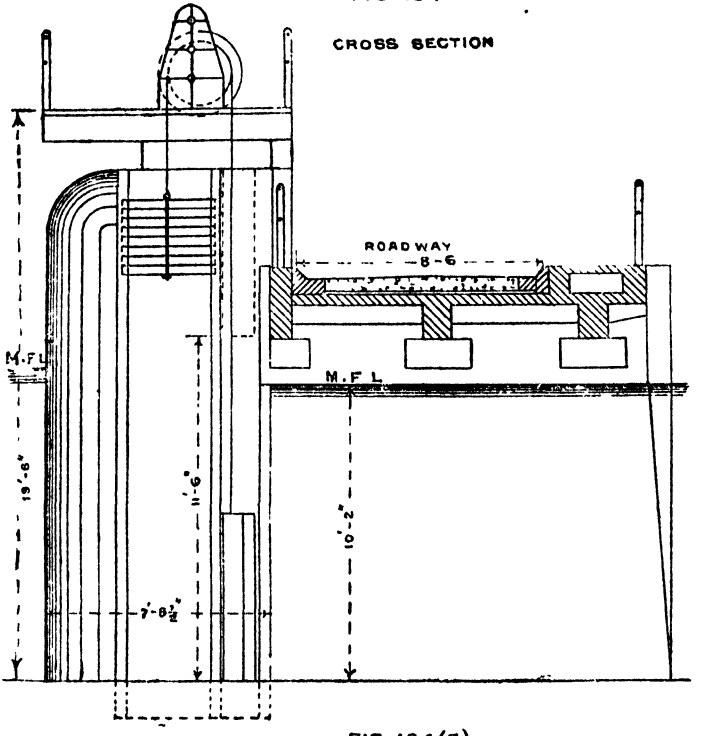
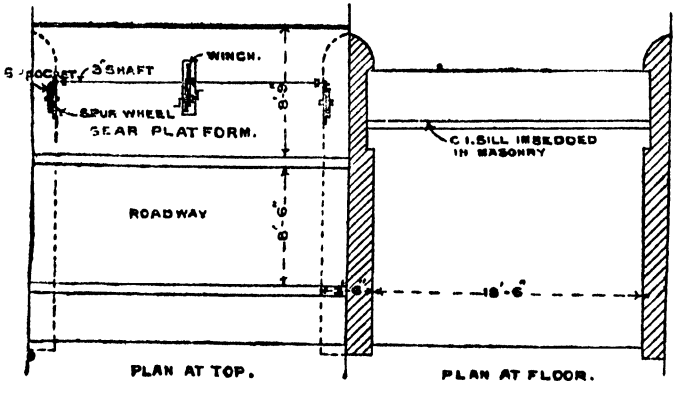


FIG. 134(a)



rollers themselves and for this reason the roller cage is suspended from a pulley block through which is passed a ½" diameter wire rope one end of which is fixed to beams of the lifting platform and the other end to the movable counterweights ; this ensures that the travel of the cage shall be half of the shutter. The sprocket

FIG. 134(b).

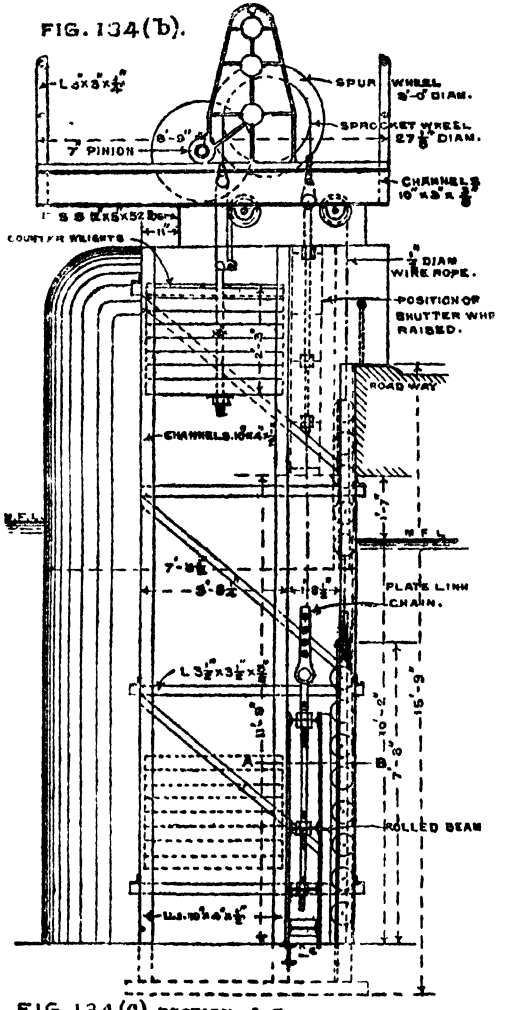
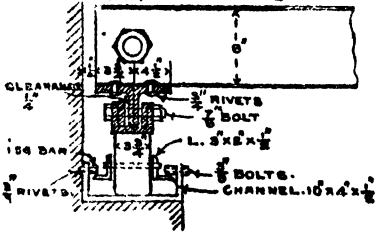


FIG 134(C) SECTION A.B.



wheels are connected by spur gearing and 3′ shafting with a winch fixed above the centre of the span by means of which the shulter is raised or lowered.

The roller path is loosely attached to the shutter and takes its bearing on the projecting edge of the angle irons of the shutter frame. This arrangement is introduced to ensure that the load shall bear truly on the roller surfaces and not be affected by any deflection of the shutter under the load

Similar arrangements of free rollers and gear are applied to reservoir shutters of comparatively small span worked under great heads. Such sluices are generally of small spans not exceeding 6 to 7 feet and the necessary area of vent is obtained by increasing the vertical height of the sluice so far as may be necessary.

Reference to figures 30 and 30-(*a*), Chapter VII, shows the general arrangement of the Stoney sluices in the Assuan dam.

The arrangements as regards lifting gear, counterweights, etc., for shutters running on fixed rollers may be similar to the above

When both are in good order the effort required to move the shutter under any pressure on fixed rollers is much greater than on free rollers and in both cases is only a small fraction of the effort required to move the shutter along plane surfaces against sliding friction. Mr. Ashford states that the following proportions of effort to pressure have been found by experiment.—

Free Rollers $\frac{1}{360}$

Fixed Rollers $\frac{1}{240}$

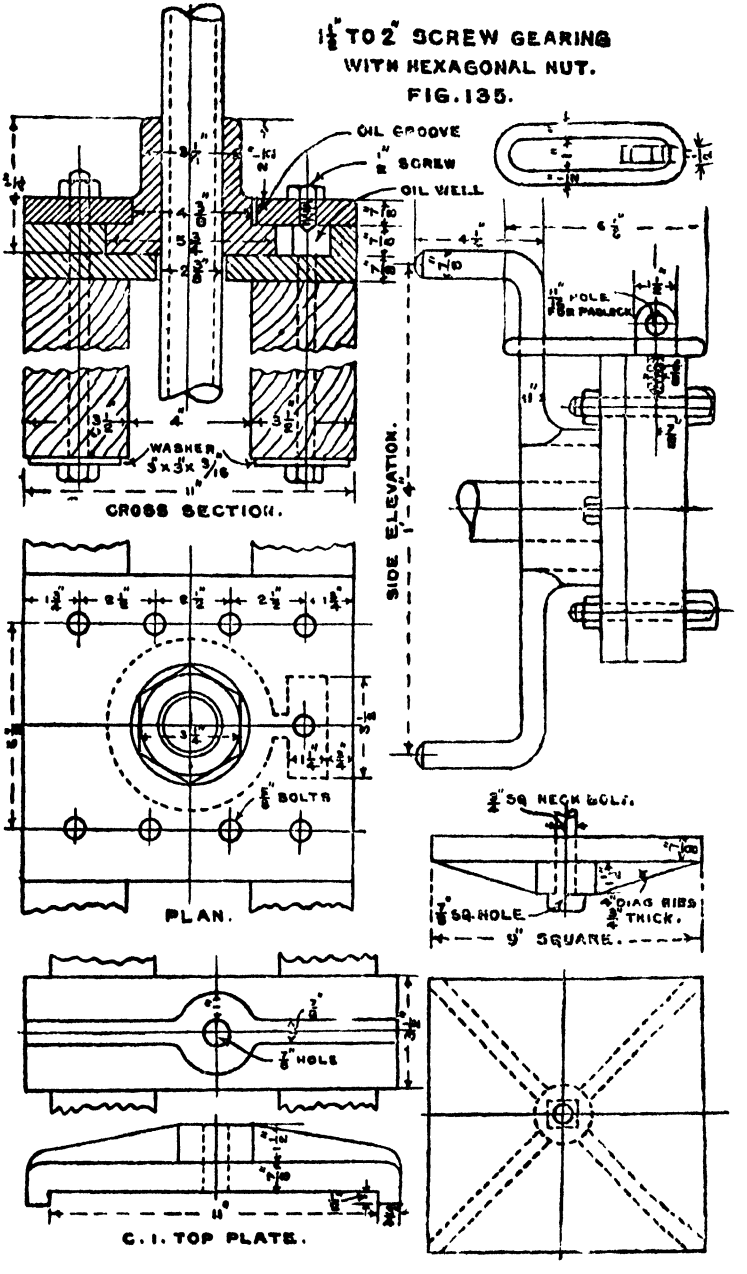
Sliding friction $\frac{1}{5}$.

The bearings of fixed rollers are however difficult to keep in good running order when subjected to immersion in silty water and the axle friction develops rapidly and after a time the rollers fail even to rotate ; free rollers are not subject to axle friction and are but little affected by silt or rust A removable 'roller box for sluice gates' has been invented by Mr Ashford which can be easily removed for cleaning and repairs and which he claims on account of the saving in cost to be more suitable for medium sized gates than Stoney rollers ; for gates of 20 feet span and upwards the free roller system should be adopted.

581. Screw geared sluice gates moving on plane bearing surfaces.—Sluices, shutters or gates, of spans up to 10 feet working under moderate heads of water are in South India almost universally operated by screw gear, the force being transmitted through a single screw shaft or spear, attached to the centre of the gate. On the upper end of this shaft a square screw thread is cut for the length necessary to secure the required travel of the shutter and a screw nut to fit this thread is housed in a box in which it can freely revolve, the box being fixed to the sluice platform. In order

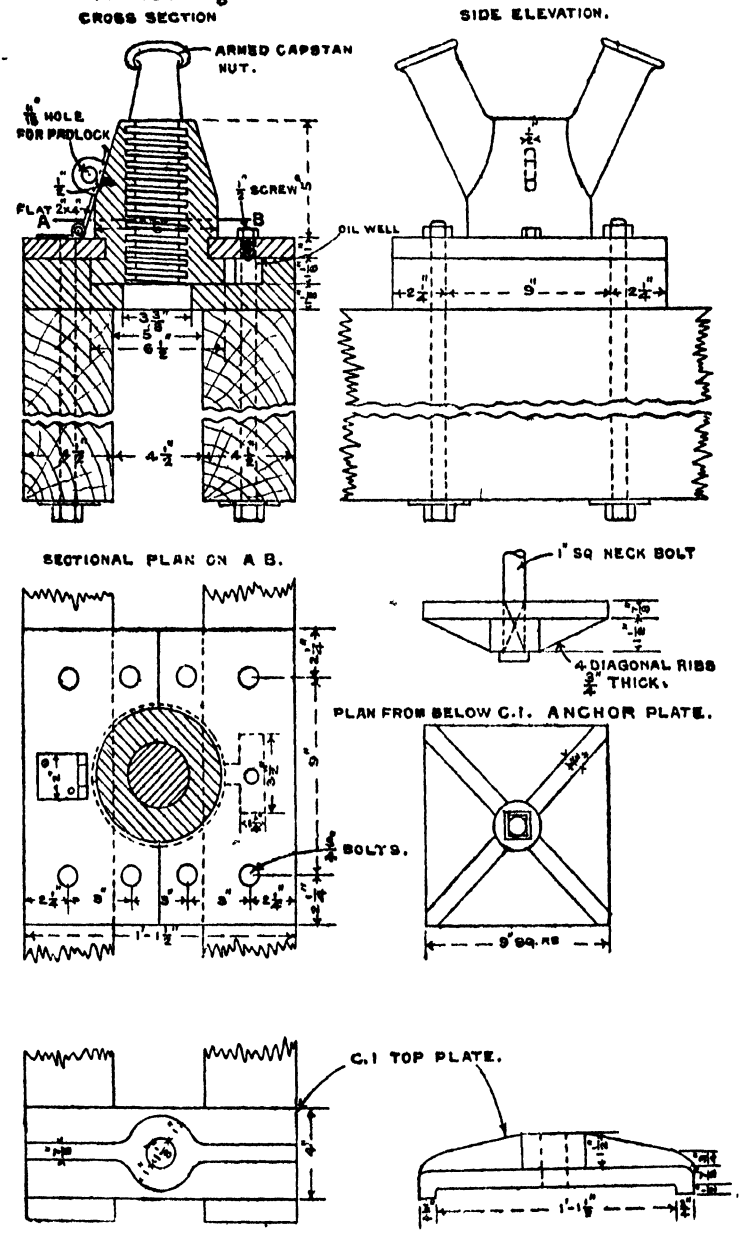
to lift or lower the shutter, the nut is revolved by means of a spanner figure 135 or capstan head figure 136.

Screw gear is extremely simple, it is not liable to sudden failure or injury by unintelligent operation, it is essential however that the



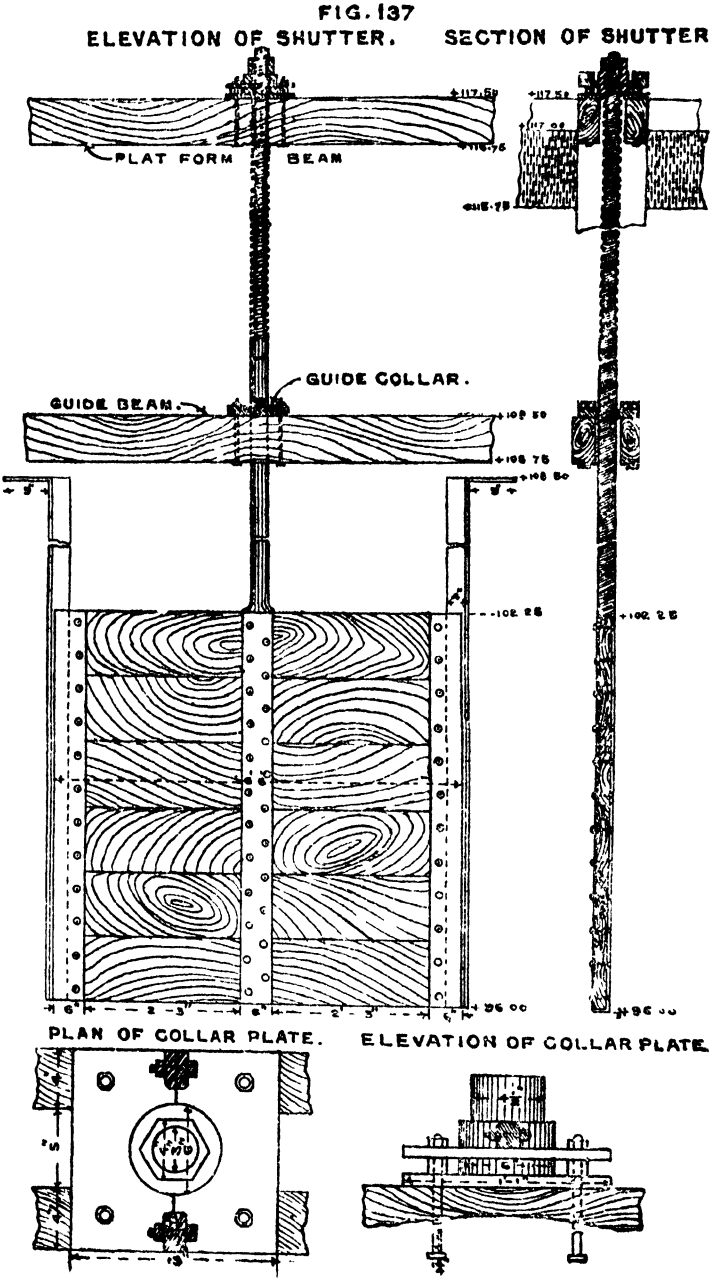
threaded length of spear should be kept lubricated and free from rust. A heavy oil should be used for lubricating the gear as this forms a protective skin over the exposed metal and protects it from corrosion.

FIG. 136. $2\frac{1}{8}$" TO 3" SCREW GEARING WITH CAPSTAN NUT.



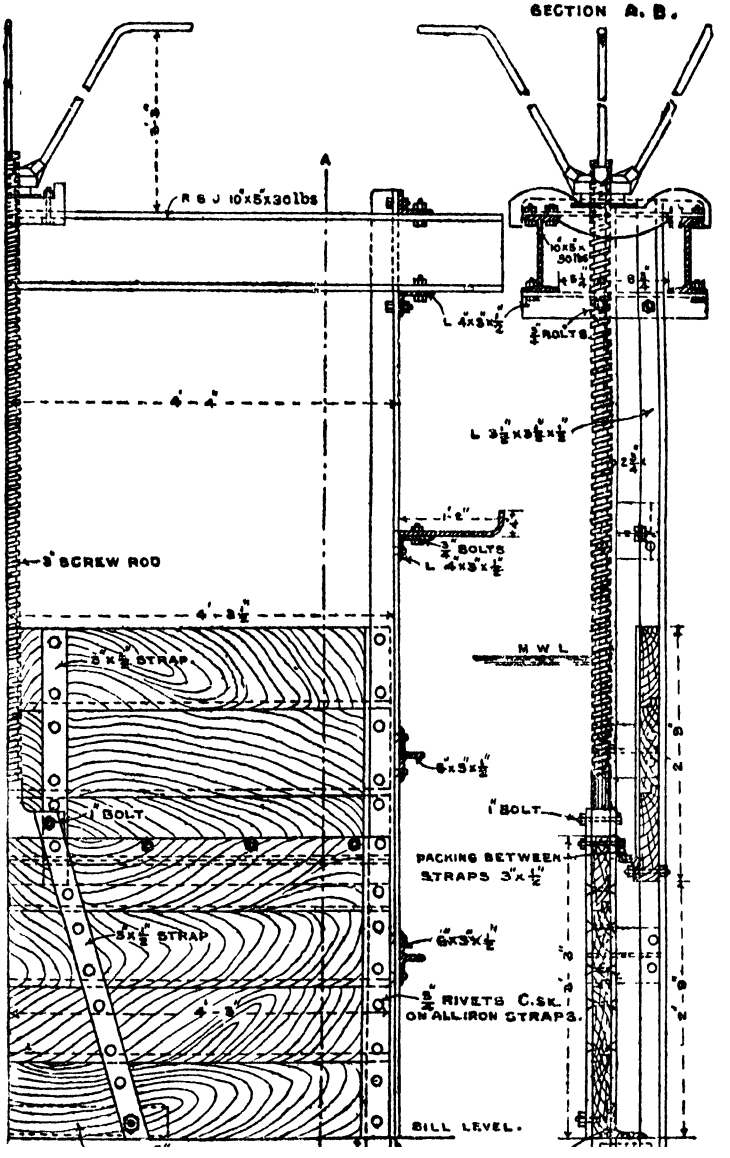
Figures 137 and 138 show the details of screw geared shutters the first being the usual type of single shutter with wooden platform beams and the second in two tiers lifted by a single screw as in the regulator in figure 9), Chapter XII, above. In this case the shutters

FIG. 137



must be capable of travel right up to the platform beams which are steel beams to which are bolted the angle iron groove linings on which the shutters slide; under this arrangement there is no necessity for anchor bolts to secure the platform beams to the sluice platform. When the shutter need not be lifted up to the gear beams it is usual only to carry the groove linings to the

FIG. 138. DOUBLE SHUTTER FOR REGULATOR.



top of the travel of the shutter, and to secure the platform beams to the body of the sluice by bolts passing through anchor plates bedded in the masonry and passing through top plates which overlie the beams (vide figure 136).

In figure 137 the travel of the shutter is 6 feet while the height from the sill to the platform beams is 20 feet. The smooth part of the spear passes through a collar plate which serves as a guide and stiffens the spear thus allowing it to be made of a smaller diameter than wood otherwise be necessary.

Similarly guides are provided at 4 feet interval for the spear of the tank sluice gate, Chapter X, figure 77, above.

In figure 138 the spear must be threaded throughout its length and no effective guide can be introduced.

582. Dimensions of spears and screw threads.—The greatest stress is placed on spears when the shutters of sluices are being forced down under the pressure of the maximum head of water to which they can be subjected.

The following equation for finding the diameter of solid mild steel screw rod which is based on Gordon's formula for struts hinged at both ends is in use in the Public Works Workshops, Madras :—

$$A \times H = \frac{82000d^4}{350d^2 + 144L^2} \quad \ldots\ldots\ldots\ldots\ldots (71)$$

where A = the area of the shutter in square feet,
H = Maximum head on the shutter in feet,
L = the maximum unsupported length of screw shaft *in feet.*
d = *least* diameter of above *in inches.*

This allows a factor of safety of 6 and a co-efficient c in Gordon's formula of $\frac{1}{5500}$.

The following table gives dimensions in inches of the size of spears and pitch of screws commonly cut for screw rods in the same workshops :—

Serial number.	Finished diameter overall D	Least diameter (d)	Depth of thread (d_1)	Pitch of thread P
1 ..	3 5/8	2 7/8	3/8	1
2 .	3 5/8	2 5/8	1/2	1 1/2
3	3 1/4	2 1/8	3/8	1 1/2
4	3	2 3/8	5/16	1
5 ..	3	2 3/8	5/16	1/4
6	2 7/8	2 1/4	5/16	1
7 .	2 7/8	2 1/8	3/16	1
8 ...	2 1/8	1 7/8	5/16	1
9 .	2 3/8	2 1/8	1/8	1
10	2 3/8	1 7/8	1/4	1/2
11 .	2 3/8	1 3/4	5/16	1
12	2 1/4	1 3/4	1/4	1
13	2	1 1/2	1/4	3/4
14 ...	2	1 5/8	3/16	1
15 ..	1 7/8	1 5/8	1/4	3/4
16	1 1/2	1 1/8	1/8	1/4

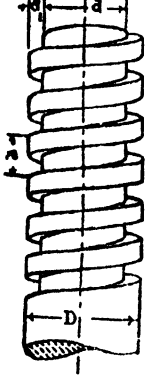


FIG. 139 SQUARE THREADED SCREW.

583. Non-rising screw spears.—Where the height of the sluice platform above the sill is great and the travel of the shutters is less than half this height it is sometimes convenient to instal screw gear so that the spears do not rise above the sluice platform. This is effected by fixing the revolving nut in the lower end of a pipe of sufficient length to receive and enclose the screw rod, the nut being revolved by turning the whole length of pipe.

FIG. 140.
HEAD SLUICES OF GODAVARI CENTRAL DELTA
SECTION THRO: PIPES.

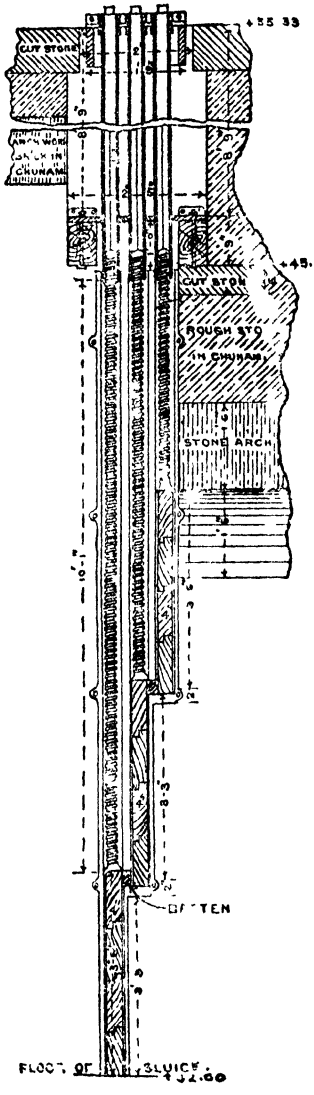


Figure 140 is a part section of the 3 tiers of shutters of the Gōdāvari Central Delta head sluice showing the screw gear which is of this type.

584. Needle shutters.—Regulators and sluices are sometimes regulated by vertical planks generally about six inches wide the lower ends of which rest on the sill while the upper ends are supported by a platform beam, sometimes one or more beams between the platform and the sill give intermediate support to the needles; usually there is a step in the floor of the sluice up to the sill level forming a ledge against which the lower ends of the planks rest

Such planks are termed 'needles' and this type of shutter is termed needle shutter.

Before the introduction of screw gearing, needles were used for regulating all the most important regulators and sluices in South India, and they are still occasionally used for works of minor importance. The objections to needle shutters are that they take much time and labour to operate and the leakage past them is considerable. Needles can be manipulated in depth of water up to 10 or 12 feet, and under considerable heads, and they are still used for regulators and weir crest shutters in many countries.

585. Regulation by horizontal planks.—Horizontal planks sliding in grooves are also used on weir crests and in regulators

and sluices and are manipulated by the help of rings fixed at the ends of each plank.

In Madras horizontal planks sliding in grooved standards or piers are occasionally used on weir crests or in regulators and escape sluices up to limiting height of three to four feet. Except in shallow water horizontal planks are more difficult to handle than needles and except in the hands of experienced and reliable establishment there is great danger of failure to work them effectively; this type of shutter is not recommended for works which are subject to floods submerging them to any considerable depth.

INDEX.

A

Abutment of Aqueduct, 321
,, ,, Regulator, 510
Absorption, 102, 103 (see 'Percolation')
,, in canals, cross sections of level, 434
Acre-foot, 13
Alluvial formations, 148, 149
Alluvium, 148
,, deposit of, in lakes, 150
,, ,, ,, the sea, 151 (see 'Deltas')
Anicut, 175 (also see 'Weirs')
Aqueducts, 501, 502, 532
,, bank connexions of, 512
,, contraction of waterway in, 524 525, 528
,, formula for heading up due to contraction of, 526
,, formula for discharge of culverts under, 519
,, formula for thickness of abutments of, 510
,, formula for thickness of arches of, 509
,, headway required for, 505
,, Gunnaram, 531
,, sites for, selection, 507, 508
,, syphon (see 'Syphons')
,, types of, 503
,, types of selection of suitable, 504
Area, catchment, 51
,, curve, 76
,, irrigated in Madras, 10
Austin overflow dam, 274
Automatic water level recorder, 74
Ayacut, 445

B

Balancing depth of cutting, 427, 438
,, ,, ,, formula for, 428
Bank, canal and channel, dimensions of, 426
,, causes of failure of, 352–354, 357, 358
,, connexions of drops, 470, 474, 476
,, ,, irrigation works, 502, 397, 470, 476, 512, 532
,, ,, aqueducts, 512
,, drains in or under a, 364
,, execution of earthwork of a, 371
,, foundations of a, 369
,, flood, 175, 184
,, hydraulic gradient of a, 112, 113, 363
,, junction or joint in a, 372
,, saturation gradient of a, 112, 113
,, scour holes in or below a, 373
,, of syphon or aqueduct, 512, 532
Bench-mark stones, 449
,, ,, in connexion with a gauge, 72
Berms, canal, width of, 428
,, ,, ,, formula for, 428
Branch canals, 417
Breaching section, 366, 408
Bridges, canal, 493
,, on navigation canals, 573
Bund, tank, dimensions of, 360, 368
,, ,, free board of, 365
,, ,, top width of, 365
,, testing soil for a, 374
,, types of, 351, 359, 361, 362
,, revetment of a, 367

C

Calibration curve or table, 87
,, of a gauge, 87
Calingulah, 399
Canals and channels, 416–452
,, alignment of, 418, 419
,, balancing depth of, 427, 438
,, bench-mark stone along, 449
,, berms, width of, 428
,, branch, 417
,, capacity of, 444, 447
,, command of, 445
,, classification of, 417
,, contour, 420, 421, 431
,, cross section, selection of, 446
,, curves in, 430
,, depth of bed above a drop in, 445
,, ,, change of, 448
,, ,, excavation, 442
,, Kutter's co efficient for, 439
,, masonry works in, 452
,, mile stones, 449
,, non-silting, characteristics of, 437, 438
,, offtakes from, 421
,, profile of least absorption, 434
,, proportion, bed width to depth of, 433, 435, 436, 437, 440, 441
,, side slopes of, 429
,, single banked, 450, 451
,, standard dimensions of, 426, 436
,, system of, typical, 422
,, table of dimensions of, 436
,, velocities of flow of, 432
,, watershed, 418
Capacity, storage, computation of, 409, 412
Catchment area, 51
,, basin, 51
Channels, 416 to 452 (also see 'Canals')
,, berms of, 428
,, classification of, 417
,, depth of, change in, 448
,, distributary, 417
,, dimensions of, 426, 436

Channels, field, 417, 425
,, Kutter's co-efficient for, 439
,, major, 417
,, minor, 417
,, masonry, works of, 452
,, side slope, 418
,, spring, 349
,, standard dimensions of, 426
,, watersned, 418
Chenab weir, 306
Command, 175, 176, 445
Crest free board of dam, 216
,, ,, tank bund, 365
,, level of weir, 176
,, shutter of weir, 314
,, 'shutter crest level,' 176
,, width, of arched dam, 240
,, ,, dam, 215
,, ,, tank bund, 365
,, ,, weir, 259
Crops, dufassil, 5
,, duty of water for, 23 to 35
,, garden, 5
,, irrigated dry, 1, 2, 5
,, ,, wet, 4, 5
Crossings, cart, 495, 496
,, cattle, 495, 496
Culvert drainage, 504
Current meters, 81
,, ,, measuring velocities with, 81–84
,, ,, operating, 85
,, ,, price, 85
Curves of canals and channels, 430
Cusec, 13

D

Dam, arched, 236
,, ,, buttress, 249-252
,, ,, crest width of, 240
,, ,, theoretic profile of, 239
,, Ashokan, 232
,, Assuan, 233
,, Australian arched, 243
,, Bear Valley, 242
,, buttress, 246 to 248
,, conditions of stability of, 201
,, comparison of arched and gravity, 253
,, dividing, 482
,, earthen (see 'Bank')
,, elementary profile of, 205
,, essential divisions of a joint of, 223
,, free board of, 216
,, Furens, 232
,, gravity, crest width of, 215
,, ,, design of 'low' dam, 217, 218
,, ,, design of 'high' dam, 219-222
,, ,, 'high', 214
,, ,, 'low', 214
,, Marikanave, 232
,, 'middle third' rule, 202, 203
,, overflow, 191
,, path-finder, 244
,, Periyar, 232
,, regulating, 480
,, stability of, by method of moments, 225, 226
Dam, stability of graphical construction, 228 to 231
,, sliding, security against, 204
,, stress in arched, 237, 238, 241, 251
,, stress in horizontal joint of, 224
,, stress, maximum in joint of, 207, 208, 209
,, stones, 399
Definition of aqueduct, 501, 502
,, base of duty, 18
,, bed silt, 136
,, calingulah, 397
,, catchment area, 51
,, catchment basin, 51
,, contour canal, 418
,, crop period, 18
,, dam, masonry, 196
,, dam, high, 214
,, dam, low, 214
,, dam, overflow, 196
,, drop, 453
,, duty of water, 12
,, escape, natural, 398
,, fall, 453
,, field to field irrigation, 425
,, free board, 358
,, ground water gradient, 111
,, hydraulic water gradient, 112
,, inlets, 534, 535
,, inverted filter, 295
,, irrigation, 1
,, irrigation works, 1
,, korambu, 348
,, level crossing, 533
,, natural escape, 398
,, outlets, 534
,, percolation gradient, 110
,, plane of saturation, 112
,, regulator, 315
,, run-off, 52
,, rainfall loss, 55
,, scour, 169
,, shutter crest level, 176
,, silt in suspension, 136
,, side slope channel, 418
,, sluice, 315
,, spring channel, 349
,, superpassage, 501, 502
,, syphon, 501, 502
,, syphon aqueduct, 501, 502
,, watershed, 418
,, weir, 196
,, wings, canal, 512
,, ,, drain, 512
Delta, distinctive features of a, 153
,, drainage difficulties in a, 158
,, formation of, 151
,, irrigation, facilities in a, 158
,, rivers, characteristics of, 156
,, ,, formation of bars in, 152
,, ,, gradual flattening of slope of, 154
,, ,, liability to change of, 155
Description of (see 'Definition of')
Dhukwa weir, 272
Diagram of distribution of pressures in a joint, 199
,, effective weight of a weir apron, 291, 293

Diagram of pressure on the sloping faces of dams or weir, 227
,, stability of dam, 228, 230, 231
,, ,, regulator, 223
,, stability of weir, 270
,, units of force in, 210
Direct irrigation, 9, 10
Discharge, maximum, from catchment, 89–100 (also see 'Flood discharge')
,, Trapezoidal notch, 461
Distribution works, 416, 417, 452
,, typical system of, 422 to 425
Diversion works, 175
,, best sites for, 185.
Divide groynes, 181
,, wall, 181
Drains, banks of, 550
,, bed fall of, 550
,, capacity of, 552
,, catch, 546
,, combined with irrigation channels, 553
Drainage culvert, 504
,, of embankments, 364
,, irrigated area, 548 to 553
,, outfall sluices, 551
Drops, 453 to 478
,, aprons of, length, 468
,, ,, testing for uplift, 472-475
,, ,, testing for thickness, 468
,, ,, testing for width, 468
,, bank connexions of, 476, 474, 476
,, body wall of, 465
,, combined with aqueducts, 502
,, control of velocity of approach to, 456, 457
,, depth of canal bed above, 443
,, design of a, 455, 464, 471, 474, 475
,, notches in, 468, 465
,, pipes, 504
,, places of existing, 454
,, pitching, length of, 469
,, revetment, length of, 469
,, sites of, 454
,, splayed wings for, 473
,, trapezoidal notches in, 460 to 462
,, water cushion, depth of, 467
,, well, 478
Duty of water, 12 to 36
,, American notation for, 15
,, at the field, 19
,, at the head works, 19
,, base of the, 18
,, effect of rainfall on, 34
,, example of working out the, 35
,, for 'dry' crops, 23 to 25
,, for rice and wet crops, 26 to 35
,, for seed beds, 36
,, in Kistna and Gōdāvari deltas, 29, 31
,, in terms of acres per cusec. 13
,, acre feet, 15
,, depth over area, 16
Duty of water, place of measurement of, 19
,, relation between different notations for, 17, 21
,, requirements for precise statement of, 20
,, during transplantation, 27
,, after transplantation, 28, 31

E

Earth connexions, 302, 397, 470, 476, 512, 532
Earths (see 'Soils')
Elementary profile or triangle, 205
,, ,, pressures on the, 212
,, ,, limit of depth of, 213
Embankment (see 'Bund')
Equation (see 'Formula')
Escapes, surplus (see 'Surplus works')
,, tail, 547
Evaporation combined with absorption, 129, 130
,, measurement of, 127, 128
,, rate of, 126

F

Falling shutters, Ashford's, 579
,, Fouracres', 577, 578
Falls (see 'Drops')
Fender piles, 571
Fetch, definion of, 216
Field channels 417, 425 (also see 'Channels')
Field to field irrigation, 425
Filter, inverted, 295
Flood banks, 175, 184
,, discharge of combined catchments, 411
,, ,, catchments, large, 89
,, ,, catchments of moderate size, 93
,, ,, co-efficients for Ryves' formula, 95 to 98
,, ,, formulæ for, 91, 99
,, ,, recording levels of, 100
,, ,, standard area for computing, 94
,, ,, Table of, 92
Floors of irrigation works, 302
Flotation, 234, 235 (also see 'Uplift')
Flow, measurement of, 63 (also see 'Stream flow')
Formula for aqueducts and syphon aqueducts, abutments, thickness of, 510
,, aqueducts and syphon culverts and discharge of, 519
,, aqueducts and syphon heading up due to contraction of, 516
,, balancing depth of cutting, 427
,, berms, width of, 428
,, dams, masonry, arched, arch stress, 238
,, dams, masonry, arched, extra arch stress due to weight of masonry, 251

Formula for dams, masonry, arched, crest width of, 240
,, dams, masonry, crest width of, 215
,, dams, masonry, elementary profile, base width, 205
,, dams, masonry, elementary profile limit of height, 213
,, dams, masonry, stress, in joints of, 197, 209, 211, 224
,, dams, masonry, stress in high dams, 220
,, dams, masonry, stresses, relation between vertical and oblique, 207
,, duty of water and depth over area, 21
,, discharge, flood, 91
,, discharge combined catchment, 411.
,, discharge, Dickens', 91
,, ,, Ryves' 91
,, ,, trapezoidal notch, 461
,, drops, apron, length of, 466
,, ,, thickness, 468
,, ,, width, 466
,, ,, pitching, lengths of, 469
,, ,, revetments, lengths of, 469
,, river regulators, apron width of, 321
,, river regulators, designs of, 321
,, percolation gradient, 110
,, ,, velocity (Hazen's), 115
,, relation between 'fetch' and height of waves, 216
,, pressure and stress in masonry, 197, 209
,, sluices, head, design of, 321
,, ,, scouring apron, width of, 336
,, ,, design of, 321
,, shutters falling, position of pivot of, 578
,, screw spear, diameter of, 582
,, water cushion, cistern, depth of, 469
,, water cushion, cistern, length of, 468
,, water cushion, cistern, thickness of, 468
,, weirs, moments of overturning, 262, 263
,, weirs, moments of resistance, 268
,, weirs, moment of pressure on face, 257
,, weirs, apron width of, 283
,, weirs, crest width of, 259
,, weirs, percolation, length of, line of, 283
,, tank, weir, apron, width of, 396
Free board, 216, 358, 365, 426
Free rollers, 580

G

Garden crops, 5
Gates, sluice (see 'Shutters')
Gauge area curve or table, 76
,, bench mark in connexion with, 72
,, calibration of a, 87
,, erection of a, 71
,, outlets (see 'Outlets')
,, readings, 67
,, ,, computing flow from, 75
,, ,, recording instruments, 74
,, ,, inaccuracies of, 68
,, ,, selection of site for a, 69, 70
well, 73
zero of a, 72
Gradient, ground water, 111
,, hydraulic, 112, 363
,, percolation, 110, 112
,, saturation, 112, 113, 363
Grit soil, 108, 109, 355
Ground water gradient, 111
,, uplift pressure, 114
Groyne, hanging, 280

H

Hanging groynes, 280
Hazen's formula, 115
Head sluices, 175, 179, 341
,, Canals minor, 348
,, ,, with spring channels, 349
,, dimensions for design of, 321
,, foundations and floors of, 347
,, Godāvari Eastern delta, 346
,, Kistna Western, 346
,, operating system of, 183
,, Sirhind canal, 345
,, sites for, 344
,, surface water, arrangements for drawing, 342
Head works, 174 to 195
,, sites favourable for, 185, 186
,, typical plan of, 182
Hydraulic gradient, 110, 112, 113, 363

I

Inlets or infall, 534, 535
,, and outlets, 534, 539, 540
,, objections to, 540
,, sluice, 535
,, surface, 535
Irrigated 'dry' crops, 2, 5
,, wet, 4
Irrigation, direct flow, 9, 60
,, definition of, 1
,, field to field, 425
,, flood, 7
,, flow, 6, 9, 10
,, inundation, 7
,, lift, 6
,, perennial, 7, 8
,, reservoir, 9
,, tank, 9, 10

K

Kennedy channels, table of, 436
Kennedy's critical velocity, theory of, 160, 161
,, gauge outlets, 491, 492
,, Vo reduced values of, 164
,, Vo variations in values of, 165
Khanki weir, 306
Kiari, 23
Korambu, 348
Kutter's co-efficients, 439

L

Level crossings, 533
Lock, canal, 559
,, chamber, 561
,, chopella, 562
,, culverts, 563
,, double, 567
,, gates, 564
,, floors, 560
,, mooring posts, 568
,, sides sloping, 566
,, side walls, 560
,, site, 569
,, sluices, 563
,, tidal, 565
,, weirs, 558, 569, 570

M

Masonry works of distribution system, 452
Measurement of evaporation, 127
,, silt, quantity of, 139
,, stream flow, 63 (also see 'Stream flow')
Meter (see 'Current Meter')
Mile stones, 449
Moments, method of, for computing stability, 225, 226, 267
Moment of overturning on a weir, 262, 263
,, pressure on a face, 257
,, resistance of a trapezoidal weir, 265, 268

N

Narora weir, failure and restoration of, 305
Natural escape, 398
Navigation canal, 554
,, ,, bridges, 573
,, ,, headway required, 573
,, ,, locks, 559 to 571
,, ,, velocities of, 555
,, ,, water required for, 557
,, ,, widths and depths of, 556
,, ,, combined with irrigation, 574
,, ,, special works required for, 558
Needle shutters, 584

O

Outfalls (see also 'Outlets')
,, drainage, 549 to 551
,, sluices, 551
,, tidal sluices, 551
Outlets, 534, 537 to 549
,, in connexion with inlets, 534, 539, 540
,, in single banked canals, 542
,, Kennedy's gauge, 491, 492
,, objections to, as cross drainage works, 540
,, pipe, defects of, 490
,, ,, discharges of, 488, 489
,, ,, sizes of, 487
,, sluices, 538
,, surface, 537

P

Palar anicut, 303
Percolation, 101 to 103
,, and absorption, 102
,, barrier for checking, 125
,, combined with evaporation, 129
,, effect on irrigation supplies, 105
,, formula for velocity of, 119
,, gradient, 110
,, length of line of, under works, 283
,, limit of safe velocity of, 120, 121
,, losses from canals and channels, 132 to 134
,, ,, from tanks, 130
,, mean annual, 104
,, through embankments, 122
,, ,, and under irrigation works, 118
,, ,, rock, 107
,, ,, soils, 106
,, uplift due to, 123
,, velocity of, 112, 115
Perennial irrigation, 7, 8
Piles fender, 571
Pipes (see 'Outlets')
"Piping" 124
Plastic soil, 108, 109, 355
Pressure, diagram of, on a sloping face of a dam, 227
,, distribution of, in a horizontal joint of a dam, 197 to 200
,, ,, in the 'elementary profile,' 212
Paddle in a canal bed, 313

R

Rainfall, 37 to 50
,, annual fluctuations of, 47
,, average, 43
,, ,, of a bad year, 50
,, disposal of, 55

Rainfall, loss in, 5
,, mean estimating, 46, 49
,, observing and registering, 40
,, proportion of maximum and minimum to mean, 48
,, run-off estimating, 57 to 62
,, ,, maximum (see 'Flood discharge')
,, statements of, 49
,, stations, 41
,, variations of, 42, 44, 47
Rain gauge, 38
,, setting up, 39
Recorder, water level, 74
Regulator, canal, 479 to 481
,, head (see 'Head sluices')
,, cross thrust on piers of, 328
,, different conditions of stress on, 323, 325
,, dimensions of parts of, 320, 321
,, foundations, pressure on, 327
,, river, 317
,, ,, aprons, width of, 331
,, ,, example of design, 322, 324, 327, 328
,, roadway over, 318
,, Toludur, 329, 332
Reservoir (see 'Tanks')
,, component works of, 190
,, dams, 191
,, irrigation, 9
,, site, best conditions of, 194
Retaining walls, 497 to 500
Revetments of bunds, 367
,, of 'drops' length of, 469
Rice (see 'Wet crops')
,, area cultivated in Madras, 10
Rupar weir head regulator, 345
,, scouring sluices, 339
Ryves' formula, 91

S

Sand, classification of, for weir foundations, 282
,, effective size of, 115
,, mechanical analysis of, 116, 117, 141, 142
Sunkesala Anicut, 272
Saturation gradient, 112, 363 (also see 'Percolation')
Scour, 169 to 173
,, effect of, on irrigation work, 170, 171
,, face, 107
,, holes, treatment of, 373
,, power of clear water to, 173
,, protection against, 172
Scouring sluices, 180, 333 to 340, 543
,, aprons, width of, 336
,, capacity of, 183
,, canal, 543, 544
,, dimensions of component parts of, 321
,, face walls of, 340
,, operating system of, 183, 337
Scouring sluice, Rupar weir, 339
,, ill levels of, 334
,, vents, size of, 335
,, ,, ,, Toludur Regulator, 338
Screw-gear, 581 to 583
Screw spear or shaft, dimensions of, 582
Seed beds, 36
Seepage, 103
Shutters, Ashford's patent drop, 579
,, crest, for weirs, 576
,, Fouracres' falling, 577, 578
,, horizontal plank, 585
,, lift, for large spans, 580
,, needle, 584
,, screw geared, 581 to 583
,, ,, with non-rising screws, 583
,, sluice, canal, 484
,, ,, tank, 383, 384
Silt, 136 to 168
,, bed, 136, 147
,, classes of, 136
,, deposit, at canal head, 166
,, ,, in lakes, 150
,, ,, in sea (see 'Deltas')
,, ,, in tanks, 167, 168
,, Kennedy's theory about, 160, 161
,, measurement of, 139, 147
,, mechanical analysis of, 116, 117, 141, 142
,, non-silting velocities, 160, 161
,, quantity carried in Cauvery, 144
,, ,, ,, in Kistna, 144
,, ,, ,, various rivers, 143, 146
,, samples of taking, 140
,, suspended, 136
,, transporting power of water, 159
,, weight and volume of, relation between, 145
Slipping of banks, 354
Sluices—barrel or culvert of, 377 to 380, 483
,, canal, 483 to 485
,, ,, scouring, 543, 544
,, drainage, 551
,, head (see 'Head sluices')
,, inlet, 535
,, outlet, 538
,, pipe, 486 (see 'Outlets, pipe')
,, shutters for (see 'Shutters')
,, Screw gear for, 581 to 583
,, Scouring (see 'Scouring sluices')
,, Tank, cistern of, 385
,, ,, head wall, 381
,, ,, plugs, details of, 383, 384
,, ,, sites for, 386
,, ,, surplus, 400, 401
,, ,, tower head, 382
,, ,, well head, 382
,, tidal, 551
,, vents, dimensions of, 483
Soakage, 103
Soils-grit and plastic, 108, 109, 355
Specific gravity of masonry, 206
Spring channel, 349
Stability, by method of moments, 225, 226

Stability, diagrams (see 'Diagrams')
Standard area of maximum rainfall, 94
Stoney shutters, 580
Storage works, 189, 350 to 415 (see 'Tanks')
Storage capacity, 409, 412
Strange's tables of run-off, 58, 61
Stream-flow computing, 79
,, ,, from gauge reading, 68, 74
,, gauging, 63, 64, 65, 67
,, ,, methods of, 77
,, ,, sites for, 76
,, ,, by weirs, 86
,, measuring by intercepting reservoirs, 88
,, velocity of, measuring by current meter, 81, 82
,, velocity of, measuring by floats, 78
,, velocity of, measuring by rods, 80
Stress, condition of greatest, on a weir, 260, 264, 266
Stress different conditions of, on regulators, 323
Stress in an arched dam, 237, 238, 241, 251
Stress in a horizontal joint in a masonry dam, 207 to 211
Superpassage, 501, 502
Surplus works, canal, sluices, 538
,, ,, outlets, 534 to 542
,, single banked canals, 450, 542
,, tanks, 388 to 407 (also see 'Weirs')
,, tanks, sites for, 407
,, tail escapes, 547
Syphon, 501, 502, 532
,, aqueduct, 501, 502
,, ,, abutments, thickness of, 510
,, ,, bank connexions of, 512
,, ,, contraction of waterway of, 524 to 526, 528
,, ,, culverts, headway for, 506
,, ,, design of, 509 to 523
,, ,, drainage, 504
,, ,, plans of existing, 531
,, ,, selection of, 504
,, ,, sites for selection of, 507, 508
,, ,, types of, 503
,, ,, type I, 530
,, culverts, arches, thickness of, 509
,, ,, discharge of, 519
,, ,, number of, required, 509
,, ,, uplift in, 514, 516, 518
,, ,, velocity through, 520, 532
,, ,, width of waterway of, 505
,, head on, 520
,, irrigation, 508, 532
,, road, 494
,, types of, 503
,, uplift on, 514, 516, 518

T

Table, water, 105
Tables of discharges of tank surplus works, 413
,, ,, pipe outlets, 489
,, maximum flood discharges, 92
,, non-silting canals and distributaries, 436
,, rainfall in Madras Presidency, 49
,, run-off from monsoon rainfall, 58
,, ,, daily rainfall, 61
Tables, working, 415
Tank, 350 to 415
,, banks or bunds, causes of failure of, 352 to 354, 357, 358
,, ,, dimensions of, 360, 365
,, ,, drains in, 363, 368
,, ,, execution of earth work of, 371, 372
,, ,, foundation of, 369
,, ,, freeboard of 365
,, ,, revetment of, 367
,, ,, slopes of, 356, 368, 370
,, ,, testing soil for, 374
,, ,, top width of, 365
,, ,, types of, 351, 359, 361, 362
,, bed, testing permeability of, 375
,, breaching section in, 366, 408
,, calingulah, 399
,, capacity, 409, 412
,, component works of a, 190
,, flood absorbing capacity of a, 410
,, formed on a single banked canal, 450, 451
,, flush escapes, 398
,, grouping of, 408
,, irrigation, 9, 10
,, natural escapes, 398
,, outlets (see 'Sluices')
,, scour holes below, 373
,, silt deposit in, 167, 168
,, sites favourable for, 194
,, sluices, supply, 377 to 387
,, ,, shutters for, 583, 584
,, ,, sites for, 386
,, sluices, surplus, 400, 401
,, supply channels, 414
,, surplus water, disposal of, 403, 404
,, weirs and surplus works, 388 to 407 (also see 'Weirs')
,, weirs and surplus works, aprons, widths of, 396
,, weirs and surplus works, dam stones on, 399
,, weirs and surplus works, sites, plans of, 405, 406
,, weirs and surplus works, sites, selection of, 407
,, weirs and surplus works, types of, 388
,, weirs and surplus works, types o selection of suitable, 393
,, working tables for, 415