THE PRACTICAL DESIGN OF IRRIGATION WORKS

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THE PRACTICAL DESIGN OF IRRIGATION WORKS

SECOND EDITION

PREFACE TO SECOND EDITION

In this edition considerable emendations and additions have been made to the book, the principal of which is the treatment accorded to the theory of the stability of weirs or other works founded on a sand stratum. In the first edition this stability was made dependent on two considerations—first, the weight of the structure, and, secondly, the enforced length of percolation. Exception was taken to this view by professional critics, weight as an agent effecting reduction of pressure or head being ruled out as unsound. The author now frankly admits that his previous view is untenable, and Chapter VI. has consequently been entirely recast, the first factor, that of weight, except as regards upward hydrostatic pressure, having been eliminated from the equation of resistance.

The next main alteration effected is in Chapter V., where the tables of discharge of submerged weirs founded on theory have been replaced by others worked out from the formula introduced by Herschel, an American hydraulician. These are based on experimental data which, though on the usual unsatisfactory small scale, still must be preferable to the use of coefficients obtained from any purely theoretical consideration. Chapter I. remains as before. Chapters II. and III. have been recast and rewritten *de novo*. Gravity dams and weirs are now united in Chapter II., while the arched and panel deck varieties are dealt with in Chapter III., with many additional examples. Chapter IV. is little altered. Chapters V. and VI. have already been referred to. The remaining chapters have had additional matter inserted and some excised, the most important alterations being in Chapter XII. (*olim* XIII.), which has been practically rewritten. The increase in the number of illustrations all through is considerable.

The work has been brought by this means thoroughly up to date. While its original characteristics, namely critical examination of existing works, are retained, its scope has been enlarged and its value, it is hoped, as a medium of instruction thereby enhanced.

The author wishes to express his obligations to the following works bearing on the same subject:—"The Irrigation Works of India," R. Buckley, C.I.E., London; "Reservoirs for Irrigation Water Power and Domestic Water Supply," James De Schuyler, New York; "Irrigation: Its Principles and Practice," Sir Hanbury Brown, K.C.M.G., London; "Irrigation Engineering," H. M. Wilson, New York; "Indian Storage Reservoirs," W. L. Strange, London; "Treatise on Hydraulics," M. Merriman, New York; "Public Water Supplies," F. E. Turneaure and H. L. Russell, New York; "Waterworks Engineering," Tudsbury and Brightmore, London.

W. G. BLIGH.

Toronto, October, 1910.

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LIST OF FORMULAS

CHAPTER II

Section I

NO.	PAR.	FORMULA.	SUBJECT.
1	5	$b = \frac{H}{\sqrt{\rho}} = \frac{2}{3}H.$	Base width of dams.
2	7	$s = s_1 \left(1 \pm \frac{6c}{b}\right).$	Relation of maximum and mean stress.
3	10	$W = \frac{H^2 w \sqrt{\rho}}{2} = \frac{H^2}{48}$ Tons.	Weight of Δ prism (reservoir empty).
4	10	$R = W\sqrt{\frac{\rho + 1}{\rho}} = 1{\cdot}2\,W = \frac{H^2}{40}$ Tons.	Value of R (reservoir full).
5	10	$s^2 = Hw\rho = \frac{H}{16}$ Tons.	Maximum unit stress (reservoir empty).
6	10	$s = Hw(\rho + 1) = \frac{H}{11{\cdot}1}$ Tons.	Maximum unit stress (reservoir full).
7	10	$H^{\lambda} = \frac{\lambda}{w\,(\rho + 1)}.$	Limiting depth.
8	12	$c = \sqrt{H}.$	Crest width.
9	13	$b = \frac{H}{\sqrt{\rho}} \times \frac{1}{\sqrt{1 + 2r^2 - 2r^3}}.$	Base width of dams when $r = \frac{c}{b}$ in pentagonal profile.
10	14	$h = 1{\cdot}5\,F + (2{\cdot}5 - \sqrt[4]{F}).$	Weatherboard.

Section II

NO.	PAR.	FORMULA.	SUBJECT.
11	41	$A = H\left(\frac{H + 2d}{2\rho}\right).$	Area of trapezoid of water pressure.
11a	41	$A = H^1\left(\frac{H + 2d}{2\rho}\right).$	Area of trapezoid when back of wall is inclined.
12	41	$h = \frac{H}{3}\left(\frac{H + 3d}{H + 2d}\right).$	Height of centre of water pressure above base.
13	42	$b = \frac{H + d}{\sqrt{\rho}}.$	Approximate base width of weirs.
14	42	$c = \sqrt{H} + \sqrt{d}.$	Crest width of weirs.
15	43	$b = \frac{H + d}{\sqrt{\rho}} \times \frac{1}{\sqrt{r + 1 - r^2}}.$	Base width when $r = \frac{c}{b}.$

NO.	PAR.	FORMULA.	SUBJECT.
15a	43	$b = \cdot 6\ (H + d).$	Base width when $r = \cdot 25$ and $\rho = 2\frac{1}{4}$.
16	47	$M = \frac{H^2}{6}(H + 3d).$	Moment of trapezoid of water pressure.
17	49	$Q = 100cD^{\frac{3}{2}}\sqrt{S}.$	Approximate discharge formula of channels.
18	63	$b = \frac{H + d}{\rho}.$	"Hybrid" weir, base width.

CHAPTER III

NO.	PAR.	FORMULA.	SUBJECT.
1	3	$b = \frac{RHw}{s_1}.\ s_1 = \frac{RHw}{b}.$	Base width and mean stress "short" formulas.
2	3	λ, or $s = \frac{RHw}{b} \times \frac{2R}{R+r}.$	Maximum stress.
3	3	λ, or $s = \frac{2Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)}.$	Maximum stress in terms of R and b. "Long" formula.
4	3	$b = R\left(1 - \sqrt{1 - \frac{2Hw}{\lambda}}\right).$	"Long" formula for base width.
5	11	$c = \frac{1}{2}\sqrt{H}.$	Crest width.
6	23	$s = R_1\ w\rho\ \sin.\ \theta.$	Stress induced by weight of arch ring when inclined.

CHAPTER IV

NO.	PAR.	FORMULA.	SUBJECT.
1	13	$P = wrt.$	Horizontal thrust of arch.
2	15	$t = \cdot 2\ r + \cdot 1\ V + 2.$	Thickness of abutments.
3	17	$t = \cdot 4\sqrt{r}.$	Thickness of arch.

CHAPTER V

NO.	PAR.	FORMULA.	SUBJECT.
1	1	$V = c\sqrt{2gH}.$	Discharge from orifice.
2	1	$Q = cl\,\frac{2}{3}\sqrt{2g}\,(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}).$	Discharge from orifice.
3	1	$Q = cA\sqrt{2g\,H}.$	Discharge from orifice.
4	3	$Q = cl\ 5\cdot 35\left\{(H\sqrt{H} - h_1\sqrt{h_1}) + \frac{3}{2}\sqrt{H}\,(h_2 - H)\right\}.$	Submerged orifices with velocity of approach.

NO.	PAR.	FORMULA.	SUBJECT.
5	4	$c_1 = c\sqrt{1 + \frac{h}{H}}.$	Modified coefficient.
6	4	$Q = cA\sqrt{2g(H+h)}.$	Approximate formula orifices.
7	10	$Q = \frac{2}{3}\,cl\sqrt{2gd} \times d = 5{\cdot}35\,cld^{\frac{3}{2}}.$	Free overfall.
8	12	$Q = 3{\cdot}333\,ld^{\frac{3}{2}}.$	Free overfall.
8a	16	$q = 3{\cdot}33\,(nd)^{\frac{3}{2}}.$	Submerged fall discharge.
9	13	$c_1 = c \times \left\{\left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}\right\}.$	Modification of coefficient, velocity of approach.
10	14	$Q = 3{\cdot}33\,d^{\frac{3}{2}}\left\{\left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}\right\}.$	Discharge of free overfalls with velocity of approach.
11	17	$Q = cl\left\{\sqrt{2gH} \times \frac{1}{3}(3d - H)\right\}.$	
12	17	$Q = cl\sqrt{2g}\,[(d+h)\sqrt{H+h} - \frac{1}{3}\{(H+h)^{\frac{3}{2}} + 2h^{\frac{3}{2}}]\}.$	Submerged falls, theoretical formula.
13	27	$Q = 100\,Ac\sqrt{RS}.$	Discharge of channels.
14	27	$c = \frac{\sqrt{R}}{100\,n} \times \frac{(m + 1{\cdot}811)}{(m + \sqrt{R})}.$	Kutter's formula.
15	31	$V = {\cdot}705 \times {\cdot}003\,v^2.$	Wagner's mean velocity formula.
16	33	$Q = \frac{Aq\sqrt{R}}{a\sqrt{r}} \times \frac{C}{c}.$	Deduction of discharge from that at a different level.
17	45	$l = \frac{d_2 - d_1}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right) \times \left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right].$	Backwater curve.
18	48	$l = -\frac{d_1 - d_2}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right) \times \left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right].$	Falling surface curve.
19	51	$l = \frac{(100c)^2}{4\,Q^2}(d_1^4 - d_2^4) - \frac{(100c)^2}{g}(d_1 - d_2).$	Curve when bed is level.
20	50	$Q = k_1\,100\,K^{\frac{3}{2}}.$	Ryves' flood discharge, second-feet.
21	50	$Q = k\,100\,K^{\frac{3}{4}}.$	Dickens' flood discharge, second-feet.
22	50	$Q = 200\,K^{\frac{5}{6}}.$	Fanning's flood discharge, second-feet.

NO.	PAR.	FORMULA.	SUBJECT.
23	67	$H = M \times \frac{V^2}{2g}$.	Head required for syphons.
24	67	$V = 8{\cdot}035\sqrt{\frac{H}{M}}$.	Velocity due to head.
25	67	$H = M \times \frac{V^2}{2g} - \frac{Va^2}{2g}$.	Head, including velocity of approach.
26	67	$V = \sqrt{\frac{2gH + Va^2}{M}}$.	Velocity, including that of approach.

CHAPTER VI

NO.	PAR.	FORMULA.	SUBJECT.
1	21	$W = 4c \times \sqrt{\frac{H^a}{13}}$.	Width of apron. H^a is height of weir shutter crest above floor. In sloping apron H^b, or solid crest above L.W.L., is used.
2	24	$b = \frac{H^c + d}{\sqrt{\rho}}$.	Base thickness of weir wall. H is height of masonry wall.
3	25	$t = \frac{4}{3}\,\frac{H - h}{(\rho - 1) \text{ or } \rho}$.	Thickness of apron. H = head, h = neutralised head.
4	27	$L = 10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$.	Distance of talus from crest wall with sloping aprons; $10\frac{1}{2}c$ or $11c$ may be used. q = unit flood discharge.
5	62	$W = 3\,(H^c + d)$.	Width of floor on a clay foundation.

CHAPTER VII

NO.	PAR.	FORMULA.	SUBJECT.
1	13	$L = 15c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$.	Distance of toe of talus from head work.
2	13	$W = 7c\sqrt{\frac{H^a}{13}}$.	Width of apron.
3	30	$t = \sqrt{\frac{3H}{2}}$.	Thickness of floor.

SUMMARY OF TABLES IN CHAPTER V

THE PRACTICAL DESIGN OF IRRIGATION WORKS

CHAPTER I

RETAINING WALLS

(1) RETAINING WALLS, whose function is the retaining or holding back of earthen banks, form one of the most considerable items of the component parts of masonry structures. The design of such should satisfy conditions comprising the utmost economy of material in combination with absolute efficiency. To effect this desirable end with any degree of accuracy and in

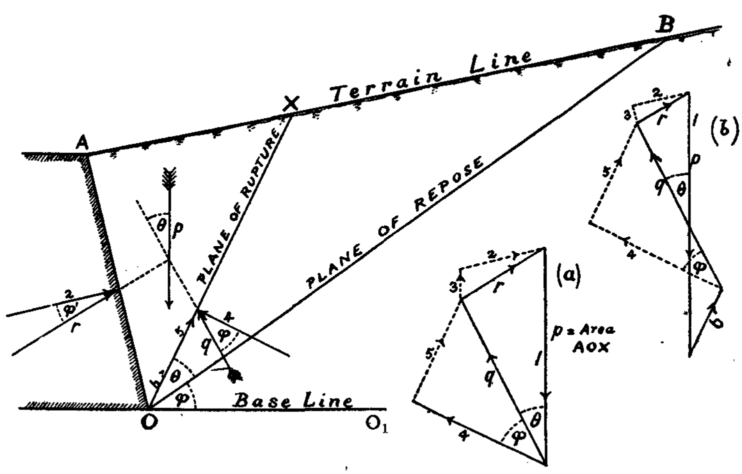


FIG. 1.

accordance with scientific principles, it will be necessary to possess a thorough understanding of the theoretical problem involved, without which the designer has to fall back upon empirical rules of doubtful value.

(2) The diagram in Fig. 1 presents a graphical statement of the theory of earth pressure. AO is the back of the retaining wall, AB the terrain line, or top surface of the bank which is upheld, and OO_1 the horizontal base line representing the natural surface of solid ground. OB is the plane of

repose of the earth forming this backing, the angle BOO_1, or ϕ, being thus the angle of repose, *i.e.*, the steepest slope at which the earth will stand when unsupported. The triangle AOB represents, therefore, the largest prism of earth held up by the wall.

In case of failure of the wall, the earth backing will break away in some line OX in the prism AOB, intermediate between AO and OB, and OX is termed the plane of rupture. Of all possible positions of OX, that one which ensures the maximum thrust on the wall has alone to be considered as the true plane of rupture, and thus the prism AOX is the portion of the earth backing tending to overturn the wall, and will be found, as hereafter explained, to have a much greater effect than the larger whole prism AOB, on the supposition that the plane of rupture is coincident with that of repose, which is not the case.

(3) By reference to Figs. 1a and 1b, the forces in equilibrium will be found as follows:—

(1) The weight of the prism AOX acting vertically.
(2) The normal reaction of the retaining wall acting at right angles to the direction of the back of the wall OA.
(3) The friction of the prism on the inner surface of the retaining wall acting upwards.
(4) The normal reaction of the plane of rupture OX.
(5) The friction of the prism AOX on the plane of rupture acting upwards.
(6) The cohesion of the earth in the plane of rupture acting upwards.

(4) Fig. 1a represents the force polygon of forces 1 to 5; Fig. 1b the same, but inclusive of (6). In both diagrams forces 2 and 3 are components of r, and 4 and 5 of q. The polygon is thus resolved into the triangle of forces $p\ q\ r$. Of these p is the weight of the prism AOX, represented by the area of the triangle AOX; q is the reaction of the plane of rupture, and r the resultant pressure on the back of the wall, both q and r being modified in direction by angles of friction. The direction of q is determined by the angle θ, which it forms with the vertical p; the length of r is dependent on the value of this angle θ. Now θ is also the angle between the planes of repose and of rupture, or the divergence of OB and OX. Hence the primary necessity of ascertaining the true value of this angle, on which that of the force r is dependent.

(5) The two diagrams, 1a and 1b, shows clearly the great reducing influence exerted on the value of r by the introduction of the comparatively small force (6) representing cohesion. No definite value can possibly be assigned to cohesion, as it must necessarily vary with differing soils and conditions of moisture, consolidation, etc.; for instance, from pure sand, which is almost devoid of the property of cohesion, to stiff clay in which this property is highly developed. Consequently we are compelled to omit

this force, though always present in a greater or less degree, from inclusion in the graphical statement.

Although this omission will apparently somewhat vitiate the value of the theory of earth pressure, as it presupposes a state of things in the condition of the earth which does not actually exist, yet it will be found that the neglect of cohesion will not preclude the successful use of the remaining five forces to which approximately correct values can be assigned, provided that the unknown value of cohesion be considered as supplying a definite, though not precisely ascertainable, factor of safety in favour of the stability of the wall.

It is well known that retaining walls, which according to received theory are designed of too weak a section to be safe, are in reality possessed of a large factor of safety. This point was fully brought out in the discussion on retaining walls in the "Minutes of the Proceedings of the Institution of Civil Engineers," Vol. LXV. Consequently the theory of earth pressure, though not reducible to absolute exactitude as in the case of fluid pressure, still will be found an indispensable and reliable guide in the economical design of retaining walls. The sections need only be designed of sufficient statical properties to withstand the theoretical earth pressure. That is to say, the moment of stability of the wall need be only just equal to that of the aggressive forces, or in equivalent graphical terms, the resultant centre of pressure of the combined forces can be allowed to pass just within the outer toe of the base of the section, leaving no factor of safety beyond that supplied by the unknown influence of cohesion and further, designs based on a slight modification of this principle will be found in agreement with commonly received and established practice.

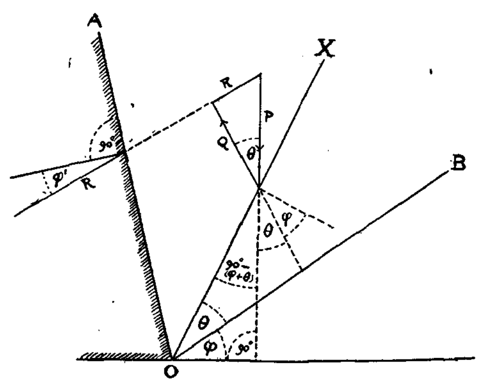


FIG. 2.

(6) Fig. 2 illustrates more fully the triangle of forces $p\ q\ r$ with their angles of divergence. The resultant r, it will be seen, forms an angle with the normal to OA of the assumed value of ϕ_1, tan. ϕ_1 being the co-efficient of friction of earth against the wall. This force, which acts in favour of the wall, causing a downward deflection in the direction of r, is neglected by some theorists, but without tangible reason. It is evident that friction must develop on this surface, as well as on the plane of rupture, on any movement of the wedge of earth AOX; consequently it cannot be ignored. The value of ϕ_1 is, however, not considered as equal to ϕ, and in Chalmers' "Graphical Determination of the Stresses in the Structures of Engineering," an authority on such subjects, the value of ϕ_1 is assessed at one half that of ϕ. This proportional value will be adopted. As regards ϕ, a consensus of authorities place

the slope of repose at $1\frac{1}{2}$ to 1, which is what ordinary soil naturally assumes. Some clays, however, have a much flatter slope of repose, as 2 or 3 to 1; in such cases, however, the backing should be formed of prepared material, which will not have a flatter angle of repose than $1\frac{1}{2}$ to 1. Sir Benjamin Baker, in the paper on the pressure of earth on retaining walls above referred to, assumes $1\frac{1}{2}$ to 1 as the slope of repose, although dealing with London clay with a natural angle of 3 to 1.

(7) The earth backing is thus theoretically considered as a fluid, having the specific gravity of earth, but subject to modifications by the influence of friction on the surfaces on which it slides. The reactions of the back of the wall and of the plane of rupture, in accordance with the properties of fluids, must be normal to those surfaces, but the action of friction causes obliquity in the directions of q and of r from the lines normal to OA and OX.

(8) In all subsequent investigations the value of ϕ will be assumed at $1\frac{1}{2}$ to 1, ϕ_1 at 3 to 1. With regard to p, the weight of earth will be taken as 1 cwt. per cubic foot, or specific gravity 1·8. Some clays weigh as much as 120 lbs. or 125 lbs. per cubic foot, or have a specific gravity of 1·9 or 2·0, but they are exceptional and can be met by special design. In hot countries, where alone irrigation is practised, the heavy clays common in Europe, which are due to glacial action, are rarely met with. Most soils do not weigh over 100 lbs. per cubic foot.

Position of the Plane of Rupture.

(9) It has already been observed (*vide* par. 2) that the position of OX is somewhere intermediate between OB and OA. By reference to Fig. 2, it will be seen that if the plane of rupture be assumed to coincide with that of repose, the triangle AOX will be coincident with AOB, and consequently p will be at a maximum value. The angle θ, however, disappears, and with it r. On the other hand, were OX coincident with OA, p would disappear. Some intermediate position of OX must be found which will satisfy the condition of giving a greater value to r than any other possible one. In works where this subject is treated by analytical methods, a formula is given whereby the angle θ can be found by calculation, but it is so involved as to be useless for practical purposes. In Chalmers' work the problem is solved by graphical process, but except in the case of a horizontal terrain line, the system adopted is too abstruse and complicated to be of practical value, though doubtless suitable for students as an exercise of ingenuity. The process adopted in this work for finding the correct value of θ, where the terrain line is not horizontal, is experimental in method and as simple as possible. Various positions of the plane of rupture are tentatively assumed, the value of r deduced in each case, and that inclination giving the greatest value to r is adopted as the correct value of θ.

(10) When the terrain line AB is horizontal and the angle of friction is neglected, OX will bisect the angle AOB. The proof of this can be found

in any work on the subject. When the friction on AO is taken into account, the direction of r will be more inclined, and thus will tend to slightly modify the position of OX, but to so small an extent that for all practical purposes it may be entirely neglected.

(11) In cases where the terrain line AB is inclined upwards or downwards, as shown in Figs. 3 and 4, the position of OX has to be found by a tentative process as exhibited below.

Fig. 3 is a case with a terrain line inclined upwards at a slope of 2 to 1. On AB any number of points, X_1, X_2, etc., are taken; these joined by the dotted lines with O, form so many planes of rupture, each having different

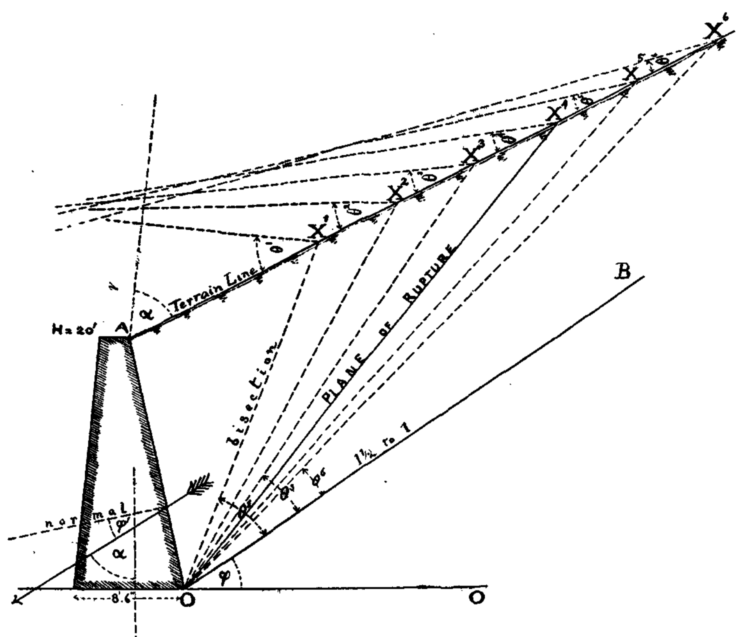


FIG. 3.

values of θ, viz., the angles X_1OB, X_2OB, etc. We have now to find which of these directions will ensure the greatest value of R, the resultant pressure on the wall. As the triangles AOX_1, AOX_2, etc., have the common base AO, their areas are directly proportional to their several sides, AX_1, AX_2, etc., consequently these sides can be taken as representative of their areas, *i.e.*, of p_1, p_2, etc., the weights of the prisms held up by the wall.

To avoid constructing a separate figure the force polygon can be joined on to the diagram by assuming AX as the vertical load line. At the point A the direction of r is set off, making an angle with p equal α on the figure, in the same way at the points X_1, X_2, etc., the angles θ_1, θ_2, which OX_1 and OX_2 make with OB are set off toward the line r. These lines, which represent the oblique reactions of the planes of rupture, cut r in several places. These several intercepts of the line above the point A give the value of r for each trial plane of rupture, and that angle effecting the largest intercept is to be adopted as the correct value of θ. In the case in question the line 4 gives that greatest value to r. OX_4 is then the correct plane of rupture, and X_4OB the correct value of the angle θ.

(12) In Fig. 4, where the terrain line slopes downwards, the process of finding the plane of rupture is identical with that already described. In this example the intercept of θ_3 is the largest, consequently OX_3 is the true plane of rupture. It will be noticed that with a rising terrain, as in Fig. 3, the position of the plane of rupture lies beyond the bisection of the angle AOX, whereas with a falling terrain line in Fig. 4 it lies within the bisection.

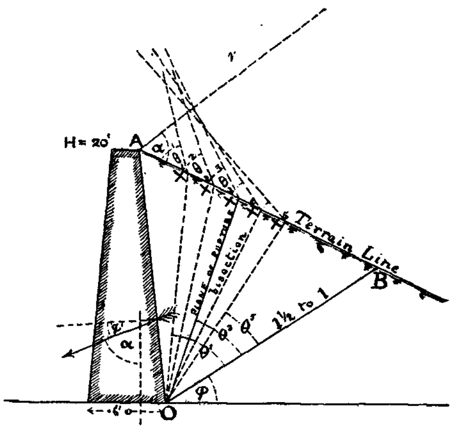


FIG. 4.

(13) A third case is illustrated in Fig. 5.

Here the terrain line is horizontal, but above this is superimposed a depth of water. This is an example of the common case of a masonry weir wall with earth backing and water passing over the crest. It is clear that the presence of the water will not modify the position of OX, which will bisect the angle AOB (*vide* par. 10). For purposes of simplification the area of the water rectangle aX has to be reduced to the corresponding weight of earth. Taking the specific gravity of the earth backing as 1·8, if the depth Aa of the water rectangle be divided by 1·8 and AA_1 made equal to this, the area of the smaller rectangle A_1X will then represent the corresponding weight of earth. In the example before us the depth of water Aa is 4 ft. AA_1 is then scaled off $= \frac{4}{1\cdot8} = 2\cdot22$, or this division can better be effected by graphical process. Make the horizontal $Ac = 1\cdot8$ on any scale, and $Ad = 1$, join cd, mark off $Ae = Aa$ and draw eA_1 parallel to cd, AA_1 will then be $\frac{aA}{1\cdot8}$. The horizontal through A_1 should be drawn, meeting OX continued, at X_1;

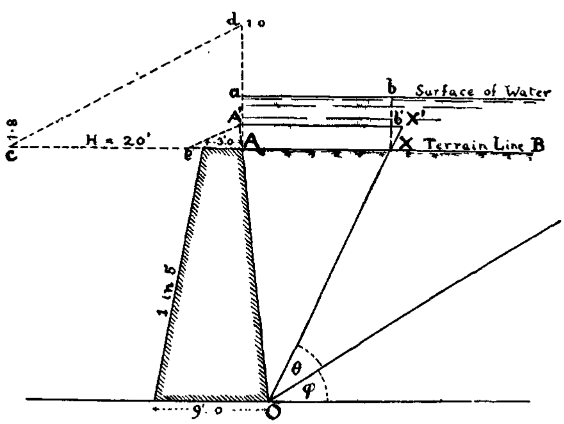


FIG. 5.

the enlarged triangle A_1OX_1 will then represent the combined weight of earth and water acting on the wall, as if composed of the former material.

The slight excess in the area of the superimposed parallelopiped AX_1, which is somewhat larger than the rectangle A_1X, is too trifling for consideration, though capable of exact adjustment by further lowering the line A_1X_1.

(**14**) Process of drawing resultant line of pressure to base of wall profile.

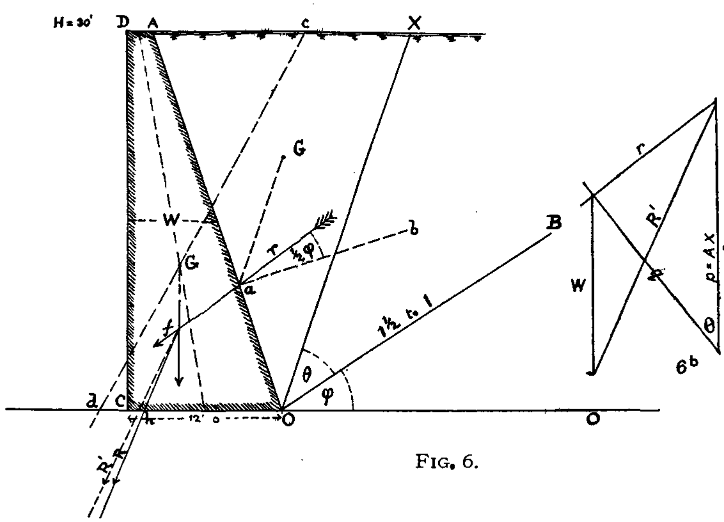


FIG. 6.

Fig. 6 represents a trapezoidal section of wall 30 feet high, 2 feet wide at crest, and 12 feet at base. The face is vertical. The object of drawing the resultant line of pressure through the base of the wall is that the position of the point where this intersects the base forms the criterion of the stability of the section adopted. If it falls at the outer toe, the section is of just sufficient statical stability; if outside, the wall will overturn. The further the point of intersection falls within the outer toe, the greater the stability. Excess, however, would lead to waste of material. As already noticed in par. 5 the unknown force of cohesion forms a large but indefinite factor of safety. It is, however, deemed advisable to ensure a certain small additional precise factor by designing the section so that the resultant line of pressure will intersect the base within the outer toe at a distance having a certain arbitrary proportion to the width of base. This will obviate any remote possibility of crushing the material at the edge of the base, which would be liable to occur were the maximum point of pressure concentrated at this point. This inset being a fraction of the base will ensure a uniform system of design applicable to all

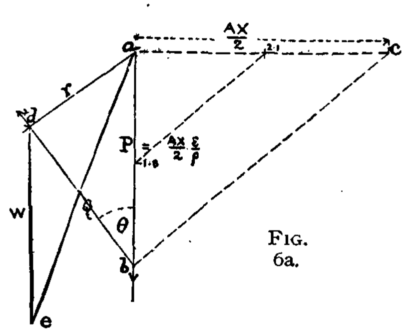


FIG. 6a.

sections and one which is in agreement with practical examples of known stability, the weight of these trapezoidal walls thus bearing a constant ratio to the base width b, the top width being a fixed dimension.

This fraction will be fixed at one-eighth the width of base, and has been decided on with due regard to assimilation of design to the proportions of retaining walls such as are commonly followed in practice. The problem of design now consists in adjusting the base width of the sections so that the resultant line of pressure will cut the base at a point $\frac{b}{8}$ within the toe. This can only be effected by a tentative process of trial and error. As the section is determined entirely by the base width, any alteration in the latter affects the back slope of the wall, and with it the oblique direction of r, which is at a fixed angle from the normal to the back; furthermore the area of the triangle of earth pressure, the position of OX and value of θ, are all modified by any alteration in the base width of the profile of the wall.

To express this in analytical terms would necessitate the use of formulas of so complicated a form as to prove too cumbersome by far for practical purposes, and the time spent in working them out would greatly exceed that employed in making the graphical solution, besides the increased liability to error.

(15) As already noted in par. 8 the specific gravity of earth backing is taken as 1·8 for these investigations. With regard to the walls, carefully built rubble masonry will weigh from 140 lbs. to 145 lbs. per cubic foot, *i.e.*, have a specific gravity of $2\frac{1}{4}$. This value is adopted in "Water Works Engineering"* as applicable to masonry dams, where it is stated that the specific gravity of the stone used with careful construction should, in cases of walls subject to water pressure, bring the weight of the built masonry to not less than 145 lbs. per cubic foot.

For retaining walls of ordinary construction a specific gravity of $2\frac{1}{4}$ is deemed too high, and in this work 2·1 will be adopted for earth-retaining walls and 2·25 for stone dams and weirs.

Granite masonry has a specific gravity of $2\frac{1}{2}$, or even 3 or more, with Portland cement mortar. Brickwork varies from 100 lbs. to 125 lbs. per cubic foot, *i.e.*, from specific gravities 1·6 to 2·0. In these investigations 1·8 will be adopted as the specific gravity of brickwork. The following is a *résumé* of the foregoing :—

Earth Backing, Specific Gravity ...	1·8
Brickwork, Specific Gravity... ...	1·7 to 1·8 in lime mortar
Ordinary Stone Masonry	2·1
Special...	2·25
Granite	2·5 to 3 in cement
ϕ	$1\frac{1}{2}$ to 1
ϕ_1	3 to 1

* "Water Works Engineering," by Tudsbery and Brightmore.

(16) To revert to Fig. 6. OB is the plane of repose, set out at $1\frac{1}{2}$ to 1 from the base line OO_1. The terrain line AX being horizontal the plane of rupture bisects the angle AOB, and $\theta = \frac{AOB}{2}$. The prism of earth AOX, which exerts active pressure on the wall will do so by sliding in its base OX. The resultant line of pressure of this weight will act through the centre of gravity of the triangle AOX, and a line, shown dotted on figure, drawn parallel to the base of the triangle AOX through G cuts the back of the wall AO at the point marked a. Oa is clearly one-third of AO, and thus we see that whenever the apex of the triangle of earth pressure is coincident with the inner point of crest of wall, the resultant earth pressure invariably acts at one-third the vertical height of wall above base or at $\frac{H}{3}$. In Fig. 5 the apex of the corresponding triangle is not at A but at A_1 higher up; consequently the centre of pressure will be at a greater height above the base.

(17) Having found a, the point of application of r, its direction is found by drawing ab at right angles to AO and from it setting off r upwards at an inclination of ϕ_1, or 3 to 1 from the normal ab. The line r should be continued through the profile. This force will now have to be combined with that consisting of the weight of the masonry acting through its centre of gravity.

By far the easiest method of finding the centre of gravity of a trapezoidal figure is as follows :—From A measure off Ac = base OC and similarly from C, Cd = crest AD, join cd and also the points of bisection of AD and OC. The intersection of these lines is G, the centre of gravity of the profile $ADCO$. From this point G draw a vertical, cutting r at the point f.

The directions of the two forces r and W are now connected on the profile, and the next step is to find out their values.

Let H denote the vertical height of the wall. Then the weight of the prism AOX taken as one unit wide is $\frac{H}{2} \times AX \times w$, w being the weight of a cubic foot of the earth backing. Similarly that of the wall will be $H \times W \times w_1$, W being the half width of the wall. Let ϵ be the specific gravity of earth, ρ that of masonry, then the weight of a cubic foot of water being 62·3 lbs. $w = \epsilon \times 62{\cdot}3$ and $w_1 = \rho \times 62{\cdot}3$. Hence the two expressions which have to be graphically equated become

$$H \times \frac{AX}{2} \times \epsilon \times 62{\cdot}3$$

$$H \times W \times \rho \times 62{\cdot}3$$

eliminating the common factors we have as representative of the loads on either side

$$\text{Earth} = \frac{AX}{2}$$

$$\text{Masonry} = W\rho$$

Reducing the earth expression to the base of the specific gravity of masonry, *i.e.*, by dividing both expressions by ρ, we have

$$\text{Vertical earth pressure} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$$

$$\text{Vertical wall pressure} = W.$$

(**18**) This arrangement for the simplification of expressions is commonly adopted in graphical calculations.

The force polygon is shown in Fig. 6a. Here the vertical force is represented by $ab = \frac{Ax}{2} \times \frac{\epsilon}{\rho}$. To effect this fractional multiplication graphically, ac is drawn horizontally $= \frac{AX}{2}$. On ab mark off on any scale, 1·8 the specific gravity of earth and on ac, 2·1, that of masonry, join these points and through c draw cb parallel to the last line. Then by similarity of triangles, ab or p will $= \frac{ac \times 1{\cdot}8}{2{\cdot}1} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$. From b set off bd inclined at the angle θ from p, and through a draw ad parallel to the line r in the profile Fig. 6. The triangle abd will then represent the forces p q and r. Having now obtained r in quantity as well as direction, it has to be combined with the force W. This can be best effected on the same diagram. The procedure is as follows:—Draw de vertically $= W$, *i.e.* $= 7$ ft. and join ae. The triangle ade will then be the force triangle required, ad being r and de W; the resultant of these two forces R will be ae, the third side of the triangle ade, in direction and magnitude. R is then the resultant line of pressure on the base which is required. In Fig. 6 from the point f draw R parallel to ae in Fig. 6a, cutting the base at the point h.

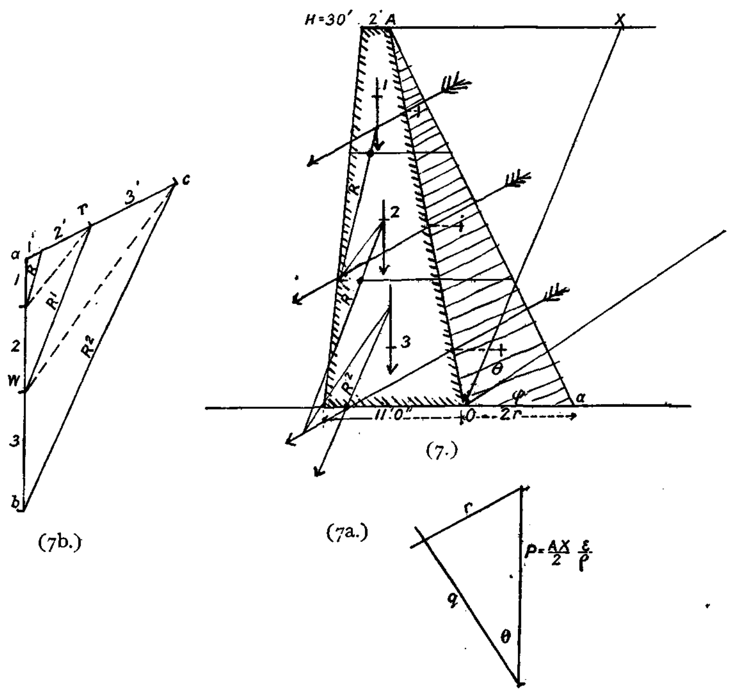
(**19**) The distance of h from the outer toe C will be found to be close to 1 foot 6 inches or $\frac{1}{8}$ base CO. When the wall is of brickwork of the specific gravity 1·8, the same as that of the earth backing, the force p is drawn equal to $\frac{Ax}{2}$. This is shown in Fig. 6b. The resultant R_1 reciprocal to that in Fig. 6b is drawn as a dotted line on the profile Fig. 6. Its incidence on the base CO is nearer the toe C than that of R. To bring the incidence of R_1 back to the point h, which latter is at $\frac{b}{8}$ distance from C, the base width will have to be increased by half a foot and be in terms of H; $\cdot 4H + \frac{1}{2}$, instead of $\cdot 4H$, as it stands in the drawing.

The Reciprocal Triangle of Earth Pressure.

(**20**) Hitherto we have only been concerned with the point of intersection of all the forces engaged on the base line of the retaining wall, the position of which point demonstrates the stability of the section, and which is identical in result with the analytical system of taking moments about the outer toe, or some other fixed point in the base line.

This procedure is sufficient in most cases, as it can be safely assumed that if the wall at the weakest part, viz., the base, is in stable equilibrium, the upper part will be still more so. When, however, retaining walls are of exceptional height, or are subjected to abnormal conditions of pressure, or are of unusual section, it will become desirable that the line of pressure be traced right through the section of wall from crest to base, thus enabling the designer by inspection of the weak points to modify the profile as may be expedient.

(21) In order to be in a position to draw the line of pressure, the distribution of the earth pressure on the back of the wall has to be



FIGS. 7, 7a, and 7b.

ascertained, the previous method only giving the point of application of the resultant on the base.

To effect this the triangle of earth pressure reciprocal to that of the earth pressing on the wall has to be drawn, reduced to a masonry base, so that ordinates drawn parallel to the base of this triangle to any point on to the back of the wall will truly measure the intensity of pressure at such point.

In Fig. 7 the triangle AOX is the area of earth backing, whose weight presses on the wall. The pressure developed by this prism is *nil* at the crest A, and is a maximum at the base, hence the form of the area of pressure will be that of a triangle having its apex at the crest A. The area of this triangle should equal the total earth pressure, and each ordinate drawn parallel to its base will measure the intensity of unit pressure at that point. Now the total earth pressure in terms of unit weight of masonry is, as we have already seen, rH, r, as in Fig. 7a, being the resultant of the earth pressure divided by H (the weight being unity), thus the area of the triangle must $= rH$, *i.e.*, its mean width will be r and its base will measure $2\,r$. In Fig. 7, if Oa is made $= 2\,r$ then the triangle AOa will be the reciprocal triangle of earth pressure, its area being rH. The base Oa being horizontal, the measure of the intensity of unit pressure at any point on the back of the wall AO will be the horizontal ordinate of the triangle AOa. The actual direction of the pressure, however, is oblique, being inclined to the normal to the back AO.

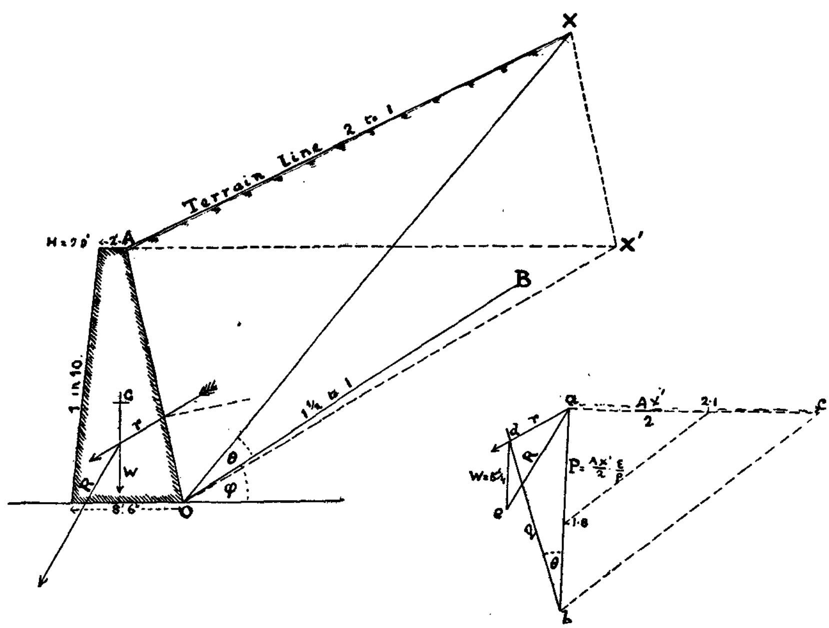


FIG. 8.

(22) The procedure of drawing the line of pressure will be briefly described. The wall and triangle of pressure are divided into laminas of equal depth, in this case three. Their common depth $\frac{H}{3}$ can then be eliminated from the graphical calculation, the weights of the laminas can then be represented in the force diagram 7b by their respective mean widths.

Thus in Fig. 7b the load line ab or W is composed of the three half widths of the wall 1, 2, and 3. In the same way ac or r is composed of the three half widths, 1_1, 2_1 and 3_1, of the laminas of the triangle of pressure. The incidences of r_1, r_2 and r_3 on the back of the wall are at the intersection of lines drawn through the CG's of the laminas. From these points the directions of the forces are inclined parallel to r in Figs. 7a and 7b. From the intersection of 1_1 with 1 in Fig. 7, R is drawn parallel to its reciprocal R

in 7b, the point where this cuts the end of the base of the lamina 1 is a point on the line of pressure. From the intersection of R continued, with 2_1, a line is drawn back parallel to its reciprocal in Fig. 7b, the resultant of the three forces 1, 1_1 and 2_1. Again from its intersection with the force 2 the line R_1 is drawn to meet the inclined force 3_1 parallel to its reciprocal R_1 in the force diagram, this intersects the base of lamina 2, giving a second point in the line of pressure and terminates as before, at its junction with the inclined force 3_1. The procedure as above described with regard to the reverse line gives the starting point of the final resultant R_2 which intersects the base of the wall. This latter point is clearly identical with that obtained by the simpler procedure formerly given.

(**23**) We will now proceed to show the method of obtaining the incidence of the resultant line of pressure on the base of a trapezoidal wall in cases where the terrain line is not horizontal and also where it is surcharged with a weight above the level of the wall crest.

Fig. 8 represents a case with the terrain inclined upwards. First the point X, or the intersection of the line of rupture with the terrain is obtained by the means described in par. 11. The area of the prism of earth pressing on the rear of the wall AOX cannot be represented as was done hitherto by $\frac{AX}{2} \times H$, but if the line XX_1 be drawn parallel to the back of the wall till it intersects the horizontal line AX_1 and the point X_1 joined with O, then the triangle AOX_1 is evidently equal to AOX and its area is equal to $AX_1 \times H$.

The centre of gravity of the prism is evidently on a line drawn parallel to the base OX, consequently the incidence of the inclined force r on the back of the wall will be at one-third of H. The same is the case with any triangle having its apex at the point A. The procedure of finding r is identical with that already described, p being made equal to $\frac{AX_1}{2} \times \frac{\epsilon}{\rho}$, and q set off at the angle θ from its extremity b, which intercepts the inclined line r drawn from the origin a.

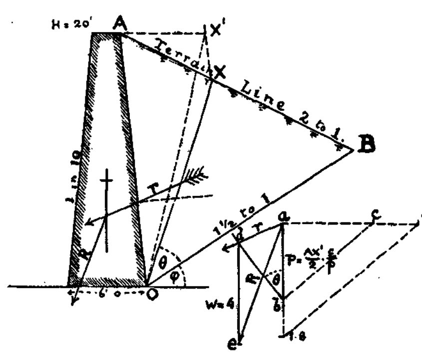
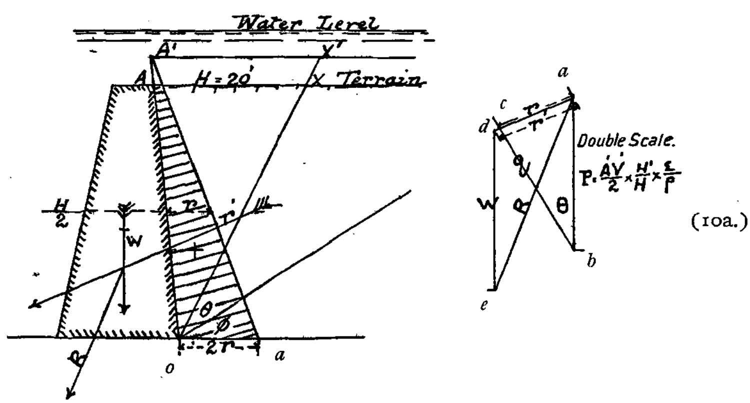


FIG. 9.

(**24**) In Fig. 9 we have a similar case, but with the terrain line inclined downward. The procedure is identical with that in the last case.

(**25**) Fig. 10 is a replica of Fig. 5, in which the terrain line is loaded with water. As already noticed in par. 13 the prism of earth acting on the wall is the triangle A_1OX_1. Consequently in the force polygon Fig. 10a, p is made either by graphical or arithmetical process equal to $\frac{A_1X_1}{2} \times \frac{\epsilon}{\rho} \times \frac{H_1}{H}.$ The last fraction enables H to be eliminated as a common factor. By setting off q at the angle θ from the base of the load-line the value of r is obtained, as also the base Oa of the triangle of pressure AOa, which is $2r$. The earth pressure is represented not by the triangle A_1Oa, but by the hatched trapezium AOa, consequently r, in Fig. 10, is not representative of the mean pressure, but r_1, the half-width of the trapezium which is greater than r, does so. Hence in Fig. 10a, r_1 has to be measured off from the point of origin a, to d, and the vertical W or de is drawn through this point d, equal to the half-width of the trapezoidal wall; ae is then the direction of R. In the profile Fig. 10 the inclined force r_1 cuts the back of the wall at the inter-



FIGS. 10, 10a.

section of a horizontal through the centre of gravity of the trapezium AOa, not at that of the triangle A_1Oa.

(26) The consideration of further abnormal conditions to which retaining walls are subject will now be deferred until the principles of design are worked out in regard to trapezoidal walls of various heights and face batters under ordinary conditions of earth pressure. For this purpose Figs. 11 to 15 have been prepared, being a series of diagrams of sections of equal strength but of varying face batters, placed side by side for purposes of comparison. Inspection of these diagrams will at once demonstrate to the eye the influence exerted in the economical design of the sections by face batter, the diagrams also furnishing reliable types of sections for practical use, based on which formulas easy of application can be deduced.

In all cases the specific gravity of the material of the wall is taken as 2·1, that of the earth backing as 1·8, the top width at crest is a fixed dimension of 2 feet, and each section is so designed that the centre of pressure falls close to the outer toe at a distance of one-eighth of the base from that point. As noted in par. 19, if the wall is of brickwork with a specific gravity the same as the earth backing, the incidence of the resultant on the base will be beyond $\frac{b}{8}$ from the toe at a point about $\frac{b}{10}$ from the same point.

Thus brick walls built of these proportions will be safe though of less stability, or if the same statical condition as was deemed requisite in the case of stone walls be insisted on, the base should be increased from 6 inches in the higher to 4 inches in the lower sections, right through.

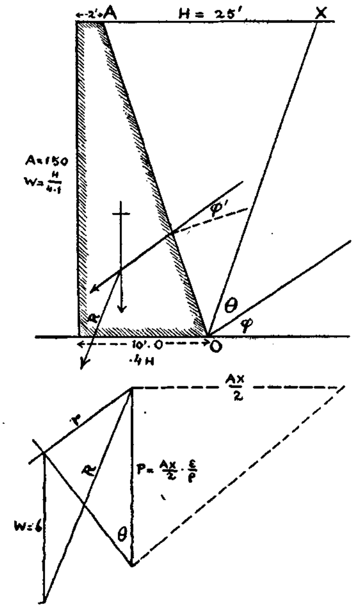


FIG. 11.

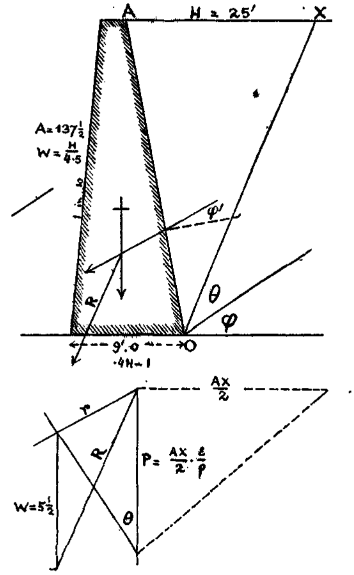


FIG. 12.

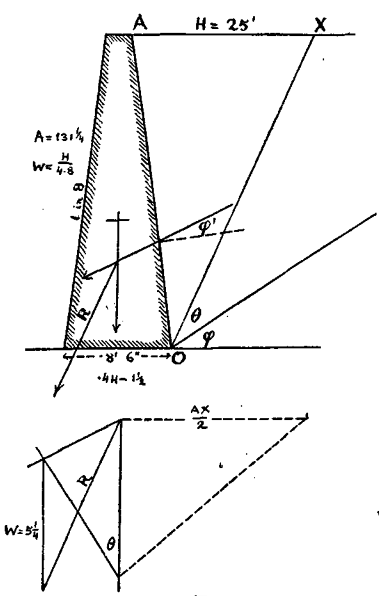


FIG. 13.

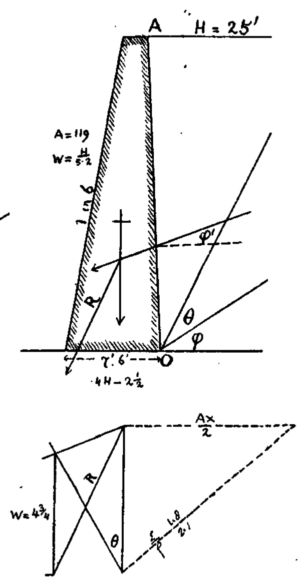


FIG. 14.

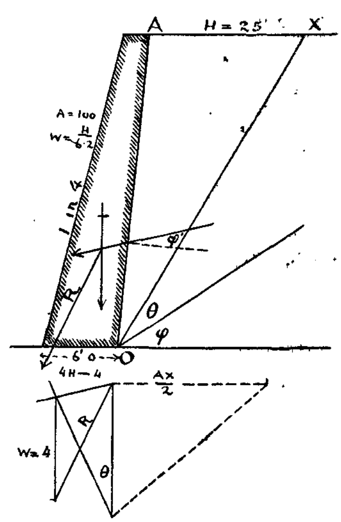


FIG. 15.

Reference to the diagram series will show that with the change of face batter the consequent gradual reduction of the base width causes the back slope to become steeper, till at last in Fig. 15 it vanishes and the back slope becomes inward in sense. This diminishing of the base is so regular and so clearly governed by the height that the base width can well be expressed in terms of H, and rules can be framed giving the required base width of any wall as a fraction of its height. In this, the base for a vertically faced wall, as in Fig. 11, will be used as a standard, the whole of that series working out to $\cdot4H$. The base width of Fig. 12 works out to $\cdot4H-1$, of Fig. 13 to $\cdot4H-1\frac{1}{2}$, of Fig. 14 to $\cdot4H-2\frac{1}{2}$, and of Fig. 15 to $\cdot4H-4$.

Sections have been drawn out for every depth from 30 to 10 feet for each face slope, though not given here, and the tabular statement below is the result.

TABLE I.

Face.	H.	Base.	Area square feet.	$\frac{H}{W}$	Height (H) when back becomes vertical.	Face.	H.	Base.	Area square feet.	$\frac{H}{W}$	Height (H) when back becomes vertical.
	Feet.				Feet.						Feet.
Vertical.	30	$\cdot4H$	210	$\frac{1}{4\cdot3}$	5	Batter 1 in 10.	30	$\cdot4H-1$	195	$\frac{1}{4\cdot6}$	10
	25	$\cdot4H$	150	$\frac{1}{4\cdot2}$			25	$\cdot4H-1$	$137\frac{1}{2}$	$\frac{1}{4\cdot5}$	
	20	$\cdot4H$	100	$\frac{1}{4}$			20	$\cdot4H-1$	90	$\frac{1}{4\cdot4}$	
	15	$\cdot4H$	60	$\frac{1}{3\cdot8}$			15	$\cdot4H-1$	$52\frac{1}{2}$	$\frac{1}{4\cdot3}$	
	10	$\cdot4H$	$27\frac{1}{2}$	$\frac{1}{3\cdot4}$			10	$\cdot4H-1$	25	$\frac{1}{4}$	
Batter 1 in 8.	30	$\cdot4H-1\frac{1}{2}$	$187\frac{1}{2}$	$\frac{1}{4\cdot8}$	12·8	Batter 1 in 6.	30	$\cdot4H-2\frac{1}{2}$	$172\frac{1}{2}$	$\frac{1}{5\cdot2}$	17·3
	25	$\cdot4H-1\frac{1}{2}$	131	$\frac{1}{4\cdot8}$			25	$\cdot4H-2\frac{1}{2}$	119	$\frac{1}{5\cdot2}$	
	20	$\cdot4H-1\frac{1}{2}$	85	$\frac{1}{4\cdot7}$			20	$\cdot4H-2\frac{1}{2}$	75	$\frac{1}{5\cdot2}$	
	15	$\cdot4H-1\frac{1}{2}$	49	$\frac{1}{4\cdot6}$			15	$\cdot4H-2\frac{1}{4}$	43	$\frac{1}{5\cdot4}$	
	10	$\cdot4H-1\frac{1}{2}$	$22\frac{1}{2}$	$\frac{1}{4\cdot4}$			10	$\cdot4H-2$	20	$\frac{1}{5}$	
Batter 1 in 4.	30	$\cdot4H-4$	150	$\frac{1}{6}$	Back is sloped inward throughout.						
	425	$\cdot4H-4$	100	$\frac{1}{6\cdot2}$							
	20	$\cdot4H-4$	60	$\frac{1}{6\cdot7}$							
	15	$\cdot4H-3$	38	$\frac{1}{6}$							

NOTE.—H = vertical height. W = mean width of walls (both in feet). $\rho = 2\cdot1$.

(27) In practice many revetment walls, as wings of bridges, have a crest sloping down at $1\frac{1}{2}$ or 2 to 1 in uniformity with the side slope of an embankment which they support. If designed of equal strength throughout, in accordance with the base widths in the Table, the end portion of the wall in some cases would have an inward and the upper an outward back slope. Although it is possible to build the wall in this way, the outer slope gradually changing from inward slope to vertical and again outward, yet it would be a somewhat awkward construction, and for the sake of simplicity it would be as well to continue the wall as soon as the outer slope is merged into the vertical as one with a vertical back; this will cause the lower portion of the wall to have an excess of strength and material. This fact would militate against the use of excessive batter as 1 in 4, or even 1 in 6, for crest sloping walls, as the saving effected in the higher portion would be discounted by the excess in the lower. In the tabular statement given above the height at which the back slope disappears and the back becomes vertical is noted in the last column of the Table.

(28) The great economical advantage of face batter to retaining walls is clearly proved by the sections given and by the Table. For instance, the vertically faced wall in Fig. 11 has a sectional area of 150 square feet and a proportion of $\frac{W}{H}$ or $\frac{\text{mean width}}{\text{height}}$ of $\frac{1}{4\cdot 2}$. Whereas in Fig. 15, of equal strength, the area is 100 square feet, and the fraction $\frac{W}{H}$ is $\frac{1}{6\cdot 2}$. In this case the back of the wall has an inward slope, thus approximating to a pitched slope.

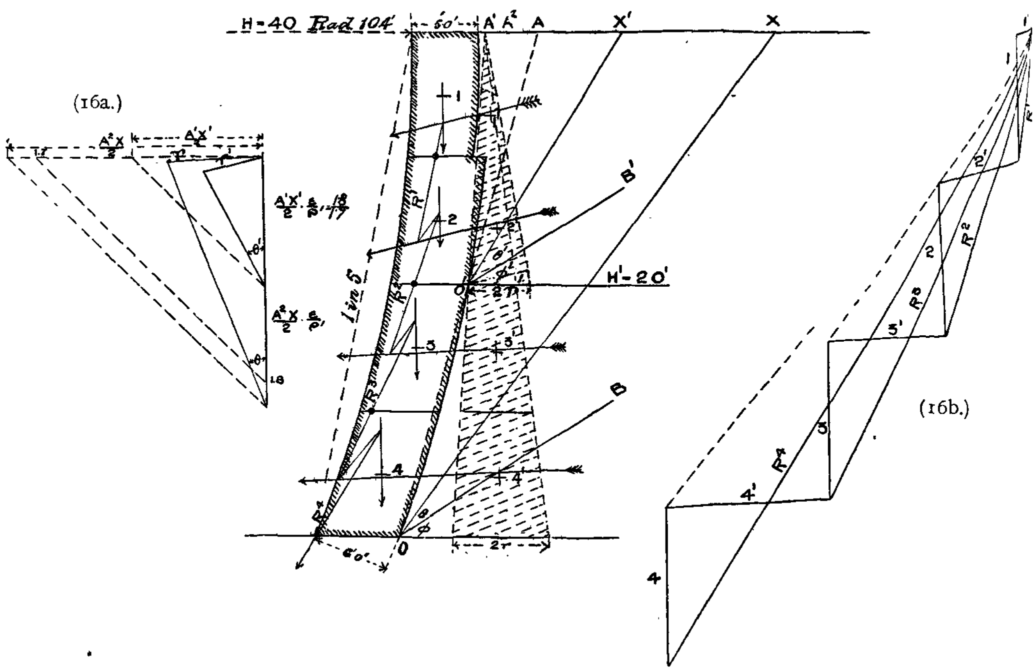
(29) Brunel's curved walls had a value of $\frac{W}{H}$ of nearly one-seventh, and leaned over to that extent that the centre of gravity of the wall fell outside the heel of the base. A specimen of the section of one of these retaining walls is given in Fig. 16.

For the graphical calculation, the back of the wall is considered as consisting of two planes A_1O_1 and O_1O, O_1 being at half the height or 20 feet above the base. As these portions of the back have different inclinations to the vertical, the directions of their several planes of rupture and values of θ and r will also vary. Thus a separate graphical calculation will have to be made for each of the portions, viz., the whole AO and the part A_1O_1.

The lower portion of the back OO_1 is continued up to meet the terrain line at A. The planes of repose OB and O_1B_1 are then set out at $1\frac{1}{2}$ to 1 with the horizontals from O and O_1, and the planes of rupture OX and O_1X_1 are found by bisecting the angles AOB and $A_1O_1B_1$. The angles θ and θ', are obtained by this construction. The area of earth pressing on the whole wall is the four-sided figure A_1O_1OX, or the triangle AOX + the triangle AO_1A_1. If the apex A be moved to A_2, so that $AA_2 = AA_1 \times \frac{H_1}{H}$, then the enlarged triangle A_2OX, will equal the combined area of the two triangles just noted. H_1 being $= \frac{H}{2}$, the position of A_2 will be at the bisection of AA_1. Hence for purposes of measuring the area of the earth triangle, A_2X will be considered as the base in lieu of AX.

With regard to the upper portion, the triangle $A_1O_1X_1$ represents the earth prism pressing on the upper half back A_1O_1, A_1X_1 will then measure the comparative weight of this part.

(**30**) To find the values of r for each division the force triangle 16a is now constructed. The vertical load line is made $= \frac{A_2X}{2}$, for the whole depth of wall, and on the same line the value of p for the upper half, viz., $\frac{A_1X_1}{2}$, is



FIGS. 16, 16a, 16b.

marked off. From the extremity of the load line and also from the higher point in it the angles θ and θ_1 are set off, intercepting r and r_1 on the two lines representing the directions of these forces, which lines are inclined at angle ϕ, above the normal to the planes AO and A_1O_1 respectively.

The next step is to construct the reciprocal triangles of earth pressure. With regard to the upper portion, $2r_1$, drawn horizontally from the point O_1, will form the base of the triangle, its apex being at A_1 (*vide* par. 21).

The lower base could in similar manner be measured $= 2r$ from the point O, the apex of this larger triangle of pressure being at A_2. It will be more convenient, however, to make the inner side of this triangle coincide with that of the upper, viz., with the line A_1O_1 and A_1O_1 produced. The base $2r$ will then be measured from the intersection of A_1O_1 produced with the base line. It is clear that the area of the trapezium below O_1 will be the same, whatever inclination is given to its side, as all will have the same base and lie between the same parallels.

The procedure now closely follows that already described in the case of Fig. 7. First the wall is divided into parts of equal height, in this case 1, 2, 3 and 4. The triangle and trapezium of earth pressure forming the corresponding divisions of the wall and of the earth pressure are next found, and vertical lines drawn through the former and horizontal lines, *i.e.*, parallel to their bases through the latter. Through the points where these horizontal lines cut the back of the wall, the four feathered lines representing the directions of r and r_1 are drawn.

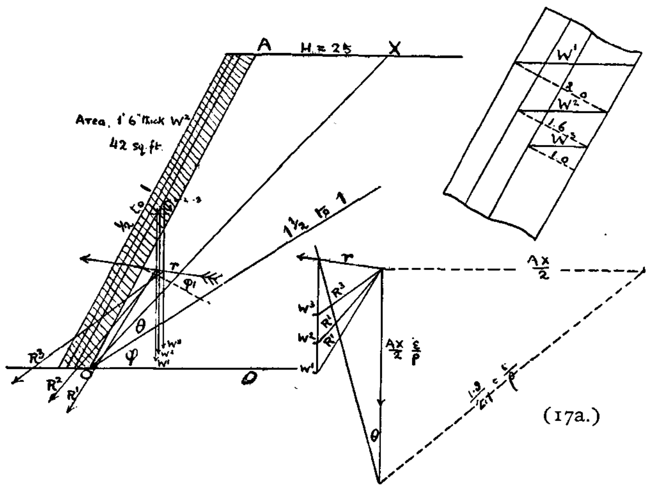
(**31**) We are now in a position to construct the Haesslers force and ray polygon, which, owing to the varied inclinations of r, presents some difference to that illustrated already in Fig. 7b. First, from the nucleus O (Fig. 16b), the first inclined force 1_1 is measured in its proper direction and equal in length to the half width of the triangle 1_1 on Fig. 16. From its extremity, the vertical force 1, is set out equal in length to the half width of division 1 of the wall section. In the same manner the other forces 2_1, 2, 3_1, 3 and 4_1, 4 are set out. The resultants, R_1, R_2, R_3 and R_4, are clearly combinations of 1_1 1, 1_1 1 2_1 2 and so on, and the dotted rays drawn from the outer corners are the resultants of the odd numbers 1_1 1 2_1, 1_1 1, 2_1 2 3_1, etc. The drawing of the reciprocals on the wall section is identical with that already described with reference to Fig. 7, and need not be repeated. The intersections of R_1, R_2, R_3, and R_4, with the base lines of the divisions 1, 2, 3 and 4 give four points on the line of pressure. The relative position of these proves the stability of the wall. The weakest point is at the base, and it is evident that the stability of the wall would be considerably improved by the addition of an off-set from the face near the lower end.

(**32**) The centre of gravity of the whole wall falls without the inner toe. It is not shown in Fig. 16. Thus without the earth backing the wall would fall backwards. This was provided for during construction by tightly packing the back of the wall with rammed earth as the work proceeded. Several walls of similar light pattern were constructed by Brunel on the Great Western Railway.

A curved face batter is very suitable for walls at or over 30 feet in height, as it corresponds more closely with the line of pressure and results in a more economical distribution of material. For irrigation works, however, this section is not very suitable except in the case of lock walls, which really belong to navigation canals.

(**33**) Retaining walls, which overfall to such an extent as to be supported entirely by the earth, instead of *vice versâ*, are termed *pitched* or *riprapped* slopes. Figs. 17, 18 and 19 are examples in question.

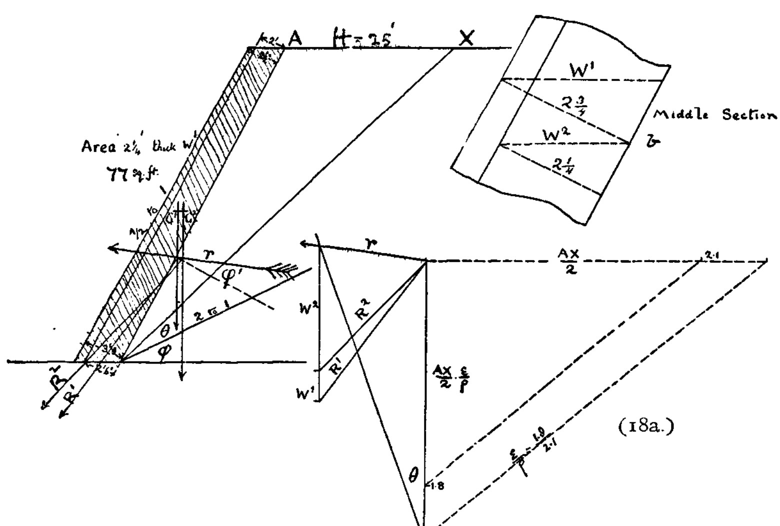
In Fig. 17 the pitched slope is ½ to 1, and the diagram exhibits tentative methods of ascertaining what thickness of pitching or riprap will be required. The procedure is identical with those already described for obtaining the centre of pressure on the base of retaining walls.



FIGS. 17, 17a.

Trial is made of three thicknesses: 2 feet, $1\frac{1}{2}$ feet and 1 foot.

W in the force polygon Fig. 17a has therefore three different values and consequently R has likewise three different directions and values. Of the



FIGS. 18, 18a.

three, R_2 intersects the base, consequently the intermediate thickness of $1\frac{1}{2}$ feet is sufficient for purposes of stability.

In Fig. 18 the slope is likewise $\frac{1}{2}$ to 1, but ϕ, the angle of repose, is given an inclination of 2 to 1. In this example, two trial mean thicknesses of $2\frac{3}{4}$ and $2\frac{1}{2}$ feet are adopted, the pitching being made 6 inches thicker at the base than at the surface. Of the two thicknesses the greater, or R_1, will be the correct one to adopt.

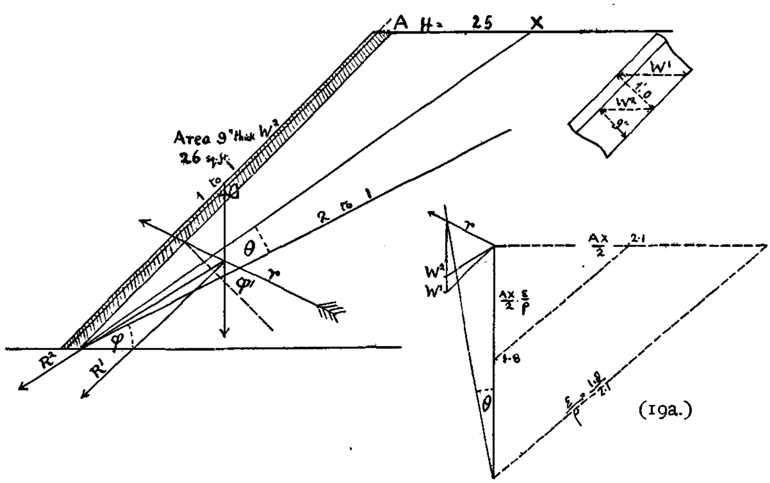
In Fig. 19 the slope is 1 to 1 with the angle of repose 2 to 1, showing that a 9-inch thickness of pitching is more than sufficient.

These examples prove the economy in the substitution of a masonry pitched slope for a retaining wall. A river weir designed on this principle is illustrated in Fig. 24.

(34) Varieties of solid retaining walls used in irrigation works may be classified as below as regards section :—

A, walls with horizontal crests with earth pressure to base.

B, walls with sloping crests with earth pressure to base.



FIGS. 19, 19a.

C, walls with earth pressure for a fraction of height above foundations.

D, walls in parallel pairs termed land wings or straight returns.

E, single land wings or straight return stop walls with earth pressure equal on either side.

F, surcharged walls.

G, walls with unequal earth pressure on both sides.

H, weir walls with earth backing to near crest which is overlaid by water.

Of these classes C, E and H are peculiar to irrigation works, not being found component parts of ordinary railway works. As regards class C this peculiarity is due either to a fall in the canal bed, at the canal cross work, when, as is generally the case, the foundations of the up-stream walls are taken down to the same level as that of the down-stream walls ; the portion below the upper bed being thus built in a trench in the solid ground is not subject to earth pressure below that level. Walls of class E are used in falls, either river or canal, where there is no regulating or traffic bridge, and the weir wall modified in section and raised to height of bank is run straight on into the embankment on either side of the overfall gap. Class H has already been investigated in part (*vide* Fig. 10).

(**35**) Figs. 20 to 24 contain outline plans and elevations of various dispositions of walls in cross canal works.

Fig. 20 represents the skeleton outline of a level floor opening, either for a bridge to pass traffic or a regulating bridge with grooved piers and gates. The archway, piers, etc., are not shown as the object of the sketch is merely to illustrate the different systems of retaining walls and the arrangement

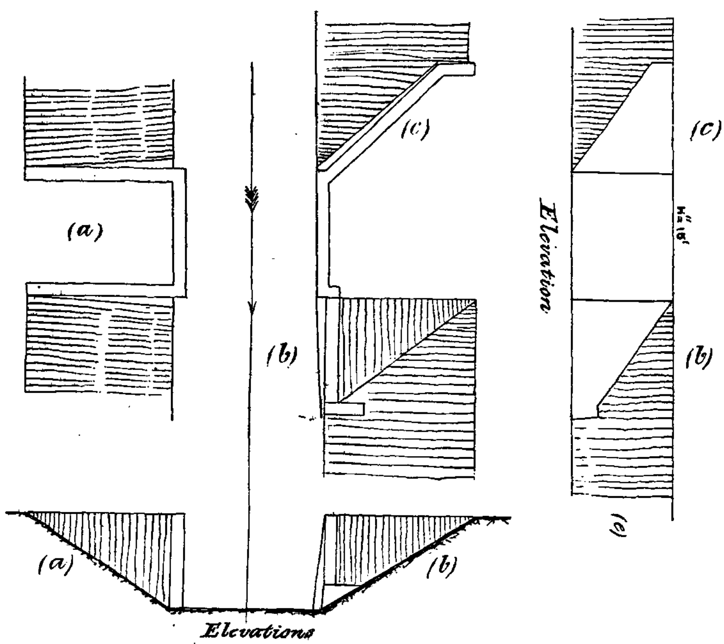


FIG. 20.

of the earth slopes. Figs. 20a are half plan and elevation of straight return or land wings similar to those used in railway over-bridges. These run back at right angles to the axis of the work at either side of the abutment, forming with the latter what is sometimes termed a U abutment.

This direct return type has this in its disfavour: that the crest being horizontal, the walls are of the same heavy section right through and, as ordinarily built, are not so economical as the sloping crest flank wall illustrated in Figs. 20b to 23b. Further, in this latter disposition the flank walls form a guide for the water, a function which is valuable in case of increased current, as occurs below a fall.

(**36**) Figs. 20c, 21c and 23c represent a type of splayed wing now in much favour for various reasons for use up stream, generally in combination with (*b*) down stream.

This wall is naturally more expensive than a splayed and crest-sloped wall; but it has the advantage of great flank protection against soakage or percolation under a head of water, and in addition, by causing the widening of the embankment, enables the abutment to be made much shorter, or if the latter is already wide, carrying arches of a bridge, it forms a level space close to the work and clear of the roadway, which space is found useful for many purposes. It further guides the current into the opening, which is always less than the water surface of the channel.

With regard to (*b*) in plan and elevation in Fig. 20, these are views of a sloping wing with crest parallel to the axis of the work, the base or spring line of the batter being splayed outward at an inclination to the crest, of ratio of batter × ratio of fall of crest. In this sketch, the face batter being 1 *in* 10 and slope of crest $1\frac{1}{2}$ to 1, the divergence of the base line from axis will be

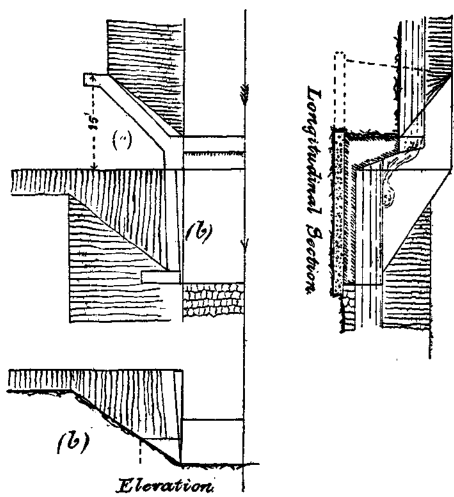


FIG. 21.

1 in 15. The wing is provided with an end return 4 feet high, whose length should equal its height × ratio of slope of bank, *i.e.*, $4 \times 1\frac{1}{2} = 6$ feet. This return wall is surcharged, but it will be strong enough in section if built of the same thickness as the end of wing abutting on to it and given a face batter.

The advantage of a water wing of this description is obvious: it guides the water issuing through the opening in a straight course down the channel and also protects the banks down stream from contact with water, except possibly at its lower entremity, which is removed far from the main work.

Where, as is often the case, a masonry floor extends down stream beyond the abutment, flank wings of this description are a necessity, or else with pitched slope a dwarf wing can be substituted, as shown in Fig. 24a.

(**37**) In Fig. 21 a similar disposition of walls is exhibited as the *b* and *c* series in the last figure. In this case the floor is not level, but a drop of

$4\frac{1}{2}$ feet occurs in the canal bed. It will be seen that the abutments are very narrow, only sufficiently wide to cover the end of the cross drop wall. Here a direct return could not be adopted, or if so the abutment would have to be lengthened to contain the proper top width of bank approach. The efficient protection of the work from outflanking and the provision of the necessary width of bank are clearly effected in a satisfactory and economical manner by the splayed wall (*c*) (termed *flaring* wings in America).

Down stream the sloping crested water wings (*b*) display a slight difference in the way the batter is arranged. The base lines are parallel with the axis of the work, consequently the crest lines diverge inwards. This looks somewhat awkward on elevation, but there are cases where even a small outward splay of the water wings would be deemed objectionable, it being essential that the current be guided in an absolutely axial direction.

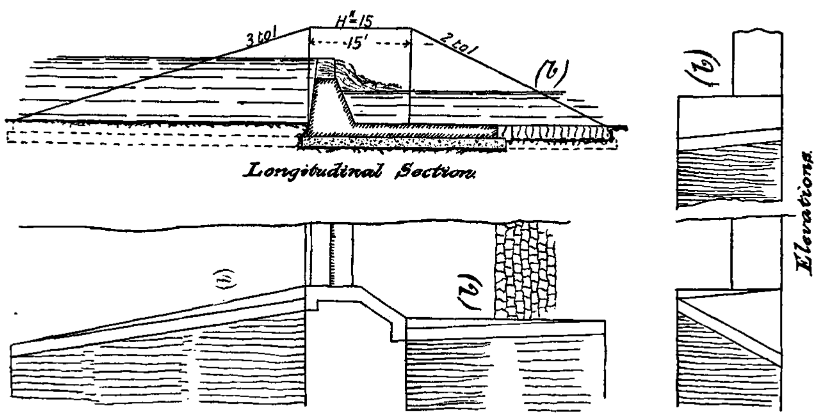


FIG. 22.

(38) Fig. 22 illustrates the case of a tank fall built in a depression which has the surface of ground at about floor level. In such cases the splayed wings up stream are not suitable, being necessarily of heavy section, and the connection with the tank embankment, which in this case has a flat inner slope of 3 to 1 and runs at right angles to the axis of the work, is rather awkward. An abutment of width equal to that of the embankment, combined with splayed sloping wings as illustrated, is about the best arrangement. In this case the down-stream wings (*b*) are shown set back, the outlining of the abutment being modified by having the corner bevelled off to prevent sharp corners and consequent eddies in the current. The dog-legged pattern down-stream wings shown in the next figure is a still better type.

The water wings (*b*) are both shown without returns, running right down to the toe of the embankment on either side. They are sometimes constructed splayed as far as end of down-stream floor, and thence continued parallel to axis, or else curved as railway under-bridge wings, the latter arrangement coinciding with the old Madras practice. The curved walls are, however, rightly tabooed in modern works as being conducive to pooling, *i.e.*, a destructive rotary movement in the water which has to be guarded against.

(39) Fig. 23 is a modification of the conditions prevailing in the last example, showing the level of natural ground as 5 feet higher, the down-stream channel being in cutting. In cases such as this the splayed wings (*c*) can be employed up stream with advantage, the length of abutment being only just sufficient to contain the end of the weir wall and the splayed returns arranged so as to afford the necessary width to the embankment.

The down-stream wings are of the dog-legged pattern, the abutments being also battered as well as the wings. This arrangement, which has

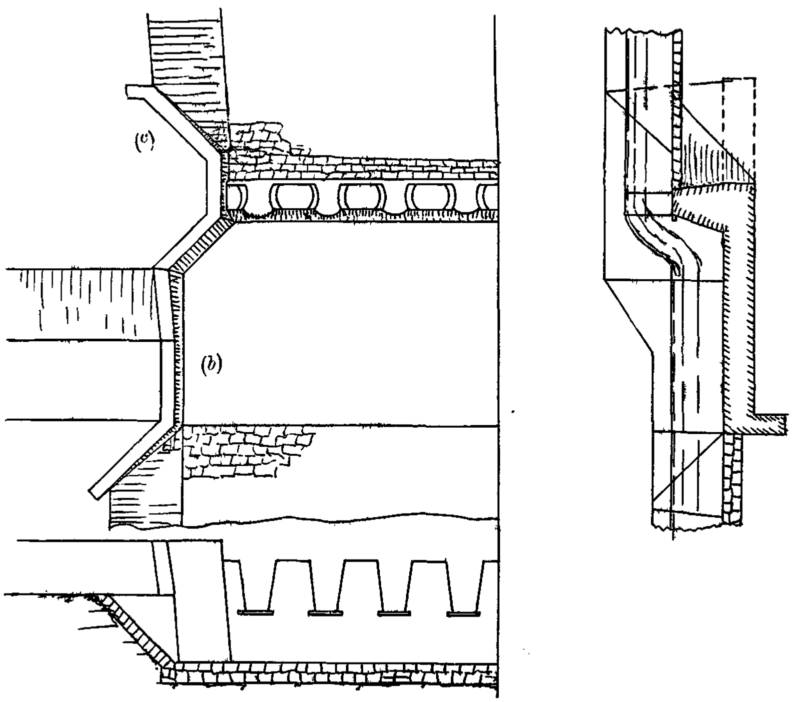


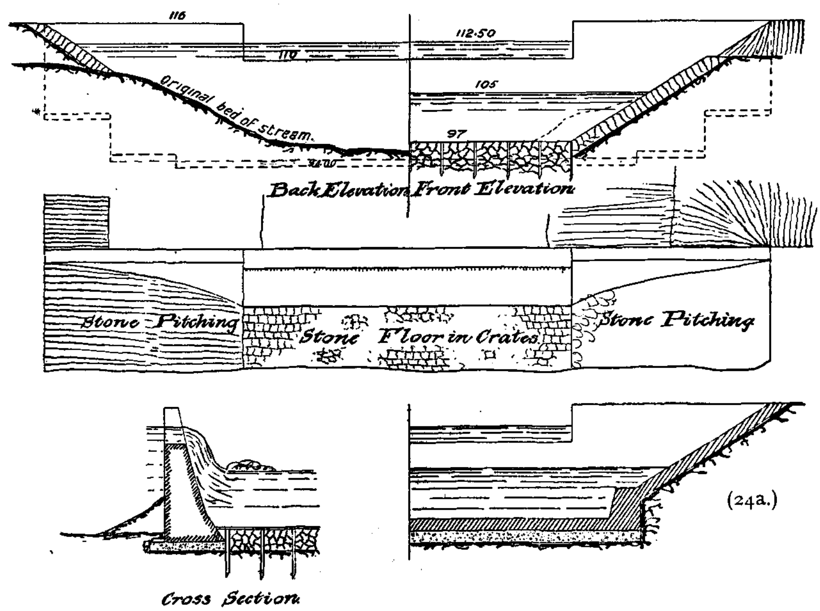
FIG. 23.

been adopted in the designs for canal falls given in Chap. IX., is probably the best for this purpose.

(40) Fig. 24 contains elevation plan and section of a weir across a river which was constructed by the author in Burma. This illustrates class E. There being no wing walls proper, the throw back of end of embankment is effected by continuing the weir in either direction with a raised crest, the single flank walls thus answering the purpose of the double land wings. In this case the flank continuations were stepped up in foundation, and, owing to the water pressure to which they are subject, were carried on of the same section as the weir proper. A photograph of this work is shown in Chap. IX.

The banks below this weir were pitched with stone, and the floor was formed of cribbed stone planked over and further protected by a subsidiary crib weir of loose stone, so as to form a water cushion. This style of construction was necessitated by circumstances, and can hardly be deemed otherwise than a temporary arrangement, although it has lasted some years. The half section marked 24a shows the probable eventual substitution of a masonry floor flanked by dwarf walls and stone pitching laid in mortar. A similar design for a canal fall is exhibited in Fig. 4, Chap. IX.

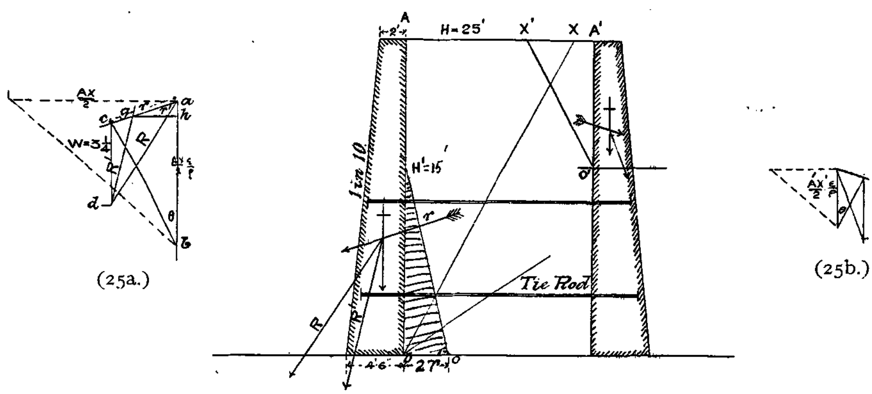
Having thus touched upon the subject of the disposition of retaining walls as are commonly used on canal works on plan and elevation, it is now proposed to continue the investigations of walls under abnormal conditions of earth pressure.



FIGS. 24 and 24a.

(41) Fig. 25 represents the section of a pair of land or direct return wings, such as are commonly employed in railway over-bridges. It will be seen that OX, the plane of rupture, clears the opposite wall, and consequently the earth pressure on AO is identical with that which would take place were the opposite wall non-existent. This would apply equally to either wall, and so, notwithstanding the reduced width of the earth retained, no diminution of the section of the walls is admissible. This fact causes this type of wall, if of any considerable height, to become an expensive construction, and so it is superseded where practicable, as already noted, by sloping wing walls. Owing, however, to the small space between the return wings, which seldom exceeds 20 feet, all undue earth pressure can be neutralised by the simple device of connecting the walls together with iron tie-rods. If this is done, a very light section of wall only need be provided, and thus equipped, the land wings can compete successfully as regards economy of section with the other types.

(42) In Fig. 25 the walls are shown of reduced section, the face batter being 1 in 10, with back vertical. In Fig. 25a the usual triangles of forces *abc* and *cda* are constructed, giving the values of r and R. The reciprocal of the latter resultant transferred to the section falls well beyond the outer toe of base. In designing the section of the tie-rods required, credit should be given to the wall for the proportion of the horizontal component of R absorbed by it, the balance being provided for by the tension in the tie-rods. A simple graphical process will solve the point. According to the rules adopted for the design of retaining walls, the resultant R should cut the base one-eighth of its width recessed within the toe. If this proportion be set off on the section, and a line R_1 drawn from this point to the intersection of the feathered arrow r with the vertical through the centre of gravity of the section, R_1 will



FIGS. 25, 25a, 25b.

then represent the proper inclination of the modified resultant. Now on Fig. 25a draw a line *dg* parallel to R_1 from the extremity of W, intersecting r at the point g. The intercept *cg* then represents the proportion of r which is absorbed by the wall, and the remaining *ga* that which has to be neutralised by the tension of the tie-rods. As, however, the tension is horizontal, the horizontal equivalent of this inclined force need only be considered; *gh* or r_1, drawn horizontally, will then represent this force.

Reverting to Fig. 25, with a value of $H_1 = 15$ feet, the line of pressure will fall at about the required distance within the toe of that base, so that above this point there is no necessity for extraneous assistance. Below, however, the line of pressure will trend towards the face, eventually falling outside altogether, as is shown by a final resultant R. The reciprocal triangle of horizontal earth pressure, reduced to the corresponding area of masonry, will be a triangle H_1OO, having its apex at H_1, 10 feet below crest, and the width of its base being marked off $= 2r_1$ on Fig. 25a.

(43) The total horizontal stress to be carried by the ties per unit length will be the area of the triangle $H_1OO \times 62{\cdot}3 \times \rho$, but as it is proposed to place

the ties 5 feet apart, the stress to be provided for will be the above expression multiplied by 5, or $\frac{3 \cdot 8}{2} \times 15 \times 2 \cdot 1 \times 0 \cdot 28 \times 5$ tons $= 8 \cdot 4$ tons. The quantity ·028 is the weight of a cubic foot of water expressed in fractions of a ton, as 62·3 is the weight in lbs.

Two tie-rods will be required; the lower one will be placed at the centre of gravity of the pressure triangle, *i.e.*, 5 feet above the base of the walls. This tie will neutralise the whole of the horizontal stress, viz., 8·4 tons, but another will be required at a higher level to counteract the overturning movement above the 5 feet level, as the wall might break off here, and fail by revolving on its outer toe. The area of that portion of the triangle lying above the lower tie level is one-half of the whole; of this one-third will be transmitted to the apex, two-thirds to the base. Hence the upper tie will carry one-sixth of the whole, *i.e.*, 1·4 tons, and the lower the remaining five-sixths, or 7 tons.

Taking 5 tons as the safe tensile strength of iron, the upper tie rod should have a sectional area of ·3 square inches and the lower 1·4 square inches. One ⅝ inch diameter rod would answer for the upper and 1⅜ inch for the lower, placed at 5 feet intervals.

(44) The reduced area of land wings is 82½ square feet, as against 137½ of the normal section (*vide* Table I.), and $\frac{W}{H}$ is one-thirteenth only. This shows the great economy effected by the introduction of cross ties, the cost of which is trifling. The ties should be provided with large plate washers, and have a head at one end and screw nut at the other. The washers need not appear outside the wall, being placed in a recess near the face, which can be closed up when the final tightening has taken place.

The reduction in section rendered possible by this arrangement leads to the more extended use of this description of wall, particularly in the case of masonry pitched slopes being substituted for water wings, a style of construction used for river works, but which could be well adopted for canal falls or other cross-canal works.

(45) In canal works in which a drop exists in the bed, it is a common practice to take the foundations of the up-stream wings down to the same level as those situated below the fall. The object of this arrangement is to obviate any possibility of saturation of the bank and percolation of water through the flank of the work. In Figs. 21 and 23 this is shown in the splayed up-stream returns (*c*) common to both plans. As canal falls are located so as to be in moderate cutting, the lower portion of the splayed walls is subjected to no earth pressure below the level of the bed of the upper reach, and reinforced concrete sheet piling could be adopted below this level instead of deepening the wall foundation.

These walls, then, belong to class C, *i.e.*, walls subjected to earth pressure for only a portion of their depth. As misapprehension undoubtedly exists regarding the action of forces in such cases, as is clear from many existing examples, it will be well to devote more than usual attention to the design of walls under similar conditions.

(**46**) Fig. 26 illustrates the typical case of a vertical wall 30 feet high, the lower 10 of which is sunk in a trench in the solid ground, and can

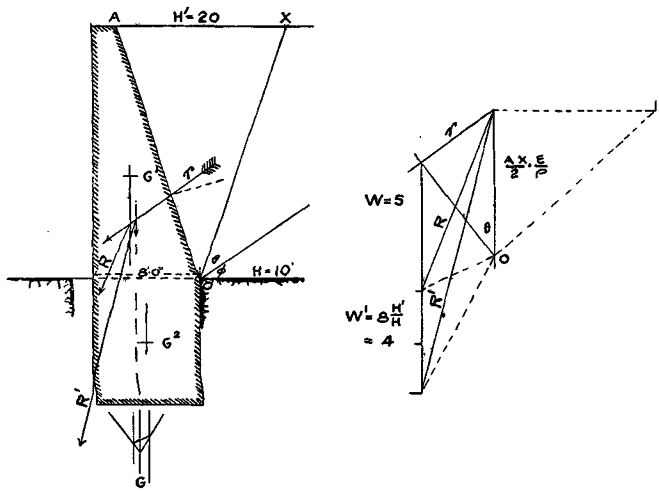


FIG. 26.

consequently be safely considered as free from earth pressure below this level. The base width at O is made $\cdot 4H$, in accordance with the tabular

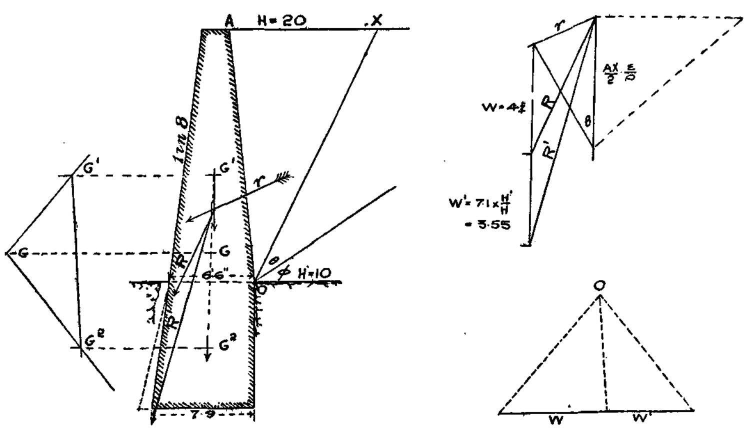


FIG. 27.

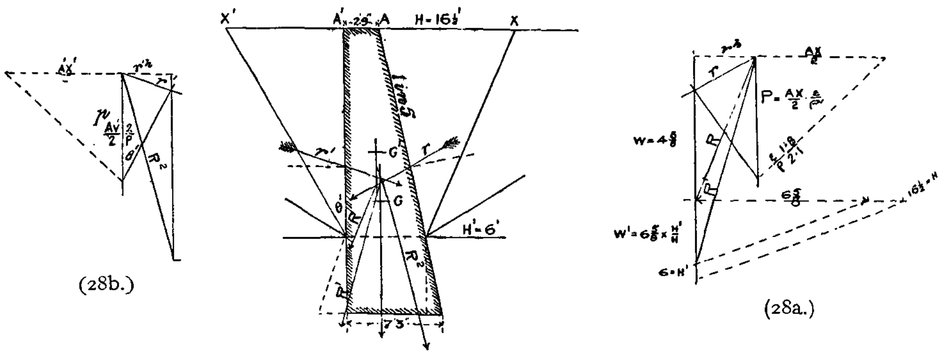
formula, and the foundations are carried down vertically. The resultant of the upper portion R cuts its base at the proper point, but the final resultant R_1 falls without the base of the whole wall.

(**47**) Fig. 27 represents exactly similar conditions, but in this instance, the wall face being battered at 1 in 8, the base width is $\cdot 4H—1\frac{1}{2}$, and below the level H_1 the back is carried down vertically, while the face batter continues. In this case R_1, the final resultant, cuts the base of the whole wall within the outer toe, proving the stability of the section, though it would be improved by still further widening the base, giving an increased batter below H_1, as is shown by the dotted line.

The area of the wall in Fig. 26 is 180 square feet, that in Fig. 27 is 156 square feet; the former is in unstable and the lesser section in stable equilibrium. If the base of Fig. 26 were widened to bring R_1 within the toe at the proper place, the discrepancy of areas would be still further intensified in favour of the lighter section. This proves the absolute necessity of having walls of class C designed with a face batter.

(**48**) As illustration of the too common neglect of this point, a few examples of actual sections will now be given and analysed.

Fig. 28 represents the section of a retaining wall on an existing canal fall with vertical face and sloping stepped back, which latter has been



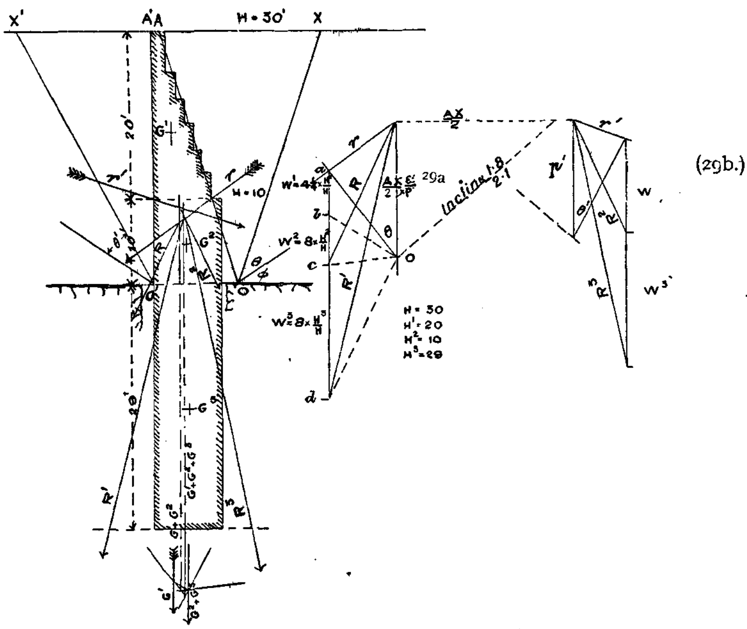
FIGS. 28, 28a, 28b.

converted, for simplicity sake, into an equivalent batter. The wall is of brickwork, and has been credited in the graphical calculation with the usual specific gravity of 1·8, that of the earth being taken at the same value. It will be noticed that the final resultant R_1 falls just without the outer toe of the base. The wall should therefore have a further widening of the base in front and corresponding reduction behind. The front addition would have to be about double that lopped off the back, as the latter operation would tend to throw the centre of gravity forward, and this tendency would be further accentuated by the addition made in front.

(**49**) Now, supposing the wall reversed in section, *i.e.*, keeping to the same dimensions. Let the earth pressure be on the vertical side. The effect is duly worked out in Fig. 28b, and R_2 is the final resultant. This, it will be seen, cuts the base 2 feet within the outer toe. The great effect of the reversal of the section is plainly manifest. Not only is the shape of the section thus reversed more suitable to sustain the thrust of the earth, but the area of the triangle of earth pressing on the wall is also much reduced. The greater the back slope, the larger is also AX and r. This is somewhat discounted by the greater inclination of r to the horizontal when the back is inclined; this inclination is the graphical allowance for the weight of earth supported on the back slope of the wall.

With a vertical back, p is less than with an inclined back, but the great difference in the two values of r and r^h, the horizontal component, is due to the angle θ, which is at a minimum with a vertical back.

(50) Thus we see that a vertically backed wall is the most economical, the batter being all thrown out on the front: it is only surpassed when the back, as in the case of Fig. 15, has an inward deflection. Exactly the same,



FIGS. 29, 29a, 29b.

as will be seen later, applies to dams subject to water pressure, the vertical backed wall being the most economical design.

In Fig. 28, with pressure acting on the right side of the wall, the section is too weak; whereas if the earth pressure were to act on the opposite side, the wall would show excess of strength and could be sensibly reduced in section. The base at H_1 need not exceed 4 feet, *i.e.*, $4H—2\frac{1}{2}$ (*vide* tabular statement). The area of the section would then be, with 2 feet top width, about $76\frac{1}{2}$ square feet; whereas in Fig. 28 it is $112\frac{1}{2}$ square feet, and, further, the stability would be greater. The section in question could not have been designed on any principle except that of making the base one-third of the height, which it scales almost exactly. It is just strong enough for a height of H, but not for $H+H_1$, although the earth pressure stops short of the lower base.

(51) Another actual example of defective design is exhibited in Fig. 29. This section has a slope of earth in front of the lower 10 feet; this is the tail end of the fore slope of a canal spoil bank which runs down the wall in question. At the commencement of the wall this slope disappears. Thus for half the length of the wall this slope has no appreciable influence in favour of its stability, and so has been omitted from consideration. The designer, however, has evidently treated it as a solid bank with horizontal terrain and has started the verticality of the back of wall from this point instead of from 10 feet lower down, where the surface of solid ground is reached; consequently that portion of the wall under direct earth pressure has been designed for a value of H of 20 feet, instead of 30 feet, and the 8 feet base is thus too narrow, R_1, the resultant line of earth pressure, as shown in Fig. 29, falling without the base. The value of $\frac{W}{H}$ is barely one-sixth, a ratio far too low for a vertically faced wall. At the base of the whole wall, which is 58 feet high, the final resultant R_1 falls over 5 feet outside the outer toe. The error here consists as usual in altogether ignoring the action of the earth pressure below the surface line.

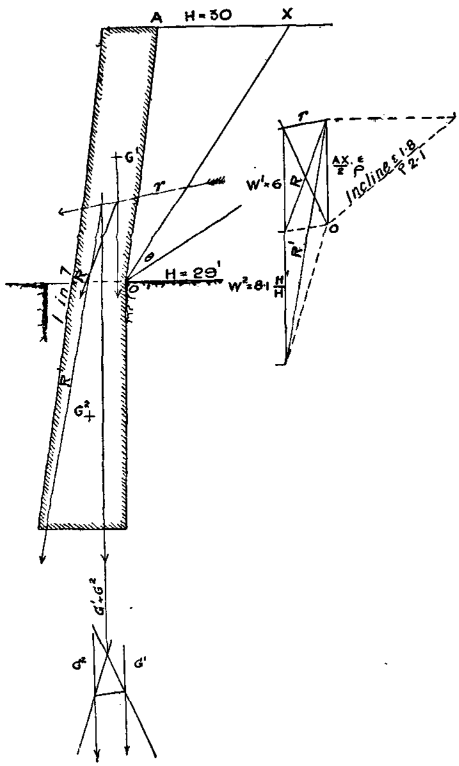


FIG. 30.

If the wall were reversed the resultant R_3 reciprocal to R_3 in Fig. 29b, would also fall outside the base, but not to so great an extent as before.

(52) An attempt will now be made to show how the wall might have been designed, of much the same sectional area, but irreproachable on the point of stability, and under exactly the same conditions as before.

The wall profile in Fig. 30 is the result of two or three trials. A continuous face batter of 1 in 7 is given to the front. The back above the point O is made parallel to the face, and below that point it is vertical. The object of making the upper part a rectangle and not a trapezium is in order to keep the base thickness of very moderate dimensions, as otherwise, the back below the point O being vertical, the lower section would become unnecessarily heavy. The inward back batter also reduces the earth pressure. The double back slope could be avoided if the portion below the point O, *i.e.*, in the trench, were also built with an inward batter. A vertical back line for this part is, however, considered more suitable. The two resultants in this design fall well within the base in both cases. The area of the wall in Fig. 20 is 397 square feet, that of Fig. 30 is 412 square feet, a slight excess.

(53) As already observed, it is a common practice peculiar to irrigation works to take the foundations of the up-stream wings in case of a drop in canal bed, right down to the lower level; consequently walls above the drop wall or weir are not subjected to earth pressure below the level of the canal upper bed.

Fig. 31 represents a case in point and is a section of the splay up-stream wall (*c*) shown in Fig. 21. That figure represents a canal notch fall. The drop in the bed of canal is $6\frac{1}{2}$ feet, the depth of water passing over weir, owing to the notches, is upheld to the normal depth, *i.e.*, 4 feet; another 4 feet is given as free board; the total height of wall above concrete then comes to 16 feet 9 inches, or say 17 feet. Of this the upper 8 feet is subjected to continuous earth pressure. Fig. 31 is a simple section designed to meet these conditions. Its strength is greater than necessity requires and could be economised if the top width were diminished, or an inward back slope given, neither of which are desirable.

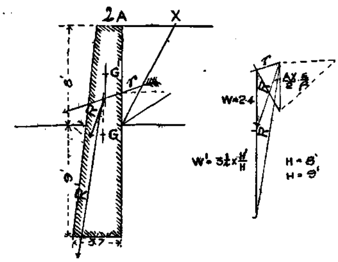
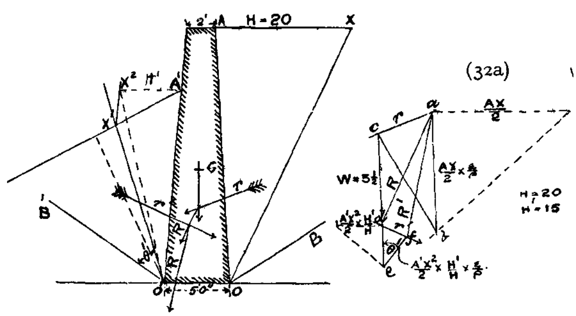


FIG. 31.

(54) One further problem is required to be solved before closing the subject of the sections of retaining walls, and that is the case where earth pressure exists on both sides of a wall acting in opposition, viz., class G. The method of combining these forces with the weight of the wall is illustrated in Fig. 32.

The profile represents a 20-foot wall with earth pressure up to crest at the back, while part of the face is covered by a falling slope. This case is identical with that prevailing in connection with land wings, at the ends of which the earth slope on the face reaches up to the outer crest and pivoting round on this point forms a half cone, pressing on the face of either land wing, thus forming in elevation a slope against the face.



FIGS. 32, 32a.

In Fig. 32a, *a b c d* forms the usual combination of force triangles closing in *R*. We have now to combine *R* with r_1, the inclined force acting in the opposite direction. This combination can be effected on the same figure by continuing the load line *c d* so that *d e* equals the relative area of earth weight which acts on the face of the wall. This will be $\frac{A_1X_2}{2} \times \frac{H_1}{H} \times \frac{\epsilon}{\rho}$. Then from the extremity e, the θ angle is set off, intersecting the inclined line r_1 drawn through d, at the point f. This latter, joined with the starting point a, gives the direction and comparative value of R_1, the final resultant, *i.e.*, the resultant of r_1 and R. In Fig. 32, R is drawn as usual from the intersection of r and G, till it meets the line r_1. This last point is the starting point for the final resultant R_1, which is drawn through the base Fig. 2. When the earth pressing on the wall has a horizontal terrain, as is the case in many direct return wings, the procedure is the same except that O_1X_1 bisects the angle $A_1O_1B_1$.

Sloping Wings on Plan and Elevation.

(**55**) The construction of trapezoidal wings with sloping crests in exact accordance with the proportions tabulated is a matter of no practical difficulty

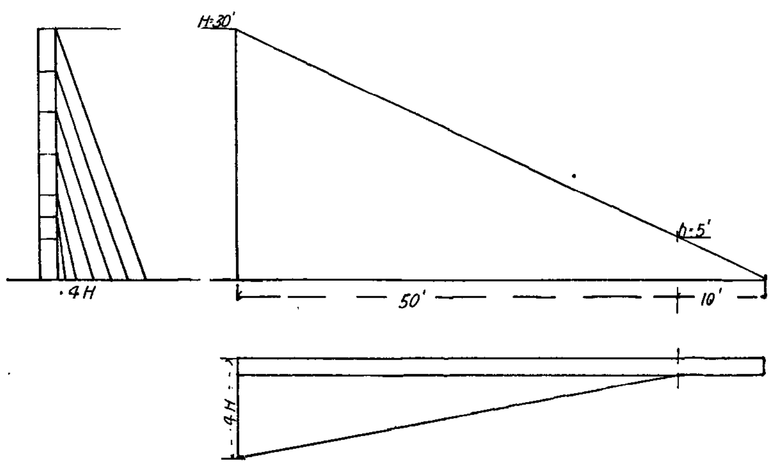


FIG. 33.

whatever. Figs. 33 to 36 contain sections and side elevations of a series of walls, the slope of crest being taken at 2 to 1. Reference to the sections of each height, at intervals of 5 feet, from 30 feet to 10 feet, which are joined in one figure (1a) clearly show that the back batter varies regularly, becoming slightly steeper with each decrease in height, till at a certain point it becomes vertical. The height of wall where the back batter ceases is given in Table I. For instance, with series A having a vertical face, the height is 5 feet. This, with a slope of crest of 2 to 1, will fall at a distance of 2 × 5, or 10 feet from the toe of slope, or if measured from the abutment the distance will always be 2 (H-h), H being the extreme height at beginning of wall. The position of h is marked on each of the Figures 33 to 36 on elevation and plan.

(**56**) To align the winding back slope of the walls, all that is necessary is to mark the foundation outline, and erect one profile of the batter at the abutment end, and another, vertical, at the point h. If the top ends of these batter profiles be joined by a taut cord representing the inner slope line of the crest, a straight edge applied to the cord above, and to the outline of base of batter will give the inclination at any point. The building of the back batter will be just as easy as that of the face. If the courses are inclined as should be the case, *i.e.*, normal to the face, each single course or two courses can be set back in accordance with the profile, thus affording a slightly stepped or roughened face, which is of no disadvantage and is concealed from view. Walls built with inclined backs are in fact easier to construct than the stepped backs, which custom has prescribed for retaining walls, with their numerous insets and offsets, all requiring measurement, whereas in the former case no measurements of varying thicknesses are needed, the shape of the wall following the profiles automatically, as previously described.

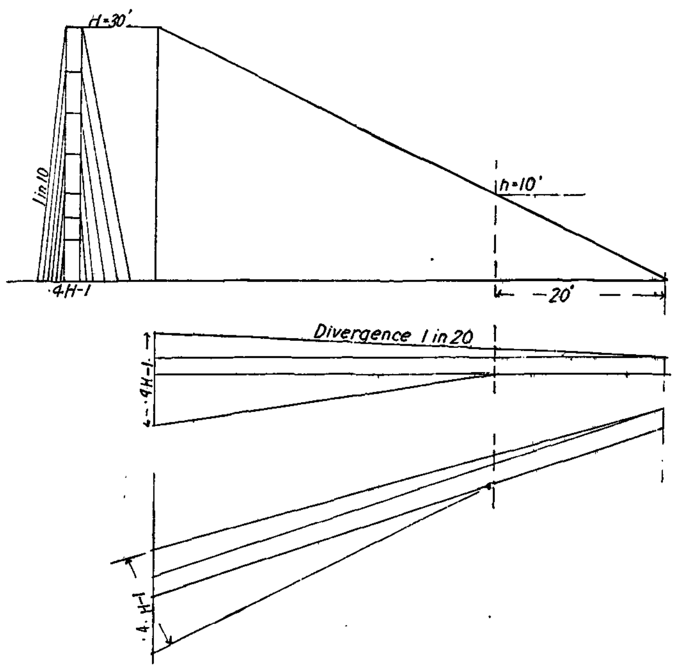


FIG. 34.

(57) The author is strongly in favour of the adoption of inclined backs for retaining walls instead of vertical stepped backs. In the former case the earth backing, on settlement, closes firmly against the inclined surface of the wall, whereas with stepped backs the settlement is necessarily uneven and hollow spaces are certain to be formed; this has been noticed in actual cases, deep holes having been found adjoining the wall, the bank in settling having evidently cut itself clear of the offsets of the back, leaving spaces down which the water finds its way, causing further settlement and disintegration.

The advocates of stepped backs for retaining walls allege that the system possesses the following advantages: firstly, of increasing the friction of the earth against the wall, thus adding to its stability; and secondly, of affording a more solid support to the superincumbent earth backing, which can thus be credited to the wall as an increment to its weight. Both these assumptions are purely chimerical. The unequal

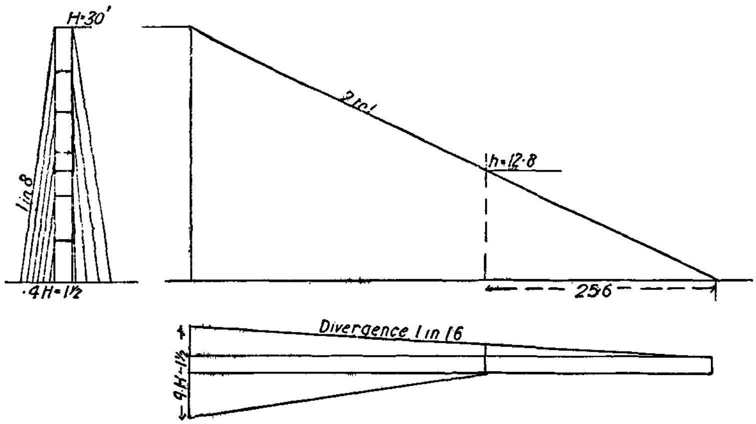


FIG. 35.

settlement which is sure to occur with stepped backs to the wall must tend to decrease instead of increase the co-efficient of friction. As, instead of the whole of both surfaces being in close contact, they will be so only at intervals, the second consideration is not based on scientific grounds.

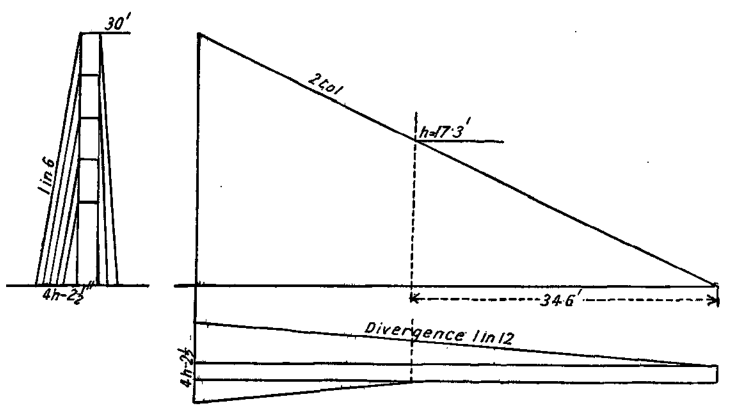


FIG. 36.

The battered back, whether in steps or in one plain surface, performs identically the same statical function, viz., its resistance to external forces must be normal to its surface. Whether this surface is composed of several horizontal and vertical portions, or is in one plane, its reflex action is the same, as the vertical and horizontal forces if resolved are identical in direction with the force of resistance normal to the plane surface. The weight of earth supported is duly credited to the wall by the normal inclination of the force r to the back, which deflects the direction of this force downwards. With a wholly vertical back the direction of r would be horizontal, if the friction of the two surfaces were neglected. In such cases no earth rests on any part of the wall. Thus it is clear that the stepped back possesses no advantages whatever over the plane batter, while it is undoubtedly more troublesome to design and to build.

The sectional area of retaining walls can be largely reduced if the panel counterfort type of reinforced concrete is adopted, and economy in cost will also result provided the labour conditions and absence of local supplies of lime and material render the use of Portland cement necessary and desirable. Where, however, such abnormal conditions do not obtain, the adoption of a slighter section composed of the more expensive material will not present any advantage in point either of cost or efficiency. An example of R. C. retaining walls is given in Chap. IX.

CHAPTER II

GRAVITY DAMS AND WEIRS

(1) DAMS of masonry or concrete can be classified into four distinct types, in the design of which different principles are involved. These are

A, Gravity dams and weirs.
B, Arched dams and weirs.
C, Arch and buttress dams and weirs.
D, Reinforced concrete panel box dams.

In addition to these, earth, rock-fill and hydraulic-fill dams will be noticed elsewhere.

(2) Where water and not earth is upheld by retaining walls, its properties as a fluid and its weight being definitely known, there can exist no uncertainty on the subject of the actual pressure exerted.

A dam may be defined to be a wall of masonry or concrete which upholds a mass of water at its rear, while its face or lower side is free from the presence of water to any appreciable extent.

The waste water of the reservoir formed by the dam is disposed of in another direction by means of a waste weir or bye-wash, or in rare cases by means of sluices through the body of the dam.

Weirs, though often confounded with dams, differ from the latter in the following points—viz., that water overflows the crest, and in consequence tail water is formed below the wall. These two facts modify the conditions which are applicable to dams proper, and consequently weirs demand separate treatment.

SECTION I.—GRAVITY DAMS

(3) The pressure of water on the back of a wall at any point varies directly as the depth, so that the water pressure can be represented by an equilateral triangle with its base equal to its height, the base being normal to the rear face of the wall.

The unit pressure exerted on any point in the back of the wall is represented by the ordinate of the triangle of pressure drawn parallel to its base, and the whole pressure by the sum of these ordinates, *i.e.*, by the area of the triangle or prism.

When the back of the wall is vertical this area will be $\frac{H^2}{2}$, H being the vertical depth of water and the pressure $\frac{wH^2}{2}$, w being the unit weight of water, one thirty-sixth of a ton per cubic foot.

When the back of the wall is inclined, the pressure will be $\frac{wH_1H}{2}$, H_1 being the inclined height of the back of the wall, which being always greater than H, the vertical depth, the latter expression is also greater.

(4) Sections of gravity dams are invariably designed on the well-known principle of the "middle third." This expression signifies that the profile must be such that the resultant pressure lines, or centres of pressure, due, first, to the weight of the dam considered alone, and, secondly, to that of the water pressure in addition, must both fall at or within the middle third of the section. These two conditions of stress are usually designated as "Reservoir Empty" and "Reservoir Full."

This stipulation ensures the fulfilment of three obligatory provisos, which are, first, the absence of tension anywhere in the section, secondly, the limiting of the maximum compressive stress in any plane to a proportion not exceeding twice the mean or average stress on the same. It further usually ensures that the angle of inclination of the resultant pressures with the horizontal exceeds that of the angle of friction of the material; this is with reference to shearing stress. A limiting value to the maximum stress allowable in the masonry of the dam is also a further necessary condition that has to be observed.

(5) The theoretically correct profile of a dam subjected to water pressure under the conditions above outlined, and also of the most economical dimensions possible, is that of a right-angled triangle having its back vertical and its apex at the surface level of the water. It can be proved that the proper base width of this triangle is expressed by the simple equation

$$b = \frac{H}{\sqrt{\rho}} \qquad (1)$$

in which H designates the vertical height of the triangle and the Greek letter ρ (rho) the specific gravity of the material in the dam. This profile will be termed the Elementary Profile, and is shown on Fig. 1.

A base width of such proportional dimension ensures the exact incidence of the vertical resultant force W of "Reservoir Empty" and also that of the inclined resultant R of "Reservoir Full" at the inner and outer third divisions of the base respectively. The same will naturally occur on any horizontal plane.

The fore slope, or hypothenuse of the Elementary Profile, will be as $1 : \sqrt{\rho}$.

The value usually assigned to ρ, the specific gravity of the material, masonry or concrete, of which the dam is composed is $2\frac{1}{4}$ or $\frac{9}{4}$.

In many dams this is exceeded and sometimes reaches as high a figure as 2·7, but the calculations for base width are generally founded on this value of $2\frac{1}{4}$.

As $\sqrt{2\frac{1}{4}}$ is $1\frac{1}{2}$, $\frac{1}{\sqrt{\rho}}$ will be $\frac{2}{3}$, and the correct proportion of the base will be $b = \frac{2}{3}H$, and the fore slope or batter will be as 2 horizontal to 3 vertical.

(6) The weight of a cubic foot of the material with $\rho = 2\frac{1}{4}$ is expressed by $w\rho$, w being the weight of a cubic foot of water, that is $\frac{1}{36}$ ton nearly, $w\rho$ will then $= \frac{1}{16}$ ton.

The system adopted throughout this work in representing pressure in tons per square foot is clearly superior to the antiquated method still adopted in many technical works of using pounds per square inch.

Large denominations are more easily grasped and retained than small ones, thus the representation of the contents of a reservoir in acre-feet appeals to the mind better than gallons which run into millions. The use

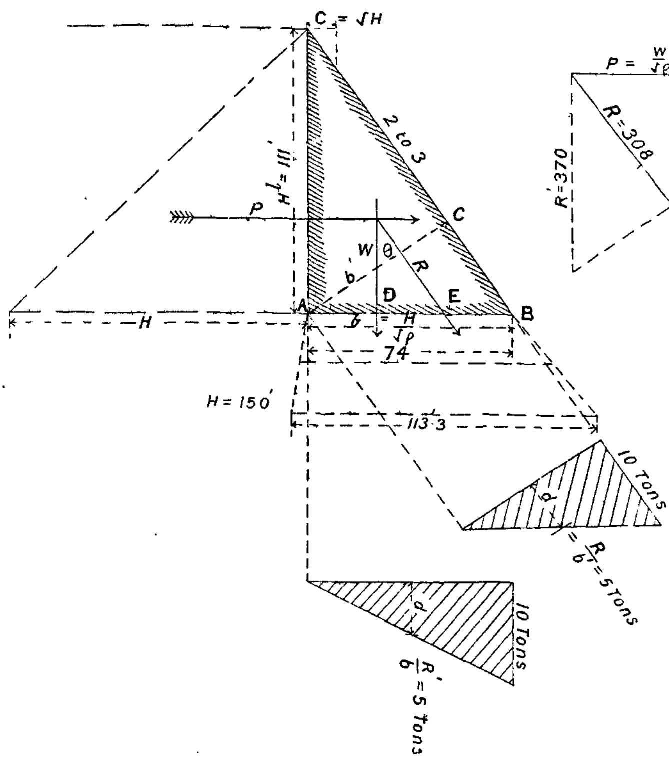


FIG. 1.

of pounds per square inch should be confined to steam or air pressures and mechanical engineering generally, for which a small denomination is clearly suitable. It may be noted here that $15\frac{1}{2}$ lbs. per square inch is equivalent to 1 ton per square foot.

(7) In the design of a section of a dam, pier, or retaining wall, the distribution of pressure on a plane in the section and the relations that exist between maximum stress (s) and the mean stress (s_1) are matters of the utmost importance.

The mean or average stress in any lamina is that which acts at its centre, and is in amount the resultant stress, the incidence of which may be at some other point, divided by the width of the lamina in question. Such width must be measured on a plane normal to the direction of the resultant force. For example, in Fig. 1 the vertical line W represents the weight of the dam acting through its centre of gravity on its base b. The intensity of the mean stress induced at the centre of b, which being horizontal is normal to W, will be $\frac{W}{b}$. The maximum stress occurs at that end of the base nearest to the incidence of W, that is, at the point a, the "heel" of the base. When, as in this case, the incidence of W is at the middle third point, this maximum unit stress will equal twice the mean, or $s = 2\, s_1 = 2\, \frac{W}{b}$.

This fact is due to the application of a well-known law, defining the relations between maximum and mean stress, which is expressed by the following formula

$$s = s_1 \left(1 \pm \frac{6c}{b}\right) \qquad (2)$$

In this c is the distance of the centre of the lamina from the centre of pressure, or in other words from the incidence of the resultant stress. When the plus sign is used, the equation gives the value of the maximum unit compressive stress at the end nearest to the centre of pressure, whether it be W or R, and when the minus sign is used that of the unit stress, which occurs at the other extremity. If the result is a minus quantity it represents a tensive, not a compressive stress set up at this extremity.

If the greater stress be termed P and the less P_1, and s_1, being as we have already seen, $= \frac{W}{b}$ when the incidence of the resultant stress W is at the centre of the base, then $c = o$ and the equation becomes

$$P = \frac{W}{b}(1 + 0) = \frac{W}{b}$$

and

$$P_1 = \frac{W}{b}(1 - 0) = \frac{W}{b}.$$

That is, the maximum is equal to the mean stress.

Again when the incidence of W is at the third division point, as is the case in Fig. 1, then $c = \frac{b}{3}$ and

$$P = \frac{W}{b}\left(1 + \frac{2b}{b}\right) = 2\frac{W}{b}$$

$$P_1 = \frac{W}{b}\left(1 - \frac{2b}{b}\right) = \text{nil}.$$

Thus the maximum is double the mean. Lastly, if the incidence of W is at the extremity of the base, $c = \frac{b}{2}$ and

$$P = \frac{W}{b}\left(1 + \frac{3b}{b}\right) = 4\,\frac{W}{b}$$

$$P_1 = \frac{W}{b}\left(1 - \frac{3b}{c}\right) = -\,2\,\frac{W}{b}.$$

In this case the maximum unit compressive stress is four times the mean while at the further end tensile stress is set up equal to twice the mean unit stress. The graphical process of obtaining the same results will be explained in par. 24 later.

(8) Now with regard to R, or the resultant pressure "Reservoir Full"; this force being always a greater quantity than W, whenever the question of the maximum permissible stress in the masonry of a dam comes under consideration, it is this force and not W that is the ruling influence.

With regard to the maximum pressure induced in the masonry by the force R, this plane b is not normal to the direction of R, but another plane (marked on Fig. 1) b_1 is so; consequently the mean stress induced by R is not $\frac{R}{b}$, but $\frac{R}{b_1}$; the mean unit stress is therefore greater than $\frac{R}{b}$ as b is greater than b_1 and equals $\frac{R}{b\ cos.\ \theta}$, θ being the inclination of R to the vertical. On the horizontal base, however, $s_1 = R \div b$.

In graphical computation it is more convenient to increase R, using b as denominator, the result being identical.

In the force polygon 1a, if a line be set out at right angles from the extremity of the force R, the intercept $N_1 (= R\ sec\ \theta)$ will represent the increased value assigned to R; the mean induced unit stress on the base b will then be

$$s_1 = \frac{N_1}{b} = \frac{R}{b_1}.$$

This is graphically demonstrated by the hatched areas below the profile in Fig. 1, in which for R_1 read N_1.

(9) Designs of parts of masonry works have often to be manipulated so as not only, as in this case, to bring the centre of pressure at the middle third point of the base, but so as to reduce the maximum unit stress to a less proportion than one of double the mean, and this can be effected by manœuvring the position of the incidence of the greater resultant R to a point as near the centre of the base as possible, with a view to equalising the value of s with that of s_1.

(10) The values of the resultant and of the maximum induced unit stresses due to "Reservoir Full," or "Reservoir Empty," are catalogued below in terms of w and of ρ. The values when expressed in tons assumes $\rho = 2\frac{1}{4}$.

$$W \text{ or the weight of the triangular prism} = \frac{H^2 w \sqrt{\rho}}{2} = \frac{H^2}{48} \text{ Tons} \quad (3)$$

$$R, \text{ the resultant (Reservoir Full)} = W\sqrt{\frac{\rho + 1}{\rho}} = 1 \cdot 2\ W = \frac{H^2}{40} \text{ Tons} \quad (4)$$

$$s^a \text{ or maximum unit stress } (R.\ E.) = Hw\rho = \frac{H}{16} \text{ Tons} \quad (5)$$

$$s^b \text{ ,, ,, ,, ,, } (R.\ F.) = Hw(\rho + 1) = \frac{H}{11 \cdot 1} \text{ Tons} \quad (6)$$

If the limiting permissible unit stress be designated by the Greek letter λ (lamda), the limiting height of the profile will be from (6)

$$H^{\lambda} = \frac{\lambda}{w\,(\rho + 1)} \text{ or when } \rho = 2\tfrac{1}{4} = 11{\cdot}1\,\lambda \ (7).$$

Thus if	λ be	8	Tons	H^{λ}	will be	89	feet
,,	,,	10	,,		,,	111	,,
,,	,,	15	,,		,,	165	,,
,,	,,	20	,,		,,	220	,,
,,	,,	25	,,		,,	275	,,

The area of the elementary triangular profile is $\frac{H^2}{2\sqrt{\rho}}$, and that of the triangle of water pressure reduced to a masonry base is $\frac{H^2}{2\rho}$.

(11) In graphical computations the base of the triangle of water pressure is invariably made equal to $\frac{H}{\rho}$ not to H. This has the effect of reducing the pressure area from that of water, of specific gravity = unity, to the same denomination as that of the masonry of the wall, or of ρ, with the result that $w\rho$ becomes a common factor in both the masonry and water pressure areas. In the triangle of forces, the value of W and of P respectively can thus be represented by the half widths of their respective areas.

This procedure simplifies construction, as the common factors H, as well as $w\rho$ are eliminated, H being also common to both triangles. In cases where H is not common, and consequently cannot be eliminated, the values of W and of P will have to be represented by their areas, the common factor $w\rho$ being then the only one that can be discarded. The same procedure was observed in Chap. I. Whenever actual values in tons are required, the measured length of the resultants in the force polygon have to be multiplied by those eliminated factors, *i.e.*, by H and by $w\rho$.

(12) In actual practice a dam must be provided with a crest of definite width and not terminate in the apex of a triangle. The imposition of the crest increases the stability of the section, but throws the incidence of W (the resultant "Reservoir Empty") a small space outside the middle third. This can be adjusted by slightly widening the base towards the heel and taking a corresponding slice off the fore slope. As, however, this adjustment only equals about one-sixteenth of the imposed crest width, it can well be neglected altogether, as is the actual case in several examples of dams which will be exhibited.

An empirical rule for the crest width of a dam is

$$c = \sqrt{H} \ (8).$$

This gives results in accordance with usual practice, although examples are not uncommon in which the crest has to be made wide enough to carry a roadway for cart traffic across the dam. In such cases the upper part can be lightened by causing part of the width to overhang the face line, supported by arches which spring from pillasters. This procedure is exemplified in Fig. 12 of the Chartrain Dam, and produces a light, but stiff and elegant crest.

The face line of the crest should be vertical, and be joined to the battered face by an easy curve.

This is shown in Fig. 4 (par. 22), which is a profile of a weir 100 feet high. The outline thus formed is termed the "Pentagonal Profile."

(13) Another method of determining the crest width of a dam is to make it proportional not to H, as in formula (8), but to the base width b. It will then become a function of ρ as well as of H. If this ratio or $\frac{c}{b}$ be designated r, the exact base width required is expressed by the following formula:—

$$b = \frac{H}{\sqrt{\rho}} \times \frac{1}{\sqrt{1 + 2r^2 - 2r^3}} \quad (9)$$

A suitable value for r will be about ·2. Thus supposing $H = 60$ feet, and $\rho = 2\frac{1}{4}$

$$b = \frac{40}{\sqrt{1 + \cdot08 - \cdot016}} = \frac{40}{1\cdot03} = 38\cdot83 \text{ feet}$$

c, the crest width, will then $= 38\cdot83 \times \cdot2 = 7\cdot77$ feet. This closely corresponds to $\sqrt{H}$, which $= 7\cdot75$ feet. A ratio of $\cdot15r$ is also suitable.

(14) The crest has to be raised above actual full reservoir level by an extent equal to the calculated depth of water passing over the waste weir, or through the bye-wash, as the case may be. This extra "weather board" depth, which adds considerably to the cost of a work where the dam is of great length or is connected with extensive earthen banks, can be avoided by the adoption of automatic waste gates, as was done in Lakes Whiting and Fife in the Bombay Presidency, in which cases full reservoir level and high flood level are merged into one.

In addition to the above, allowance has to be made for wave action, the height of which is obtained by the following formula

$$h = 1\cdot5\,F + (2\cdot5 - \sqrt[4]{F}) \quad (10)$$

In this, F is the pitch or longest line of exposure of water surface to wind, expressed in statute miles.

Thus if $F = 4$ miles, the extra height above maximum flood level will be

$$1\cdot5 \times 2 + (2\cdot5 - 1\cdot4) = 3 + 1\cdot1 = 4\cdot1 \text{ feet.}$$

If $F = 10$ miles, the height works out to $5\frac{1}{2}$ feet. If the height of the crest is not made higher than the allowance necessary for wave action, the apex of the triangle of water pressure should correspond with this level. If on the other hand the crest is designed well clear of this point the apex will be at a lower level.

As the depth of a dam often varies very considerably, the crest width, to be consistent, should also vary in width as the latter is a function of the height. Such, however, is not the usual practice, consequently waste of material in the shallower parts, where it is least wanted, must result.

With an uneven bed, it would be well to adopt a uniform crest width, based, however, on the average, not the maximum depth, and thickening it to the proper extent just where the deepest portion occurs.

(15) To revert to the pentagonal profile, if the back of a dam is not vertical, but canted forward, the stability will be injuriously affected, the incidence of R falling without the middle third. Thus we see that the vertically backed wall is the most economical profile for water, as it has already been shown to be for earth pressure. In weirs alone it is not so.

(16) We have seen that the elementary profile, or its modification the pentagonal, satisfies all imposed conditions of stability up to a certain point, which is the pressure limit; below this the simple profile must be departed from and the base widened out in order that the stipulated pressure limit be not exceeded while at the same time the other conditions are maintained. This widening adds immensely to the cost of a dam; consequently as high a pressure limit as is consistent with safety should be employed.

Of late years the adopted values of λ, the limiting stress, have largely exceeded what was previously deemed the safe limit. For this bold innovation we are mainly indebted to American engineers, who have proved without doubt that pressures of 16 to 20 tons, *i.e.*, double the old values, are quite practicable in gravity dams, whereas in arched dams even higher pressures than these are safe.

In the Roosevelt Dam (Fig. 20), one of the highest in the world, the elementary profile is strictly adhered to right down to the base, a depth of 230 feet. From formula (6), we obtain

$$s = \frac{H}{11\cdot1}, \textit{i.e.}, \frac{230}{11\cdot1}, \text{ or over 20 tons.}$$

In contrast to this, the Assuan Dam has an imposed limit unit pressure in its piers of 5·4 tons only.

Design of Dams below the Limiting Depth H^{λ}.

(17) We have seen in par. 10 that the section of a dam can be carried down in accordance with the elementary triangular section or with such modifications of it as are advisable, until the depth has a value of $\frac{\lambda}{w(\rho + 1)}$, after which the procedure becomes more complicated, the maximum stress on the masonry, which must not be exceeded, adding another factor to the problem. Not only has one line of pressure, reservoir full, viz. N, to fall at the middle third, but the base width will have to be increased in a greater ratio to satisfy the conditions of stress limit. This could be effected by means of trial and error by graphical process, but would be very troublesome, and figured calculation from formulas deduced by analytical methods will be found easier to use. This difficult subject has been very ably treated in the "Principles of Water Works Engineering," and an excellent example of the method of working out the varying width is given in the above work, which will be reproduced below in a somewhat condensed form.

The solution of this problem is based on the following :—If a rigid body rest on a horizontal surface, the distance of the centre of pressure from the extremity of the base at which the maximum stress occurs is—

$$l = \frac{b}{3}\left(2 - \frac{s}{2s_1}\right) \quad \text{(A)}$$

where b is the width of the base, s the maximum stress intensity in tons, the average pressure on the base being s_1, from which the following formula giving values of l is deduced :—

$$b = \frac{wH^3}{\lambda}\left(1 + \frac{w^2 H^4}{4N^2}\right) \quad \text{(B)}$$

λ the limiting and s the maximum stress being in this case identical. Here $w =$ weight of water per cubic foot in tons, viz. $\frac{1}{36}$ ton, and $H =$ depth of water. $N =$ the vertical forces, viz., the weight of the masonry wall and of the water over the inner face where it is inclined, *i.e.*, Reservoir Full.*

From this equation b is found readily with a high degree of accuracy if N be known even approximately.

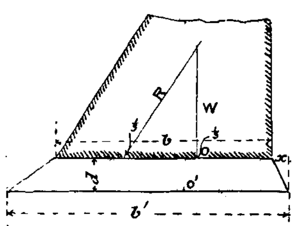


FIG. 2.

Having found b, or b_1, the next step is to ascertain how much of it must be under the inner face of the dam, *i.e.*, within the vertical through the crest, in order to bring the incidence of the resultant weight of the whole superincumbent mass of the masonry and water to a distance of $\frac{b}{3}$ from the inner toe. This is arrived at by equating the moments of the vertical forces about O_1, the point in question (*vide* Fig. 2), to zero, by which means we obtain the following equation, the solution of which gives the value of the distance x_1, *i.e.*, the projection of the base at heel of wall :—

$$\left.\begin{aligned} &\frac{\rho wd}{24}\left(3b^2 - b_1^2 + 6x_1(b + b_1) + 2bb_1\right) - \frac{wx_1}{12}(H + H_1) \times \\ &\qquad (2b_1 - 3x_1) - N\left(\frac{b_1 - b}{3} - x_1\right) = O \end{aligned}\right\} \quad \text{(C)}$$

H is depth of water at the base b, and $H_1 = H + d$, d being depth of lower lamina.

This formidable-looking quadratic equation is not so difficult to work out as one would judge from its inordinate length.

From formula (B) the value of b_1 is obtained, *i.e.*, the bottom width of the first strip to be added below the base, and from (C) x_1 is found, which fixes the position of the new lamina with regard to the base above.

(18) The following illustration of the practical working out of the formulas (B) and (C), is given below.

Fig. 3 represents the whole section of the dam, part of which is shown in Fig. 2, the value of ρ being $2\frac{1}{4}$ and λ 10 tons, whence H, the limiting depth, will be equal to 111 feet (par. 10).

* See note at end of Section II.

The base width at this depth will be equal to $x + 74{\cdot}0 = 74{\cdot}6$, or excluding the back strip 74·0. The face line corresponds with that in the elementary triangular profile, no deductions having been made.

Now the weight of the section, including that of the water overlaying the

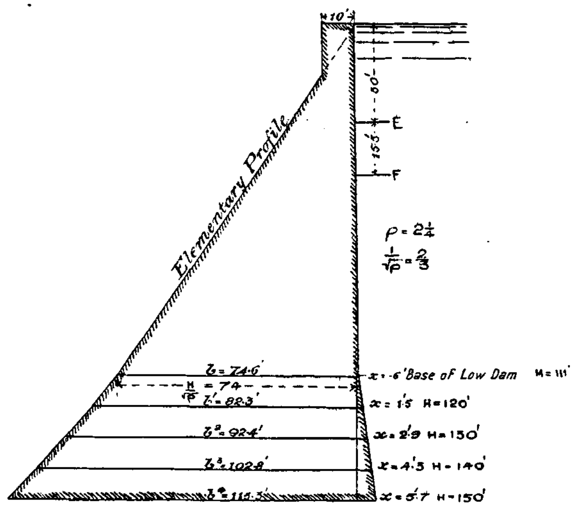


FIG. 3.

portion EF of the back, is nearly 265 tons. The successive values of H to be considered will be:—

From crest to base of low dam			H	111 feet.
,,	,,	1st lamina	H_1	120 ,,
,,	,,	2nd ,,	H_2	130 ,,
,,	,,	3rd ,,	H_3	140 ,,
,,	,,	4th ,,	H_4	150 ,,

The respective values of b and x (Fig. 13) may be conveniently designated by b, b_1, b_2, and x, x_1 and x_2; also the total weight of the dam and superincumbent water may be by N, N_1, and N_2.

First Lamina.—At the base of the low dam (Figs. 2 and 3)

$$N = 265 \text{ tons}, \quad H = 111 \text{ feet}, \quad b = 74{\cdot}6 \text{ feet}, \quad x = O.$$

If the profiles above b be produced down to the level $H_1 = 120$ feet, *i.e.*, 9 feet deeper, the weight of the trapezoid of masonry thus added is

$$\frac{9 \times 74{\cdot}6\left(1 + \frac{120}{111}\right)}{2 \times 16}, \text{ say, equal to 44 tons.}$$

Then, as a first approximation,

$$N_1 = 265 + 44 = 309 \text{ tons.}$$

By equation (B), par. 17, introducing the proper value of λ, or s of 10 tons

$$b_1 = \sqrt{\frac{(120)^3}{360}\left(1 + \frac{(120)^4}{5184 \times (309)^2}\right)}$$
$$= 82{\cdot}5 \text{ feet.}$$

The next step is to find x_1 corresponding with this value of b_1.

By equation (C), we have

$$\frac{9}{16 \times 24}[3 \times 5565 - 6806 + 6\,(157\cdot1)\,x_1 + 2 \times 6154]$$
$$- \frac{x_1}{36 \times 12}[231\,(165 - 3x_1)] - 265\,(2\cdot63 - x_1) = 0\,;$$

that is
$$1\cdot6x_1^2 + 199x_1 - 177 = 0\,;$$

therefore
$$x_1 = \frac{-199 + \sqrt{39{,}601 + 1133}}{3\cdot2},$$
$$= \frac{3}{3\cdot2} = 0\cdot9 \text{ foot.}$$

The values of b_1 and x_1 thus found, enable us to obtain a closer approximation to the weight of the trapezoid under consideration, and to determine the weight of water overlying its inner face. The sum of these two quantities may be found to be 47 tons.

Then a second approximation gives

$$N_1 = 265 + 47 = 312 \text{ tons.}$$

Introducing this value of N into equation (B), par. 17, we find, as a second approximation,

$$b_1 = 82\cdot3 \text{ feet;}$$

which value of b_1 is so nearly that previously used in applying equation (3) as not to affect the already found value $x_1 = 0\cdot9$ foot.

Thus we have at the base of the first lamina below the low dam—

$$N_1 = 312 \text{ tons, } H_1 = 120 \text{ feet, } b_1 = 82\cdot3 \text{ feet, } x_1 = 0\cdot9 \text{ foot.}$$

(19) *Second Lamina.*—Proceeding as before, produce the profiles immediately above b_1 down to the level $H_2 = 130$ feet; the weight of the trapezoid of masonry thus added, together with that of the water overlying its inner face, is

$$\frac{10}{16}\left[82\cdot3 + \frac{1}{2}\cdot\frac{10}{9}(82\cdot3 - 74\cdot6)\right] + \frac{120 + 130}{2 \times 36} \times 1\cdot0 = 58 \text{ tons.}$$

Then, as a first approximation,

$$N_2 = 312 + 58 = 370 \text{ tons.}$$

Hence
$$b_2 = \sqrt{\frac{(130)^3}{360}\left(1 + \frac{(130)^4}{5184\,(370)^2}\right)}$$
$$= 92\cdot5 \text{ feet.}$$

Applying equation (C), par. 17, and introducing these values of b_2 and N_2, we find

$$x_2 = 1\cdot4 \text{ feet.}$$

Correcting the weight of the trapezoid and its superincumbent water for this value of x_2, we have as a second approximation

$$N_2 = 312 + 59 = 371 \text{ tons,}$$

whence a re-application of equation (B), par. 17, gives the corrected value

$$b_2 = 92\cdot4 \text{ feet.}$$

Thus, at the base of the second lamina,

$N_2 = 371$ tons, $H_2 = 130$ feet, $b_2 = 92{\cdot}4$ feet, $x_2 = 1{\cdot}4$ feet.

(20) *Third Lamina.*—Produce the profiles above b_2 down to the level $H_3 = 140$ feet.

As a first approximation

$$N_3 = 371 + 67 = 438 \text{ tons.}$$

$$b_3 = \sqrt{\frac{(140)^3}{360}\left(1 - \frac{(140)^4}{5184\,(438)^2}\right)}$$
$$= 102{\cdot}8 \text{ feet.}$$

By equation (C), par. 17,

$$x_3 = 1{\cdot}4 \text{ feet.}$$

This introduces no sensible correction in the value of N_3; and we therefore have, without further calculation, at the base of the third lamina,

$N_3 = 438$ tons, $H_3 = 140$ feet, $b_3 = 102{\cdot}8$ feet, $x_3 = 1{\cdot}4$ feet.

Fourth Lamina. — Produce the profiles above b_3 down to the level $H_4 = 150$ feet.

As a first approximation,

$$N_4 = 438 + 75 = 513 \text{ tons.}$$

$$b_4 = \sqrt{\frac{(150)^3}{360}\left(1 + \frac{(150)^4}{5184\,(513)^2}\right)}$$
$$= 113{\cdot}3 \text{ feet.}$$

By equation (C), par. 17,

$$x_4 = 1{\cdot}4 \text{ feet.}$$

Again no sensible correction is introduced into N_4, and we have at the base of the fourth lamina

$N_4 = 513$ tons, $H_4 = 150$ feet, $b_4 = 113{\cdot}3$ feet, $x_4 = 1{\cdot}4$ feet.

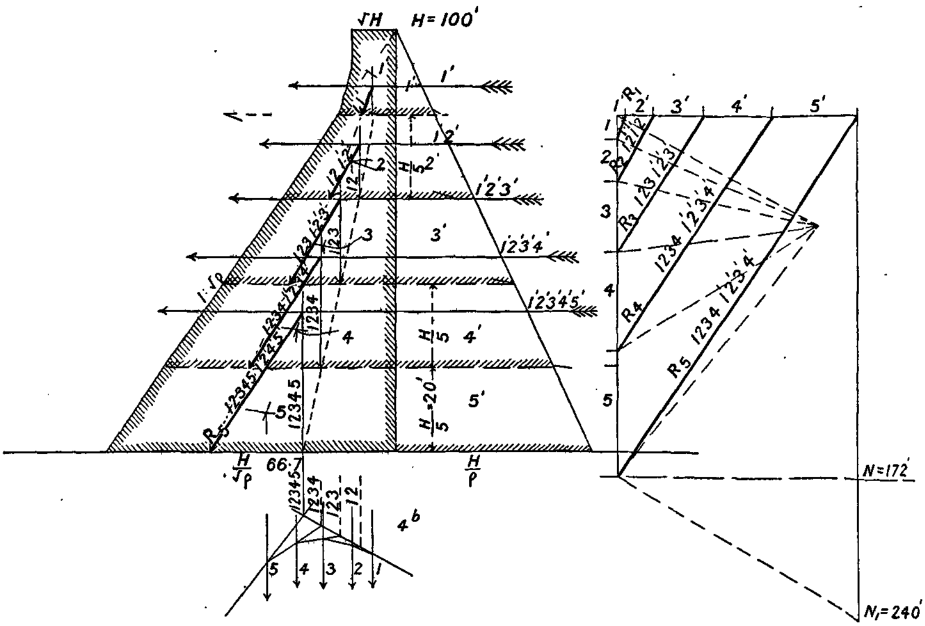
Summarising these particulars, we have :—

Depth below crest of dam (H).	Breadth of base (b).	Weight of masonry.	Breadth measuring from axis under inner face (x).	Weight of water over inner face.
Feet.	Feet.	Tons.	Feet.	Tons.
0	10	—	—	—
111	74·6	264	0·6	$\frac{2}{3}$
120	82·3	308	1·5	4
130	92·4	362	2·9	9
140	102·8	424	4·3	14
150	113·3	493	5·7	20

(21) The foundations of dams are often carried for a great depth below the surface of the ground, and if a square concrete base with vertical sides is provided, closely adhering to the original cutting face or soil or rock, this portion may be considered as free from water pressure.

As a general rule, however, the dam is designed for full water pressure down to its actual base. Below this base a comparatively narrow trench filled with concrete and puddle has often to be excavated to great depths. This is simply a curtain wall to stop any chance of percolation, and is not designed to withstand water pressure.

(22) An example of the pentagonal profile of a dam 100 feet high, designed in accordance with the rules already given, is exhibited in Fig. 4. The base is made $\frac{H}{\sqrt{\rho}}$ or $\frac{2}{3} \times 100 = 66{\cdot}7$ feet. The crest is $\sqrt{H}$, or $\cdot 15b = 10$ feet. The

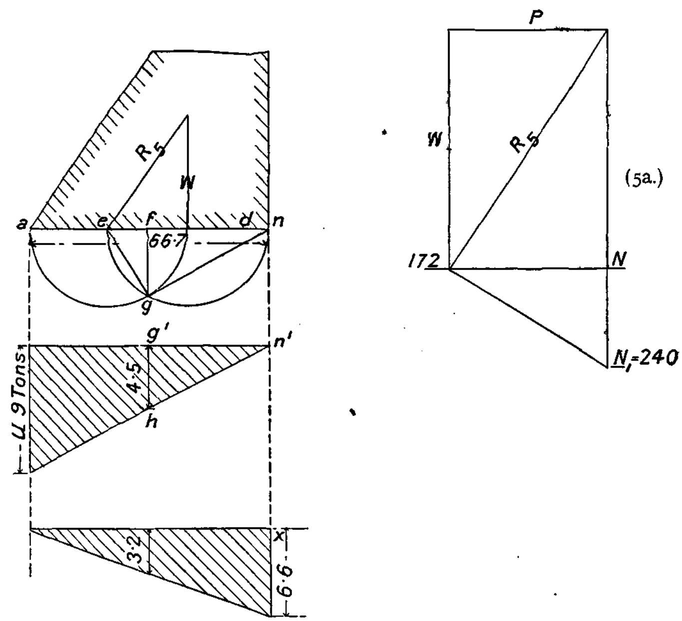


FIGS. 4 and 4a.

base of the water pressure triangle is made $\frac{H}{\rho}$ or $\frac{4}{9} \times 100 = 44{\cdot}4$ feet for reasons previously explained.

The procedure of drawing the line of pressure in Fig. 4 is that usually adopted for showing the points of pressure reservoir empty and full. The system consists of dividing the profile and triangle of water pressure into a certain number of equal laminas, in this case 5, numbered accordingly 1 to 5, while the corresponding divisions of the triangle of pressure are numbered 1_1 to 5_1. This latter having a horizontal base, all half-widths will be measured parallel to it. A series of independent combinations are now formed in the force triangles in Fig. 4a, viz., of 1_1 and 1; 1_1, 2_1, with 1, 2; 1_1, 2_1, 3_1, with 1, 2, 3 and 1_1, 2_1, 3_1, 4_1, with 1, 2, 3, 4. The centres of pressure of these three last combinations are discovered by use of the funicular 4b, and the intersection of the projected vertical resultants with the horizontal and inclined forces of each combination. The intersection of those same verticals, with their respective base lines, give points on the line of pressure, reservoir empty. Thus each pair of forces is independently dealt with ; this arrangement is so far advantageous in that error is not perpetuated. The system, however, can only be used when the back is in one straight line.

(**23**) In the force polygon Fig. 4a, R_5 is the final resultant. As explained in par. 8, by setting out a line at right angles from its lower extremity, the intercept cut off from a vertical line drawn from the upper extremity gives the value N_1, which is the actual maximum resultant pressure on the base.



FIGS. 5 and 5a.

Another intercept N is formed by a horizontal line drawn from the extremity of the load line W. This N is the vertical component, Reservoir Full.

In this case, the water pressure forces being all horizontal, $N = W$, which otherwise would not be the case. In order to obtain the actual values of N and of N_1, the measured lengths in feet will have to be multiplied up by the eliminated common factors, *i.e.*, by $\frac{H}{5}$ or by 20 and by $w\rho$, or by $\frac{1}{16}$ ton. Now N, or W measures 172 feet and its actual value will be $172 \times 20 \times \frac{1}{16} = 215$ tons. This is the vertical weight per foot run of the dam. N_1 measures 240 feet and similarly its actual value will be $240 \times 20 \times \frac{1}{16} = 300$ tons.

(**24**) The graphical method of ascertaining the distribution of pressure on the base of the masonry wall, which has already been dealt with analytically in par. 7, is exhibited in Figs. 5 and 5a, which are a reproduction of part of Figs. 4 and 4a. The procedure is as follows :—Two semi-circles are struck on the base with their centres at the two-third division points, and with radius $\frac{b}{3}$. From e, the point of incidence of R_5, the line eg is drawn to g, the intersection of the semi-circle. Again from the point g, a line gn is set off at right angles to eg, cutting the base or its continuation at the point n. This point n is the antipole of e, or the neutral point at which pressure is nil in either sense.

Below the profile another projection of the base is made. From g a perpendicular is let fall cutting the new base line in g, and its continuation g_1h is made equal to the mean unit pressure or to $\frac{N_1}{b}$. Another perpen-

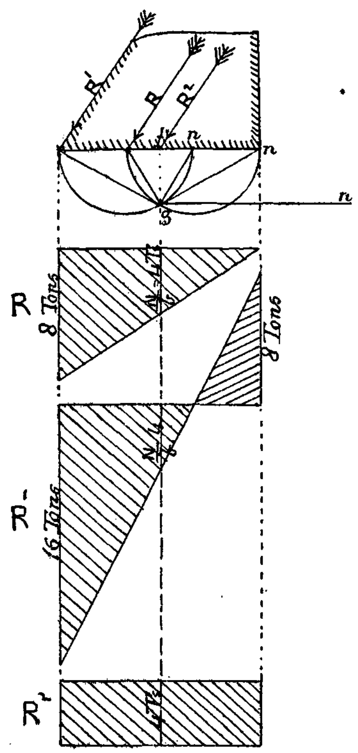


FIG. 6.

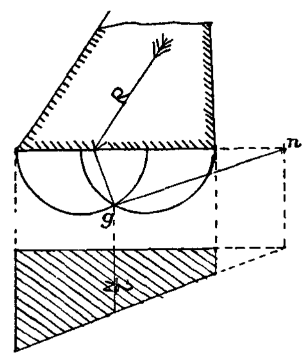


FIG. 7.

dicular is then let fall from n, cutting the new base in h_1, and then the points n_1 and h are joined and the line continued till it meets the perpendicular marked u from the toe of the base. Where n does not coincide with d, the heel of the base, a further perpendicular must be drawn from the heel. The hatched trapezoid thus enclosed represents the distribution of pressure. In similar manner the pressure due to W or N, the vertical load, is shown.

(25) We have already seen (par. 23) that the value of N_1 in Fig. 4a is 300 tons. The mean pressure will then be $\frac{N_1}{b}$ (par. 7) or $\frac{300}{66 \cdot 7} = 4 \cdot 5$ tons. This is marked off from g to h and u measures double the mean, or 9 tons.

In this diagram n happens to correspond with d because of the incidence of R_5 at the outer third division point.

In the hatched area the intensity of pressure at any point in the base is measured by the vertical ordinate of the triangle.

A similar pressure area for the load N is given; here $N = 215$ tons, and $\frac{N}{b} = \frac{215}{66 \cdot 7} = 3 \cdot 2$ tons, the maximum at the heel being $= 6 \cdot 4$ tons.

(26) In Fig. 6 the distribution of pressure on the base, due to the incidence of R, first at the toe, secondly at the two-third point and thirdly at the centre, is illustrated. In the first case (R_1) it will be seen that the neutral point n falls at the first third point. Thus two-thirds of the base is in compression and one-third in tension, the maximum in either case being clearly proportionate to the relative distance of the neutral point from the toe and heel of the base, the compression at the toe being four times, while the tension at the heel is twice the mean pressure or $\frac{N_1}{b}$ assumed at 4 tons.

In the second case R intersects at the two-third point, and the consequent position of n is exactly at the heel. The whole base is then in compression, and the maximum double the mean.

In the third case (R_2) the line gn will be at right angles to fg, which is vertical, and consequently is horizontal, and the position of n is indefinite; the area of pressure thus becomes a rectangle with a uniform pressure of $\frac{N_1}{b}$.

In Fig. 7 an intermediate case is exemplified.

(27) In order to illustrate Haessler's polygon as used with a curved back Fig. 8 is produced.

The profile is similar to Fig. 4, but with a projection at the rear, the back of laminas 4 and 5 being battered outwards. The face line is also recessed within the elementary hypothenuse. The section is not put forward as a model, but simply by way of illustration of the graphical system employed with a curvilinear back.

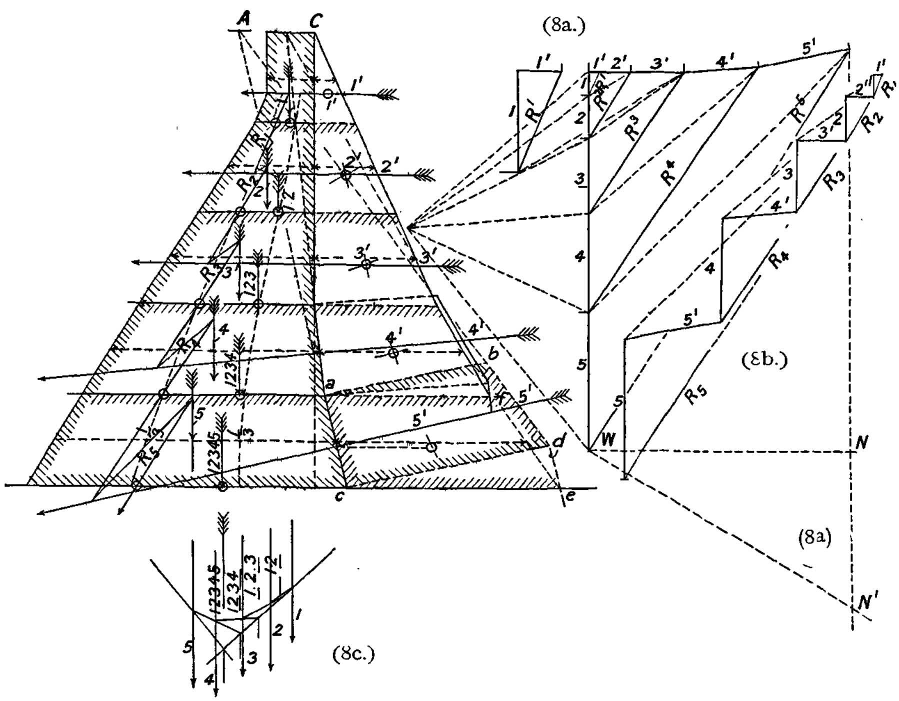
Two force polygons, Figs. 8a and 8b, are given, illustrative of two methods of graphical construction, which have identical results. The corresponding force line in both figures are parallel and of the same lengths. Either can be used. The small detached triangle adjoining Fig. 8a is merely an enlargement of the first triangle of forces $1, 1^1, R_1$, which is on too small a scale for accurate transmission to reciprocal lines on the profile.

The process having been already described in par. 30, Chap. I, need not be repeated. It differs from that shown in Fig. 4 in that the forces are not grouped in sets, each pair, with their resultant, being independent of the remainder; but the whole system forms a combination of all the several parts. The resultant lines of water pressure belong only to each lamina separately, and are drawn normal to the back of the wall. In the former case the water pressure lines belong to a group of several laminas, and consequently if the back of the wall is not in one plane any direction given will be erroneous.

(28) As the laminas of the water pressure areas have inclined, not horizontal, bases, their mean widths multiplied by their vertical depth, *i.e.*, by $\frac{H}{5}$, do not represent their areas; consequently $\frac{H}{5}$ not being a common factor, cannot be eliminated; all the forces then would have to be represented by areas not half widths.

This can be avoided by the following simple device, which will be found frequently used in subsequent diagrams:—

In Fig. 8 let the back of the lowest lamina *acdb* be produced upwards to



FIGS. 8, 8a, 8b, and 8c.

A, and *Ad* joined. *Acd* is then the triangle of water pressure, on the supposition that the back of the wall had the same inclination throughout as *ac*, because *cd* has originally been made $= \frac{H}{\rho}$; the trapezoid *acdb* also represent the water pressure on the plane *ac*.

Draw *de* parallel to *ac* and join *Ae*. Then the triangle *acd* and *Ace* are equal, being on the same base *Ac* and between the same parallels. But the area of the triangle *Ace* is equal to *ce* $\times$ *H*, consequently also the horizontally-based trapezoid *acef*, which has $\frac{H}{5}$ for its vertical depth, is equal to the original inclined area *acdb*, *i.e.*, it represents the water pressure acting on *ac*. The same procedure is followed with regard to the upper lamina. With this alteration, which takes longer to describe than to effect, the horizontal half-widths of the two newly-formed pressure areas will truly represent their areas, as their depth $\frac{H}{5}$ is a factor common to all others.

In Fig. 9 a clearer illustration of the working is afforded, the horizontally-based triangle ABD being substituted for its equal ABC; the half-width FG will then represent the area of either when H is eliminated.

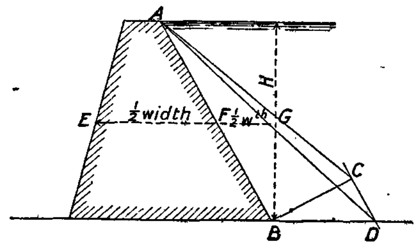


FIG. 9.

(29) Two further examples of dam sections are given in Figs. 10 and 11. In both of these the back of the wall is given a slight batter to compensate

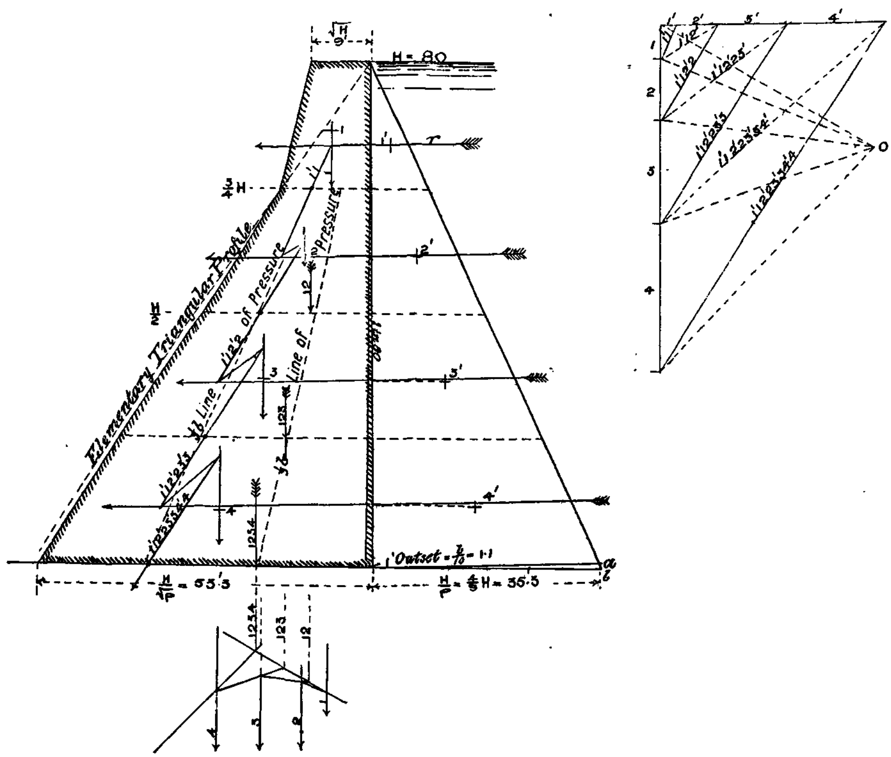


FIG. 10.

for the imposition of the crest. The latter face also is not carried down vertically, but is joined to where a height of $\frac{3}{4}H$ intersects the hypothenuse of the elementary triangle. This thick-necked profile is preferred by some authorities as affording resistance to the great temperature stresses to which the upper quarter of a dam is subject, as well as to the pressure of ice.

In Fig. 10 the two lines of pressure are drawn through the profile, Haessler's system being employed.

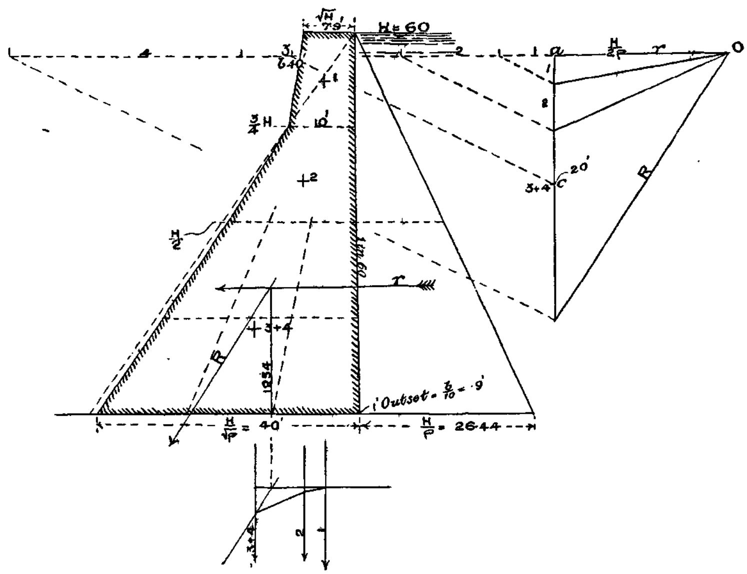


FIG. 11.

In Fig. 11 the dotted lines show the graphical reduction of the actual half-widths, which are reduced from a horizontal line, to a smaller scale on

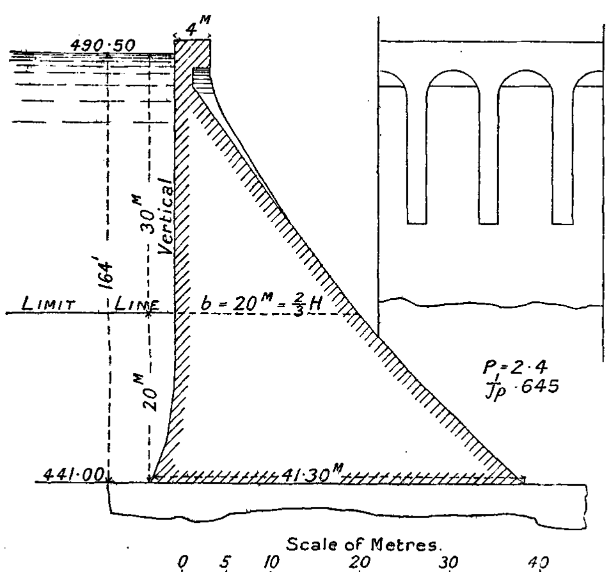


FIG. 12.—Chartrain Dam.

the load line. The final resultants of R and W on the base are alone shown. The drawing of pressure lines at end is really only necessary below the limiting depth, but has been done in Fig. 10 and other cases in order to illustrate the graphical methods employed.

(30) Some examples of actual sections, confined to strictly modern types, will now be given.

Fig. 12 is of the Chartrain dam, erected some fifteen years ago. This dam may be considered as the latest exposition of French practice, of which the Furens dam was the first to be designed on strictly correct scientific principles. The crest is remarkably light and yet stiff, and the roadway partly carried on arches overhanging the fore slope, which arrangement greatly adds to the architectural effect of the elevation.

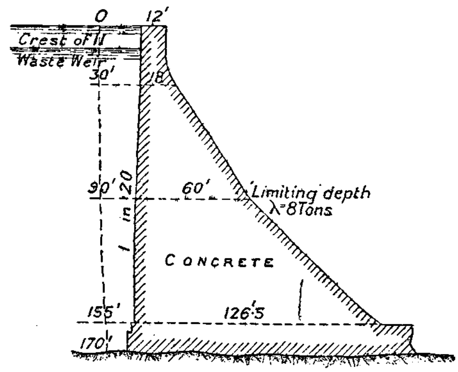


FIG. 13.—Periyar Dam.

The limit pressure is 9·1 tons, and the value of ρ is 2·4. The limiting depth is marked on the profile. The back is vertical up to this point, beyond which it widens out in accordance with the principles and method adopted in this work. The height H governing the base in the formula

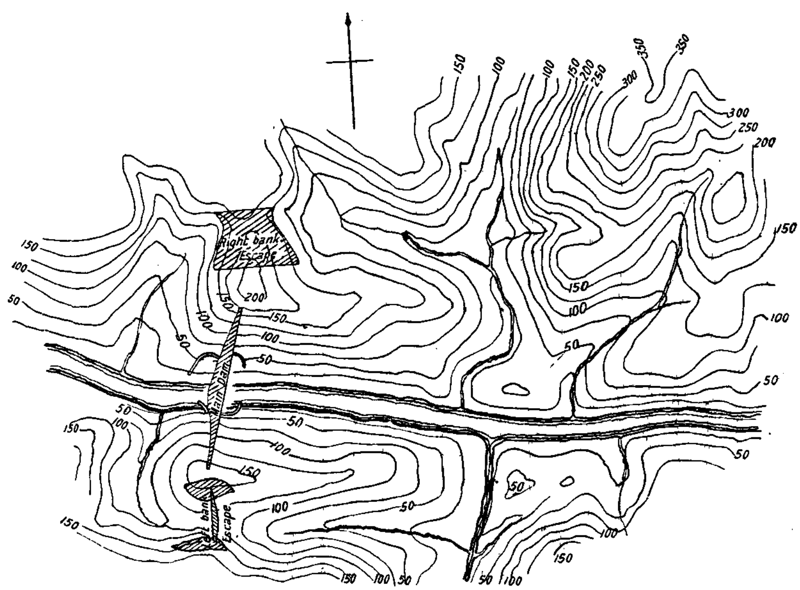


FIG. 13a.—Periyar Dam. Site plan.

$\left(b = \frac{H}{\sqrt{\rho}}\right)$ is taken half-way between full reservoir level and the actual weir crest. The dam is built to a curve of 1,300 feet radius, and is over a tributary of the Loire.

(31) Fig. 13 is the section of the Periyar dam in the Madras Presidency.

The Periyar river is fed by the heavy rainfall of the western Ghauts. By means of the dam the natural outlet of the river is blocked and the supply is thus diverted by a tunnel cut through the watershed of the Peninsula into the comparatively rainless tracts east of the mountain barrier. It debouches into the channel of the Vigar river, a stream running towards the east coast which will form the supply to a large canal and tank system.

Surplus water is passed over a waste weir on the left in addition to which there is a wide bye-wash or waste-way cut in the solid rock on the right flank of the dam. The position of the work is shown in the site plan of Fig. 13a, excepting that of the tunnel. This work was constructed under great difficulties due to the unhealthiness and remoteness of the site, and also due to the timidity of the Irrigation Department in not sanctioning the construction of a sluice through the body of the dam so as to allow drainage to pass, during the progress of the work, which greatly increased the cost and the difficulties to be overcome. It will impound 156,500 acre-feet.

This great work is one of the few storage projects undertaken in India. With exception of the large Lakes Fife and Whiting, reservoirs in the Bombay Presidency and of the works on the Betwa river in Upper India, there are hardly any storage works of magnitude. The great irrigation works in India consist almost exclusively of large canal systems fed from perennial and often snow-fed rivers from which the low supply only is utilised, leaving the flood waters to escape to the sea. As soon as this source of supply is finally exhausted it will be necessary to undertake large storage works in the mountainous sources of the rivers so as to impound part of the hitherto untouched flood waters.

(32) Fig. 14 is a presentment of the section of the Marikanave dam situated in the native State of Mysore.

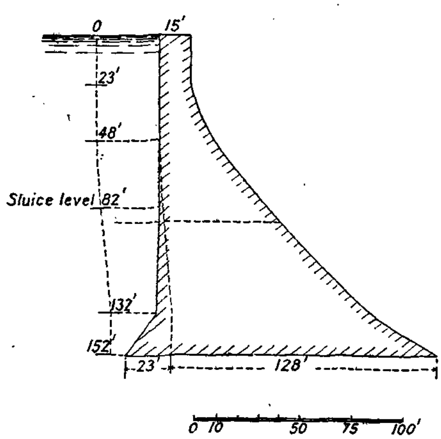


FIG. 14.—Marikanave Dam.

It has the marked peculiarity of having its escape for surplus water below full requirement, the dam having been constructed so high above the ordinary working capacity of the reservoir as to enable half the highest floods to be absorbed therein. It was found cheaper to do this than to provide an escape of the full flood capacity.

In this dam the value of ρ is 2·4 that of λ is 8 tons. It was built of rubble masonry, set in "Kunkur" lime mortar, an excellent native nodular limestone, possessing hydraulic property, which is found over large areas in gravel quarries, composed of nothing but this material. Kunkur not only supplies all the lime, but is used for the metalling of roads and other similar functions. In this dam the extreme water level is estimated at 142 feet above bed level, leaving a weatherboard of 10 feet. The elementary profile drawn on the section has its apex at the crest level, as this outline appears to be more in consonance with the actual profile. This work is the latest Indian dam. It is 1,200 feet in length. It will impound 183,500 acre-feet.

(33) Fig. 15 is a section of the Bhatgarh dam, which forms part of the Vira Canal Head works in the Bombay Presidency. Its height is 119 feet, and it has the appearance of having been designed according to the water level at RL 111, the divergence of its face from the hypothenuse of the elementary triangle with apex at 119·0 being too great. It is provided with fifteen low-level vents, each 4 feet wide by 8 feet deep, and also turbine sluices.

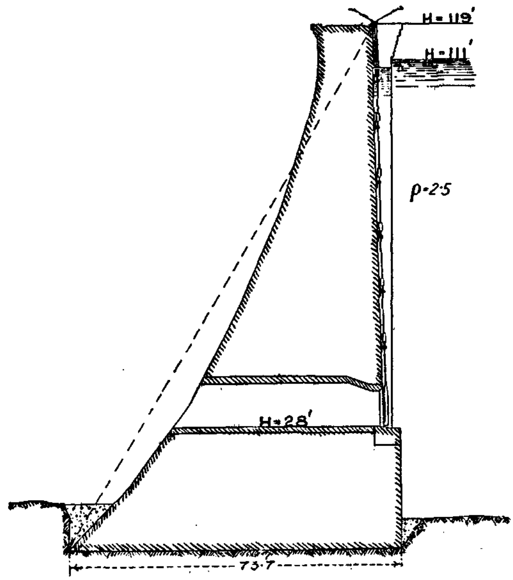


FIG. 15.—Bhatgarh Dam.

This dam is 3,020 feet long and 127' feet high above lowest point in foundations. The prescribed limiting pressure, the vertical component only being considered, is 8 tons per square foot, and the average weight of the structure is stipulated not to be less than 160 lbs. per square foot. Although the greater part is composed of concrete, yet this limit was actually exceeded, some of the stone put in weighing as much as 190 lbs. per cubic foot. The concrete was mixed with large blocks of loose stone, which were inserted in the work as it proceeded. These saved mortar and added to the weight and also efficiency of the concrete mass. The sides were lined with ashlar, and the lower portion was built with rubble masonry. The sluice ways were lined with cut granite ashlar blocks. On the section, the elementary triangular profile has been drawn, the base $\frac{H}{\sqrt{\rho}}$ being 71 feet, ρ having a value of over

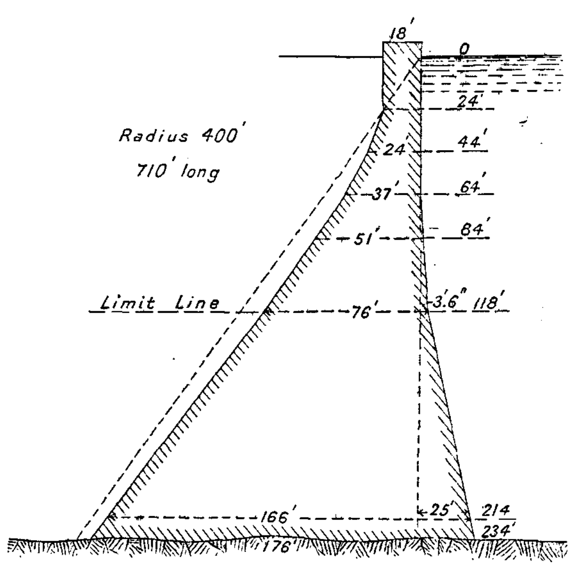


FIG. 16.—Cheeseman Dam, Colorado.

$2\frac{1}{2}$. The actual base is 74 feet, probably to allow for compensation for the vantage.

The fifteen sluices are spaced 17 feet apart, so they do not require any

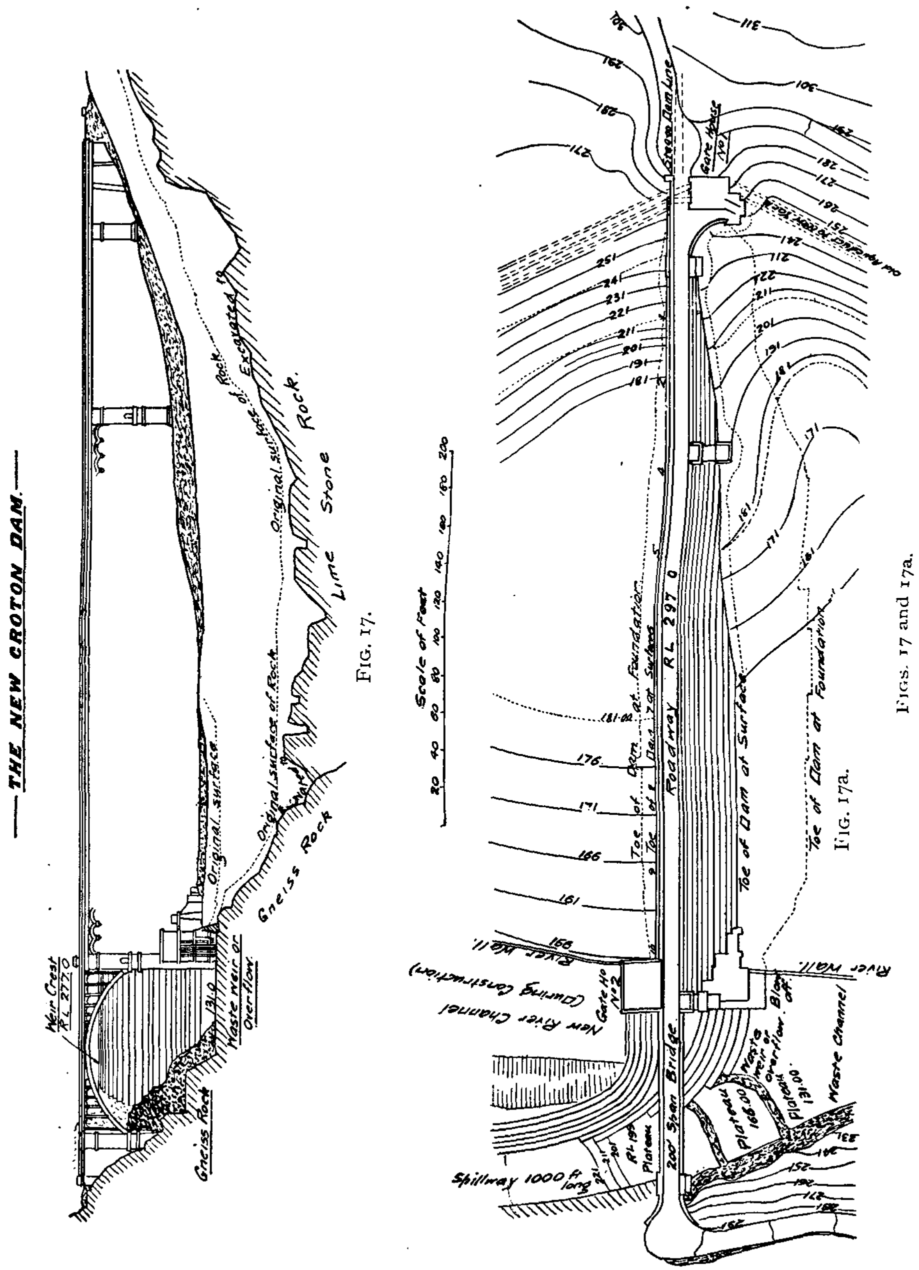


FIG. 17.

FIG. 17a.

FIGS. 17 and 17a.

special precautions of limit of pressure with regard to the piers, such as are necessary in the Assuan Dam. It will impound 106,500 acre-feet.

(34) Of gravity dams the Cheeseman dam in Fig. 16 is the highest existing example, barring the Roosevelt which is still under construction. It is built on a curvature of 400 feet radius, and is for the water supply of Denver City, Colorado. The spillway for surplus flood water is a bye-wash constructed in a narrow gap in the granite ridge, and is 300 feet wide.

The hypothenuse of the elementary profile is shown dotted in the section till it meets the actual base. At this level the value of *s* works out to 17·4 tons. This graphical calculation is not shown on the figure.

Calculated as an arch from the "Long" formula given in Chap. III., the pressure comes to 18 tons.

This section is deemed too thin in the upper quarter and would be better if the elementary profile were more closely adhered to, as in the Roosevelt dam.

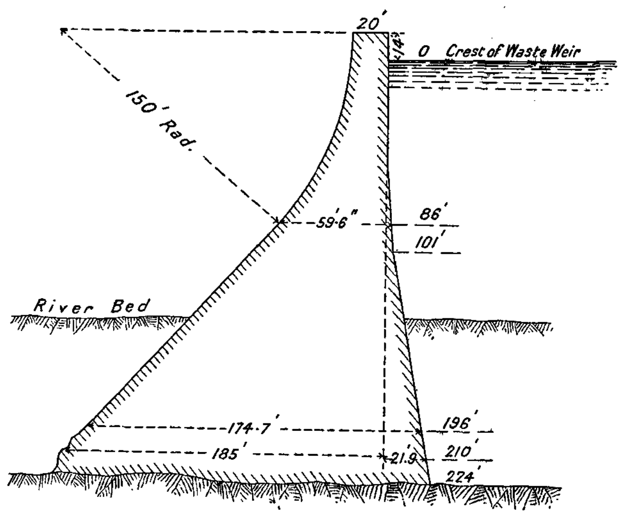


FIG. 17b.—New Croton Dam, N.Y.

(35) The New Croton dam for the water supply of New York City is a recently constructed work of great magnitude. The works consist of a dam carrying a roadway 20 ft. wide on the crest and at right angles to it the waste weir is set, forming one side of the reservoir. By this remarkable arrangement one flank of the river valley is utilised for the weir and the escape channel which is walled off from the actual river channel below the weir. The original waterway down stream is filled up to a high level and utilised as a park or recreation ground.

The dam portion of the work is 1,168 feet and the weir 1,000 feet in length. The plan and elevation of the work, for which the author is indebted to the *Engineer*, are given in Figs. 17 and 17a, the section of the dam in Fig. 17b. The section of the dam is remarkable for its immense size and the fact that the greater portion of its depth is buried out of sight below the bed of the reservoir. The maximum unit pressure is 11 tons. Owing to a fault in the rock foundation, it was deemed essential to carry the solid masonry down to this immense depth, through 150 feet of sand and gravel. The rock eventually met with formed probably the bed of the ancient glacial river and the material dug out was deposited in that remote age. This material although porous cannot but be deemed a most excellent foundation in a masonry superstructure, and certainly superior to made earth provided percolation was stopped. This, it is maintained, could have been effected with perfect safety by excavating a trench up stream through the material and well into the rock and filling the same up afterwards with cement concrete. If deemed insufficient, for further security a second curtain could be constructed down stream of the dam, but there are objections to this. The arrangement is shown in Fig. 17c.

If the core wall has been properly carried down well into the solid rock there is no reason why an earthen or hydraulic fill dam should not have answered, provided the original river-bed was left undisturbed and the embankment built on top of it. There are many instances of reservoir embankments thus constructed overlying porous material, which latter is rendered water-

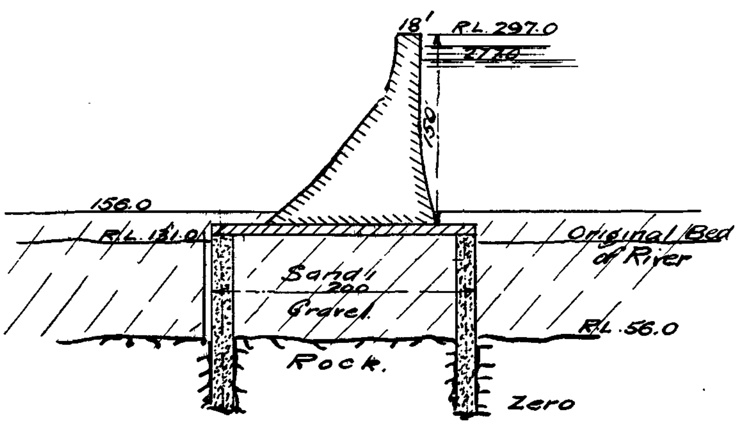


FIG. 17C.

tight by means of a core trench wall of puddle or concrete. In the Fig. 17c the horizontal platform should not overlap the vertical core walls.

(36) The Cross River Dam (Fig. 18) is another New York work, but on different lines to the two previous dams, having a closer correspondence with the elementary profile. The limiting stress is apparently 10 tons only. The neck of this dam is designed thicker than in the last two examples on account of ice.

(37) Fig. 19 is a section of the Ashokan Dam now under construction for the New York Waterworks extension. The section is a very thick one, extending in the upper part 5 feet beyond the hypothenuse of the elementary profile, with the apex of the latter placed at crest level; if the apex were placed lower at half-way up the weatherboard, which is no less than 20 feet deep, the discrepancy would amount to 10 feet. Both lines have been drawn on. This dam is fitted with a vertical line of porous blocks connected with two inspection galleries. This is in accordance with recent German practice to enable any leakage which would induce hydrostatic pressure to be stopped

and drained off. It is probable that the introduction of this refinement is the cause of the unusual widening of the section. The dam is constructed

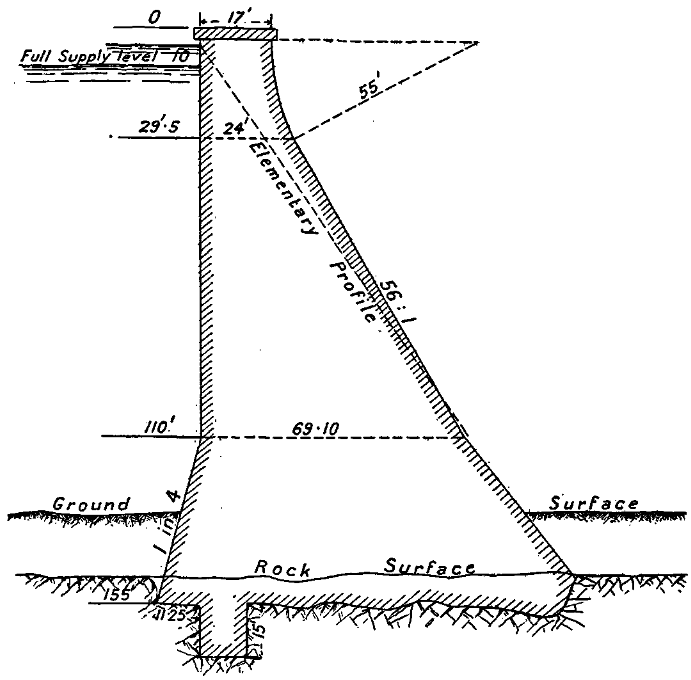


FIG. 18.—Cross River Dam, N.Y.

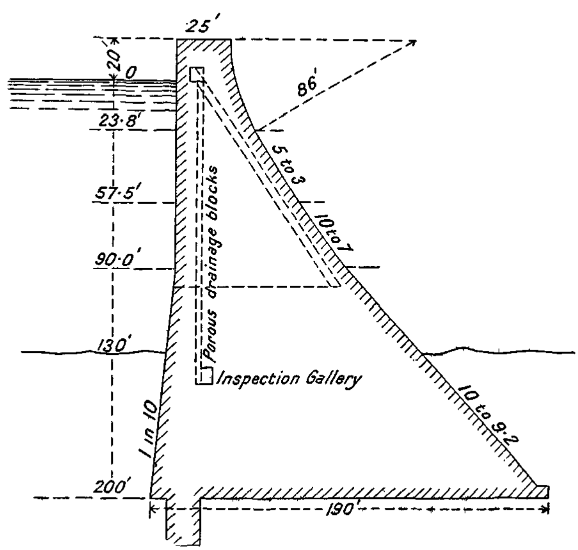


FIG. 19.—Ashokan Dam, N.Y.

of cyclopean masonry set in P.C. concrete. With the great interests involved, it was probably the determination of the designer to be absolutely on the safe side. Where money is "no object," as was also the case in the Assuan Dam, timid counsels are often apt to prevail.

(38) The Roosevelt Dam is given in Fig. 20. This grand work is still in process of construction, and when completed will be the highest gravity dam

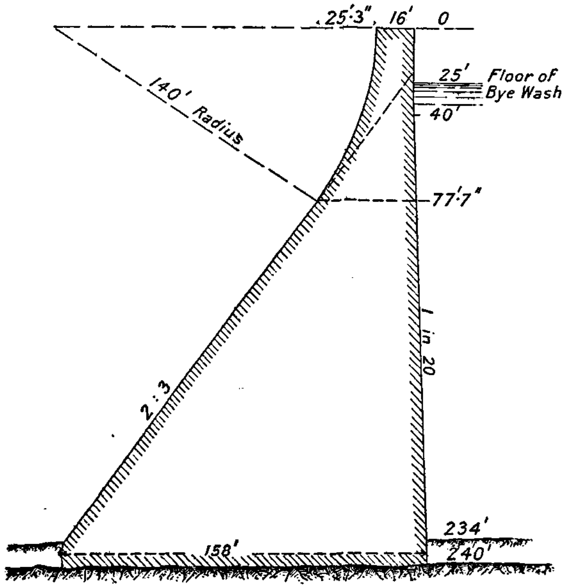


FIG. 20.—Roosevelt Dam.

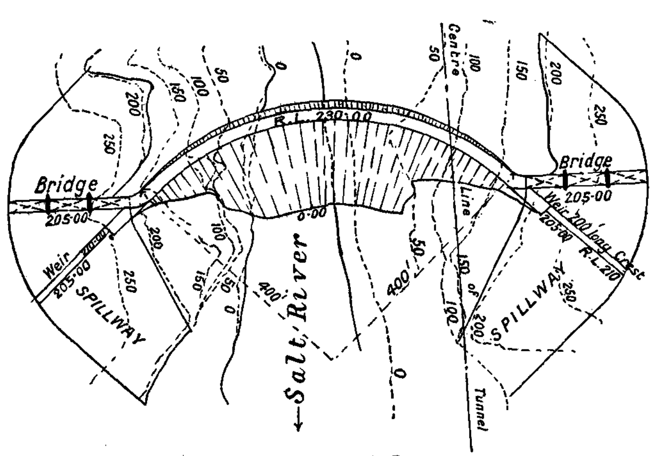


FIG. 20a.—Roosevelt Dam.

in existence. It is built on a curve of 410 feet radius to centre of crest, and 418 feet to extrados of the arch. It is, however, considered as a gravity, not as an arched dam, the curve being introduced for greater security, as it allows of freer movement under changes of temperature than the straight alignment. However that may be, if the apex of the elementary triangle were situated, as is probably the actual case, at 10 feet above the floor of the bye-wash or waste-way, the value of H will be 230 feet, requiring a base width of 154 feet according to formula 6, which is close to what it actually scales. The maximum unit stress has been worked out and amounts to 16 tons. If the stress were calculated as if the dam were an arch, it would work out by the long formula No. 3, given in the next chapter, to 20·8 tons, so that the stresses are not identical whichever view is taken of the status of the dam. The profile is free from the faults of being too thin or too thick in the neck, as is apparent in the last four examples, and is considered unexceptionable in every respect. As noted above, the section adheres closely to the elementary profile and forms a powerful advocacy in favour of the simple style of pentagonal profile favoured in this work.

The site plan given in Fig. 20a, forms an instructive example of the arrangement of spillways cut in the solid rock out of the shoulders of the canyon, the material thus obtained being used in the dam itself. These spillways are each 200 feet wide, and are excavated to 5 feet below the crests of the waste weirs which cross them. This allows of a much greater discharge passing under a given head than could be the case were a simple bye-wash provided without any drop-wall. The afflux is consequently diminished.

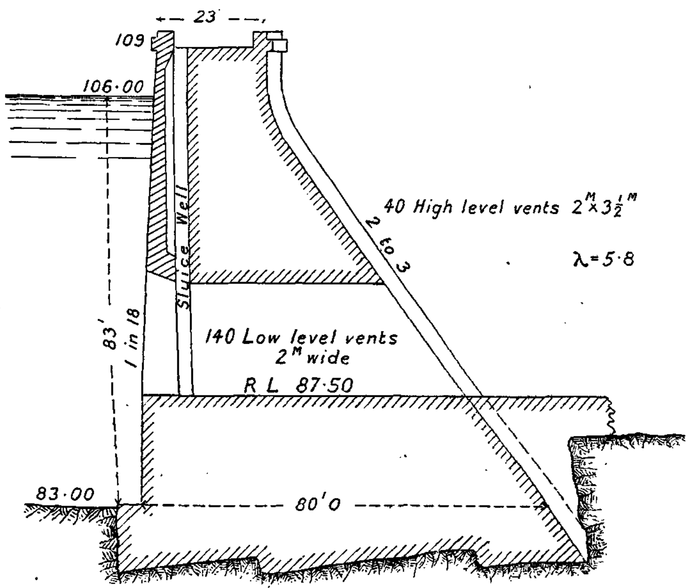


FIG. 21.—Assuan Dam.

This dam will impound the enormous quantity of 1¼ million acre-feet. It spans the Salt River in Arizona, and is a U.S. reclamation project. On the same river below this work is situated the Granite Reef Weir, of which mention is made in the next section.

(39) Fig. 21 is a section of the Assuan Dam at its deepest part. This great work is built across the Nile at the first cataract. Its height is only 85 feet, but, like most Eastern works, it makes up in length what it lacks in height, which former is 6,400 feet.

The area now impounded is 863,000 acre-feet, but it is intended to raise the crest to R.L. 116·0, *i.e.* by 7 metres. This will double the capacity of the reservoir, which will exceed that of the Roosevelt Dam. Even then its height will be only 106 feet, one-third of that of the Shoshone arched Dam. This work is principally remarkable as being the only solid dam which passes the discharge of a large river like the Nile through its body, for which purpose it is provided with 110 low-level sluices, each 23 feet deep by $6\frac{1}{2}$ feet wide, and 40 feet high-level sluices 110 feet high and 6 feet wide. They are capable of discharging 500,000 second-feet, the

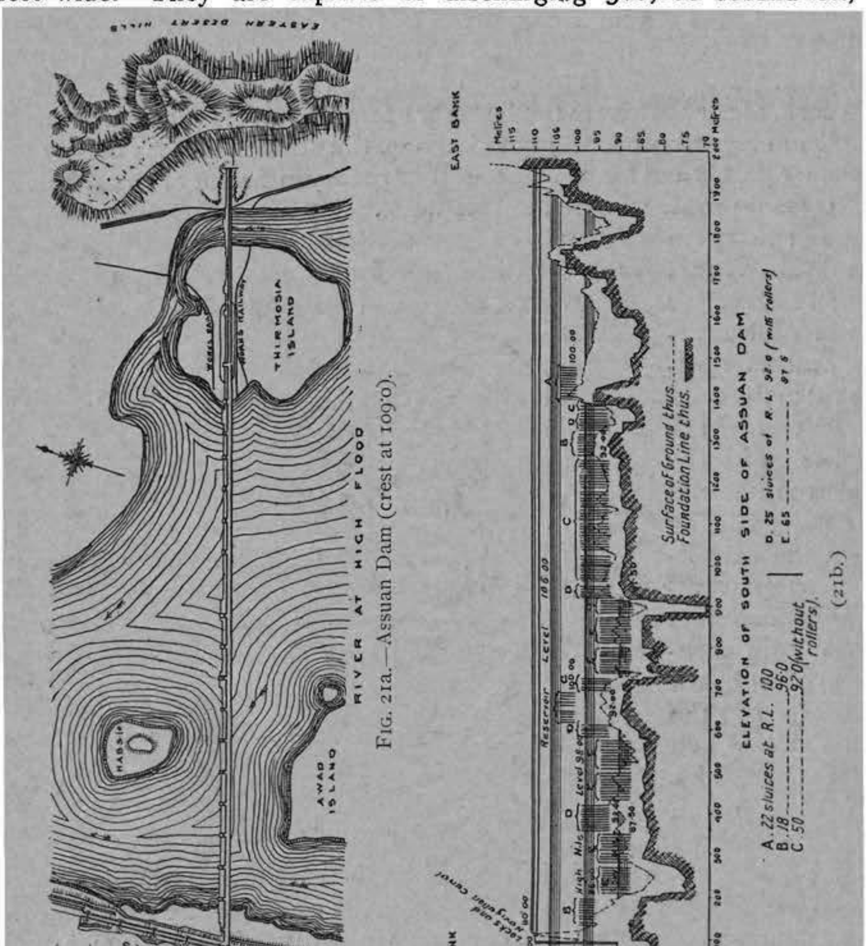


FIG. 21A.—Assuan Dam (crest at 109·0).

(21b.)

velocity in the low-level sluices being as high as 20 feet per second. As a matter of fact, however, the rise and the fall of the Nile being very regular, the level in the reservoir can be so adjusted as never to strain the sluices to anything like this extent. (See note, p. 83.)

The profile is by no means a bold one, the unit pressure in the piers being limited to 5·8 tons only.

Even when the dam is raised, the unit stress in the piers would not exceed 10 tons. However, it is intended to widen the whole base in order to maintain the restriction in unit pressure which is now in force. To effect this, an outer skin will be built on the down-stream side, and the space between it and the present face of the dam will be filled up with cement concrete.

Other sections of this dam, showing details of the "Stoney" sluice gates and frames, are given in Chap. XIV.

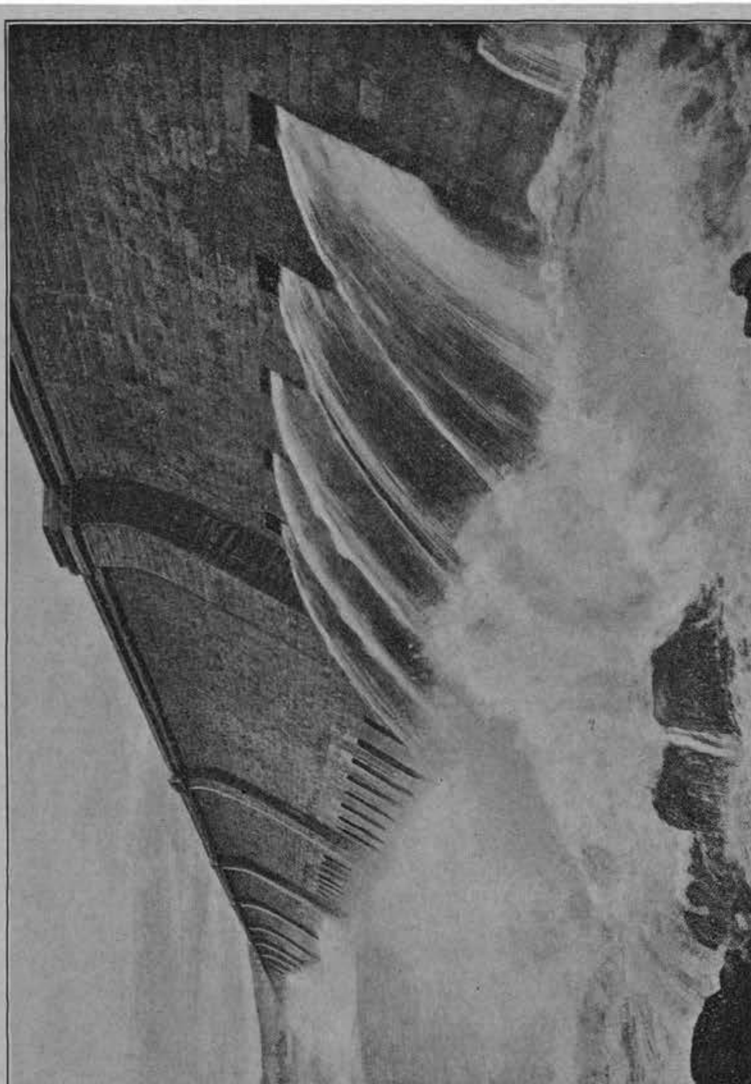


FIG. 22.—Assuan Dam, Egypt.

The photograph Fig. 22 shows some of the sluices in operation, and gives a good idea of the extent of the work.

SECTION II.—GRAVITY WEIRS.

(40) When water overflows the crest of a dam it is termed a weir, and some modification in the design of the section generally becomes necessary,

and not only that, but the kinetic effect of the falling water has to be provided for by the construction of an apron or floor which in many cases forms by far the most important part of the general design. This is so pronounced in the case of dwarf diversion weirs over wide sandy river beds that the weir wall itself forms but an insignificant part of the whole section, and a separate chapter will have to be devoted to the elucidation of the complicated problem presented for solution. At present the section of the weir wall alone will be dealt with.

(**41**) Fig. 23 is a typical section of a trapezoidal weir wall with water passing over the crest. The height of the crest of the wall as before is designated by H, that of the depth above crest to reservoir level by d. The total height to the upper water level will therefore be $H + d$. The triangle of water

(23a.)

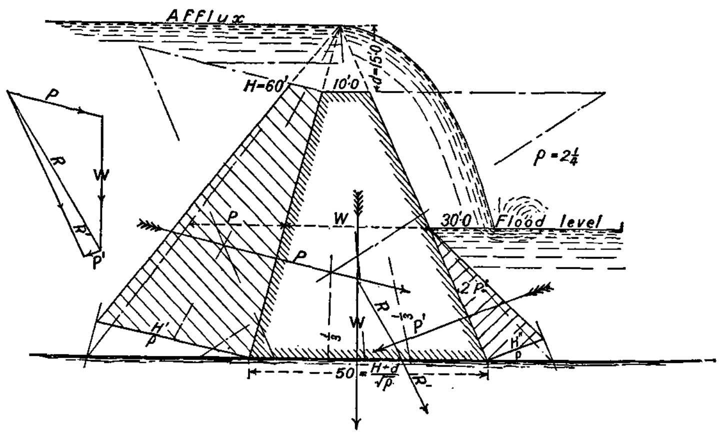


FIG. 23.

pressure will have its apex at the surface, and its base will be $\left(\frac{H+d}{\rho}\right)$ or $\frac{H'}{\rho}$, as previously, but it will be truncated at the crest level, and the actual water pressure acting on the wall is the lower part of the triangle, a trapezoid whose base is $\frac{H+d}{\rho}$ and whose top width is $\frac{d}{\rho}$. Its area will therefore be, if the back of the weir is vertical,

$$A = \left(\frac{H+d}{\rho} + \frac{d}{\rho}\right)\frac{H}{2} = H\left(\frac{H+2d}{2\rho}\right). \quad (11)$$

If the back is inclined, the area will be

$$A = \left(\frac{H+2d}{2\rho}\right) \times H^1. \quad (11a)$$

H^1 being the inclined length of the back of the weir wall. The distance of its point of application above the base

$$= \frac{H}{3}\left(\frac{H+3d}{H+2d}\right). \quad (12)$$

(**42**) With regard to the weir wall, the elementary triangular profile, so useful in the case of dams, will be found to be equally so as a guide to design in this case. When applied, its apex will in like manner be at the upper level, or at $H + d$, and be similarly truncated at the crest.

The trapezoid thus formed will have a base width of

$$b = \frac{H + d}{\sqrt{\rho}} \qquad (13)$$

which formula will be found approximately correct, as a general rule, and with a top width of $\frac{d}{\sqrt{\rho}}$ its area will be $\frac{H + 2d}{2\sqrt{\rho}} \times H$.

The crest width of $\frac{d}{\sqrt{\rho}}$ will, however, be found in practice much too narrow for actual requirements unless the depth of water above the crest or d is exceptionally great. The following rule will be found to provide a suitable crest width for most ordinary cases, viz.,

$$c = \sqrt{H} + \sqrt{d} \qquad (14)$$

with $\frac{d}{\sqrt{\rho}}$ as a minimum. This will also apply to submerged weirs; for instance, in the Narora Weir (Fig. 24), H is 10 feet and d is 8 feet, whence $c = \sqrt{10} + \sqrt{8} = 3{\cdot}2 + 2{\cdot}8 = 6$ feet. In the insubmerged La Grange Weir (Fig. 29) $H = 120$ and $d = 16$; the crest width would be $11 + 4 = 15$ feet.

In many cases, however, the necessity of providing space for falling shutters or for cross traffic during times when the weir is not acting renders obligatory the provision of an even wider crest. With a moderate width, as obtained by formula (14), a trapezoidal outline has to be adopted in order to give the requisite stability to the section. This is formed by joining the edge of the crest to the toe of the base by a straight line, the base width of $H + d \div \sqrt{\rho}$ being retained.

This is done in Figs. 24 and 29 (*post*). When the crest width exceeds the dimensions of formula (14) the face of the crest should drop down vertically till it meets the hypothenuse of the elementary profile, as is the case with the pentagonal profile of dams. An example of this is given in Fig. 28 of the Dhukwa Weir. The tentative section thus outlined should be tested by graphical process, and if necessary the base width altered to conform with the theory of the middle third.

(**43**) In American practice with trapezoidal weir profiles, the width of the weir crest is made some fixed proportion of that of the base. To be universally applicable, however, the value of c should be also some function of d, and it is possible that a formula might be devised, which should include all three influencing factors, viz., H, ρ, and d.

With values of d up to 10 feet the following formula for base width for a trapezoidal profile will be found closely approximate to requirements, $\frac{c}{b}$ or r being taken as $\cdot 25$

$$b = \frac{H + d}{\sqrt{\rho}} \times \frac{1}{\sqrt{r + 1 - r^2}}, \qquad (15)$$

$$\text{or } \rho = \frac{2}{3} \text{ and } r = \cdot 25,\ b = \cdot 6\,(H + d). \qquad (15a)$$

Formula (15) is strictly applicable to dams of trapezoidal profile, H being substituted for $(H + d)$. When applied to weirs, the value of c in the ratio $\frac{c}{b}$ is the crest width on the supposition that the weir is a dam whose height $H = H + d$. The actual crest width at the lower level will naturally be wider than the assumed value of c at the higher level.

When a reservoir wall has its crest lowered for a portion of its length, it acts both as a dam and a weir. Its outline is then usually trapezoidal, when formula (15) for base width will apply equally to both portions. The value of b will, however, be barely sufficient for the weir section were it not influenced favourably by the reverse pressure of the tail water.

(**44**) A weir is subject to the effect of the reverse pressure of the tail water down stream, to which a dam is usually exempt.

This reverse pressure modifies the position of the centre of pressure on

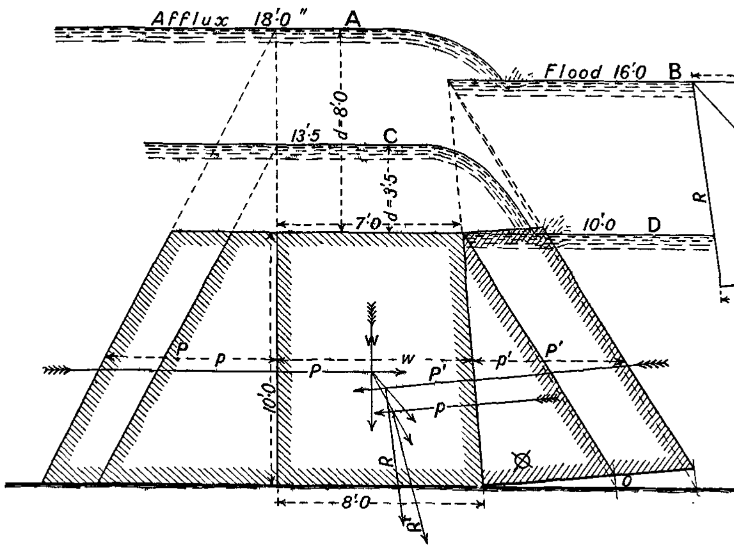


FIG. 24.—Narora Weir Wall.

the base in a manner usually, but not invariably, favourable to stability. As the moments of the horizontal pressure of water on either side of the weir wall vary very nearly with the cubes of their height, it is evident that a comparatively low depth of tail water will have but small influence and may well be neglected. This is accentuated by the slope given to the face when a vertical back is adopted by which the normal water pressure is given a downward inclination that reduces its capacity for good. The small result thus produced is illustrated in the force diagram Fig. 23a, where the reverse is shown combined with the back pressure.

(**45**) Calculations of the depths of water passing over the weir or rather the height of reservoir level above the weir crest, designated by d, and of its reciprocal depth D in the tail channel, are often necessary for the purpose of ascertaining what height of water level up stream, or value of d, will produce the greatest effect on the weir wall. In low submerged, or drowned weirs, the highest flood level has often the least effect, as at that time the difference of levels above and below the weir are reduced to a minimum. This is graphically shown in Fig. 24, which represents a section of the Narora dwarf weir wall, to which reference will subsequently be again made in Chap. VI. In this profile two resultant pressures, R and R_1 are shown, in which R_1, due to the much lower water level of the two "states" under comparison, falls nearer the toe of the base.

(**46**) The rise of the river water produces, with regard to the stress induced on the weir, three principal situations or "states" which are enumerated below.

(1) When the *head water* is at weir crest level, then, except in cases where a water cushion exists, natural or artificial, the tail channel is empty, and the conditions are those of a dam.

(2) When the *tail water* is at weir crest level; in this case the reciprocal depth of the head water above crest is found by calculation.

(3) At *highest flood level*, the difference between the head and tail waters is then at a minimum. (2) applies only to submerged weirs.

In an insubmerged weir, the greatest stress is generally produced during state (3). In a submerged weir the greatest stress is produced during state (2).

(**47**) The horizontal water moments on either side of a wall are related to each other in proportion to the cubes of their respective depths. In cases where the dividing wall is overtopped and one or both of the water pressure triangles are truncated, the moment of the resulting trapezoid of pressure will be the product of formulas (11) and (12) and will be

$$M = \frac{H^2}{6}(H + 3d). \qquad (16)$$

The moment of the opposing tail water will be, if a triangle, $\frac{D^3}{6}$, or if a trapezoid formula 16 will again apply, d^1 or $(D - H)$ taking the place of d in that expression. The difference of the two moments of water pressure up and down stream will then be the balance moment acting on the wall.

For example, in the case of Narora Weir:

During "state" (1) $H = 10$, $D = 0$ feet. Balance moment will then be $\frac{1}{6}(10^3 - 0) = 166{\cdot}6$ feet.

During "state" (2) $H = 10$, $d = 3{\cdot}5$ and $D = 10$ feet. Then from formula (16) the balance moment will be $\frac{1}{6}\{(100 \times 20{\cdot}5) - 1000\}$ $= \frac{2050 - 1000}{6} = 175$ feet.

During "state" (3) $H = 10$, $d = 8$, $D = 16$ and $d^1 = 6$ feet. The balance moment will be $\frac{1}{6}(100 \times 34) - \frac{1}{6}(100 \times 28) = 100$ feet.

Thus we see that state (2) produces the greatest effect, the least being state (3).

In spite of these obvious facts many weir wall sections have been designed under the erroneous supposition that the overturning moment is greatest when the upper water is at crest level and the tail channel empty, *i.e.*, at a time when the difference of levels above and below the weir is at a maximum.

(48) In order to obtain the proper value of d, during the second "state," it will be necessary to make a calculation of the discharge per foot run of the channel below the weir at the level of its crest.

This can be effected by use of the formula $Q = 100\ Ac\sqrt{RS}$ (*vide* Chap. V.) or, as it is often expressed, $Q = AC\sqrt{RS}$ where $C = 100c$.

In this expression A is the area of the channel, c a coefficient varying with R, with n the rugosity of the channel, and with S, and its value is obtained from tables worked out for that purpose. R is the hydraulic mean depth, or the area divided by the wetted perimeter $= \frac{A}{WP}$, and S is the ratio of slope of the bed of the channel.

(49) In the case of submerged weirs over rivers with sandy beds, the base of the weir and its floor often coincide with the average bed level of the river. The river also is of considerable width in proportion to its depth ; consequently the formula $100\ Ac\sqrt{RS}$ can be simplified without sensible error by substituting D for R.

As the discharge per unit length, or foot run only, is required, the area A will equal $D \times 1 = D$, and, further, if the friction at the sides of the river be neglected, WP, or the wetted perimeter, will equal the base of the water section, *i.e.*, unity, whence $R = \frac{A}{WP} = \frac{D}{1} = D$. The formula then simplified becomes

$$Q = 100\ cD^{\frac{3}{2}}\sqrt{S}. \quad (17)$$

The coefficient c can be obtained from Jackson's "Hydraulic Manual," Table XII., Part 4, D being taken as R, or from Higham's Tables. To facilitate calculation the following Table I. gives the formula worked out in terms of c and D for different bed slopes :—

TABLE I.

S or Slope of Bed.	S per 1,000.	$100\sqrt{S}$.	Discharges per ft. run or Q.
1 in 2,500	·4	2·00	$2cD^{\frac{3}{2}}$
1 in 5,000	·2	1·414	$1{\cdot}414cD^{\frac{3}{2}}$
1 in 6,666	·15	1·247	$1{\cdot}247cD^{\frac{3}{2}}$
1 in 10,000	·1	1·000	$cD^{\frac{3}{2}}$
1 in 20,000	·05	·707	$·707cD^{\frac{3}{2}}$

The following Table II. will be useful for purposes of comparison of the effect different slopes and values of n have on the ratio $d : D$:—

TABLE II.

S = 1 in 2,500 or ·4 per 1,000.													
n = ·0275.									n = ·030.				
D	$100\sqrt{S}$	$D^{\frac{3}{2}}$	$100\sqrt{S}D^{\frac{3}{2}}$	c	Q	V	d	$\frac{d}{D}$	c	Q	v	d	$\frac{d}{D}$
7½	2	20·54	41·08	·768	31·6	4·2	4·2	·546	·711	29·2	3·9	4	·533
12	2	41·57	83·14	·826	68·7	5·7	7·1	·583	·767	62·3	5·2	6·7	·56
15	2	58·77	117·53	·850	99·9	7·0	9·0	·6	·791	93·0	6·2	8 6	·573
S = 1 in 5,000 or ·2 per 1,000.													
n = ·0275.									n = ·0250.				
7½	1·414	20·54	29·04	·779	22·6	3	3·5	·467	·850	24·7	3·3	3·7	·500
12	1·414	41·57	58·78	·843	49·6	4·2	5·9	·492	·914	53·7	4·5	6·1	5·1
15	1·414	58·77	83·10	·870	72·3	4·8	7·5	·500	·942	78·3	5·2	7·7	·513
20	1·414	89·44	126·48	·905	114·5	5·7	10·2	·510	·977	123·5	6·7	10·5	·525
S = 1 in 6,666 or ·15 per 1,000.													
n = ·0275.									n = ·0250.				
7½	1·247	20·54	25·61	·785	20·1	2·7	3·2	·426	·856	21·9	2·9	3·4	·453
12	1·247	41·57	51·84	·853	44·2	3·7	5·4	·450	·925	48·0	4·0	5·75	·480
15	1·247	58·77	73·29	·883	64·7	4·3	7·1	·473	·955	70·0	4·7	7·3	·490
20	1·247	89·44	111·54	·920	102·6	5·1	9·5	·475	·993	110·8	5·5	9·8	·490
S = 1 in 10,000 or ·10 per 1,000.													
n = ·0275.									n = ·0250.				
7½	1·000	20·54	20·54	·797	16·4	2·2	2·8	·373	·868	17·8	2·4	3	·400
12	1·000	41·57	41·57	·873	36·3	3·0	4·7	·392	·946	39·3	3·3	5	·417
15	1·000	58·77	58·77	·906	53·2	3·5	6·2	·413	·980	57·6	3·8	6·5	·433
20	1·000	89·44	89·44	·949	84·9	4·2	8·4	·420	1·023	91·5	4·6	8·8	·440
S = 1 in 20,000 or ·05 per 1,000.													
n = ·0275.									n = ·0250.				
7½	·707	20·54	14·52	·825	12·0	1·6	2·4	·320	·899	13·1	1·7	2·4	·320
10	·707	31·62	22·36	·819	18·3	1·8	3·2	·320	·960	21·5	2·25	3·1	·310
12	·707	41·57	29·39	·921	27·0	2·3	4·0	·333	·998	29·3	2·5	4·2	·350
15	·707	58·77	41·55	·965	40·1	2·7	5·0	·333	1·043	43·3	2·9	5·4	·360
20	·707	89·44	63·24	1·022	64·6	3·2	7·0	·350	1·101	69·6	3·5	7·5	·375

(**50**) In Table II. the expressions in the final column of Table I. are worked out for different values of D and of n, the coefficient of rugosity, by which the discharge Q in column 6 is obtained.

The next column V, or the mean velocity of current, is obtained by dividing Q by D as $V = \frac{Q}{A}$ and, as we have seen, $D = A$.

The value of d reciprocal to D is given in the eighth column. This is obtained from the formula for discharge of free overfalls, viz.,

$$= 3{\cdot}333 d^{\frac{3}{2}} \left\{ \left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\}$$

which is worked out in Table II., Chap. V. For example, in the fourth series of Table II., where $S = \frac{1}{10,000}$ and $n \doteq {\cdot}0275$, when $D = 12$, $Q = 36{\cdot}3$, and $V = 3{\cdot}0$. The corresponding value of d is found by examining the discharges per foot run in the seventh column, Table I., Chap. V., under $V = 3$. The value of d, as will be seen, lies between $4\frac{1}{2}$ and 5 feet, the discharges at which depths are 32·739 and 38·233, the difference being 5·494.

Now the difference between the lower of these two quantities corresponding to $d = 4\frac{1}{2}$ and the given discharge of 36·3 is 3·56, consequently the required value of d will be $4{\cdot}5 + \frac{3{\cdot}56}{5{\cdot}49} \times \frac{1}{2} = 4{\cdot}5 + {\cdot}24$ (nearly) $= 4{\cdot}74$.

(**51**) The values of d obtained from the above Table II. in this chapter must always be maximum values, and are really only applicable when the widths of the river bed and length of weir are such that the non-inclusion of the sides of the water section in the wetted perimeter has no appreciable effect on the result. The ratio of d to D is dependent on the width of the water section, which latter influences the value of WP and consequently that of R. In the following Table III. the discharges are worked out by the ordinary formula of $100\ Ac.\sqrt{RS}$ for two bed widths of 200 feet and 1,000 feet. The length of weir is assumed as equal to the bed width, the side slopes are taken as 1 to 1; consequently the wetted perimeter will be $b + 2\,D\,\sqrt{2} \doteq b + 3D$ nearly.

Table III.

S = 1 in 5,000.										
$n = {\cdot}0275$ $b = 200$.										
D	$A = (b+D)\,D$	$WP = b + 3D$	$R = \frac{A}{WP}$	c	$100\sqrt{RS}$	$V = 100\sqrt{RS}$	$Q = AV$	$\frac{Q}{b}$	d	ratio $\frac{d}{D}$
12	2544	236	10·8	·829	4·5	3·73	9489	47·44	5·7	·473
15	3225	236	12·4	·847	4·9	4·15	13383	66·90	7·1	·473
Same with b = 1,000 ft.										
12	12144	1036	11·7	·839	4·83	4·06	49256	49·25	5·8	·483
15	15225	1045	14·5	·866	5·38	4·66	70948	70·95	7·5	·500

As will be seen, the ratio of d to D falls short of that given in the previous Table, and must vary indirectly with b. The former Table, however, is useful as a guide where b exceeds 1,000 feet. When actual velocity observations are taken with a representative water section extending up to highest flood level, the discharge at any level can be obtained with much greater accuracy than by using any formula. Surface velocities should be reduced to mean velocities, and the discharge at any level other than that at which the velocity observations were taken can be obtained, as explained in Chap. V., by using the equation $Q = \frac{ACq\sqrt{R}}{ac\sqrt{r}}$ there given. With the discharge and mean velocity thus obtained, the value of d can be found from Table I., Chap. V., and the ratio $\frac{d}{D}$ or the proportion of the respective rises in the water levels above and below the weir ascertained.

(52) In the above calculations for the approximate base width of a weir the pressure on the weir due to the velocity of approach of the stream has not been taken into consideration. This velocity is the mean velocity of approach, not that of the actual film passing over the crest.

The depth of the actual film at the weir crest is always less than the depth d we have been considering, which latter is the height of the reservoir level some short distance above the weir, measured up from the crest, *i.e.*, before the break in the water level, which always occurs on the current approaching an overfall, causes a diminution in the depth, owing to increased velocity.

The velocity of approach of the current $Va = Q_1 \div A_1$, in which Q_1 is the discharge and A_1 the effective area of the water section at the weir. With dwarf weirs on sand Va can be assumed equal to V, the mean normal velocity.

Its effect on the weir is that of the corresponding head of water which causes this velocity, and when V is the mean velocity of the stream, this head will be $h = \cdot 0155 V^2 = V^2 \div 2g$.

The following Table gives the values of h due to various mean velocities of approach :—

TABLE IV.

TABLE OF VALUES OF HEAD DUE TO VELOCITY OF APPROACH.

Mean Vel. V.	V^2.	Multiplier.	h.	Mean Vel. V.	V^2.	Multiplier.	h.
2	4	·0155	·06	6	36	·0155	·56
$2\frac{1}{2}$	6·25	,,	·10	$6\frac{1}{2}$	42·25	,,	·66
3	9	,,	·14	7	49	,,	·76
$3\frac{1}{2}$	12·25	,,	·19	$7\frac{1}{2}$	56·25	,,	·87
4	16	,,	·25	8	64	,,	·99
$4\frac{1}{2}$	20·25	,,	·31	$8\frac{1}{2}$	72·25	,,	1·12
5	25	,,	·39	9	81	,,	1·40
$5\frac{1}{2}$	30·25	,,		10	100	,,	1·55

These should, strictly speaking, be added to the height H_1 or that on which the calculation for the base width is dependent, but for moderate velocities of approach, which mostly prevail when $D = H$, and the bed does not exceed 1 in 10,000, h can safely be neglected.

(**53**) When deep channels occur in the water section or the bed is very uneven, D, the depth at any part will not indicate the mean depth, and the

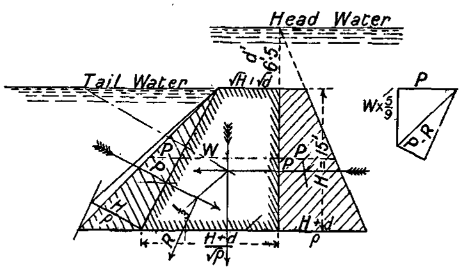


FIG. 25.

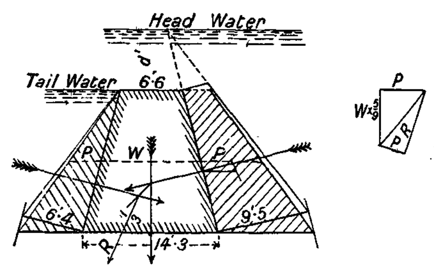


FIG. 26.

ratio r^0 will not be the fraction $\frac{d}{D}$. In such cases the ratio of rise of head and tail water can only be ascertained by working out the river discharges at two suitable levels of the tail water. The values of d reciprocal to these two levels can then be obtained from Table II., Chap. V., and the required ratio will be $\frac{d - d}{l - l_1}$, l and l_1 in formula being the two levels assumed of the tail water.

(**54**) An equiangular is sometimes preferable to a vertically backed section in order to permit of water shooting clear of the weir face. When the depth of the tail water D is considerable relatively to H, the vertical profile can be canted forward without any increase to the base width of $H + d \div \sqrt{\rho}$. When, however, the depth below is not much, this base width may prove insufficient.

In the case of a dam without any tail water, or in that of a weir with the tail water reaching to only half its height, a vertical backed profile is the most economical. Where the reverse pressure of the tail water is considerable, it is a distinct advantage to adopt the equiangular profile, as thereby the fore shape is made steeper, reducing the overlying weight of water, which weight on the toe of a dam or weir is decidedly detrimental to stability. When the lower part of the face of the wall has a flat inclination, the tail water will affect the value of S, necessitating a greater base width. Consequently, when designing the lower part of a dam below the pressure limit, the presence of tail water, if any, should not be neglected, but should properly be included in the calculation for the lower base width.

(**55**) Figs. 25 and 26 are examples of two low weir walls under similar conditions of water pressure. Fig. 25 is a vertically backed wall. Fig. 26 is of equiangular section. The bed slope of the river, or S, is assumed at 1 in 10,000, and by reference to Table II., par. 49, when $D = 15$ feet, which is the value given to H in the figure, the reciprocal depth over crest, namely, *d*, will be 6·5 (with $n = \cdot 0250$). The resultants are worked out on the supposition that the weir wall is on a porous sand foundation, and is thus

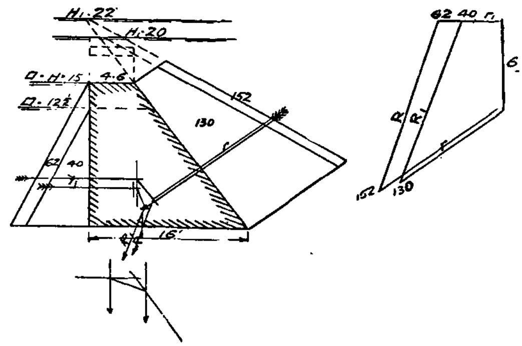


FIG. 27.

subject to loss of weight by flotation. This is the almost invariable status of dwarf diversion weirs. The incidence of the resultants on their bases go to show that the canting forward of the profile makes no practical difference

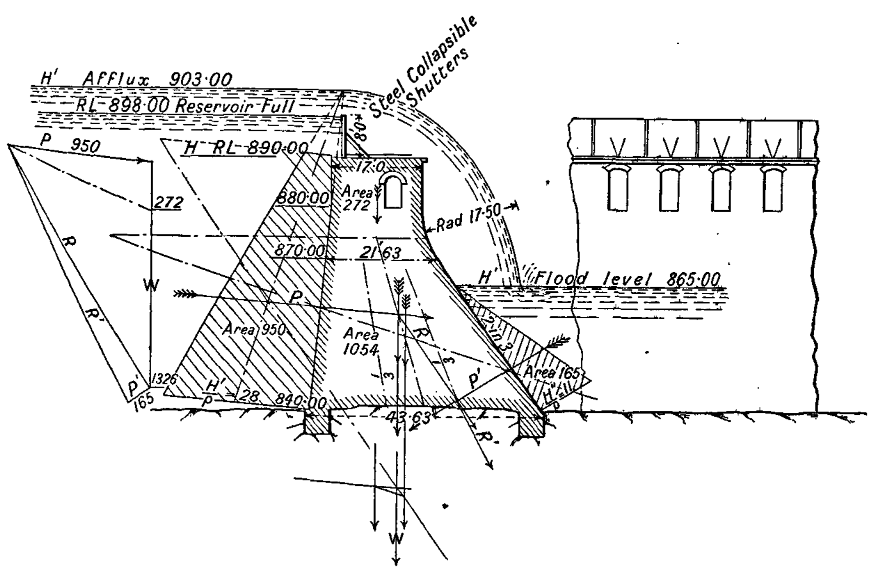


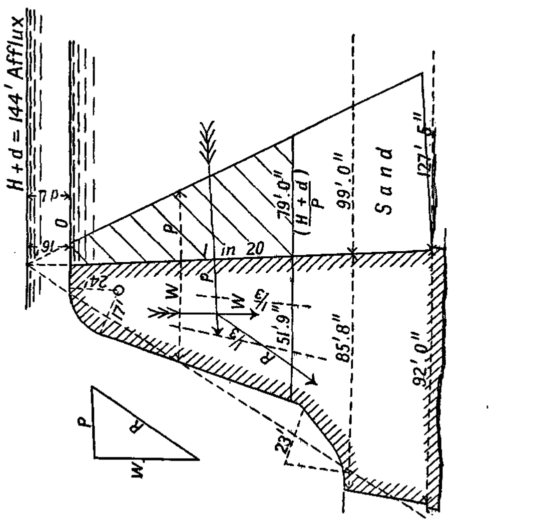
FIG. 28.—Dhukwa Weir.

in the stability of the wall, but the reversal does, as shown in Fig. 27, which, however, is under somewhat different conditions. The reduction of weight due to immersion is equivalent to multiplying the area by $\frac{\rho - 1}{\rho}$ or by $\frac{5}{9}$.

Some examples of high weirs will now be given.

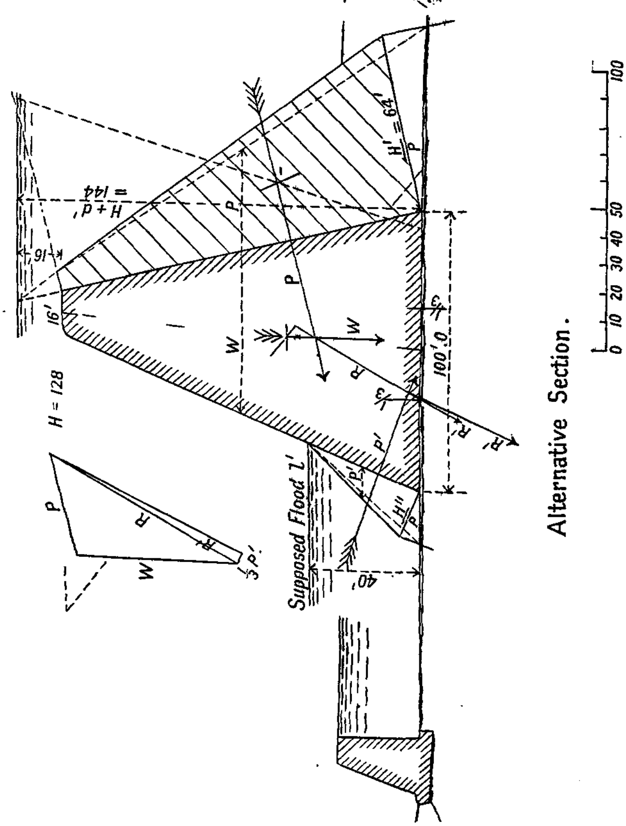
(56) The profile of the Dhukwa Weir (Fig. 28) is that of a recently projected work to be thrown across the Betwa River in Bundelkund, Upper

India, with the object of forming a second reservoir to supplement the existing one at Parichha, the head of the Betwa Canal. It will be further described with reference to the project and to the weir shutters in Chap. VI.



LA GRANGE WEIR.

FIG. 29.



Alternative Section.

FIG. 30.

In this place it will be sufficient to examine the section only. The tail water at full flood does not rise above half the height of the weir; consequently, as noted in par. 47, state (3) of maximum flood will give the correct value of d to be used in the formula (13), $b = \frac{H+d}{\sqrt{\rho}}$. Here H = 50 feet and $H + d$ = 63 feet. The base width should then be $\frac{2}{3} \times 63$, or 42 feet, which it scales almost exactly. The crest width is necessarily very wide in order to accommodate the high collapsible weir shutters and their gear. According to formula (14) par. 42, the ordinary width would be $\sqrt{H} + \sqrt{d}$, or $\sqrt{50} + \sqrt{13} = 7 + 3{\cdot}6$, or 11 feet.

In a case like this the flood velocity of approach, which must be considerable, should be ascertained and the height sufficient to produce this velocity, which is given in Table IV., added to

$H + d$, making the formula for base width $\frac{H + d + h}{\sqrt{\rho}}$, and for crest width $\sqrt{H} + \sqrt{d + h}$.

If the velocity of approach were 10 feet a second, a not unlikely value in such a river, h would equal 1·5 feet. This would, however, only slightly affect the values of b and of c.

* (57) The La Grange Weir (Fig. 29) is an example of a very high overfall weir, situated in California. The velocity of the 16 feet deep overfall is estimated to be 13 feet per second. Assuming that in the reciprocal channel below to be 6 feet per second, the ratio will be 2·1 : 1; therefore with $d = 16$ feet, D will $= 16 \times 2{\cdot}1 = 34$ feet, which, if measured above the given low water level, will hardly reach half-way up the weir face. Consequently high flood state, No. 3 of par. 47, will apply. Some exception can well be taken to the section. At a depth of 79 feet below the crest, *i.e.*, with a value of $H + d$ of 95 feet, the base width is only 51·9, whereas it should be $\frac{2}{3} \times 95 = 64$ feet; consequently, as the force diagram also corroborates, the incidence of the resultant falls well outside the middle third of the base at this point. The corrected outline is shown dotted on. An alternative profile is shown in Fig. 30, alongside. The base is increased to 100 feet, a greater width than provided for by the formula, which is 91 feet, to allow of the profile being tilted forward, with the object of causing the falling water to clear the face of the dam. The crest is reduced to $\sqrt{H} + \sqrt{d}$. A subsidiary weir and water cushion is also provided. American practice still favours the more or less pronounced ogee form of overfall, although in India this has long been discarded for the clear overfall. The alternative section is not so economical as the dotted outline in Fig. 29, it is simply an example.

Experience has proved that a vertical fall on to a good cushion of water has far less erosive action on the bed and banks downstream than the great horizontal scour inevitable with a sloping overfall. However, where floating ice is encountered, the ogee or sloping rollway may be a necessary evil.

(58) The Granite Reef Weir (Fig. 31) is another example of an American weir. It is founded partly on rock and partly on boulders and sand; the section on the latter porous substratum is given in Chap. VI., as belonging to that class of river weir. The superstructure above the floor level is the same throughout, but the foundations on rock are remarkable as being founded not on the rock itself, but on a super-imposed cushion of sand. Reinforced concrete piers, spaced 20 feet apart, were built on the bedrock to a certain height, so as to clear all inequalities; these were connected by thin reinforced concrete side-walls; the series of boxes formed were then filled up level with sand, and the dam built thereon. This work was only completed in 1908, and as Schuyler's "Reservoirs," from which valuable work this account is taken, supplies no dimensions, the portion of the profile below the floor is conjectural. This construction appears to be a bold and commendable novelty. Sand in a confined space is incompressible, and there is no reason

* The profile has been since found to be incorrectly drawn; 51·9 should read 57·9, which width is nearly sufficient.

why it should not be used in like situations. A suggested improvement would be to abandon the piers and form the substructure of two long outer walls only, braced together with rods or old rails encased in concrete.

The section of the weir is exceptionally strong, its base being quite equal to its height; probably the value of d is very high.

It is an unfortunate fact that in most reference books containing record plans of irrigation works, vital statistics, such as flood levels, discharges, and other matters, are often wanting. The only work immaculate in this respect is the "Madras Irrigation Manual." The work of analysing designs is, therefore, often like making bricks without straw, as not infrequently the most important statistics are entirely unrecorded. The too persistent inroad of the photograph is also apt to vitiate the value of reference works. The photograph, useful as it may be in presenting an idea of the appearance of a work to the lay mind, can never take the place of a properly dimensioned drawing with the professional student, and the tendency is now to substitute one for the other.

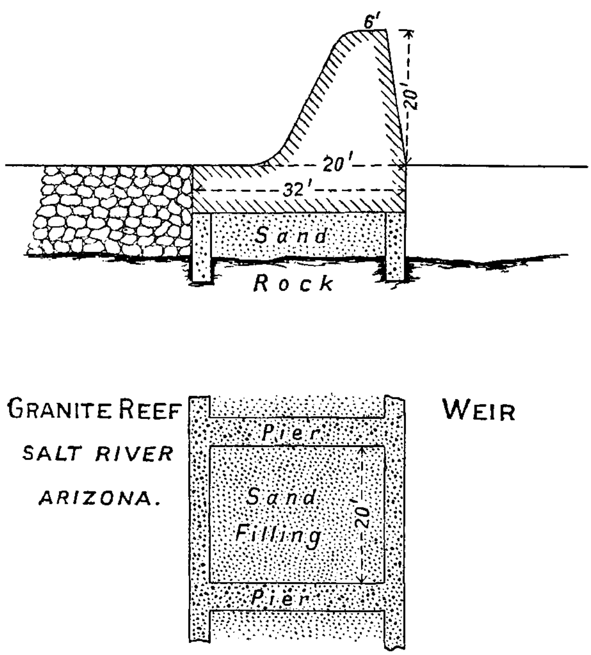


FIG. 31.

(**59**) Another typical high American gravity weir is illustrated in Fig. 32 of the Mariquina Dam, situated in the Philippine Islands. The height is 60 feet; foundation, rock. The usual ogee curve is adopted. If the value of d be assumed at 12 feet, according to formula (14) the crest width would be $\sqrt{60} + \sqrt{12} = 11{\cdot}2$ feet; it is actually 14 feet, and the base width which should be 48 feet is actually, if the toe curve be excluded, about 52 feet. On the whole the section is about 2 to $2\frac{1}{2}$ feet too thick throughout. The projection at the heel appears uncalled for. W scales 31 feet, and its value, multiplying by H and by $w\rho$, is $31 \times 60 \times \frac{1}{16} = 116$ tons. Dividing by b, or 52 feet, the mean stress will be $\frac{116}{52}$ or a little over 2 tons, so that there is no necessity for widening the base. This small calculation is merely inserted in order to familiarise the reader with the analysis of designs. This work is partly a dam and partly a weir. The crest width of the dam part, 17 feet, is out of proportion, but the probability is that a roadway is carried across with piers and arches over the weir portion. Such a work could very well be erected with perfect safety on a boulder and gravel formation if a floor of suitable dimensions were provided, but the arch buttress type illustrated in the next chapter would be about 40 per cent. cheaper.

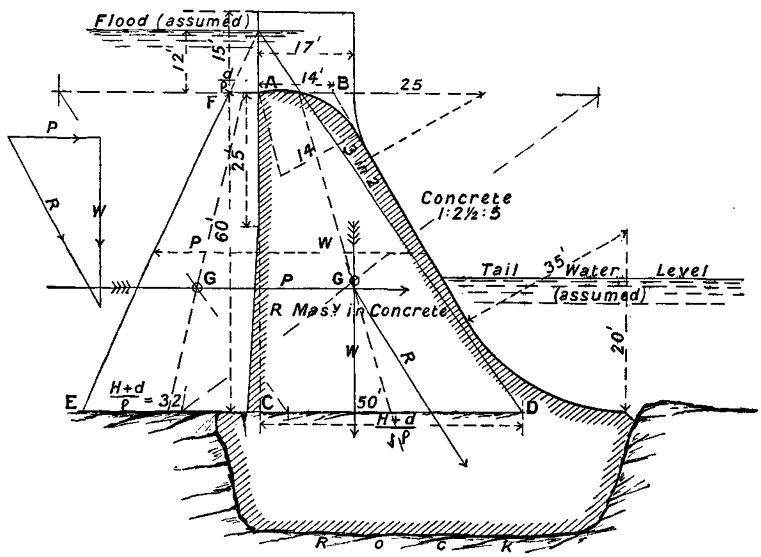


FIG. 32.—Mariquina Weir.

(60) The Castlewood weir (Fig. 33) is a high weir of peculiar construction, being formed of dry rubble stonework enclosed in an outer casing of cement masonry. There appears to be no reason why a construction of this kind

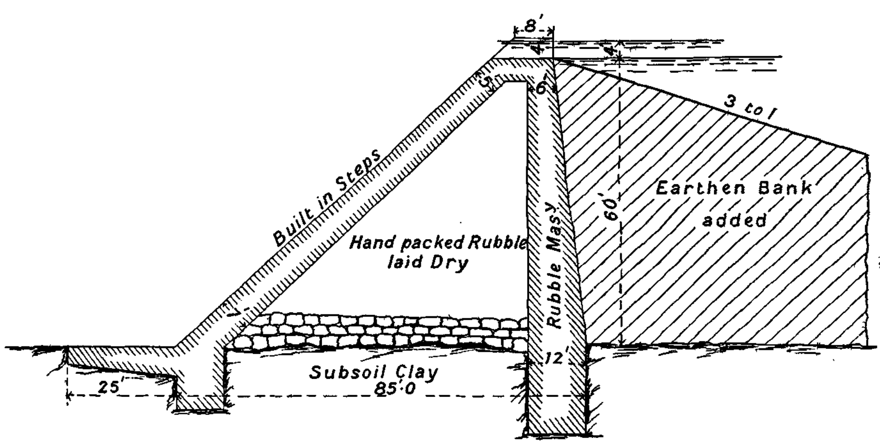


FIG. 33.—Castlewood Weir.

should not answer, but the fact remains that it did not. The structure developed serious leaks, probably owing to bad connections with the flanks of the canyon, and had to be reconstructed. This was effected by adding an earthen bank with a 3 to 1 slope on to the rear of the work, and lengthening the outlet pipe accordingly.

(**61**) An example of a weir with a very high flood passing over is the Folsom weir on the American River (Figs. 34 and 34a). The flood rises to R. L. 225, *i.e.*, 32 feet above the lower crest—this is according to "Irrigation Engineering" (Wilson).

The base width at sill of sluice will be, by formula (13) (par. 42), $\frac{2}{3} \times 81 = 54$ feet, which is just about what it scales. The crest width by formula (14) should be $\sqrt{70} + \sqrt{24} = 13\frac{1}{4}$ feet, but the minimum width $\sqrt{d}$ will be $\frac{2}{3} \times 30 = 20$ feet (par. 42). Its exceptional width of 24 feet is due to the hydraulic collapsible shutters, which form so excellent a feature in the design.

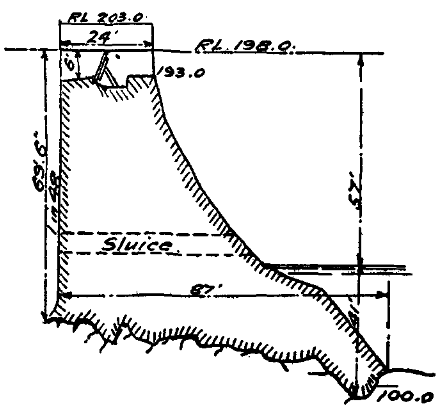
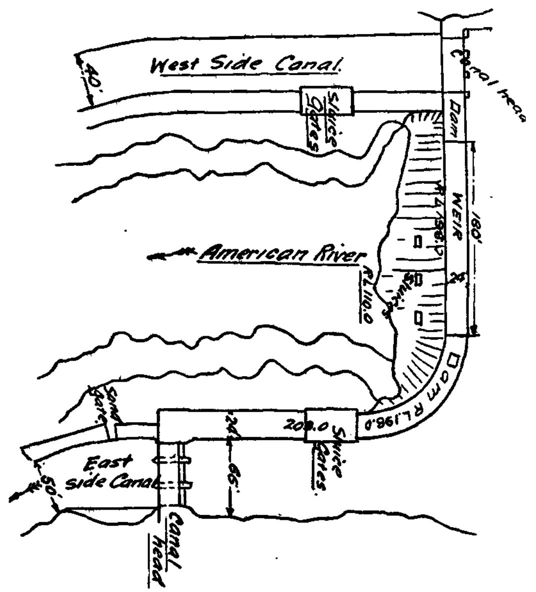


FIG. 34.



(34a.)

FIGS. 34 and 34a.—Folsom Weir.

(**62**) The highest weir in the world is that belonging to the Coolgardie water supply reservoir, near Fremantle, in Western Australia. The profile is given in Fig. 35. On account of its immense height, this structure is to all intents and purposes a dam. The tail water will have very little influence, except detrimentally, as regards load on the foundations.

The limiting stress λ is 8 tons. The base of low weir and the elementary triangular profile have been dotted on the section. It is considered that the rear projection of the heel is excessive; a more nearly vertical back would be statically better. If the crest were thrown forward to reduce the fore slope and allow the water to fall clear of the face, the sectional area would have to be somewhat increased.

(**63**) Canal falls and tank escape weirs and works of this description subjected to a fixed and moderate depth of water, if built in solid clay cutting well backed with puddle, are not considered as subject to full hydrostatic pressure as would be a wall without such solid backing; consequently the base width can be reduced to a mean between that of a retaining wall for earth and one for water. This is effected by making the base width not $\frac{H + d}{\sqrt{\rho}}$, as in formula (13), but

$$b = \frac{H + d}{\rho} \qquad (18);$$

with $\rho = 2\frac{1}{4}$ this is equivalent to $b = \frac{4}{9}(H + d)$, and with $\rho = 2$ this is equivalent to $b = \frac{H + d}{2}$.

This is termed the "Hybrid" section, and is further noticed in Chap. IX.

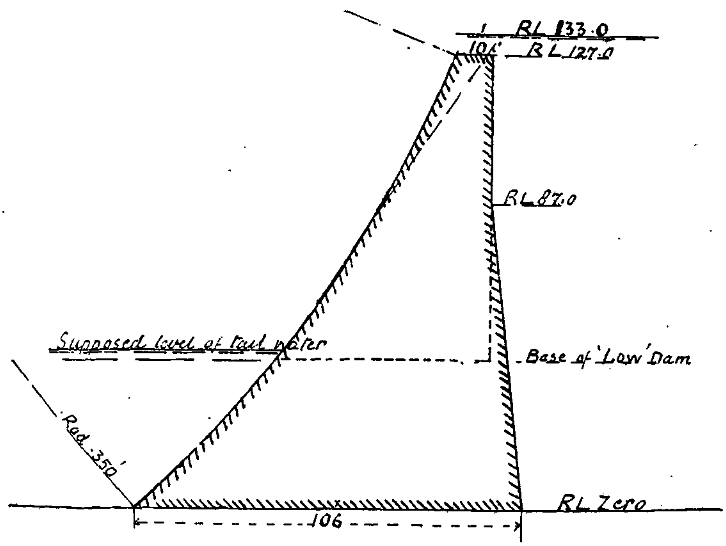


FIG. 35.—Coolgardie Reservoir Weir.

NOTE.—Formula (A) can be written $e = \frac{b}{3}\left(2 - \frac{\lambda b}{R}\right)$ because in this case S_1 is the mean pressure on the base, or $\frac{R}{b}$, not $\frac{N_1}{b}$; the same expression is $e = \frac{b}{3}\left(2 - \frac{\lambda b}{\sqrt{N^2 + P^2}}\right)$.

The base b is formed of three divisions: (1) e from the toe to the incidence of R; (2) f from the incidence of R to that of N; and (3) that from N to the heel of the base. Now $f = \frac{PH}{3N}$ obtained by taking the moments of N and of P about the incidence of R, and (3) is postulated to be $\frac{b}{3}$ whence $b = \frac{b}{3}\left(2 - \frac{\lambda b}{\sqrt{N^2 + P^2}}\right) \frac{PH}{3N} + \frac{b}{3}$. If the value of P, the horizontal water pressure, in terms of w and of H be substituted, the expression will have the form given in formula (B) par. 18.

With regard to formula (C). The vertical forces whose moments about the inner third division of the base is required are (1) the weight of water overlying the projection x, which can easily be estimated; (2) the vertical N of the overlying mass, a known quantity; (3) the weight of the new lamina, which having b and b_1 can also be closely estimated. The distances of these several forces from the incidence of N^1 at the inner third point are as follows: (1) $\frac{b_1}{3} - \frac{x}{2}$, (2) $\frac{b_1 - b}{3} - x$, (3) $\frac{3b - b_1 + 6x}{12}$. By separately estimating the weights 1, 2, and 3 the equation is very much simplified, being reduced to a simple equation, and x is found with ease. The one condition is that the incidence of N^1 must be exactly at the inner third point of the new base. This proviso insures that both W, reservoir empty, and R, fall within the middle third. The length given to b_1 also insures that $S = \lambda$ the limit stress.

CHAPTER III

SECTION I.—ARCHED DAMS (TYPE B)

(1) IN this type, designated as B, the whole dam, being arched on plan, is supposed to be in the statical condition of an arch under pressure. As, however, the base is immovably fixed to the foundations by the frictional resistance due to the weight of the structure, the lowest portion of the dam cannot possess full freedom of motion nor elasticity, and consequently must act more or less as a gravity dam subject to oblique pressure.

However this may be, experience has conclusively proved that if the profile be designed on the supposition that the whole is an elastic arch, this conflict of stresses near the base can be neglected by the practical man. The probability is that both actions take place, the true arch action, at the crest, gradually merging into transverse stress near the base ; the general result being that the safety of the dam is enhanced by this combination of tangential and vertical stresses on two planes.

(2) In this type of structure, the weight of the arch itself is conveyed to the base producing stress on a horizontal plane, while the water pressure normal to the extrados, radial in direction, is transmitted through the arch rings to the abutments. The pressure is therefore distributed along the whole line of contact of the dam with the sides as well as the ground. In a gravity dam on the other hand, the whole pressure is concentrated on the horizontal base.

(3) The average unit stress developed by the water pressure is expressed by the formula

$$s_1 = \frac{RHw}{b} \therefore b = \frac{RHw}{s_1} \quad (1), \qquad \text{"Short" formula.}$$

in which R is the radius of the extrados, sometimes measured to the centre of the crest, H the depth of the lamina, b its width, and w the unit weight of water. In this formula, ρ, the specific gravity of the material in the arch, forms no factor. This simple formula answers well for all arched dams of moderate base width. When, however, the base width is considerable, as, say, in the case of the Pathfinder Dam, the use of a longer formula giving the maximum stress (s) is to be preferred. This is derived from the same principle affecting the relations of s and s_1, or the maximum and average stresses, already referred to in Chap. II. on Gravity Dams, pars. (7) and (17). The expression is

$$\lambda \text{ or } s = s_1 \frac{2R}{R+r} = \frac{RHw}{b} \times \frac{2R}{R+r} \qquad (2),$$

or in terms of R and b

$$\lambda \text{ or } s = \frac{2\,Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)} \quad (3), \qquad \text{“Long” formula.}$$

also $$b = R\left(1 - \sqrt{1 - \frac{2Hw}{\lambda}}\right) \quad (4). \quad ,, \quad ,,$$

(4) In a manner similar to type A, the theoretical profile suitable for an arched dam is a triangle having its apex at the extreme water level, its base width being dependent on the prescribed limiting pressure. Successful examples have proved that a very high value for s (or λ), the maximum stress, can be adopted with safety. If it were not for this the profitable use of arched dams would be restricted within the narrow limits of a short admissible radius, as with a low limit pressure the section would equal that of a gravity dam.

(5) The water pressure on an arch acts normally to the surface of its back and is radial in direction; consequently the true line of pressure in the arch ring corresponds with the curvature of the arch and has no tendency to depart from this condition. There is therefore no point of rupture as in the case in a horizontal circular arch subjected to vertical, not radial pressure. This property conduces largely to the stability of an arch under liquid pressure. This condition is not strictly applicable in its entirety to the case of a segment of a circle held rigidly between abutments, as the arch is then partly in the position of a beam. The complication of stress involved is, however, too abstruse for practical consideration.

(6) As we have already seen, the correct profile of the arched dam is a triangle modified into a trapezoid with a narrow crest, and with regard to arch stresses, the most favourable outline is that with the back or extrados vertical. The reason for this is that the vertical stress due to the weight of the arch, although it acts on a different plane to the tangential stresses in the arch ring, still has a definable influence on the maximum induced stress in the arch ring. This vertical pressure produces a transverse expansion which may be expressed as $W \times E \times m$, in which E is the coefficient of elasticity of the material and m that of transverse dilatation. This tends, when the extrados is vertical, to diminish the maximum unit stress in the section, whereas when the intrados is vertical and the back inclined, the modification of the distribution of pressure is unfavourable, the maximum stress being augmented in similar degree. When the triangular profile is equiangular an intermediate or neutral condition exists. A profile with vertical extrados should, therefore, be adopted whenever practicable.

(7) In very high dams, however, the pressure on the horizontal plane of the base, due to the weight of the structure, becomes so great as to even exceed that in the arch ring; consequently it is necessary to adopt an equiangular profile in order to bring the centre of pressure at or near to the centre of the base, so as to reduce the ratio of maximum pressure, and average pressure, to a

minimum. As stated in the previous chapter, par. (7), when a vertical through the CG of the profile passes through the centre of the base, the maximum pressure equals the average or $= \frac{W}{b}$.

(8) When the back of an arched dam is inclined, the weight of the water superimposed is carried by the arch, and does not reach the base, but the abutments. The unit pressure induced in the arch ring is clearly the same whatever be its inclination to the vertical, for this reason, that the liquid pressure never exceeds Hw (H being any vertical depth), and is radial and normal to the back in direction. The area of water pressure acting on an inclined, is naturally greater than that acting on a vertical surface; but the length of the surface in question is greater in similar degree, so that the unit stress in the arch ring remains unaltered; the total stress conveyed to the buttresses or abutments, is, however, naturally greater with an inclined than

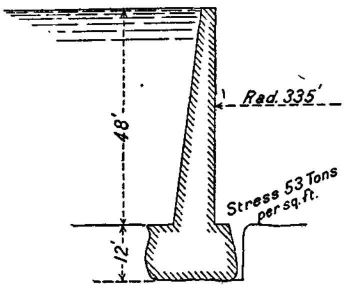


FIG. 1.—Bear Valley Dam.

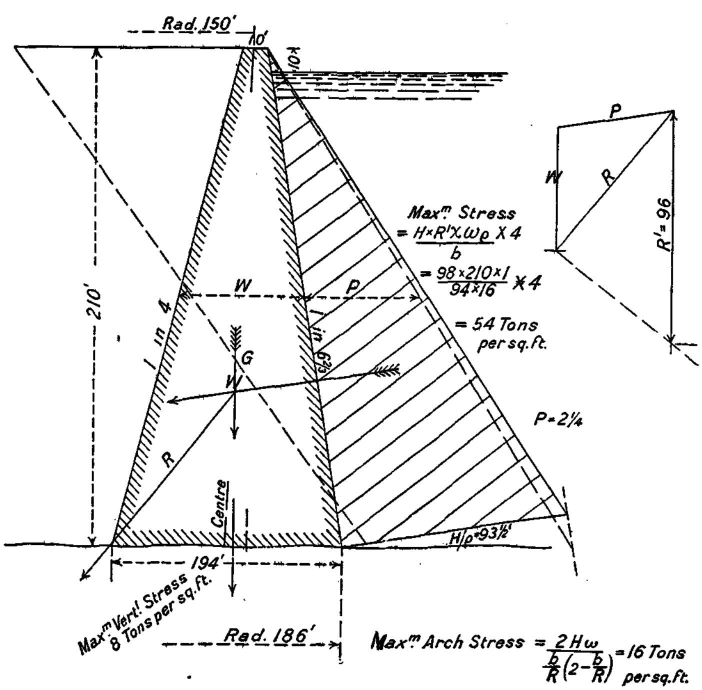


FIG. 2.—Pathfinder Dam.

with a vertically backed profile. This does not affect the problem we are now considering, but when the dam consists of a series of vertical arches with intervening buttresses as in type C this question is one that demands attention. Some examples of arched dams will now be exhibited.

Bear Valley Dam (Fig. 1).

(9) This small work is the most remarkable example of arched dam in existence, and as a precedent is most valuable; the pressure induced in the arch ring just above the widening works out as 53 tons per square foot. The section would be better reversed. This proves the capability of this type of dam to stand a pressure previously deemed impossible.

Pathfinder Dam (Fig. 2).

(10) This immense work has a radius of 150 feet measured to the centre of the crest; that, however, of the extrados at the base of the profile is 186 feet, and this quantity has to be used for the value of R in the long formula No. 9. The unit arch stress then works out to 16 tons. The average vertical stress on the base is 7 tons and the maximum about 8. If the profiles were altered to one with a vertical back, the arch stress would be less, as the radius would be reduced to 155 feet, but that on the base would be nearly double the average, viz., 14 tons. It is deemed that the existing outline is the better of the two. The stresses involved, considering the dam as a gravity dam only, have been graphically shown.

(11) The Shoshone Dam (Fig. 3) is designed on identical lines to the last example. It has the distinction of being the highest dam in the world. The value of s (vertical stress) is 14, and that in the arch 17 tons respectively. If the profile were altered to a vertically backed trapezoid, as shown dotted on the diagram, and the base extended at the toe, the superficial area would be increased from 21,000 to 22,000 square feet. The average vertical pressure would then be 10 tons and the maximum 19·6 tons, while the arch pressure will be 14 tons, the reduction in arch stress below the previous being due to the shortening of the radius at the base. This proves the soundness of the inclined backed profile which has been adopted.

The existing section has one serious defect, and that is the suspension of both batters, rear and fore, from the level where the sandy water-bearing bed of the stream is entered, thus reducing the base width. Considered strictly as an arch there is no increase of pressure from this level down to the base, and so further widening of the section is not theoretically required. The comparative effect of the reverse pressure is, however, so small with regard to volume, as will be seen from the hatched pressure area marked on the section, that the petty economy thus involved must substantially impair the stability of the dam. Overturning moment, as we have already seen, does certainly act at the base of an arched dam, though to what exact extent is not ascertainable; hence it is culpable folly to reduce the leverage of resistance by shortening the base at the most critical point of all. If the fore slope were continued down to the solid foundation the increase would be only $2\frac{1}{2}$ per cent. of the whole sectional area.

In both these dams a crest width of 10 feet has been adopted. The crest width of arched dams can be safely made much lighter than gravity dams and a rule of

$$c = \frac{\sqrt{H}}{2} \qquad (5)$$

would seem to answer the purpose.

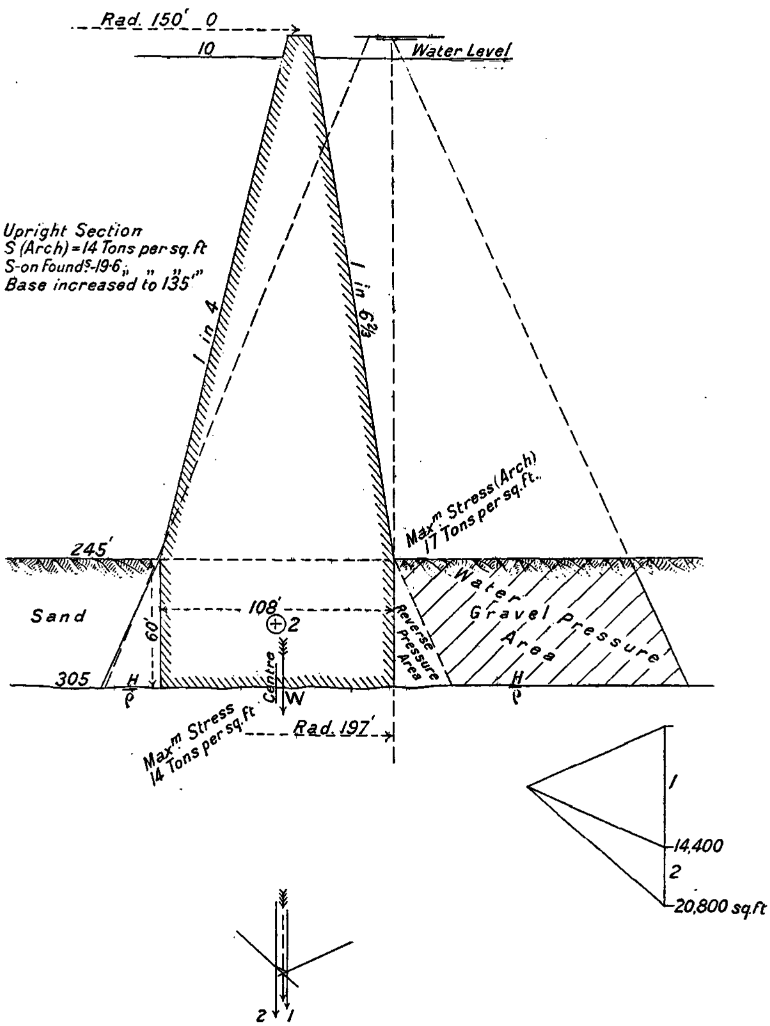


FIG. 3.—Shoshone Dam.

(12) The profile of the Sweetwater Dam in California is given in Fig. 4. This work, during a great flood was overtopped by 5 feet of water and subsequently the crest was raised by a thin wall up stream to prevent its recurrence. The maximum unit stress developed works out by the long formula 3 (par. 3) as follows:—The radius of the extrados at the base being 232

feet, $\frac{b}{R} = \frac{46}{232} = \cdot 2$. The expression then becomes by the "Long" formula

$$s = \frac{2 \times 95 \times \frac{1}{36}}{\cdot 2\,(2 - \cdot 2)} = 14\cdot 7 \text{ tons.}$$

The back is canted forward with the object of equalising the stress on the base.

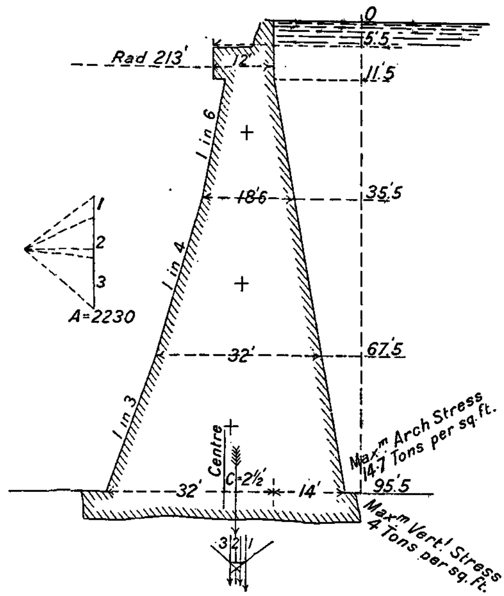


FIG. 4.—Sweetwater Dam.

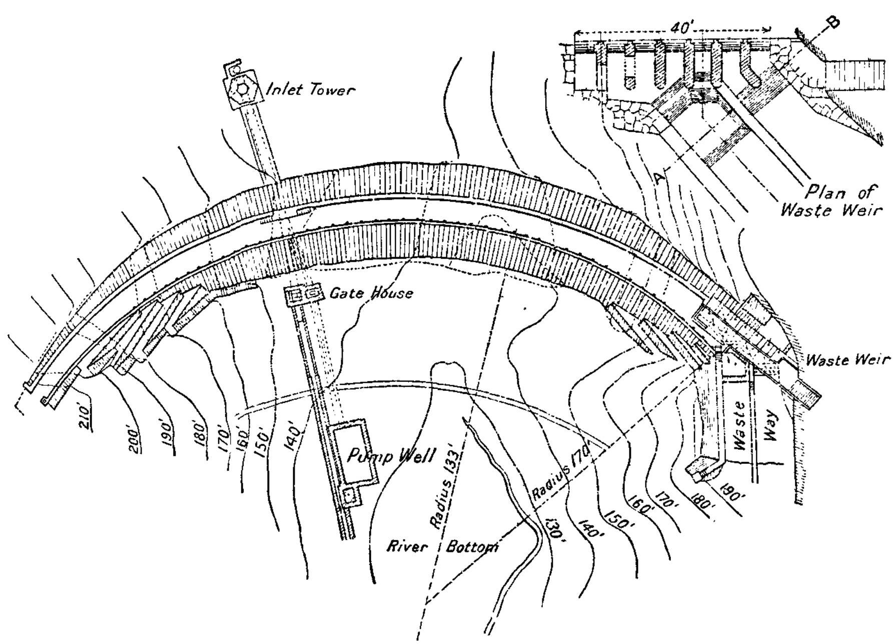


FIG. 4a.—Plan of Sweetwater Dam.

With a comparatively low height, as in this case, a slighter rear batter or a perfectly vertical back would answer better—for, as we have seen, the inclined

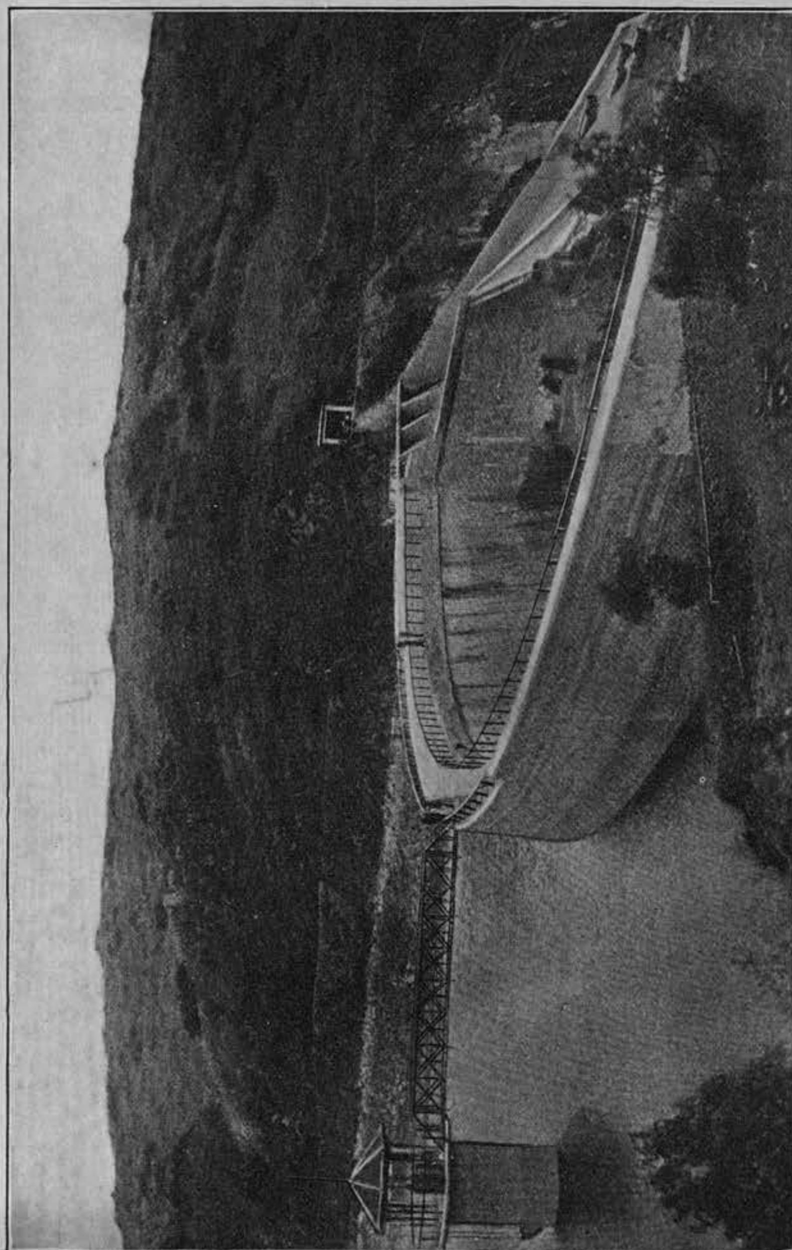


FIG. 4b.—Photograph of Sweetwater Dam.

back is not of advantage to the arch, and throwing as it does a weight of water on the dam which otherwise would be avoided. The face should undoubtedly be in one straight batter, not curved, which is hardly suitable even for gravity dams. The top thickness seems excessive for a purely arched dam as this is. The Sweetwater is a comparatively old work. The maximum unit

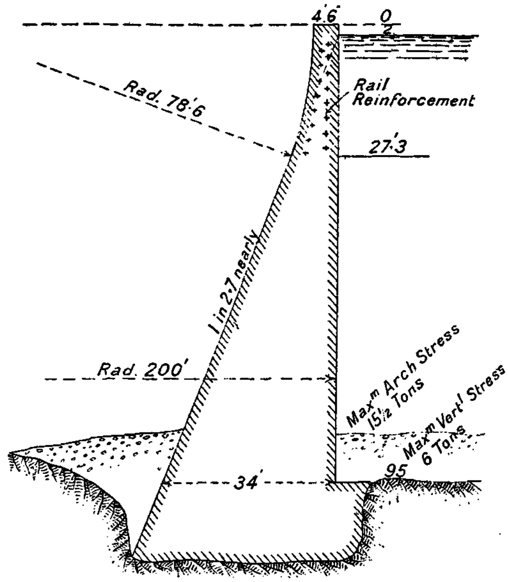


FIG. 5.—Barossa Dam.

pressure due to the vertical weight is obtained by using formula 2 (par. 7), Chap. II., W being $= A \times wp$, or, in figures, $2{,}230 \times \frac{1}{16} = 139$ tons. And c measuring 2·5 feet, the stress works out as below:

$$\frac{W}{b}\left(1 + \frac{6\,c}{b}\right) = \frac{139}{46}\left(1 + \frac{15}{46}\right) = 4 \text{ tons nearly.}$$

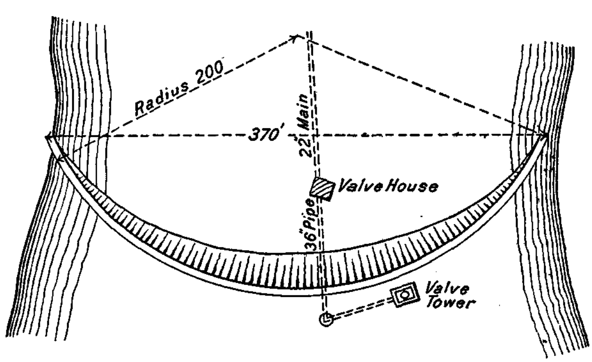


FIG. 5a.—Site Plan.

This is low unit stress, while that in the arch is high. With a vertical back the former would be increased, which it can well afford, and the latter be reduced to 14 tons owing to the shortening of the radius of the extrados—besides being otherwise in a better statical position (*vide* par. 6). For the plan photographic view of this dam, given as Fig. 4b, we are indebted to Schuyler's "Reservoirs."

(13) The Barossa Dam (Fig. 5) is an Australian work, and although of quite moderate dimensions is a model of good and bold design.

The back is vertical and the fore batter is almost 1 in 2·7. The outline is not trapezoidal but pentagonal, viz., a square crest imposed on a triangle—the face joined with the hypothenuse of the latter by a curve. The crest is slender, being only $4\frac{1}{2}$ feet wide, but is strengthened by rows of 40-lbs. iron rails, fished together, built into the concrete. The maximum arch stress works out to $15\frac{1}{2}$ tons, the corresponding vertical stress on base to 6 tons. Fig. 5a is a site plan of the work, in which the broken lines on the flanks represent earthen slopes, not water.

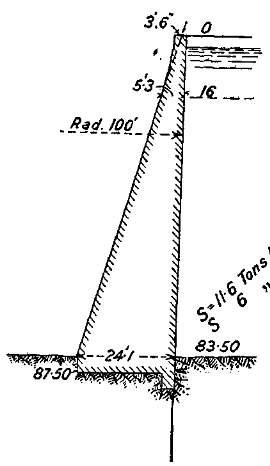


FIG. 6.—Lithgow Dam.

(14) Another example very similar to the last is the Lithgow Dam (No. 2) (Fig. 6). The arch stress in this works out by the short formula to $11\frac{1}{2}$ tons; the radius is only 100 feet.

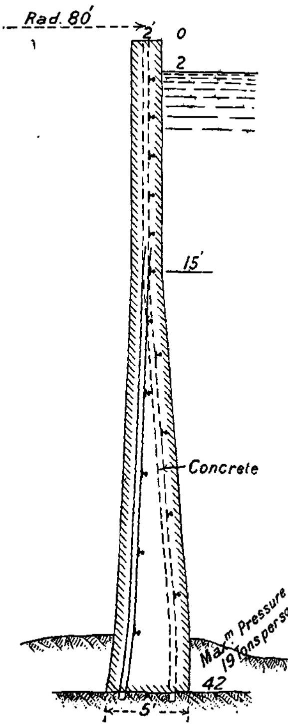


FIG. 7.—Barren Jack Dam.

Arched dams abut either on the solid rock banks of a canyon or else on the end of a gravity dam. In cases where a narrow deep central channel occurs in a river, this portion can advantageously be closed by an arched dam, while the flanks on which the arch abuts can be gravity dams aligned tangential to the arch at each end. The dam will thus consist of a central arch with two inclined straight continuations. The plan of the Roosevelt Dam (par. 38, Chap. II.) will give a good idea of this class of work.

(15) Fig. 7 is the profile of a temporary reinforced arched dam for domestic water supply at Barren Jack, Australia. The reinforcement consists of iron rails. The arch pressure at the base works out to 19 tons nearly. Reinforcement of permanent dams down to the base is not desirable as the metal may corrode in time and cause failure, although the possibility of such a contingency is often stoutly denied.

SECTION II.—ARCH BUTTRESS DAMS (TYPE C).

(16) In this type, the dam is composed of a series of vertical or inclined arches divided into spans by buttress piers. The arrangement is in fact identical with that of a masonry arched bridge, if the latter could be considered turned over on one side.

For wide rivers and moderate depths this system will in time undoubtedly supplant the heavier gravity type A, and, indeed, also in many cases the arched type B treated of in the first section of this chapter, the economy in material being very great. The following very conservative estimate has been made of the economy in cost of this type in comparison with the gravity type of dam, under the same condition of limiting stress:—

At	30	feet in height,	saving	is	31	per	cent.
,,	50	,,	,,	,,	28	,,	,,
,,	100	,,	,,	,,	23	,,	,,
,,	150	,,	,,	,,	12	,,	,,
,,	200	,,	,,	,,	9	,,	,,

In a design at the end of this chapter for a dam of type C, 64 feet in height, the saving in material works outs to 50 per cent., so that by allowing proper high stresses for arches under liquid pressure and by adopting large spans, the actual saving effected will, it is believed, considerably exceed that given above.

(17) The first example will be that of the Mir Alam Tank dam (Fig. 8). This work is situated near Hyderabad, the capital city of the Nizam's dominions in the Deccan, South India.

This remarkable structure is quite unique of its kind and forms a most instructive example of the principles governing the design of this type of dam. It was built over a century ago by some Engineer of "John Company," whose name has not been handed down by fame to posterity. The dam, which is aligned on a wide curve, consists of a series of vertical semi-circular arches of various spans. These arches abut on buttress piers. The spans vary in width from 70 to 147 feet, one of which latter is shown in Fig. 8. The height is 40 feet. Water has been known to overtop the crest.

On account of the inequality of the spans the adoption of the semi-circular form of arch is evidently a most judicious measure, for this reason, that an arch of this form under liquid pressure exerts no lateral thrust at the springing. The water pressure, as already noted in the previous section, is radial in direction, consequently the half arch is balanced and in equilibrium. Whatever thrust is exerted is not in the direction of the axis of the arch, but on that of the buttress piers, *i.e.*, at right angles to the springing line. On the other hand, if the arch were segmental in outline, as is shown dotted on Fig. 8, the tangential thrust is intermediate between the two axes, and when resolved in both directions one component of the thrust acts along the axis of the dam. This has to be met, either by the abutment, if it is the end span, or else by the corresponding thrust of the adjoining half arch. The other component is carried by the buttress; therefore, if segmental arches are used, in order to avoid inequality of thrust, the spans must be equal.

The whole work is built of brickwork, but the unit stress in the arch ring at the base, using the short formula $s_1 = \frac{RHw}{b}$ (par. 3), works out to over $10\frac{1}{2}$ tons. This dam therefore forms a useful precedent as proving what high a stress can be borne with safety by an arch of such decidedly inferior

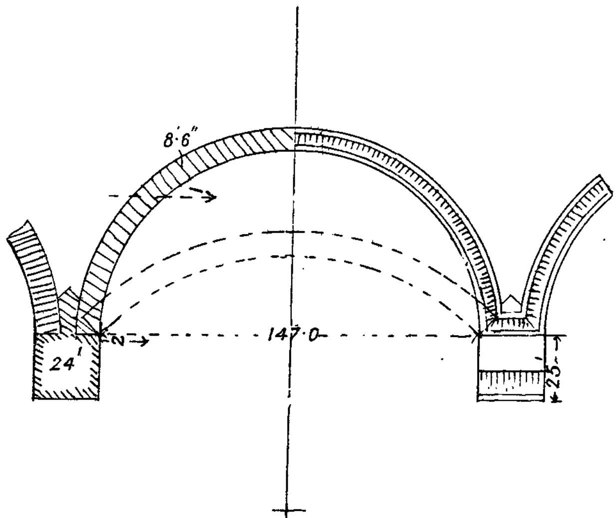
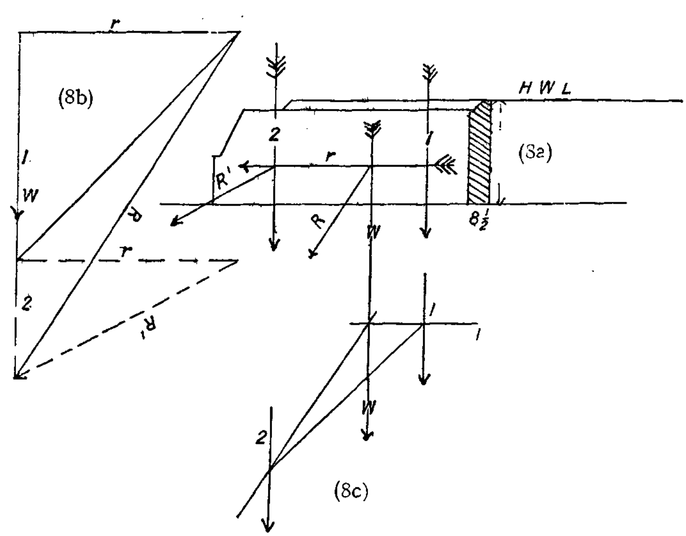


FIG. 8.



FIGS. 8, 8a, 8b, 8c.—Mir Alam Arched Panel Dam.

material as brickwork in ordinary lime mortar. The arch is vertical and $8\frac{1}{2}$ feet thick throughout, and 40 feet high.

The buttress piers in this work are very short, projecting only 25 feet beyond the spring line of the arches, and being altogether only 42 feet long. This length and the corresponding height would clearly be inadequate to withstand the immense horizontal thrust which is equivalent to $\frac{H^2}{2} \times w \times l$ $= \frac{1,600 \times \cdot 173}{2 \times 36} = 3,844$ tons. This is proved in Fig. 8a, where the resultant stress line on this supposition falls just outside the toe of the base and has a dangerously low inclination to the horizontal. It is evident that if the buttress pier slides, or overturns, the arches behind it must follow suit, for which reason the two half arches and the buttress pier cannot be considered as separate entities, but as actually forming one whole, and consequently it follows that the effective length of the base must extend from the toe of the buttress right back to the extrados of the two adjoining arches. At, or a little in rear of the spring line, the base is split up into two forked curved continuations. The weight of these arms, belonging to the adjoining arches, has consequently to be included with that of the buttress proper when the stability of the structure against overturning or sliding is estimated.

(18) In the transverse section (Fig. 8a), the graphical calculations establish the fact that the resultant line intersects the base thus lengthened almost exactly at its centre; the direction of the resultant R is also satisfactory as regards the angle of frictional resistance. The maximum unit pressure in the masonry of the pier is $\frac{R \text{ sec. } \theta \times w\rho}{\text{Area of base}}$ (par. 8, Chap. II.), which works out to 3·4 tons only. In estimating A, or the area of the base, the length scales 107 feet, but the width of the buttress portion is 25 feet, while that of the two branches combined is 17 feet. The greater curved length of the latter, however, if allowed for, would enlarge the area by about $\frac{1}{4}$, which would be equivalent to increasing the width to 21 feet. Therefore 22 feet has been adopted as an average representative width for the whole base. The actual figures are $\frac{14.300 \times 2}{36 \times 107 \times 22} = 3 \cdot 4$ tons. The specific gravity is taken as 2. The quantity 14,300 is the value of N_1, or of $R \text{ sec. } \theta$, not shown in Fig. 8b. The incidence of the resultant actually falls within the two arches, not in the buttress proper.

(19) The design could be much improved by altering the distribution of the material of the section of the arch, by making it trapezoidal instead of rectangular in profile. This is effected by increasing the base and decreasing the crest width. Thus, for the same sectional area, a much stronger profile could be produced, subject also to nearly half the unit stress to which it is liable in its present form. The base could be widened from $8\frac{1}{2}$ to 14 feet, and the crest reduced to 3 feet or $\frac{\sqrt{H}}{2}$ (par. 11). The maximum unit stress would then be lowered from 10·6 to under 6·5 tons.

(20) If the arch were altered on plan from a semicircle to a segment of a circle, the radius would of necessity be increased, and the stress with it; a thicker arch would therefore be required. This would not quite compensate

for the reduced length of arc, but on the other hand, owing to the crown being depressed, the effective base width is reduced and will have to be made good by lengthening the buttress piers. What particular disposition of arch and buttress would be the most economical is a matter which can only be worked out by means of a number of trial designs.

(21) There are not many modern examples of arch buttress dams. The Mir Alam dam has remained resting on its laurels, without a rival, for over 100 years, but the time has now come when this type will be very largely adopted. Fig. 9 is an early example of a segmental panel arch dam. It is of the Belubula dam in New South Wales. The arch crest is 37 feet above base, very nearly the same as in the last example. The arches which are inclined 60° to the horizon are built on a high solid platform which obliterates inequalities in the rock foundation. This platform is 16 to 23 feet high, so that the total height of the dam is over 50 feet. The spans are 16 feet, with buttresses 12 feet wide at the spring line, tapering to a thickness of 5 feet at the toe; they are 40 feet long. The buttress piers, which form quadrants of circles in elevation, diminish in thickness by steps from the base up, these insets corresponding with similar ones in the arch itself. These steps are not shown in the drawing; the arch also is drawn as if in one straight batter. The arches are elliptical in form, and the spandrels are filled up flush with the crown, presenting a flat surface towards the water.

(22) Some of the features of this design are open to objection: Firstly, the filling in of the arch spandrels entirely abrogates the advantage accruing to arches under liquid pressure. The direction of the water pressure in this case is not radial but normal to the rear slope, thus exactly reproducing the statical condition of a horizontal arch. The pressure therefore increases from the crown to the haunches, and is parabolic, not circular, in curvature. The arches should have been circular, not elliptical, with the spandrels left empty to allow of the radial pressure which is so beneficial. Secondly, the stepping in of the intrados of the arch complicates the construction. A plain batter would be easier to build, particularly in concrete. Thirdly, the tapering of the buttress pier towards the toe is quite indefensible; the stress does not decrease but increases towards the toe, being a maximum at the edge of the section. The whole structure is liable to overturning moment which moment is resisted by its base; so that any reduction of area at the toe of the buttress accentuates the unit stress at the most critical point.

(23) The inclination of the axis of the arch to the vertical is generally a desirable, in fact a necessary feature wherever segmental arched panels are used; the weight of water carried is of value in depressing the final resultant line to a suitable angle having regard to frictional resistance to shearing stress. As noted in Section I., the weight of the water overlying the arch does not increase the unit stress in the arch ring. Consequently, any inclination of axis can be adopted without in any way increasing the unit stress due to the water pressure. This is due to the radial and normal direction of the

pressure which causes it to be all taken up by arch stress. When, however, the arch is inclined, a certain proportion of the weight of each lamina in

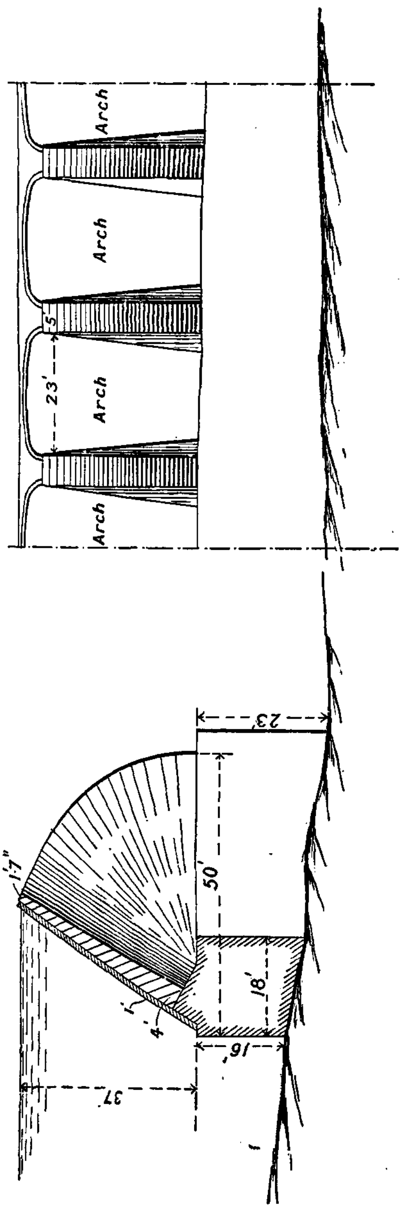
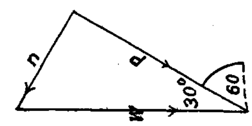
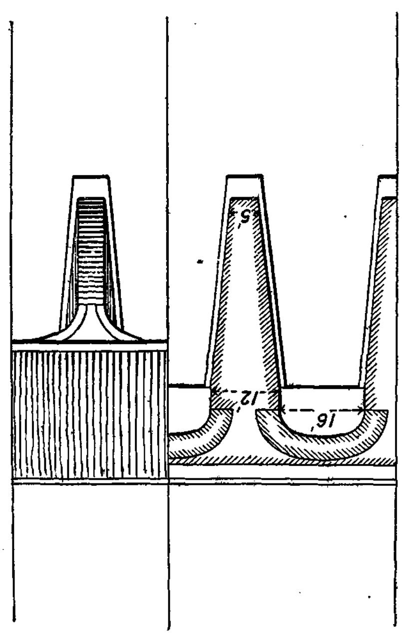


FIG. 9.—Belubula Dam.



(9a.)



which it may be considered as divided is conveyed by arch action to the abutments. In the diagram (Fig. 9a) the vertical load line W represents the weight of one unit or one cubic foot of the arch ring which is equal to $w\rho$. This force is resolved in two directions one p parallel to the axis of the arch, and the other n normal to the former. The force $n = W\ sin.\ \theta$, θ being the inclination of the arch axis to the vertical, $p = W\ cos.\ \theta$. The unit stress developed by the radial force n is similar to that produced by the water pressure which is also radial in direction and is $R_1 n$, but R_1, the radius in this case, is the mean radius, the pressure being internal, not external. The unit stress s will then be

$$s = R_1 w\rho\ sin.\ \theta \qquad (6).$$

When θ is 30° $sin.\ \theta = \frac{1}{2}$; when 45° $= sin.\ \theta = \frac{2}{3}$.

It will easily be understood that this unit stress due to n does not accumulate, but is the same at the first foot depth of the arch as it is at the bottom; the width of the lamina also does not affect it. The component p does, however, accumulate, and the expression $w\rho\ cos.\ \theta$ should be multiplied by the inclined height h lying above the base under consideration. As $h = H\ sec.\ \theta$, the unit compressive stress at the base will be $\frac{Hw\rho \times b^1}{b}$ in which b^1 is the mean width of the arch. If the arch were a rectangle, not a trapezoid, s would $=$ $Hw\rho$ simply.

(24) Fig. 10 is a design for a segmental arch panel dam, or, rather, weir. The height of the crest is 64 feet above base, with 5 feet of water passing over; the apex of the triangle of water pressure will then be 69 feet above the base. The inclination given to the axis, which is coincident with the spring line and the intrados, is 60° with the horizon.

In designing such a work, the following salient points first require consideration :—

Firstly: Width of span. This, it is deemed, should for economical reasons be never less than the height of crest above base. In the Mir Alam dam the span is over three times the depth of water upheld. In the present case it will be made the same, that is 64 feet.

Secondly : Thickness of buttress piers. As with bridge piers the width should be at least sufficient to accommodate the skew backs of the two arches, and a width of 12 feet or about $\frac{1}{5}$ span will effect this.

Thirdly : Radius and versed sine. The radius will be made 40 feet; this allows of a versed sine of $\frac{1}{4}$ span or 16 feet, which is considered a correct proportion to afford a good curvature.

Fourthly : Thickness of arch. This must first be assumed, as its thickness depends on R, the radius of the extrados, as well as on the value assigned to λ, the limiting pressure. This latter will be fixed at $12\frac{1}{2}$ tons, a value by no means excessive for arches under liquid pressure. With a base width of 7 feet, the radius of the extrados will be 47 feet. The base will be considered, not at the extreme depth of 64 feet below crest, but at the point marked D, where a line normal to the inclined intrados at its base cuts the extrados of the arch. H will therefore be 60 feet. The stress due to the water pressure, using the short formula (3) (par. 3) will be

$$\frac{RHw}{b} = \frac{47 \times 60 \times 1}{7 \times 36} = 11{\cdot}2 \text{ tons.}$$

To this must be added that due to the weight of the arch ring from formula

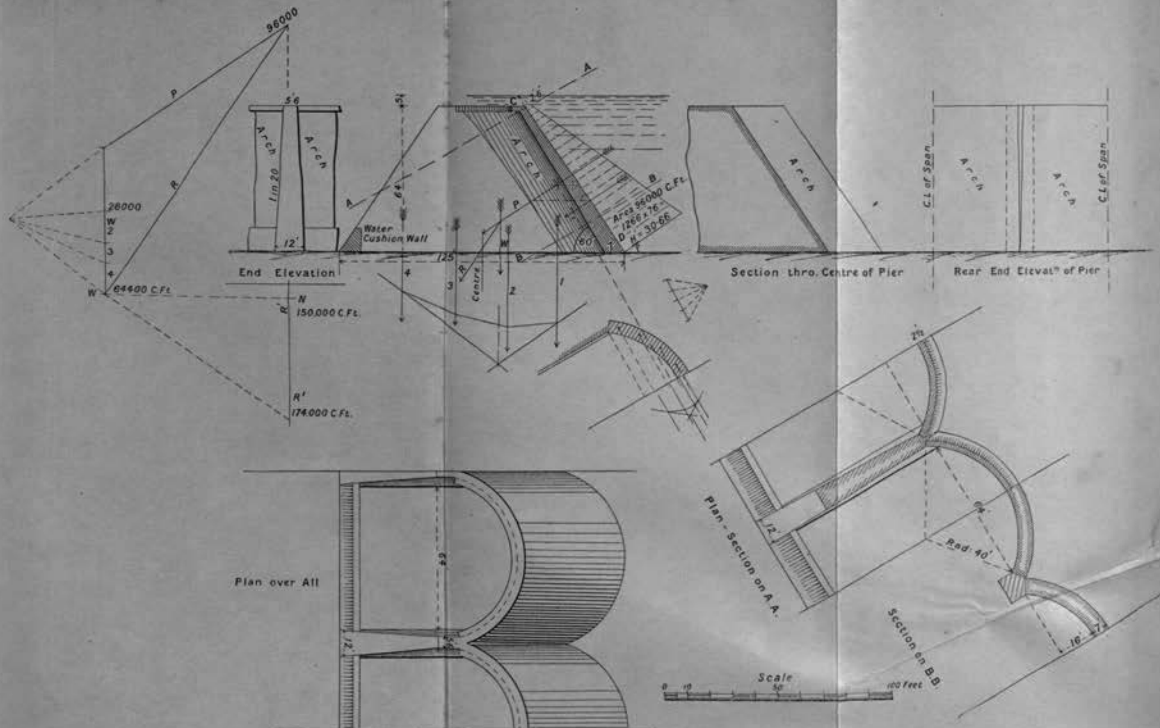


Fig. 10.—Design for Arch Buttress Dam.

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(6) (par. 23). $s = \frac{\text{[illegible]}}{2}$ (the angle θ being 30° and $sin.\ \theta = \frac{1}{2}$), which will be in figures $\frac{43 \cdot 5 \times 1}{2 \times 16} = 1 \cdot 36$ tons, the total stress being a trifle over $12\frac{1}{2}$ tons. The 7 feet base width will then be adopted. The depth of water producing this pressure is taken as 60 not as 65 feet, which is $H + d$, the reason being that the reverse pressure due to the tail water, which must at least be level with the water cushion bar wall, will reduce the effective depth to 60 feet.

(25) The reverse pressure has a great influence in the case of calculations based on the hydrostatic pressure alone, the head being the difference of levels—above and below; whereas, where overturning moment is concerned, the balance moment (*vide* formula (16), par. 47, Chap. II.) is $M = \frac{H^2}{6}(H + 3d) - \frac{h^3}{d}$ or taking the base of both triangles of pressure at the level of D, $M = \frac{3{,}600}{6}(60 + 15) - \frac{125}{6} = 45{,}000 - 21 = 44{,}979$ feet. The relation of the reverse to the up stream moment is then : 1 : 2,143 whereas the reverse pressure which affects the thickness of the arch is as 5 : 65 or : : 1 : 13.

This question of the great divergence between simple hydrostatic pressure and overturning moment produced by the same head or difference of levels is one often lost sight of. Works are stated to be subject to a certain head of water pressure which, unless the depths up stream and down stream are known, can give no correct impression of the effect produced. It would do so in the case of a floor on a porous foundation exposed to upward hydrostatic pressure, but not so when the horizontal pressure of water acting against a surface has to be considered.

(26) The crest width of the arch according to formula (5), par. 11, should be $\frac{\sqrt{H}}{2} = 3\frac{3}{4}$ feet nearly. It will be made 3 feet, with a stiffening rib or rim of 2 feet in width. It could well be made a fraction of the base width, say $= \cdot 3b$, and be reinforced wherever the thickness falls below 2 feet.

The length of the pier base is measured from the extrados of the arch, the two half arches forming, as already explained in par. 17, a forked continuation of the buttress pier base.

The battering of the sides of the pier would clearly be a correct procedure, as the pressure diminishes from the base upwards. A combined batter of 1 in 10 is adopted, which leaves a crest width of 5·6 feet. The length of the pier base, as also its outline, were determined by trial graphical process, with the object of manœuvring the centre of pressure as near that of the base as possible, so as to equalise the maximum and the mean unit stress as much as possible. This has been effected as shown by the incidence of the final resultant on the elevation of the buttress pier.

(27) *Pressure on foundations.*

The total imposed weight is measured by N in the force diagram, and amounts to 150,000 cubic feet of masonry, which at a specific gravity of $2\frac{1}{4}$ is equivalent to $\frac{150,000}{16} = 8,138$ tons. The average pressure is this quantity divided by the area of the base, or by $125 \times 12 = 1,490$ super feet, the quotient being $5\frac{1}{2}$ tons nearly. The maximum pressure will be the same owing to the incidence of R at the centre of the base. This $5\frac{1}{2}$ tons is a very moderate pressure for a hard foundation. It will be noticed that N greatly exceeds W. This is due to the added weight of water represented by

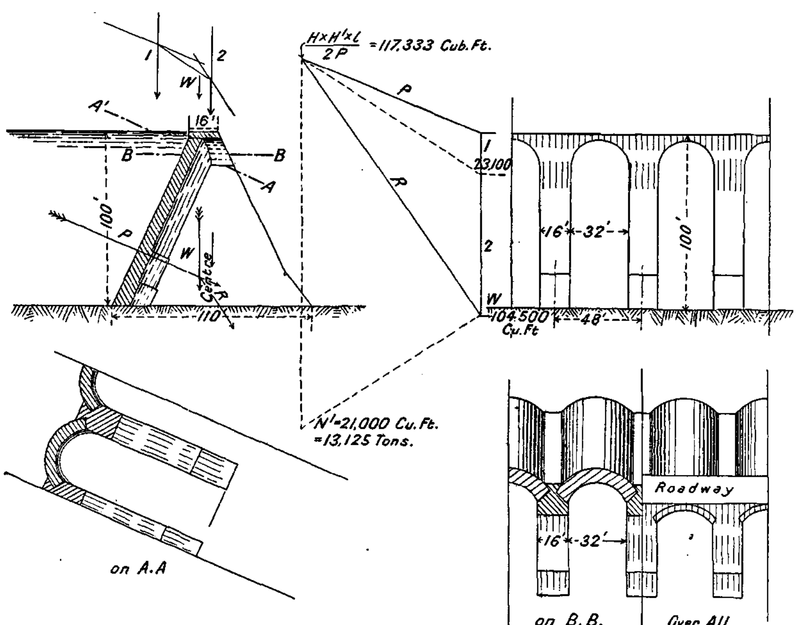


FIG. 11.—Ogden Dam, Utah.

the inclination given to the force line P, which represents the water pressure.

According to the force diagram, N_1 scales 174,000 cubic feet or 10,800 tons. Dividing this by the area of the base, or by 1,490 super. feet, the quotient 7·2 tons is the average or mean unit stress, the maximum being the same. This is also satisfactory. The cubic contents per foot run works out to $\frac{64,000}{76} = 850$ cubic feet nearly; but if the stress be increased to 10 tons in the buttress this will be reduced to 800 cubic feet.

The contents of a gravity weir with base width $\frac{2}{3}(H + d)$, and top width $\sqrt{H} + \sqrt{d}$, works out at 1,728 cubic feet; the saving in material is therefore over 50 per cent.

(28) The Ogden Dam, Fig. 11, lately erected, is a notable example of the arch and buttress type C. Its height is 100 feet, that is in excess of anything

hitherto attempted. The inclination of the arches is less than $\frac{1}{2}$ to 1, or about 25° to the vertical. The profile of the buttress is equiangular except for a small out-throw of the toe. On the whole it must be pronounced a good design, but could be improved in the following points: the arch is unnecessarily thick at the crest and could well be reduced from 6 to 2 feet, thus effecting considerable economy. The designers were evidently afraid of the concrete in the arch leaking, and so overlaid the extrados with steel plates. A greater thickness of arch, causing it to possess less liability to percolation under pressure, could have been provided by increasing the span and radius of the arches. The design would thus be improved by adopting larger spans, say 100 feet: buttresses, say, 25 feet thick, their length being dependent on the width of base required to provide sufficient moment of resistance; and, further, the inclination of the arches might require increasing to bring the centre of pressure at or close to the centre of the buttress. The finish off of the crest by another arch forming a roadway is an excellent arrangement, and is well suited for a dam; for a weir, on the other hand, the curved crest is preferable from the increased length of overflow provided. The stress diagram shows that the value of N_1 is 13,125 tons. The incidence of R on the base is 5 feet from the centre, consequently $S = \frac{N_1}{A}\left(1 + \frac{6c}{b}\right) = \frac{13,125}{110 \times 16}\left(1 + \frac{3}{11}\right) = 8{\cdot}9$ tons (*vide* formula (2), par. 7, Chap. II).

The pressure on the arch ring at the base by the short formula works out by $\frac{RHw}{t}$ to $\frac{24 \times 100}{8 \times 36} = 8\frac{1}{3}$ tons.

The contents of the dam per foot run amounts to $\frac{104,500}{48} = 2,177$ cubic feet; that of a gravity dam would be about 3,500 cubic feet per foot run, making a percentage in favour of the arched type of nearly 30. With a better disposition of the parts as indicated above, the percentage would be increased to 40 or 50 per cent. Actually the saving amounted to only 12 per cent.; this was owing to the steel-covering which, as we have seen, could have been dispensed with.

Reinforced Concrete Flat Deck Dams and Weirs (Type D).

(29) Of late years an increasing number of dams and weirs of the panel and buttress reinforced concrete type have been constructed of heights, it is believed, up to 100 feet. These are all on much the same pattern. This consists of a deck sloping at 45°, supported by buttress piers at short intervals. These piers are nearly triangular in profile, having their apex at a point above crest level. On the down-stream slope a thin covering is often placed over the ends of these piers, providing a rollway for water if a weir. The whole thus forms a hollow box with numerous partition walls. The space thus provided is utilised in some cases to accommodate the power plant and to afford cross communication. In the former case the partition walls are not

continuous across, but form an arcade, affording space for the machinery. The designs are admirable, the only point remaining to be decided is whether they can be improved upon in point of cost by substituting arches for the reinforced flat panels and converting the profile into type C. The use of a flat deck demands considerable reinforcement, as well as frequent supports. With arches, on the other hand, no reinforcement is necessary, as they are entirely in compression, and they can be of any span. There is good reason to believe that type C will prove to be the more economical of the two, not only in saving of actual material but in cost of construction.

(**30**) An example of a small work in type D is given in Fig. 12 of the Schuylerville Dam, while an alternative design in type C has been prepared in Fig. 13 and 13a. The arrangement is similar to the last example (Fig. 9), except that the piers are hollow. No doubt a wider span than 20 feet would give better results.

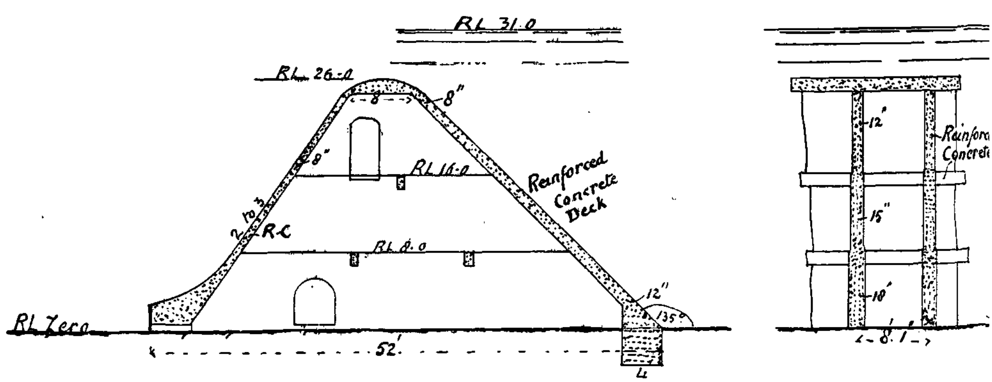


FIG. 12.—Schuylerville Weir.

The comparative quantities per foot run are as follows :—

Fig. 12. Reinforced concrete 215 cubic feet per foot run.
Fig. 13. Plain concrete 178 ,, ,, ,,

This is considerably in favour of type C.

On the other hand, a direct overfall is provided in the last, not a rollway although the direct overfall is considered the better style, barring ice floes.

(**31**) Another much larger section is given in Fig. 14, that of the Ellsworth Weir. This work is of the same height as Fig. 9, which was designedly made so for the sake of comparison.

The comparative quantities per foot run of these two can now be set out.

Fig. 14. Reinforced concrete, 11,000 cubic feet per 15-foot bay, equivalent to 733 cubic feet per foot run.

Fig. 9, as by par. 27, 800 cubic feet per foot run (with S raised to 10 tons).

The advantage in actual quantity of concrete lies with Fig. 14, but when the cost of the steel, which amounts to $8\frac{1}{2}$ tons per 15-foot bay, together with the extra expense involved in construction and in the quality of the concrete,

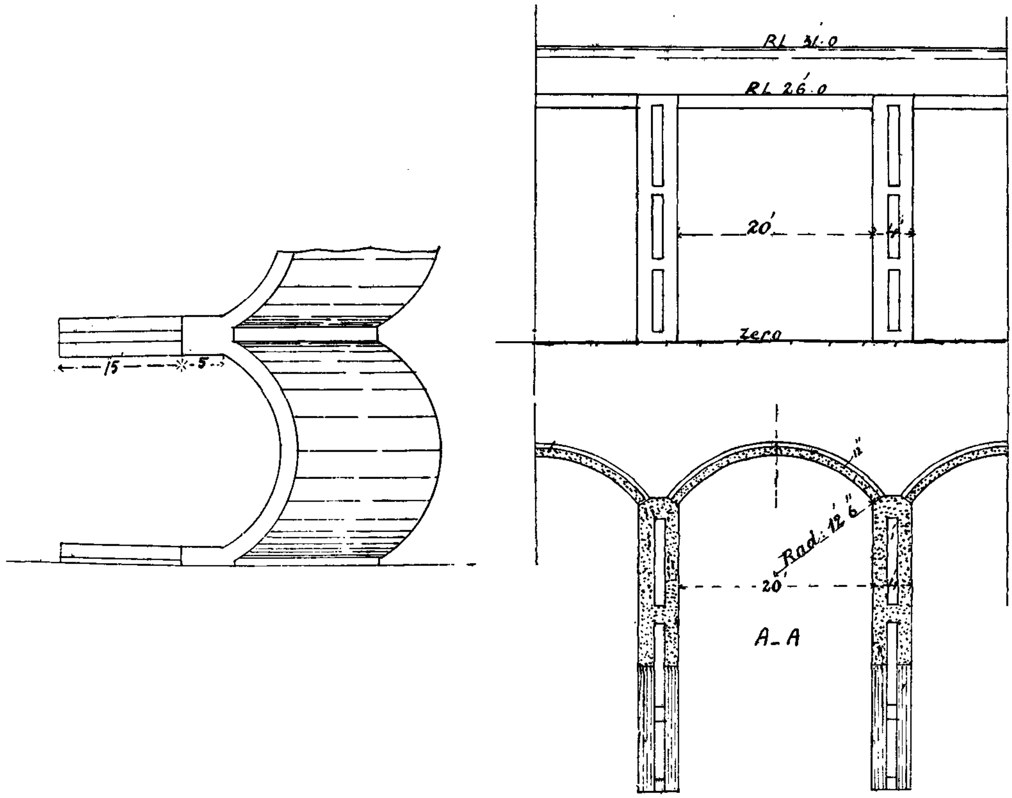
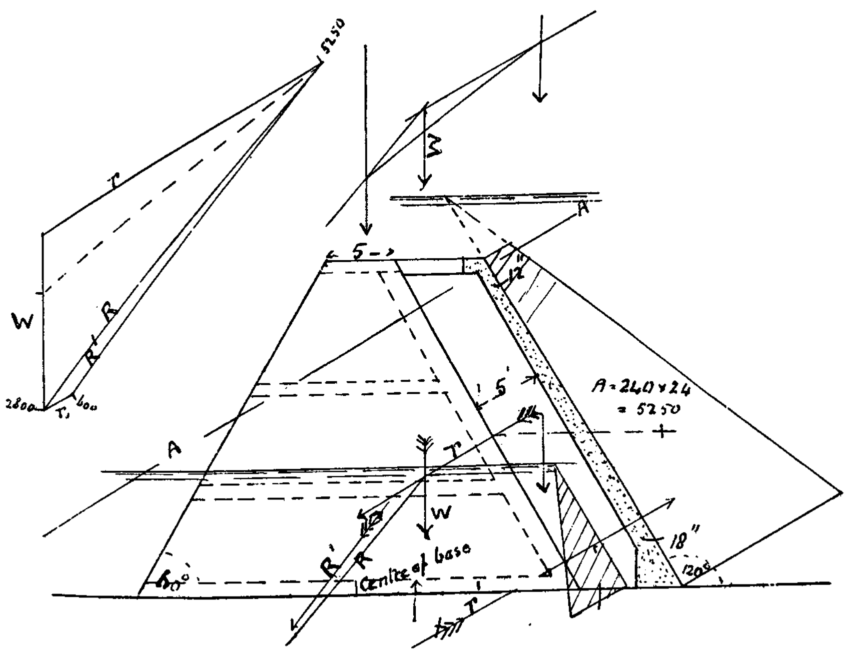

RL 31.0
RL 26.0
20'
zero
15'
5
Rad. 12' 6"
20'
A_A

FIG. 13.


5250
T
W
R' R
2800
600
5
12"
A
5'
A = 240 × 24
= 5250
A
T
W
R'
R
18"
60°
Centre of base
120°
T

(13a.)

is taken into consideration, the advantage will be with the arched type. If steel reinforcement were used in Fig. 9, the quantities of material

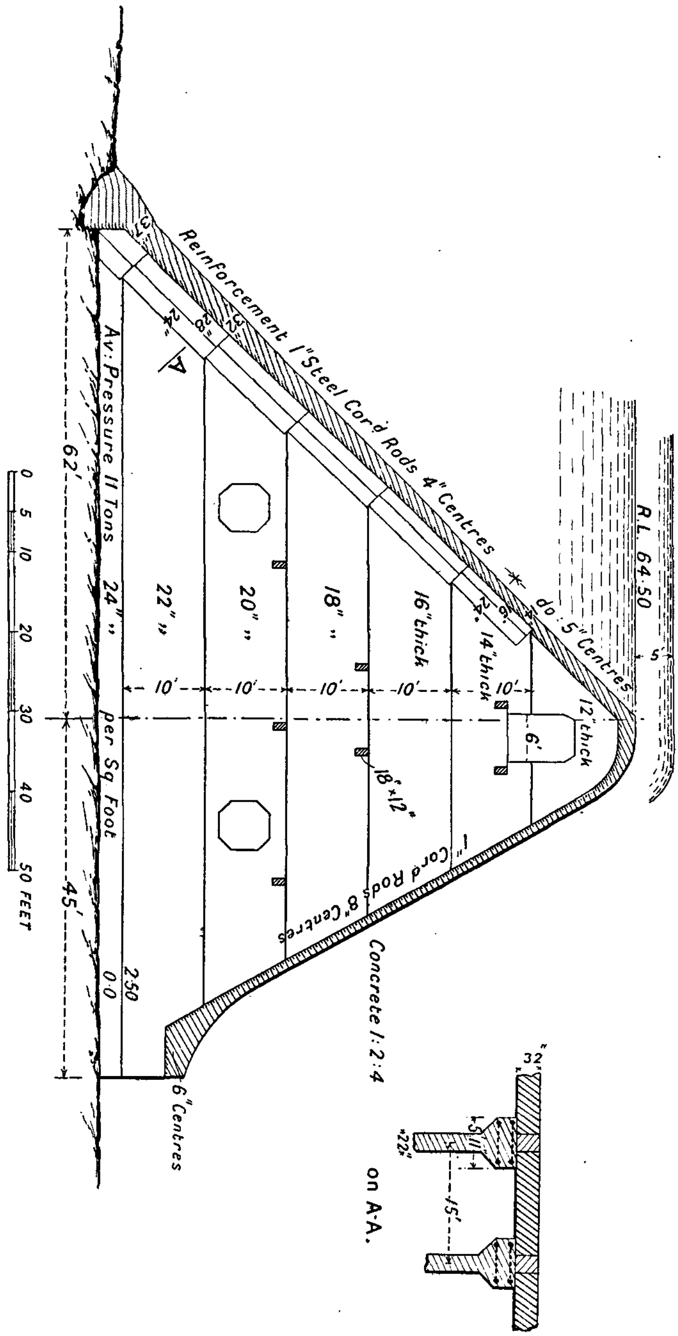


Fig. 14.—Ellsworth Weir.

could be reduced, though it is doubtful if much economy would result thereby.

(32) The deck panel type cannot be so economical as the arched. The reason is not far to seek. Flat decks are unsuitable except for very small spans, and even then have to be of considerable thickness to withstand the great pressure to which they are subject. Thus in Fig. 14 the deck for a 15-foot bay is only half the thickness of the arch in Fig. 9 with a 76-foot bay. In addition to this, numerous thin buttresses are required, the cubical contents of which mount up, and the cost of construction must be much in excess per yard of thick plain concrete buttress piers. All this together causes this type to be more expensive than Type C.

(33) A common impression is abroad that structures designed in reinforced concrete must be the most suitable and most economical of any. With regard to irrigation works, this is by no means the case. In all hydraulic works *weight* is a desideratum; consequently economy in material, by introducing reinforced concrete, is often a mistaken policy, and if comparative designs were made it would frequently be found that a better and more economical design could be produced by employing thicker walls and larger spans of ordinary masonry, than the slim buttress style of reinforced concrete structure now so much in vogue.

CHAPTER IV

PIERS, ARCHES, ABUTMENTS AND FLOORS

(1) INVESTIGATION into the stresses to which piers are subjected is naturally connected with that on arches and abutments, so that these three items will be treated together.

In ordinary bridges of fairly large span the thickness of piers is usually made from one-tenth to one-sixth of the span, and in cases of large spans, say from 40 to 80 feet, the piers should be designed with battering sides so as to produce uniformity of pressure in their masonry. Except in the case of aqueducts, large spans are, however, seldom necessary in irrigation works. Even in road bridges crossing a canal, a series of quite small spans with light arches and piers are far more economical than large spans which, except where navigation is concerned, offer no advantage of any kind. In cross-canal, or head regulators, bridged falls, etc., the spans are necessarily small, as, owing to the expense of manipulating heavy regulation gates, these have to be kept within reasonable dimensions. On the other hand, the comparatively recent improvements in non-frictional and counter-weighted roller gates allow of much larger openings being used to what was formerly the practice, but spans of 10 to 25 feet are about the economical limits in such cases.

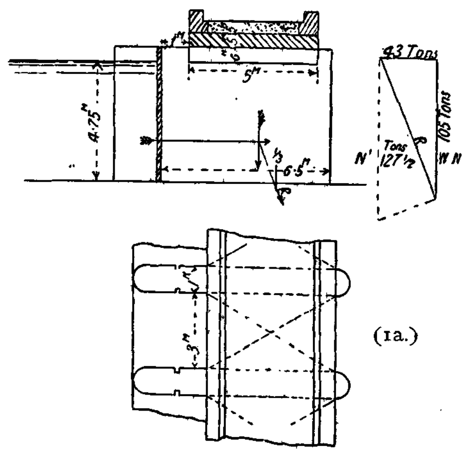
(2) In irrigation works, piers have not only to safely withstand the weight of the superstructure, and the incidence of stress due to a rolling road, but are often subjected to water pressure, not only in the direction of their length but also cross-wise, from the lateral pressure exerted when one bay of a regulating bridge is closed at the head, being then nearly empty of water, but with a full supply running in the next compartment. This latter cross-stress is a very important item and should never be lost sight of in designing works of this description, as serious failure has occurred from oversight of this point.

Even when one bay is partially full of water while the next is running full, the pier is often weakened by displacement, for which reason it is very necessary to keep the arches and superstructure well above the highest water level so as to obviate any diminution in their effective weight by flotation.

(3) We will now proceed to investigate the stresses induced in the piers arches and abutments of a regulating bridge by water, under the most unfavourable circumstances. For this purpose an actual work has been selected.

Fig. 1 is a representation of the transverse section and plan over all of one bay of the Raswaniya regulator. The dimensions are all in metres. The piers are 1 metre, *i.e.*, $\frac{S}{3}$ in thickness, the spans S being 3 metres wide. Each pier is consequently subjected to the pressure of a prism of water 4 metres in length. The depth being 4·75 metres, the total pressure will be the area of the triangle of water pressure, which is $\frac{h^2}{2}$, multiplied by the length and by w the weight of a cubic metre of water or $\frac{wh^2}{2} \times l$. Taking w as equal to 1 ton, which it is very nearly, we have $r = \frac{(4{\cdot}75)^2}{2} \times 4 = 43{\cdot}1$ tons.

Now the cubical contents of one pier with superstructure, lying to the right of the grooves, is estimated at 60 cubic metres, the weight of which, taking 1·75 for specific gravity of the brickwork, will be $1 \times 60 \times 1{\cdot}75 = 105$ tons. The portion of the pier up stream of the grooves has been omitted from consideration, as it lies beyond the plane of pressure and is subjected to flotation. Strictly it should have been included in the count.



(1a.)

FIGS. 1, 1a.

Fig. 1a is the force polygon. The incidence of the resultant R, on the base line, is almost exactly at the outer one-third division of the base. If it fell without to any extent, tension would be set up at the opposite toe, to avoid which the pier would have to be lengthened. The vertical line N_1 in Fig. 1a measures the intensity of unit pressure on the masonry at the intersection of R and equals $127\frac{1}{2}$ tons, which being situated at two-thirds the length of the pier, the maximum at the toe will be $\frac{N_1}{b} \times 2 = \frac{127{\cdot}5}{6{\cdot}5} \times 2 = 40$ tons per square metre nearly (*vide* par. 8, Chap. II.), or as a square metre equals 10·75 feet, the pressure per foot square will be $\frac{40}{10{\cdot}76} = 3{\cdot}7$ tons. This is by no means excessive. This calculation in the case of large spans with a greater head of water is very necessary, as not only may the pier be in tension at the heel, but the pressure on foundations or floor may be excessive, necessitating a lengthening of the pier down stream to increase its bearing surface.

(4) The thickness of piers is dependent on the weight carried, and consequently is best expressed as some function of the span. The depth of water regulating the height of the piers is likewise a factor which must not

be disregarded, and further the allowable pressure on the foundation, though this latter can be arranged for by widening the pier foundation below the level of the floor base. The thickness may be generally taken as not under $\sqrt{S}$, or to vary from $\cdot 35S$ as a maximum to $\cdot 25S$ as a minimum, for spans from 10 to 25 feet. The greater the span the less fractional proportion required for the pier. Thus, as in Fig. 1, the width of the pier is $\cdot 33S$. For a span of 24 feet under similar conditions the same fraction would give a width to the pier of 8 feet, which is excessive, 6 feet being a better proportion.

(5) The following Table, though naturally empirical, will, it is believed, form a good guide of thickness to adopt in design for partial regulators. The depth of water up stream of the work forms an additional factor, not the head or horizontal pressure, which latter would be provided for in the length more than in the thickness of the piers.

TABLE I.—TABLE OF PIER THICKNESSES SUITABLE FOR OPEN PARTIAL REGULATORS AND WEIR SLUICES.

Span.	Depths of Water.							
	15 ft.		20 ft.		25 ft.		30 ft.	
	M	T	M	T	M	T	M	T
10	·25	2·5	·27	2·7	·29	2·9	·31	3·1
15	·24	3·6	·26	3·9	·28	4·2	·30	4·5
20	·23	4·6	·25	5·0	·27	5·4	·29	5·8
25	·21	5·5	·24	6·0	·26	6·5	·28	6·7

M signifies multiplier. T thickness.

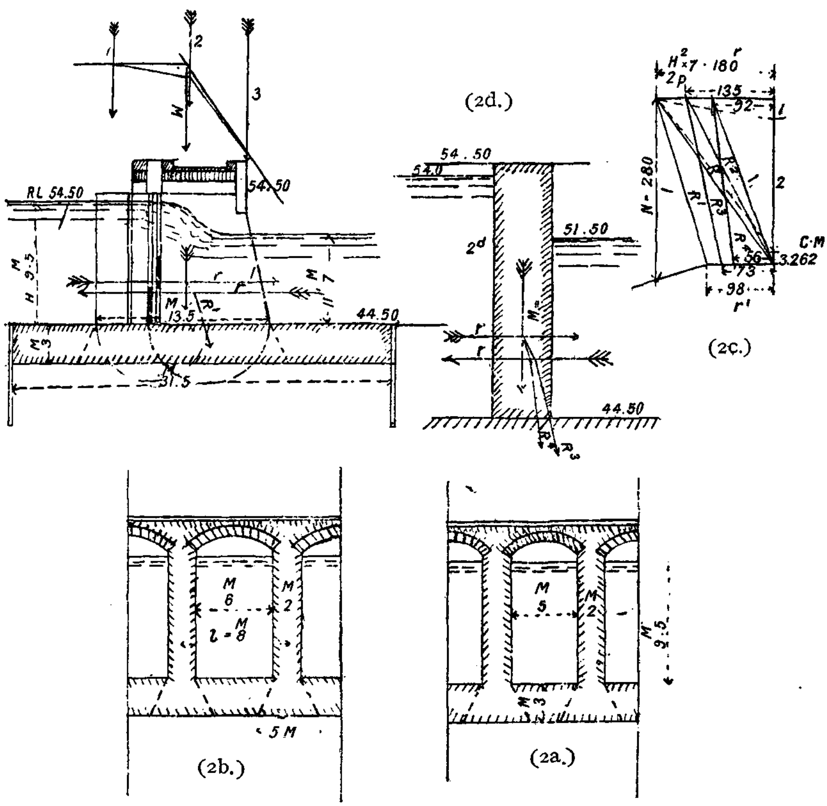
Twenty-five feet is about the outside limit of workable vents for weir sluices and similar works whose functions are limited to partial regulation, and in modern practice the least width of vent is hardly under 10 feet. For canal heads subjected to a considerable head of water the thickness of the piers should be $\cdot 4S$.

(6) Inspection of many examples of regulating works will show how far actual practice agrees with the rules for thickness set forth above. Of one quite modern example, viz., the Assiût regulators, a section is produced in Fig. 2.

The piers in this work are 2 metres in width to spans of 5 metres, giving a ratio of $\cdot 4S$. This proportion appears excessive. There seems to be no reason why the spans should not have been designed of a width of

6 metres or 19¾ feet, retaining the width of the piers at 2 metres; the ratio would then be reduced to ·33*S*, *i.e.*, close upon what is given in the Table for a depth of water of 30 feet, which would be about 6 feet to the 6·6 feet of 2 metres. Fig. 2a is a transverse section of the spans as they are, and Fig. 2b as with the suggested enlargement.

The stress diagram is shown in Fig. 2c. The total contents of one span of 6 metres is estimated to be 358 cubic metres. This reduced for flotation will be diminished to 262 cubic metres, ρ being taken as 2. This is the



FIGS. 2, 2a, 2b, 2c, 2d.—Assiût Regulator.

length of the load line *W*. The opposing water pressures are represented by the lines r and r_1, each equal to $\frac{H^2}{2\rho} \times l$, viz., 180 and 98 cubic metres (l in this case being 8 metres, the length of one span plus the pier). The projection of the resultant R_1 on the longitudinal section in Fig. 2 shows the incidence of R_1 to be just within the middle third of the base. The question of what water levels up and down stream induce the maximum stress on the piers of a partial regulator is considered in Chap. VII.

(7) With regard to lateral pressure, it is deemed that an extreme length of 6 metres is the outside exposed to this force. The values of r and r_1 in this case

will therefore be $\frac{6}{8}$ of the former values, W remaining the same. These lengths are marked in the diagram Fig. 2c and projected on to an enlarged section of the pier in Fig. 2d. The final resultant falls just at the pier toe. Thus the pier would not overturn even if not supported at the top by the arching.

Properly speaking, this support should be estimated and the base relieved accordingly. The force r lies $\frac{9 \cdot 5}{3} = 3 \cdot 17$, or, say, $3 \cdot 2$ metres above the base, while the distance of r_1 is $\frac{7}{3} = 2 \cdot 33$ metres.

The pier is supported at 10 metres height, consequently the proportion of r supported at the top of the pier will be $\frac{3 \cdot 2}{10} = \cdot 32$, and similarly that of r_1 will be $\frac{2 \cdot 3}{10} = \cdot 23$. Put into figures, the balance pressure in favour of r will be $(\cdot 32 \times 135) - (\cdot 23 \times 73)$, *i.e.*, $43 \cdot 2 - 16 \cdot 8 = 26 \cdot 4$ cubic metres, equivalent with $\rho = 2$, to $2 \times 26 \cdot 4 = 52 \cdot 8$ tons distributed over a length of 7 metres of arch, *i.e.*, $\frac{52 \cdot 8}{7} = 7 \cdot 54$ tons per metre run, or $\frac{7 \cdot 54}{10 \cdot 76} = \cdot 7$ tons per foot run of horizontal superstructure.

To revert to the base, r and r_1 will be reduced by $\cdot 32r$ and $\cdot 23r_1$ respectively, *i.e.*, from 135 and 73 to $135 - 43 \cdot 2 = 92$ and $73 - 17 = 56$. These reduced values are marked on the force diagram, Fig. 2c, and the resultant line appertaining thereto, viz., R_4, projected on to the pier section.

This falls well within the base.

(8) Next, with regard to the maximum inclined pressure on the pier section. This is represented in Fig. 2c by the line N_1, which measures 280 cubic metres, or 560 tons. Dividing by the length and width of the base, the mean pressure per square metre will be $\frac{560}{13 \cdot 5 \times 2} = 20 \cdot 74$ tons; again dividing by 10·76, we obtain mean pressure in tons per square foot, viz., $\frac{20 \cdot 74}{10 \cdot 76} = 2$ tons nearly. As the incidence of the resultant is at the boundary of the middle third of the base, the maximum intensity will be double the mean or 4 tons per square foot.

In the original design the piers were carried on circular wells of brickwork sunk in the sand. The width of these was only 2·3 metres, so that, except in length, the bearing surface on the sand was not diminished, but neglecting the weight of the wells themselves and also the skin friction, the load on the actual base would not be less than 2 tons. The modification of the original design, actually executed, is shown in Fig. 2, mass concrete of one depth throughout of 3 metres, enclosed in iron sheet-piling, having been substituted for a shallower floor and well foundations under the piers.

(9) When a pier rests on a mass foundation of this description the load is gradually spread over a larger area, and can be assumed with safety to splay out on each side in correspondence with that usually given to footings at a slope of $\frac{1}{2}$ to 1.

Thus, as shown dotted in Figs. 2, 2a and 2b, the bearing width at the base of the foundation will be the thickness of the pier plus that of the foundation, in both cases $2 + 3 = 5$ metres. The same widening of effective bearing area may be conceived to occur at each end of the piers, increasing the effective length from $13\frac{1}{2}$ to $16\frac{1}{2}$ metres.

The vertical load on the foundation is 358 cubic metres, to which must be added the contents below the floor or a piece $15 \times 5 \times 3 = 225$ cubic metres submerged, *i.e.*, equivalent to 112 cubic metres (with $\rho = 2$). The total will then be $358 + 112 = 470$ cubic metres $= 940$ tons.

This load is distributed over a base $16\frac{1}{2}$ metres long and 5 metres wide, consequently the pressure per square metre will be $\frac{940}{82 \cdot 5} = 11 \cdot 4$ tons, and reduced to per square foot $\frac{11 \cdot 4}{10 \cdot 76} = 1 \cdot 06$ tons. In the case of the actual 5-metre spans, the load will be less by a length of arching of 1 metre, *i.e.*, by some 12 or 15 cubic metres of masonry, which would clearly make no appreciable difference.

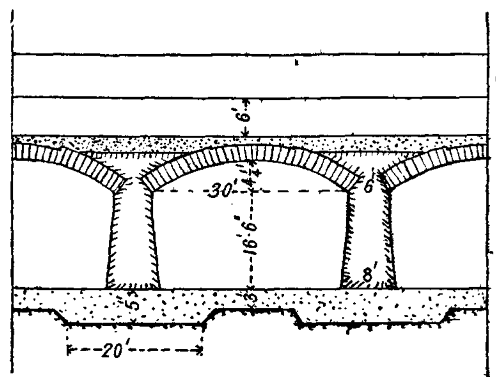


FIG. 3.

(10) We gather from this that a load of 1 ton per square foot is admissible on a foundation of Nile silt, which is of the worst possible kind. On good coarse river sand, a unit load of 2 tons, or even over, is quite allowable. As regards hard clay, 4 tons is not exceptional.

In the Nadrai Aqueduct, Fig. 3, Chap. XI., the pressure at the bottom of the well foundations was limited to 4 tons, and this on pure sand, no allowance, as usual, having been made for skin friction, which latter was ascertained by direct experiment to range from ·6 to $1\frac{3}{4}$ cwt. per square foot of surface. On the Hawkesbury Bridge foundations, also in sand, $4\frac{1}{2}$ tons was the limit pressure. In some works on clay it amounts to over 8 tons per square foot. Sand, if protected from erosion, is as good a foundation as any, when of great depth; if, however, the sand is only a thin layer overlying a clay bed, the latter should be reached by protective curtain walls or sheet piling, as otherwise the reduced frictional stability promoted by contact with a smooth hard surface might cause the sand layer to be washed out under hydrostatic pressure.

(11) If the sand foundation is efficiently protected by curtain walls or sheet piling, no object is gained by deep foundations, and the same would apply to hard clay in cutting. In the case of large spans where the floor is not sufficiently deep to afford the requisite support to the piers, the foundations of the piers must be carried lower. A good example of how this is effected in mass foundations is given in Fig. 3 of the pier bases of the Budki Superpassage.

The required depth, as we have already seen, should be such that, added to the thickness of the pier, a base width is provided suitable to the limiting pressure on the subsoil.

In regulating works, thick floors to resist the head of water are a necessity which is now thoroughly recognised, and the thickness given is often more than sufficient for pier foundations without anything supplementary in the way of special foundations being supplied underneath the piers.

In the Budki Superpassage the pressure due to the weight of one span, plus that of 6 feet of water carried, is $2\frac{1}{4}$ tons per square foot at the base of the piers, and 1·45 tons on the ground. The subsoil is believed to be sand.

(12) The proportional thickness of piers for aqueducts varies considerably in different examples, as below :—

TABLE II.

Chap. XI.	Fig.	Span.	Thickness.	Ratio.	$\sqrt{S}$
Budki	8	30	6 ft.	·25	5·4
Thora Nala	1	30	5 ft.	$\cdot 166S$	5·4
Kerai	2	20	4 ft.	$\cdot 2S$	4·4
Gunneram	4	40	6 ft.	$\cdot 15S$	6·3
Kali Nadi	3	60	7 ft.	·117	7·75

A good rule for guidance would be to make the top thickness of piers for aqueducts carrying 5 feet or 6 feet of water from 25 feet span upwards $= \sqrt{S}$, below 25 feet $= \cdot 2S$ (S being the span).

Aqueduct piers, like those of large span bridges, do not require to be vertically sided, as is necessarily the case in regulators, where draw-gates in grooves are used, and should always be constructed with either straight or curved battering sides, in order to increase the base width. There is, however, a limit to the batter adopted, as the water-way is thereby diminished. This subject is further treated in Chap. XI.

Arches.

(13) The horizontal thrust of an arch is entirely dependent on the weight of that portion of the half arch and its load lying between the crown and the joint of rupture. Whatever weight lies between the joint of rupture and the abutment does not affect the value of the thrust, as it really forms a projecting part of the abutment.

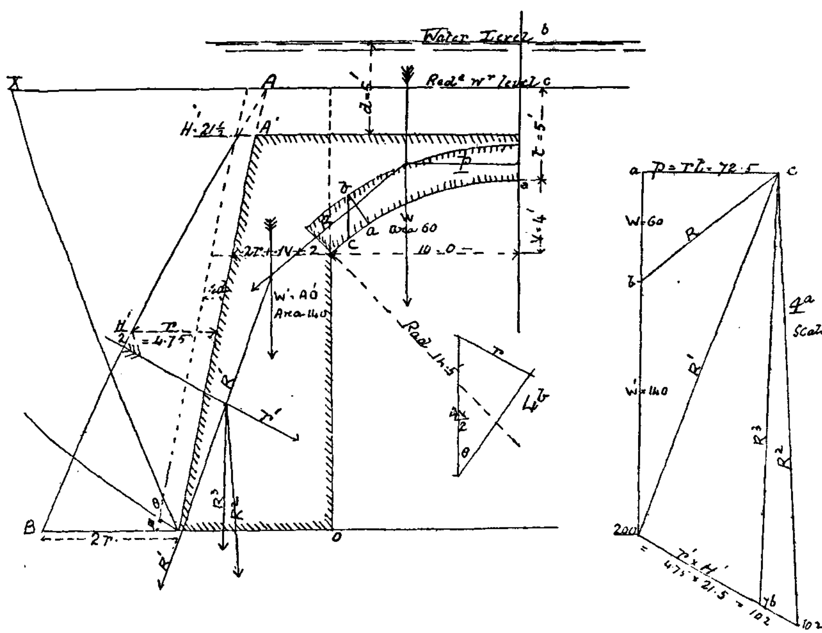
In every arch, of whatever profile, elliptical, segmental or semicircular, the point of rupture is that point where the internal line of pressure tends to go outside the middle third of the arch ring, and so produce tension in the arch. Further, at this point, compared with others, the horizontal stress is at a maximum.

The exact position can easily be found by graphical methods, but as segmental arches are as a rule only employed in modern irrigation works in which the point of rupture is at the springing, it will be superfluous to enter into this question.

The thrust of an arch (P) loaded to a horizontal line above the crown is obtained by the following formula with great exactitude :—

$$P = wrt, \qquad (1)$$

in which w is the unit weight of the material of the arch ring and filling, the latter reduced to an equivalent area of the material of the arch ring, r the radius of the intrados and t the vertical height of reduced terrain line above the intrados at the crown.



FIGS. 4, 4a, 4b.

(14) An example illustrative of the arch thrust on abutments is given in Fig. 4, which represents the segmental half arch of an aqueduct. The spandrel is built up to a horizontal line 6 inches above crown of arch. Above this lies 5 feet of water. This water area is reduced by dividing it by ρ, the specific gravity of the material in the arch, and in this case ρ is assumed as 2. The distance ac is thus the t in the formula and equals 5 feet. As the arch and abutment are considered to be of the same specific gravity, W, which equals $w\rho$, is a common factor which can be eliminated in the graphical process. The horizontal thrust will then be represented by rt, or by $14{\cdot}5 \times 5 = 72{\cdot}5$, and the weights of other parts by their areas. In Fig. 4a $ac = P = 72{\cdot}5$, and the load line W, or ab, equals the area of the half arch, viz., 60, the resultant being bc or R. In the profile Fig. 4, the position of P must be determined with reference not only to its own incidence in the arch ring at the crown, but with reference to that of R in the arch ring

in the springing. Both lines must fall within the middle third of the ring, and that position of P which ensures the most axial incidence of both R and P is the correct one.

We now come to the composition of R with the weight of the abutment, which is W_1 on the load line, representing the area of the trapezoid AO. The resultant R_1 falls just within the base.

The incidence should be somewhat more recessed within the toe, hence the abutment requires thickening at the base.

Thickness of Abutments.

(15) The thickness of an abutment at the spring line of an arch must be such as to ensure against failure by sliding due to the horizontal thrust of the arch. A suitable width is obtained by use of the following empirical formula from "Trautwine"—

$$t = \cdot 2r + \cdot 1V + 2, \qquad (2)$$

in which r is the radius of arch and V the versed sine.

In Fig. 4 the thickness is made in accordance with the formula, being $\cdot 2 \times 14\cdot 5 + \cdot 1 \times 4 + 2 = 5\cdot 3$ feet. The rear slope is given by another formula—

Ratio of slope equals $\cdot 02S$ horizontal to $\cdot 5\ V$ vertical

$$\text{or } \frac{\cdot 02S}{\cdot 5V}.$$

The base width thus obtained will be just sufficient, not counting in the assistance of the earth backing. Under normal conditions of loading these formulas would answer well, but in the case in point the heavy load of water carried would require an increase in the thickness proportional to the depth carried. The increase adopted will be $\cdot 2d$, or one-fifth the depth of water, and the formula would become for aqueducts—

$$t = \cdot 2r + \cdot 1V + 2 + 2d. \qquad (2a)$$

The increased thickness is shown by a dotted line in the profile Fig. 4.

In these calculations the abutment is assumed of the same specific gravity as that of the arch and loading. In stone bridges its specific gravity is greater.

(16) The effect of the earth backing is shown in Figs. 4, 4a and 4b. In Fig. 4b, r, or the half width of the triangle of earth pressure AOB, is obtained by the process frequently employed in Chap. I., the specific gravity of the earth being taken as the same as that of the wall.

The half width r_1 of the trapezoid of pressure A_1B, multiplied by its height gives its area, and is equal to $4\cdot 75 \times 21\cdot 5 = 102$. By this quantity being set off in Fig. 4a from the extremity of the load line in its proper direction we obtain R_2 on Figs. 4a and 4.

If the earth backing has three-fourths the unit weight of the masonry, the length of r_1 in Fig. 4a will be three-fourths less or 76, and the corresponding resultant is R_3 in both figures.

Thickness of Arches.

(17) There are several empirical formulas giving values of the crown thickness of arches, of which the simplest and best is that in Molesworth's Pocket-book of

$$t = n \sqrt{r} \text{ or } \cdot 4 \sqrt{r}. \qquad (3)$$

n is a multiplier varying from ·4 upwards.

For ordinary bridges a value of ·4 will be ample for the multiplier, but this should be increased in the case of aqueducts up to ·5 in proportion to depth of water carried (*vide* Chap. XI.), and the formula will then become

$$t = \cdot 5 \sqrt{r}. \qquad (3a)$$

This is the thickness adopted in Fig. 4.

In large arches the thickness should increase proportionately with that of the thrust from the crown to the springing. This increase can be obtained graphically as shown in Fig. 4. Here ab is a radial equal to the thickness of the crown, the vertical bc will then be the correct increased thickness at this point. The span in this particular instance being small, no increase is deemed necessary.

Buttresses.

(18) Considerable economy can be effected in the section of abutments by the use of buttresses; their effect on the stability of a retaining wall or abutment is obtained as follows:—The wall should be considered as having a base width equal to the normal thickness at base plus the projection of the buttress, but formed of two materials of different specific gravities, the solid portion being of the actual specific gravity of the masonry, and the area behind, which is partly solid buttresses and partly open space, should be considered as of a lighter specific gravity equivalent to that of a material occupying the whole of this rear area, but weighing no more than the buttresses themselves.

The specific gravity of this portion will then be $\rho \times \frac{a}{A}$, in which a is the area of the solid portion, viz., of the projecting buttresses, and A that of the whole space between a line forming the ends of the buttresses and the back of the wall from which they project.

Floors, Thickness and Length.

(19) *Firstly: Of Submerged River Weirs on Sand.*

The thickness of the floor is a matter dependent on the hydrostatic pressure to which the floor is subject owing to the porous nature of the substratum. This is fully dealt with in Chap. VI.

A minimum thickness for a floor can be taken as $\sqrt{H}$, *i.e*, of the maximum statical head which is the height of the weir plus that of the weir shutters.

The formula given in Chap. VI. is

$$t = \tfrac{4}{3}\,\frac{H - h}{\rho - 1}$$

for submerged aprons or floor, in which H is the maximum statical head and h the loss of head due to percolation up to the beginning of the floor.

The width of submerged river weir aprons or floors on sand foundations is a function not only of the height of the permanent obstruction, viz., that of the masonry weir wall, but also of the quality of the river sand represented by the so termed coefficient c, and is found by the following formula, also given in Chap. VI.

$$W = 4\,c\,\sqrt{\frac{H^a}{13}}$$

in which H^a is the height of the shutter crest. For example, if H^a be 12 feet and the river coefficient be of class 2, that is if $c = 12$, the width would be $4 \times 12\,\sqrt{\cdot 93} = 48 \times \cdot 96 = 46$ feet nearly.

(**20**) *Secondly: Canal Falls.*

The thickness of the floor of canal falls without a sunk water cushion should be $\sqrt{H + d}$ in which H is the height of the drop wall and d the depth of film, *i.e.*, the height of the upper reach above crest. The width of floor should be $2\,(H + d)$. The same applies to escape falls.

(**21**) *Thirdly: Weir Sluices and other partial Regulators on sand.*

The floors of Weir sluices are on the same level and subjected to the same hydrostatic pressure as those of river weirs, of which they form an integral part; consequently their thickness will be the same as that given to the weir apron.

If this depth is insufficient for the support of the piers of the superstructure, with a prescribed limiting pressure, it should be increased, either under each pier separately, or else the whole mass can be of increased thickness underneath the sluices, and taper off upwards to the end of the masonry floor, which can be of proportionately less thickness to allow for the excess at the head.

The head to which a weir sluice is subjected being the same as that on the weir apron, its floor could be made of the same length were it not for kinetic considerations, the rush of water through the sluices, particularly when the river is high, being very great; this length may be put down as $2W$, or as $8c\sqrt{\frac{H^a}{13}}$.

Of partial regulators, a good example is that of the Assiût regulators. Although the water is over 30 feet in depth, it is never required to be completely shut off, so that the maximum hydrostatic pressure is due to a head of about 9 feet.

In this case the thickness of the floor is 10 feet, *i.e.*, much in excess of what our formula gives, but this is by reason of the necessity of accommodating the piers, and also for the purpose of providing weight to withstand hydrostatic pressure.

The kinetic force to which these floors (Assiût) are subjected, is nothing like that in weir sluices, so that a very long masonry floor is not a *sine quâ non* as in the latter case.

(22) *Fourthly : Head Regulators or Intakes.*

These works are subjected to greater hydrostatical pressure than any others. Canal heads, as a rule, have to be entirely closed, even during the highest floods in the river, as often at this very period water is not required in the canal, and a total closure is further necessary for silt clearance and repairs. The Egyptian canal heads alone are partial Head Regulators.

The thickness of floor as a minimum should be $\sqrt{H_1}$, H_1 being the height of flood above regulator floor. Thus, with flood level 25 feet above regulator floor, the thickness will be 5 feet. The length and section of the floor and pitching and the depth of the fore sheet piling are, however, matters entirely for calculation, as will be explained in Chap. VI. Owing to the absence of kinetic considerations the length of the floor is solely dependent on that length of enforced percolation which is requisite for the stability of the sand substratum.

If a Head Regulator is founded on good clay, the floor thickness need not exceed $\sqrt{H_1}$, and the length be limited to 10 feet to 15 feet beyond the noses of the sluice piers.

For cross-canal Regulators and branch Heads, the pressure is limited to that due to a very moderate head of water, and a thickness of floor of $\sqrt{D}$ is ample (D being full supply depth), with a minimum of 3 feet. Extension of the masonry floor beyond the down stream bridge pier noses is hardly required, except when the foundation is porous sand, when the usual calculation for floor width and thickness has to be made.

(23) The old system of providing water cushions to canal falls by lowering the floor below the bed of the channel down stream involves great and unnecessary expense, in that the height of the weir and retaining walls, in fact all the parts of the structure, have to be increased by the depth of the cushions. In addition to this, the difficulty of getting in the foundations in a water bearing strata is greatly enhanced.

In most cases the water below the channel, which, except in notch falls, rises much faster than the level above the fall itself, forms an efficient water cushion ; or, if deemed insufficient, a cushion can be provided by building a dwarf subsidiary weir of suitable height across the floor, not at its extreme edge, where the action of the water would tend to damage the pitching on the earthen bed beyond the floor limit, but just sufficiently distant from the crest of the weir to ensure that the falling water drops within it. This distance is $2 \sqrt{d} \times H$ where $d =$ depth of film plus height due to velocity of approach.

This subsidiary cross wall in canal works should be provided with openings to drain the water out, when the canal is empty. With canal notch falls, the current being split up diminishes materially the velocity of the impact of the falling water.

(24) The abolition of water cushions renders it necessary that floors of large falls should be covered by durable stone set in cement mortar. To effect this economically, the surface stones, which should be large and deep, can be laid on ordinary hydraulic mortar and subsequently grouted with cement. The vertical joints will then be all of cement, and with this construction carefully done the impact and swirl of the falling water will have no damaging effect. It is clearly cheaper to do this than to go to the expense of lowering the floor for a water cushion. Ashlar, with fine joints, is the worst possible material for floors. Very exaggerated and erroneous ideas used to prevail regarding the destructive dynamical or kinetic effect of the impact of falling water, resulting in the design of the ogee curve profiles in the old Ganges canal works, which eventually had all to be altered to direct overfalls.

CHAPTER V

HYDRAULIC FORMULAS

(1) THE velocity of a jet issuing from an orifice under the head h, is

$$c\sqrt{2gh}. \qquad (1)$$

Fig. 1 represents a sluice opening in a dam with a free outlet, *i.e.*, the level of the tail water is below the orifice. In this case H is the mean head, or depth of centre of sluice-way below surface; h_1 and h_2 are the depths to the top and bottom of the orifice, l the width of the orifice whose depth is $h_2 - h_1$, or d. Then the discharge will be

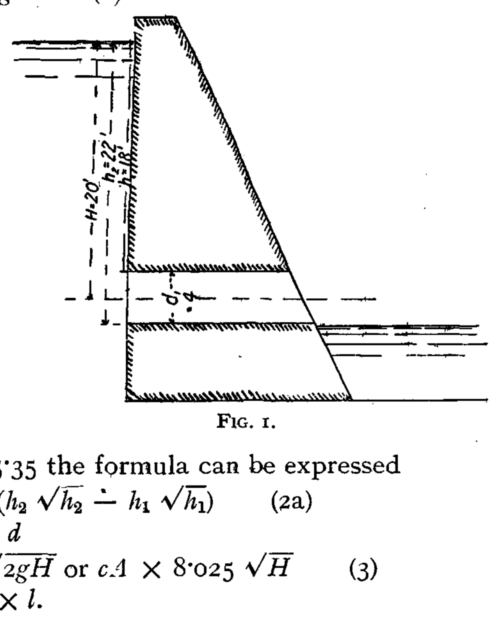


FIG. 1.

$$Q = cl\tfrac{2}{3}\sqrt{2g}\left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right) \qquad (2)$$

as $\sqrt{2g} = 8{\cdot}025$ and $\frac{2}{3}\sqrt{2g} = 5{\cdot}35$ the formula can be expressed as

$$Q = cl\, 5{\cdot}35\, (h_2\sqrt{h_2} - h_1\sqrt{h_1}) \qquad (2a)$$

When H is large compared with d

$$Q = \text{(approximately) } cA\sqrt{2gH} \text{ or } cA \times 8{\cdot}025\sqrt{H} \qquad (3)$$

A being the area of the orifice or $d \times l$.

(2) In the example given in Fig. 1, $H = 20$, $h_2 = 22$, $h_1 = 18$, $l = 4$ and $c = {\cdot}66$

whence $Q = {\cdot}66 \times 4 \times 5{\cdot}35\,(22 \times 4{\cdot}69 - 18 \times 4{\cdot}242)$ cubic feet.

$= 378{\cdot}8$ cubic feet per second by formula (2a).

The discharge by formula (3) will be

${\cdot}66 \times 16 \times 8{\cdot}025 \times 4{\cdot}472 = 379$ cubic feet per second.

Hence in this case the shorter formula, which always gives results somewhat in excess of (2), can well be used.

The velocity of the current through the orifice will be according to (2a)

$$\frac{c \times 5{\cdot}35\,(h_2\sqrt{h_2} - h_1\sqrt{h_1})}{d}$$

or, according to (3), $c\sqrt{2gH}$. In most cases, considering that the suitable value of the coefficient c is by no means known with anything like absolute precision, it would be a useless refinement to adopt the longer and more accurate formula (2) or (2a).

(**3**) Fig. 2 represents a similar case in which the sluice or orifice is submerged. Here the head H is the difference in level between the head and tail water.

The exact formula for the discharge through a submerged orifice is

$$Q = cl\ 5\cdot35 \left\{ (H\sqrt{H} - h_1\sqrt{h_1}) + \tfrac{3}{2}\sqrt{H}\,(h_2 - H) \right\} \quad (4)$$

In Fig. 2, $H = 16$, h_1 and h_2 being, as before, 18 and 22 feet,

whence $\quad Q = \cdot66 \times 4 \times 5\cdot35 \left\{ (16 \times 4 - 18 \times 4\cdot243) + \tfrac{3}{2}(4 \times 6) \right\}$

$$= 14\cdot124\ (-12\cdot37 + 36) = 333\cdot75 \text{ cubic feet.}$$

The approximate and more commonly used formula for discharge through a submerged orifice is the same as in (3), viz., $Q = cA\sqrt{2gH}$, H, however, being the difference between head and tail water, not the depth of the centre of the orifice below the surface. In this instance $Q = \cdot66 \times 16 \times 8\cdot025 \times 4 = 338\cdot98$ cubic feet, the difference being 1·3 per cent. in excess of the last. However, as already stated, in the present unsatisfactory state of hydraulic science the coefficients adopted are only approximate, so that the shorter formula is quite good enough for practical purposes.

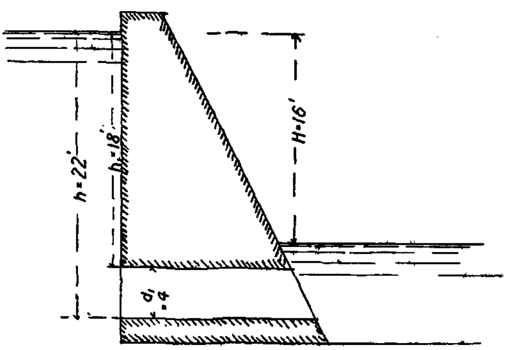


FIG. 2.

In the above cases the same coefficient, ·66, has been used for both free and submerged orifices. As will be seen later, the coefficient is higher when the orifice is submerged.

(**4**) In the two cases cited the question of the velocity of approach of the current, if any, has not been considered.

Where the section of the orifice is not very small compared with that of the channel of supply, this factor must be taken into account. The velocity of approach can be expressed in terms of the head h, expended in producing it, as follows:—

$h = \dfrac{V^2}{2g} = \cdot0155\ V^2$; in this V is the mean velocity of the current. If observed surface velocity (v) only is known, the mean velocity is commonly assumed to be $V = \cdot8v$. A more accurate formula is given in par. 31. The correct theoretical influence of the velocity of approach is best obtained by modifying the coefficient so as to include it, as follows:—

$$c_1 = c\sqrt{1 + \frac{h}{H}} \qquad (5)$$

This obviates two more formulas to replace (2) and (3).

Where the depth of the orifice is small as regards the head of water, the following will give sufficiently accurate results, viz. :—

$$Q = cA\ \sqrt{2g\,(H + h)} \qquad (6)$$

In this the head due to velocity of approach is simply added to the actual head of water, which is not strictly accurate.

(5) For example, in Fig. 1, supposing the mean velocity of the channel which supplies the sluice to be 3 feet per second, then the modified coefficient will become $c_1 = {}^{\cdot}66\ \sqrt{1 + \frac{h}{20}}$. Now h, or the head due to velocity of approach, is, as we have seen, equal to $\cdot 0155 \times 9 = \cdot 1395$, whence $c_1 = \cdot 66\ \sqrt{1\cdot 007} = \cdot 662$. The discharge will be increased in the first case of free outfall from 378·8 cubic feet to $378\cdot 8 \times \frac{662}{660}$ or to 379·95 cubic feet per second.

If formula (6) be used, the discharge with free outfall will be $\cdot 66 \times 16 \times 8\cdot 025\ \sqrt{20\cdot 1395} = 380\cdot 31$. With submerged orifice, $H = 16$ and $Q = \cdot 66 \times 16 \times 8\cdot 025 \times \sqrt{16\cdot 1395} = 340\cdot 45$ cubic feet.

These examples are useful as showing the discrepancy between the discharges worked out by the long or by the short formulas, and also showing that when H in either case is below $\frac{d}{5}$ (d being the depth of the orifice or sluice), the longer formulas (2) and (4) should be used instead of (3) and (6).

(6) In an orifice or sluice-way, contraction in the body of water passing through occurs on all four sides. The section of the fluid is thus reduced below that of the opening, and further, the velocity of the water passing is decreased, owing to the friction induced at the sides of the orifice, below what it should be according to theory. A reducing coefficient c has therefore to be used to include both these factors. The exact determination of this coefficient is a matter of great importance. For a circular orifice with thin sides, as a hole cut in a plank, the correct coefficient obtained by experiment is about ·62. As regards rectangular openings, the value of c varies with the shape of the orifice, as it has been found that an oblong hole gives a higher discharge than a square one of the same dimensions under the same head. This proves that the end contractions of the vertical sides have a greater effect than those at the top and bottom of the orifice. Hence a wide shallow orifice gives a higher discharge than a narrow and deep one. The recognition of this fact should influence the design of sluice ways, where it is desirable to reduce the head of water which must form above the sluice, so as to overcome the frictional resistance of the current in its passage. Cases illustrative of this point are noticed in Chap. XI., which deals with the design of masonry syphons.

(7) The coefficient of discharge through a submerged orifice is always higher than that with a free outfall. The same applies, but in a more

marked degree, to cases of overfalls where there is no contraction on the upper surface. Unfortunately, no experiments have been made to ascertain the exact value of the coefficient under various values of H, with which it naturally should vary. In "The Principles of Waterworks Engineering," p. 81, the coefficient for a submerged oblong orifice is stated to be experimentally ·67. The ordinary coefficient applicable to small square openings is ·62. In this particular case the opening is 2 feet × 6 inches, *i.e.*, *m*, or the multiplier = 1·05, whence for free outfall $c = \cdot 62 \times 1\cdot 05 = \cdot 65$. Deduction from these data would make the coefficient for a submerged orifice under moderate head, $\frac{\cdot 62 \times \cdot 67}{\cdot 65} = \cdot 639$, or, say, ·64.

In nearly all practical cases the orifice is submerged, which accounts for the high range of many coefficients which have been obtained by testing the discharge through openings on a large scale on actual works, and not on the minute dimensions of orifices and head, from which the original coefficients were deduced.

The rise of the coefficient due to submergence of an orifice shows that up to a certain point the rise in the back water has no influence on the discharge, which is the same as if the orifice were clear. There are no experiments which fix this limit, but it can safely be assumed at $\frac{1}{3}d$.

If c be the proper coefficient for a square orifice, then the multiplier due to the alteration of the shape to oblong will be as follows:—

when square,		$m = 1\cdot 00$
when length = twice depth	. . . .	$m = 1\cdot 03$.
" = 4 times depth	. . .	$m = 1\cdot 05$.
" = 10 "	. . .	$m = 1\cdot 07$.
" = 100 "	. . .	$m = 1\cdot 08$.

Thus, if the coefficient applicable to any case be assumed as ·8, doubling the width of the opening with same depth as before would increase the coefficient from ·8 to 1·03 × ·8, or ·824.

(8) The following are the coefficients applicable to different classes of orifices or sluices:—

	C
1. Orifices in a thin plate, such as hole in planks or small iron sluice gates not at base of reservoir, circular pipe outlets, etc.	= ·62
Same submerged	= ·64
2. Sluices without side walls, as tank sluices which open into wide culverts or discharge direct into the lower basin through a dam	= ·66
3. Sluices in lock gates or moderate sized sluice openings situated at bottom of a reservoir, as many tank sluices are . . .	= ·84
4. Narrow bridge openings, as canal heads and undersluices of small spans, say, 4 feet to 6 feet, with side walls, bottom sluices in dam	= ·90
5. Large sluice openings with side walls, as wide undersluice openings of modern type, 15 feet to 20 feet span . . .	= ·94

6. Wide openings with bed level with that of the reservoir, *i.e.*, undersluices or escape heads of exceptional size; also wide bridge openings . . . . = ·96

(9) From the foregoing it will be seen that the value of the coefficient is affected, firstly, by the size of the opening; secondly, by the provision of side walls beyond the sluice gate to guide the issuing current; and, thirdly, by the suppression of the bottom contraction which occurs when the opening is at base of the reservoir, with its floor level with that of the latter.

Thus a sluice built half way up a lock gate will have a coefficient of ·66, while the same if constructed just above floor level will have an increased coefficient of ·84. Where a dam is pierced by sluice openings, as the Bhatgarh and Assuan Dams, the sills of the sluices will not be flush with that of the river, which is uneven; consequently the bottom contraction will exist, and the lower coefficient of ·90 will be applicable. On the other hand, undersluices in weirs are built on a masonry floor, flush or below the normal river bed, and the higher coefficient of ·94 would be applicable. It would be an easy matter to definitely settle the exact suitable coefficients by instituting experiments on a large scale. This could be done with canal head regulators, which are of various sizes in many existing works in India. The exact coefficient of discharge could be found by manipulating the gates and noting the resulting level of the canal, from which the discharge could be estimated with great accuracy, and the coefficients obtained under varying conditions of size of aperture and head of water.

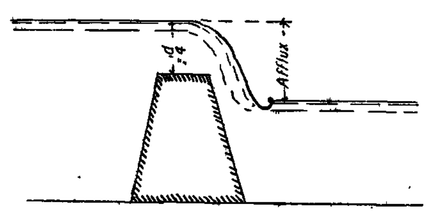


FIG. 3.

The construction of curved piers, noses, or cutwaters at both ends effect a considerable increase in the coefficient in case of a bridge; bluff pier ends, on the other hand, cause a diminution of efficiency of discharge from ·90 to ·86 (*vide* Molesworth's "Pocket Book"). Sharp curved cutwaters are almost as necessary below as above a bridge or sluice-way, as they prevent pooling, which is certain to occur where the water section is abruptly enlarged. With regard to the comparative size of orifices, it is clear that the coefficient must vary more or less with the wetted perimeter, *i.e.*, with $\sqrt{R}$, as is recognised in the case of pipes.

(10) When water falls over a weir, as in Fig. 3, the conditions are different; the top contraction here is non-existent, and in the equation (2) $h_1 = o$ and $h_2 = d$, the depth of the film, whence the discharge will become

$$Q = \tfrac{2}{3}cl\sqrt{2gd} \times d = 5{\cdot}35\, cld^{\frac{3}{2}} \qquad (7)$$

(11) If piers or obstructions are built on the weir, or the weir opening is narrow in proportion to the width of supply channel, the effective length

of overfall becomes contracted, and l_1 will $= l - \cdot 1nd$. With pointed cutwaters to piers the contraction is suppressed entirely.

In a long open weir, where l exceeds $20d$, this end contraction has an inappreciable effect and can be neglected.

(**12**) In free overfalls, the common practice is to adopt Francis' Lowell experimental coefficient of ·623. With this coefficient the discharge of free overfall will be

$$Q = 3{\cdot}333\, ld^{\frac{3}{2}} \qquad (8)$$

This coefficient is generally adopted, and is believed to be much nearer the truth than Castel's coefficient of ·666, which has been used by Jackson in the "Hydraulic Manual." This latter is undoubtedly much too high, except possibly in the case of a very small value of d, discharging over a thin plank. A coefficient varying with the depth of film would probably be more strictly in accordance with experiments. These experiments have, however, been undertaken on a very small scale, and a coefficient which may be truly applicable in cases of small depths of 2 feet or 3 feet passing over a narrow plank, the weir opening being likewise a small proportion of the normal width of the supply channel, will not prove so under actual practical conditions.

The coefficient undoubtedly does decrease to some extent in inverse proportion with the depth of film, but on the other hand with a wide flat-topped masonry weir the increment is in the opposite sense, the fall in one case balancing to a greater or less degree the rise in the other, so that, pending further exhaustive experiments on a proper large scale, the adoption of one value of c of ·623 for all depths appears sufficiently accurate for practical purposes. The letter Q signifies either the whole discharge, or in the case of a weir, that per foot run, the absence of A or of l denoting which is meant.

Besides which, this value is recognised generally as suitable, while that of Castel's, viz., ·666, is not. In the "Madras Manual," the formula having a still lower value of c, viz., $Q = 3{\cdot}25d\sqrt{d}$, is adopted. "Per foot run" is sometimes designated *unit* discharge.

(**13**) In Fig. 3 the depth of film is shown as 4 feet. The discharge per foot run of weir will then be $Q = 3{\cdot}333 \times 4\sqrt{4} = 26{\cdot}67$ cubic feet.

In this, the velocity of approach, which has always to be considered except in waste weirs to reservoirs of still water, has been neglected.

The easiest method is to modify the coefficient c, using formula (8). Let h equal height due to the mean velocity of approach, or $\cdot 0155\, V^2$, then this modified coefficient c_1 is obtained by use of the following formula :—

$$c_1 = c \times \left\{ \left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\} \qquad (9)$$

For example, supposing in the case already cited the mean velocity of the current to be $3\frac{1}{2}$ feet per second, then $h = \cdot 0155 \times (3{\cdot}5)^2 = \cdot 19$ feet, and by reference to Table I., in which formula (9) has been worked out, the multiplier of the coefficient with $d = 4$ and $h = \cdot 19$ is 1·062, and the discharge modified to include effect of velocity of approach will be $26\cdot 666 \times 1\cdot 062 = 28\cdot 32$ cubic feet per foot run. The value (·0155) is $\frac{1}{2g} = \frac{1}{64\cdot 4}$.

TABLE I.—MULTIPLIERS IN FORMULA (9) OF FREE OVERFALLS FOR VELOCITY OF APPROACH, VIZ. $\left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}$.

	Mean Velocities of Approach.										
	1	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	6	7
	Corresponding Values of h, viz.: $\cdot 0155V^2$.										
d	·0155	·035	·062	·097	·1395	·190	·248	·314	·3875	·558	·7595
$\frac{1}{2}$	1·041	1·092	1·148	1·219	1·299	1·387	1·460	1·579	1·682		
1	1·021	1·047	1·078	1·119	1·164	1·215	1·270	1·330	1·393		
$1\frac{1}{2}$	1·014	1·029	1·054	1·082	1·114	1·151	1·191	1·234	1·280		
2	1·011	1·024	1·040	1·061	1·089	1·117	1·158	1·182	1·224		
$2\frac{1}{4}$	1·010	1·021	1·037	1·056	1·079	1·104	1·133	1·164	1·198		
$2\frac{1}{2}$	1·009	1·019	1·033	1·051	1·072	1·095	1·117	1·150	1·180		
3	1·008	1·017	1·028	1·043	1·061	1·080	1·102	1·127	1·153		
$3\frac{1}{2}$	1·107	1·014	1·024	1·037	1·052	1·070	1·089	1·111	1·134		
$3\frac{3}{4}$	1·107	1·013	1·023	1·035	1·050	1·066	1·084	1·104	1·125		
4	1·006	1·012	1·021	1·033	1·047	1·062	1·079	1·097	1·118		
$4\frac{1}{2}$	1·006	1·011	1·019	1·029	1·041	1·055	1·071	1·088	1·107		
5	1·005	1·010	1·017	1·026	1·037	1·050	1·065	1·080	1·097		
$5\frac{1}{4}$	1·005	1·010	1·016	1·026	1·036	1·048	1·062	1·077	1·093		
$5\frac{1}{2}$	1·005	1·010	1·015	1·025	1·034	1·046	1·059	1·073	1·089		
6	1·004	1·009	1·014	1·023	1·031	1·042	1·055	1·068	1·082	1·114	1·151
$6\frac{1}{2}$	1·004	1·008	1·014	1·021	1·029	1·039	1·050	1·063	1·076		
$6\frac{3}{4}$	1·004	1·008	1·013	1·020	1·028	1·038	1·048	1·060	1·073		
7	1·003	1·007	1·013	1·019	1·027	1·037	1·046	1·059	1·071	1·099	1·134
$7\frac{1}{2}$	1·003	1·006	1·012	1·018	1·025	1·035	1·044	1·055	1·067	1·093	1·123
8	1·003	1·006	1·012	1·016	1·023	1·032	1·042	1·052	1·063	1·088	1·116
$8\frac{1}{2}$	1·003	1·005	1·011	1·015	1·022	1·030	1·039	1·049	1·059		
9	1·003	1·005	1·010	1·014	1·021	1·029	1·037	1·046	1·056	1·079	1·105
$9\frac{1}{2}$	1·002	1·005	1·009	1·014	1·020	1·027	1·035	1·044	1·054		
10	1·002	1·005	1·009	1·013	1·019	1·025	1·034	1·042	1·051	1·072	1·095
$10\frac{1}{2}$	—	—	1·008	1·012	1·018	1·023	1·033	1·041	1·049	1·068	1·091
12	—	—	1·007	1·011	1·016	1·022	1·028	1·035	1·043	1·060	1·080
$13\frac{1}{2}$	—	—	—	—	1·015	1·020	1·025	1·030	1·039	1·057	1·072
15	—	—	—	—	—	1·018	1·023	1·028	1·035	1·049	1·065
20	—	—	—	—	—	—	1·018	1·022	1·027	1·038	1·050

(**14**) In case actually observed surface velocities are given in lieu of mean velocities; they can be reduced to mean velocities by use of formula (15), par. 31, and the results obtained by interpolation from Table I., or else the value of c can be altered by use of formula (9).

TABLE II.—DISCHARGES PER FOOT RUN OF FREE OVERFALLS.

$$c = \cdot 623. \quad (\text{FORMULA}) \; 3\cdot 333 d^{\frac{3}{2}} \left\{ \left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\}. \qquad (10)$$

$$h = \cdot 0155 V^2.$$

		MEAN VELOCITIES OF APPROACH IN FEET PER SECOND.										
d	$3\cdot 333d^{\frac{3}{2}}$	1	1½	2	2½	3	3½	4	4½	5	6	7
½	1·178	1·227	1·287	1·353								
1	3·333	3·403	3·489	3·593	3·730							
1½	6·123	6·209	6·301	6·454	6·625	6·821						
2	9·427	9·531	9·653	9·804	10·002	10·266						
2¼	11·249	11·361	11·485	11·665	11·868	12·138	12·419	12·745	13·094	13·476		
2½	13·178	13·296	13·428	13·612	13·849	14·026	14·429	14·719				
3	17·319	17·457	17·613	17·804	18·064	18·375	18·722	19·085	19·518	19·969		
3½	21·824	21·977	22·129	22·348	22·631	22·959	23·352	23·766				
3¾	24·204	24·373	24·529	24·760	25·051	25·414	25·801	26·237	27·721	27·229		
4	26·664	26·824	26·984	27·224	27·544	27·917	28·317	28·770	29·250			
4½	31·817	31·976	32·166	32·421	32·739	33·121	33·566	34·076	34·617	35·221		
5	37·264	—	37·636	37·898	38·233	38·643	39·127	39·686	40·245	40·879		
5¼	40·094	—	40·494	40·735	41·136	41·537	42·018	42·579	42·982	43·822		
5½	42·991	—	43·422	43·636	44·066	44·452	44·969	45·528	46·129	47·075		
6	48·985	—	49·425	49·671	50·111	50·504	51·042	51·679	52·316	53·001	54·579	56·372
6½	55·234.	—	55·676	56·007	56·394	56·836	57·388	57·996	58·713	59·431		
6¾	58·451	—	58·919	59·211	59·621	60·088	60·672	61·183	61·958	62·718		
7	61·728	—	62·160	62·531	62·901	63·395	64·012	64·691	65·370	66·111	67·864	70·006
7½	68·458	—	—	69·280	69·691	70·170	70·786	71·471	72·223	73·045	74·225	76·878
8	75·417	—	—	76·322	76·624	77·152	77·831	78·585	79·339	80·168	82·061	84·203
8½	81·597	—	—	82·505	83·836	84·414	85·075	85·818	86·651	87·470		
9	89·991	—	—	90·891	91·206	91·881	92·601	93·321	94·131	95·031	97·101	99·413
9½	97·594	—	—	98·472	98·960	99·545	100·23	101·01	101·88	102·86		
10	105·398	—	—	106·24	106·77	107·40	108·03	108·98	109·82	110·67	112·96	115·44
10½	113·402	—	—	114·19	114·76	115·44	116·01	117·14	118·05	118·96	121·11	123·72
11	121·63	—	—	122·60	—	123·82	—	125·52	—	127·47	129·79	132·58
12	138·425	—	—	139·44	140·00	140·68	141·44	142·33	143·27	144·38	146·80	149·47
13	156·08	—	—	—	—	158·42	—	160·14	—	162·33	165·51	167·32
13½	165·324	—	—	—	—	167·80	168·63	169·46	170·28	171·77	174·75	177·23
14	174·43	—	—	—	—	177·05	—	178·62	—	181·06	183·15	186·64
15	193·64	—	—	—	—	—	197·12	198·08	199·05	200·41	203·12	206·22
16	213·31	—	—	—	—	—	—	217·78	—	220·56	223·55	226·96
18	254·53	—	—	—	—	—	—	258·86	—	261·15	263·93	267·00
20	298·13	—	—	—	—	—	—	303·59	304·66	306·03	309·30	313·06

(**15**) In Table II., formula (8) is worked out in connection with formula (9) *i.e.*, Table I. The second column is formula (8), the items of which are separately multiplied by the proper multipliers found in Table I. The products fill the remaining columns. For values of V above 7 feet, or d above 20 feet, special calculations will have to be made.

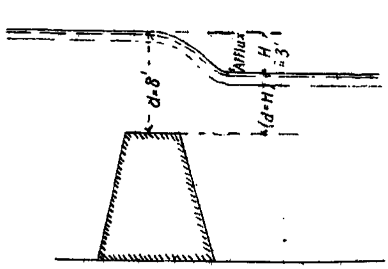


FIG. 4.

(**16**) Fig. 4 represents a submerged or drowned weir. As in the case of a submerged orifice, the head H is the difference in level of the head and tail water, or the afflux. The depth of the film is termed, as before, d, and the submerged portion of the film is $d - H$.

The passing film thus consists of two portions, the upper having a free overfall and the lower being what can possibly be considered as a submerged orifice, but without any top contraction. There can likewise be no bottom contraction or friction in the upper portion. Some authorities, as Jackson, calculate the discharge of each portion with two separate values of c, the upper by formula (8), the lower by formulas (2) or (3). Jackson uses the coefficient ·66 for the upper and ·62 for the lower. The use of so low a coefficient as ·62 for the lower orifice portion is clearly quite indefensible; it undoubtedly should vary from ·7 to ·9 or more. In the case of an ordinary orifice, the coefficient ·84 given in par. 8 would seem suitable. If, however, a high coefficient be used, the discharge will, in many cases, exceed that of a free overfall, which is clearly impossible—hence the reason of Jackson's adoption of the low coefficient.

But further, cases do occur where, by using this method and these coefficients, the resulting discharge will exceed that of a free overfall. The actual conditions of the two portions of the film are so dissimilar from the hypothetical, that this method of estimating the discharge cannot be satisfactory, and the only alternative is to adopt one coefficient for the whole film, neglecting any imaginary horizontal division in the same.

(17) The theoretical formula for the discharge of a submerged fall, using one coefficient for the whole film above weir crest, is

$$Q = cl\left\{\sqrt{2gH} \times \tfrac{1}{3}(3d - H)\right\} \qquad (11)$$

taking c and l as unity, and $\sqrt{2g}$ being 8·025, the formula becomes

$$Q = 8{\cdot}025\sqrt{H} \times \tfrac{1}{3}(3d - H) \qquad (11a)$$

When velocity of approach has to be taken into consideration, as is necessary in all cases of running water, as canals and rivers, the formula for discharge is

$$Q = cl\sqrt{2g}\left[(d + h)\sqrt{H + h} - \tfrac{1}{3}\left\{(H + h)^{\frac{3}{2}} + 2h^{\frac{3}{2}}\right\}\right] \qquad (12)$$

This unwieldy formula has been worked out in Table V., par. 22.

(18) The following method, first instituted by Herschel, an American hydraulician, from the experiments of Francis and of Fteley and Stearns, appears to give satisfactory results within certain limits, and has the great merit of simplicity.

The formula for free discharge per foot run, or $Q = 3{\cdot}33d^{\frac{3}{2}}$ is modified as follows:—

$$Q = 3{\cdot}33\,(nd)^{\frac{3}{2}} \qquad (8a)$$

The multiplier n varies in the proportion of $d : (d - H)$. The results are liable to a probable error of about one unit in the second decimal place when $(d - H)$ is less than ·$2H$, and to greater errors as the table progresses, values of n under ·7 being in particular uncertain.

The following table of values of n has been derived from Merriman's "Hydraulics":—

TABLE III.—FACTORS (n) FOR SUBMERGED WEIRS.

In formula (8a) for discharge of submerged weirs, per foot run, $Q = 3{\cdot}33\,(nd)^{\frac{3}{2}}$

$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n
0·00	1·000	0·18	0·989	0·38	0·935	0·58	0·856
·01	1·004	·20	·985	·40	·929	·60	·846
·02	1·006	·22	·980	·42	·922	·62	·836
·04	1·007	·24	·975	·44	·915	·64	·824
·06	1·007	·26	·970	·46	·908	·66	·813
·08	1·006	·28	·964	·48	·900	·70	·787
·10	1·005	·30	·959	·50	·892	·75	·750
·12	1·002	·32	·953	·52	·884	·80	·703
·14	·998	·34	·947	·54	·875	·90	·574
·16	·994	·36	·941	·56	·866	1·00	0·000

(**19**) The question of velocities of approach and their values appears not to have received attention in works on hydraulics, but is passed over in discreet silence; consequently the only way to deal with the question is to use the same values as appear in Table II., multiplying up all the columns by the new factor $n^{\frac{3}{2}}$. This has been done in the following Tables, which give a sufficiently approximate value to work on, until reliable coefficients are made available by means of experiments. This is what the profession have been waiting for during many decades, but waiting in vain. Experiments of this kind could best be made by the Irrigation Branch of the Public Works Department of the Government of India, who have an infinity of very large volumes of water to deal with and every possible description of weirs, canal falls, canal heads, and other works, the accurate discharge through which, under different heads, could easily be gauged. Possibly this has already been done, but if so, like most good things done by the Department, it is carefully buried in the musty archives at Calcutta or Simla until some benefactor to the human race, as Mr. R. Buckley, unearths them and publishes the results for the benefit of the profession at large.

In the following Tables, numbered IV., the third column contains the values of n obtained from Table III. raised to the power of $\frac{3}{2}$, the fourth column contains the products of the corresponding discharge for the same value of d as are given in Table II. multiplied by those of $n^{\frac{3}{2}}$ in the third column.

The remaining columns include the velocity of approach, and are the reciprocal amounts in Table II. multiplied also by $n^{\frac{3}{2}}$.

Table IV. (Series I.).—Table of Discharges per Foot Run of Submerged Weirs.

$H = \frac{1}{2}$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.				
			0	1	2	3	4
2	·75	·65	6·13	6·19	6·37	6·67	
$2\frac{1}{2}$	·8	·58	7·64	7·71	7·89	8·13	
3	·83	·54	9·35	9·42	9·61	10·42	
$3\frac{1}{2}$	·85	·51	11·12	11·21	11·40	11·71	
4	·87	·48	12·79	12·87	13·06	13·40	13·82
$4\frac{1}{2}$	·88	·46	14·64	14·71	14·91	15·23	15·67
5	·90	·43	16·02	16·30	16·61	17·06	17·57

Table IV. (Series II.).—Discharges per Foot Run of Submerged Falls.

$H = 1$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.							
			0	1	2	3	4	5	6	7
$1\frac{1}{2}$	·3	·94	5·7	5·8	6·1	6·4				
2	·5	·84	7·9	8·0	8·2	8·6				
$2\frac{1}{2}$	·6	·79	10·2	10·3	10·6	10·9	11·4			
3	·66	·73	12·7	12·8	13·0	13·5	14·0			
$3\frac{1}{2}$	·7	·70	15·3	15·4	15·6	16·0	16·6			
4	·75	·65	17·3	17·4	17·7	18·1	18·6	19·0		
$4\frac{1}{2}$	·77	·63	20·0	20·1	20·4	20·8	21·5	22·1		
5	·8	·59	22·0	—	22·3	22·8	23·4	24·1		
$5\frac{1}{2}$	·82	·56	24·1	—	24·4	24·9	25·5	26·3		
6	·83	·54	26·4	—	26·8	27·3	27·9	28·6	29·5	
7	·85	·51	31·5	—	31·9	32·3	33·0	33·7	34·6	
8	·87	·48	36·2	—	36·6	37·0	37·7	38·5	34·6	

TABLE IV. (SERIES III.).—DISCHARGE PER FOOT RUN OF SUBMERGED FALLS.

$H = 1\frac{1}{2}$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.							
			0	1	2	3	4	5	6	7
$2\frac{1}{2}$	·4	·93	12·2	12·4	12·6	13·0	13·6			
3	·5	·84	14·5	14·8	15·0	15·4	16·0			
$3\frac{1}{2}$	·57	·81	17·6	17·8	18·1	18·6	19·2			
4	·62	·76	20·2	20·4	20·7	21·2	21·6			
$4\frac{1}{2}$	·67	·72	22·9	23·1	23·3	23·8	24·5	25·3		
5	·7	·70	26·1	—	26·5	27·0	27·8	28·1		
$5\frac{1}{2}$	·73	·67	28·8	—	29·2	29·8	30·5	31·5		
6	·75	·65	31·8	—	32·3	32·8	33·6	34·6		
$6\frac{1}{2}$	·77	·63	30·9	—	31·3	31·8	32·5	33·4		
7	·79	·59	36·4	—	36·9	37·4	38·2	39·0		
$7\frac{1}{2}$	·8	·58	39·7	—	40·2	40·7	41·4	42·4	43·0	44·6
8	·81	·57	43·0	—	43·5	43·9	44·8	45·6	46·8	48·0
$8\frac{1}{2}$	·82	·56	45·7	—	46·2	47·3	48·0	48·9		
9	·83	·54	48·6	—	49·0	49·6	50·4	51·3	52·3	53·7
$9\frac{1}{2}$	·84	·53	51·7	—	52·2	52·7	53·5	54·5		
10	·85	·51	53·7	—	54·2	54·8	55·6	56·4	57·1	58·9
11	·86	·49	59·6	—	60·0	60·7	61·3	62·5	63·6	65·0
12	·87	·48	66·4	—	66·9	67·5	68·3	69·3	70·5	71·7

TABLE IV. (SERIES IV.).—DISCHARGE PER FOOT RUN OF SUBMERGED FALLS.

$H = 2$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.								
			0	1	2	3	4	5	6	7	8
$2\frac{1}{2}$	·2	·98	12·9	13·0	13·3	13·7					
3	·33	·92	15·9	16·1	16·5	16·9	17·6				
$3\frac{1}{2}$	·4	·89	19·5	19·7	20·0	20·5	21·3				
4	·5	·84	22·3	22·5	22·8	23·4	24·2				
$4\frac{1}{2}$	·55	·81	25·8	25·9	26·2	26·8	27·5	28·5			
5	·6	·78	29·0	—	29·5	30·1	31·0	31·8			
$5\frac{1}{2}$	·64	·75	32·1	—	32·7	33·3	34·1	36·1			
6	·67	·72	35·3	—	35·8	36·4	37·2	38·2			
$6\frac{1}{2}$	·69	·71	39·2	—	39·8	40·3	41·2	42·2			
7	·71	·68	42·0	—	42·5	43·1	44·0	45·0	46·1	47·6	
$7\frac{1}{2}$	·73	·67	45·8	—	46·4	47·0	47·9	48·9	49·7	51·5	
8	·75	·65	48·0	—	49·6	50·1	51·1	52·1	53·3	54·7	
$8\frac{1}{2}$	·76	·63	51·4	—	52·0	53·2	54·1	55·1			
9	·78	·61	54·9	—	55·4	56·1	57·0	58·0	59·2	62·6	
$9\frac{1}{2}$	·79	·60	58·5	—	59·1	59·7	60·6	61·7			
10	·80	·59	62·2	—	62·7	63·4	64·3	65·3	66·7	68·0	
11	·82	·56	68·1	—	68·6	69·3	70·3	71·5	72·7	74·2	
12	·83	·54	74·7	—	75·3	75·9	76·8	78·0	79·2	81·0	

TABLE IV. (SERIES V.)

$H = 2\frac{1}{2}$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.							
			0	2	3	4	5	6	7	8
$3\frac{1}{2}$	.28	.95	20.7	21.2	21.8	22.6				
4	.37	.91	24.4	24.7	25.4	26.2				
$4\frac{1}{2}$	.44	.87	27.7	28.2	28.8	29.6	30.6			
5	.5	.84	31.3	31.9	32.5	33.3	34.3			
$5\frac{1}{2}$	.53	.82	35.2	35.8	36.4	37.3	38.6			
6	.58	.79	38.7	39.2	39.9	40.8	41.9	42.5		
$6\frac{1}{2}$	.61	.77	42.5	43.1	43.7	44.7	45.8			
7	.64	.75	46.3	46.9	47.5	48.5	49.6	50.9	52.50	
$7\frac{1}{2}$	.66	.73	50.0	50.6	51.2	52.2	53.3	54.2	56.1	
8	.69	.71	53.5	54.2	54.8	55.7	56.9	58.2	59.8	
$8\frac{1}{2}$	.7	.70	57.1	57.7	59.1	60.0	61.2			
9	.72	.68	61.2	61.8	62.5	63.4	64.6	66.0	67.6	
$9\frac{1}{2}$	.73	.67	65.4	65.9	66.7	67.6	68.9			
10	.75	.65	68.5	69.0	69.8	70.8	71.9	73.4	75.0	
11	.77	.63	76.6	77.2	78.0	79.1	80.3	81.7	83.5	
12	.80	.59	81.7	82.3	83.0	84.0	85.2	86.6	88.2	
13	.81	.57	89.0	—	90.3	91.8	92.5	94.3	95.4	
14	.82	.56	97.6	—	99.0	100.0	101.4	102.6	104.5	
15	.83	.54	104.6	—	—	107.0	108.2	109.7	111.35	
16	.84	.53	113.0	—	—	115.4	116.9	118.5	120.3	
18	.85	.51	130.3	—	—	132.0	133.2	134.7	136.2	
20	.87	.48	142.9	—	—	145.7	146.9	148.5	150.3	

TABLE IV. (SERIES VI.)

$H = 3$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.						
			0	2	3	4	5	6	7
4	.25	.96	25.6	26.1	26.8	27.6			
$4\frac{1}{2}$	.33	.93	29.6	30.1	30.8	31.7			
5	.4	.89	33.2	33.7	34.4	35.3	36.4		
$5\frac{1}{2}$	.45	.87	34.8	35.4	36.1	37.0	38.1		
6	.5	.84	41.1	41.7	42.4	43.4	44.5	45.8	
$6\frac{1}{2}$	.54	.82	45.3	45.9	46.6	47.5	48.7		
7	.57	.80	49.4	50.0	50.7	51.7	52.8	54.3	56.0
$7\frac{1}{2}$	.60	.78	53.4	54.0	54.7	55.7	56.9	57.9	59.0
8	.62	.76	57.3	58.0	58.6	59.7	60.9	62.3	64.0
$8\frac{1}{2}$	.64	.75	61.2	61.9	63.4	64.4	65.6		
9	.66	.73	65.7	66.3	67.0	68.1	69.4	70.9	72.6
$9\frac{1}{2}$	.68	.71	69.3	69.9	70.7	71.7	73.0		
10	.7	.70	73.6	74.3	75.2	76.3	77.5	79.0	80.7
$10\frac{1}{2}$	.72	.68	77.1	77.6	78.5	79.6	80.7	82.3	84.1
11	.73	.67	81.5	81.9	82.9	84.2	85.4	87.3	88.9
12	.75	.65	90.0	90.6	91.4	92.5	93.9	95.4	97.1
13	.77	.63	98.3	—	99.8	100.8	102.2	103.9	105.4
14	.79	.60	104.6	—	106.1	107.1	108.6	109.9	112.0
15	.80	.59	114.1	—	—	116.9	118.1	119.7	121.8
16	.81	.57	121.6	—	—	124.1	125.7	127.4	129.4
18	.83	.54	137.4	—	—	139.8	141.0	142.5	144.2
20	.85	.51	152.0	—	—	154.7	155.4	157.8	159.6

TABLE IV. (SERIES VII.)

$H = 3\frac{1}{2}$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.							
			0	2	3	4	5	6	7	8
5	·3	·94	35·0	35·5	36·3	37·3	38·4			
5½	·36	·91	39·1	39·7	40·4	41·4	43·8			
6	·42	·88	43·1	43·7	44·4	45·5	46·6	48·0		
6½	·46	·86	47·5	48·1	48·9	49·9	51·1			
7	·50	·84	51·8	52·5	53·2	54·3	54·9	55·5	58·8	
7½	·53	·82	56·1	56·8	57·5	58·6	59·9	60·9	63·0	
8	·56	·81	61·1	61·8	62·5	63·6	64·9	66·4	69·0	
8½	·59	·78	63·6	64·3	65·8	66·9	68·2			
9	·61	·77	69·3	70·0	70·7	71·8	73·1	74·7	76·5	
9½	·63	·76	74·2	74·8	75·6	76·8	78·2	80·6	83·5	
10	·65	·74	78·0	78·6	79·5	80·64	81·9	83·6	85·4	
11	·68	·72	87·5	88·3	89·1	90·4	91·8	93·4	95·4	
12	·71	·69	95·5	96·2	97·1	98·2	99·6	101·3	103·1	
13	·73	·67	104·6	—	106·1	107·3	108·8	110·9	112·1	
14	·75	·65	113·4	—	115·1	116·1	117·7	119·0	121·3	
15	·77	·63	122·0	—	—	124·8	126·2	128·0	130·0	
16	·78	·61	130·1	—	—	132·8	134·5	136·4	138·2	
18	·80	·59	150·1	—	—	152·7	154·0	155·6	157·5	
20	·82	·56	167·0	—	—	170·6	171·4	173·2	174·4	

TABLE IV. (SERIES VIII.)

$H = 4$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.						
			0	2	3	4	5	6	7
6	·33	·93	45·6	46·3	47·0	48·1	49·7	50·8	
6½	·38	·90	49·7	50·4	51·2	52·2	53·5		
7	·43	·88	54·3	55·0	55·8	56·9	58·2	59·7	61·6
7½	·47	·86	58·9	59·6	60·3	61·5	62·8	63·8	66·1
8	·50	·84	63·4	64·1	64·8	66·0	67·3	68·9	70·7
8½	·53	·82	67·0	67·6	69·2	70·4	71·7		
9	·55	·81	72·9	73·7	74·4	75·6	77·0	78·6	80·5
9½	·58	·79	77·1	77·8	78·6	79·8	81·3		
10	·60	·78	82·2	82·9	83·7	85·0	86·3	88·1	89·6
11	·64	·75	91·2	92·0	92·9	94·1	95·5	97·3	99·4
12	·67	·72	99·7	100·3	101·3	102·6	103·9	105·7	107·6
13	·69	·71	110·8	—	112·5	113·7	115·3	117·5	118·8
14	·71	·69	120·4	—	122·2	123·3	124·9	126·4	128·8
15	·73	·67	129·7	—	—	132·7	134·2	136·1	138·2
16	·75	·65	138·6	—	—	141·6	143·4	145·3	147·5
18	·78	·61	155·2	—	—	157·9	159·3	161·0	162·9
20	·80	·59	175·9	—	—	179·1	180·6	182·5	184·7

TABLE IV. (SERIES IX.)

$H = 4\frac{1}{2}$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.							
			0	1	2	3	4	5	6	7
6	·25	·96	47·0	—	47·6	48·5	49·6	50·9		
$6\frac{1}{2}$	·3	·94	52·0	—	52·6	53·4	54·5	55·8		
7	·36	·91	56·2	—	56·9	57·7	58·9	60·2	61·7	
$7\frac{1}{2}$	·4	·89	60·9	—	61·6	62·4	63·6	65·0	66·1	68·4
8	·44	·87	65·6	—	66·4	67·1	68·0	69·7	71·4	73·2
$8\frac{1}{2}$	·47	·86	70·2	—	71·0	72·8	73·8	75·2		
9	·5	·84	75·6	—	76·3	77·1	78·4	79·8	81·6	83·5
$9\frac{1}{2}$	·53	·83	81·0	—	81·7	82·6	83·8	85·3		
10	·55	·81	85·4	—	86·0	87·0	88·2	89·7	91·4	93·5
11	·59	·78	94·9	—	95·6	96·6	97·9	99·4	101·2	103·4
12	·62	·76	105·2	—	105·9	106·9	108·2	109·7	111·5	113·6
13	·65	·74	115·5	—	—	117·2	118·5	120·1	122·4	123·8
14	·68	·72	125·6	—	—	127·4	128·6	130·3	131·9	134·3
15	·70	·70	135·5	—	—	—	138·6	140·3	142·2	144·3
16	·72	·68	145·0	—	—	—	148·1	151·0	152·0	154·4
18	·75	·65	165·4	—	—	—	168·3	169·7	171·5	173·5
20	·77	·63	187·8	—	—	—	191·2	192·8	194·8	197·2

TABLE IV. (SERIES X.)

$H = 5$.

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach.						
			0	2	3	4	5	6	7
7	·3	·94	58·0	58·8	59·6	60·8	62·1		
$7\frac{1}{2}$	·33	·92	63·0	63·7	64·6	65·7	67·2		
8	·37	·91	68·6	69·4	70·2	71·4	73·0	74·7	76·6
$8\frac{1}{2}$	·41	·89	72·6	73·4	75·1	76·4	77·8		
9	·44	·87	78·3	79·1	79·9	81·2	82·7	84·5	86·5
$9\frac{1}{2}$	·47	·86	83·9	84·7	85·6	86·8	88·5		
10	·50	·84	88·5	89·2	89·7	91·5	93·0	94·9	97·0
11	·54	·82	99·7	100·5	101·5	102·9	104·5	106·4	108·7
12	·58	·79	109·3	110·1	111·1	112·4	114·1	116·0	118·1
13	·61	·77	120·2	—	122·0	123·3	125·0	127·4	128·8
14	·64	·75	130·8	—	132·8	134·0	135·8	137·4	140·0
15	·67	·72	139·4	—	—	142·6	144·3	146·5	148·5
16	·70	·70	149·3	—	—	152·4	154·4	156·5	158·9
18	·72	·68	173·0	—	—	176·0	177·6	179·5	181·2
20	·75	·65	193·8	—	—	197·3	199·0	201·0	203·3

(20) The mean velocity of approach, Va, which we have been considering, is naturally some function of that of the normal river current. Now the discharge $Q = AV = A_1 Va$, in which expression A_1 is the area of the water section at or near the weir or obstruction. The value of A_1 may be more or may be less than that of A, or else, as with dwarf river weirs, it may be

considered equal to it. In deep reservoirs which do not silt up to any great extent Va is less than V. A proper appraisement of the value of A_1 is essential to the correct valuation of Va, and in estimating the area of A_1 the depth of film passing the weir crest must be assumed tentatively.

Another point to be noted is that the mean velocity of approach of a large wide river varies in different parts of the water section, and in such cases the river and the weir should be divided into parts, each with its own velocity of approach, and the multiplier varied accordingly. The reduction of observed surface to mean velocities is treated of in par. 31.

(21) In the above treatment of weirs we have dealt with open weirs, presumably across rivers; in canals, however, such weirs are not now designed, the great superiority of the notch type of weirs, which enables the water in the upper reach to be held up to normal level at whatever depth it may be, rendering its adoption universal for falls in up-to-date canals of any pretension.

In a notch fall, the triangular, or rather trapezoidal, openings are built with a converging and diverging approach, similar to the so-termed adjutage attached to an orifice. This method of construction greatly increases the value of the coefficient of discharge through the notch, so that when thus constructed the modification due to end contraction need not be used, as it would have to be were the sides of the notch left square; instead of which the coefficient should be slightly reduced.

It is considered that the coefficient for notch falls should be reduced from the ·623 of Francis' formula to one of about ·56. This will make with notch falls $Q = \cdot 3d^{\frac{3}{2}}$ per foot run. This value is just 10 per cent. below that for ordinary falls as calculated from Francis' coefficient, consequently new tables are not required. When notch falls are concerned $\frac{1}{10}$ or 10 per cent. should be deducted from the quantities given in Tables II. and IV.

(22) Some values of the long theoretical formula (12) have been worked out and are given below in Table V., although not used in this work. The coefficient is unity. The values obtained are much higher than with the simple formula (8a), for instance, in Table IV. When $H = 2$ and $d = 6$ the unit discharge without velocity of approach is 60·528 cubic feet per second with $c =$ unity.

In Table IV., Series IV., the discharge is 35·3 cubic feet. This would imply a coefficient for the whole depth of $\frac{35\cdot 3}{60\cdot 5} = \cdot 58$ nearly, which is decidedly low.

The theoretical formulas (11) and (12) are, however, but theoretical, whereas formula (8a) is based on actual observations, in addition to which formulas (11) and (12) should properly have two coefficients for the two portions in which the current is arbitrarily divided.

It is a pity that observations on which the values of n are founded are obtained from such small depths and widths. Reliable coefficients can only be deduced by observations on large canals which are 8 or 10 feet in depth and some 200 feet in width. There are many such in India.

TABLE V.—THEORETICAL UNIT DISCHARGES OF SUBMERGED WEIRS FROM FORMULA (12). $\sqrt{2g}\left[(d + h)\sqrt{H + h} - \frac{1}{3}\left\{(H + h)^{\frac{3}{2}} + 2h^{\frac{3}{2}}\right\}\right]$

$v = {\cdot}0155\ V_2$. (c and l = unity.)

$H = \frac{1}{2}$.

	Mean Velocities of Approach.									
d	$\sqrt{2gH} \times \frac{1}{3}(3d - H)$ 0	1	1½	2	2½	3	3½	4	4½	5
1½	7·566	7·727	7·923	8·183	8·528					
2	10·403	10·606	10·856	11·190	11·634	12·115				
2½	13·241	13·486	13·789	14·197	14·720	15·317				
3	16·075	16·365	16·722	17·204	17·817	18·519	19·273	20·123	21·077	22·072
3½	18·912	19·245	19·655	20·211	20·913	21·721	22·604	23·592	24·695	25·851
4	21·752	22·124	22·588	23·218	24·010	24·923	25·935	27·061	28·313	29·629
4½	24·590	25·003	25·521	26·225	27·106	28·125	29·266	30·530	31·932	33·408
5	27·427	27·883	28·454	29·232	30·303	31·326	32·597	33·998	35·550	37·186
5½	30·264	—	31·387	32·238	33·300	34·528	35·928	37·467	39·168	40·965
6	33·101	—	34·320	35·245	36·395	37·730	39·259	40·936	42·786	44·743
6½	35·938	—	—	38·252	39·491	40·932	42·590	44·405	46·404	48·521
7	38·776	—	—	41·259	42·588	44·134	45·930	47·874	50·022	52·300
7½	41·613	—	—	44·266	45·684	47·335	49·251	51·342	53·640	56·078
8	44·456	—	—	47·273	48·781	50·537	52·582	54·811	57·258	59·857
8½	47·288	—	—	—	—	53·739	55·913	58·280	60·876	63·635
9	50·124	—	—	—	—	56·941	59·244	61·749	64·494	67·414
9½	52·962	—	—	—	—	60·143	63·575	65·218	68·113	71·192
10	55·800	—	—	—	—	63·344	65·906	68·686	71·731	74·970

$H = 1$.

3	21·400	21·622	21·912	22·297	22·761	23·339	23·997	24·711	25·497	26·348
3½	25·412	25·669	25·992	26·430	26·962	27·619	28·372	29·191	30·102	31·016
4	29·425	29·710	30·071	30·563	31·162	31·898	32·746	33·271	34·707	35·685
4½	33·437	33·751	34·151	34·696	35·363	36·178	37·121	38·151	39·313	40·354
5	37·450	37·792	38·230	38·829	39·563	40·457	41·495	42·632	43·918	45·022
5½	41·462	41·834	42·310	42·962	43·763	44·736	45·869	47·112	48·524	49·691
6	45·475	45·875	46·390	47·095	47·963	49·016	50·244	51·592	53·129	54·360
6½	49·487	49·916	50·469	51·228	52·164	53·295	54·618	56·072	57·734	59·028
7	53·500	53·597	54·549	55·361	56·365	57·575	58·993	60·553	62·349	63·697
7½	57·512	57·999	58·629	59·494	60·565	61·854	63·367	65·033	66·945	68·366
8	61·525	62·040	62·708	63·627	64·765	66·133	67·741	69·514	71·550	73·035
8½	65·554	—	—	—	—	70·413	72·116	73·994	76·156	77·703
9	69·550	—	—	—	—	74·692	76·490	78·474	80·761	82·372
9½	73·562	—	—	—	—	78·971	80·864	82·954	85·367	87·041
10	77·575	—	—	—	—	83·251	85·239	87·434	89·972	91·709

$H = 1\frac{1}{2}$.

4½	39·315	—	39·752	40·428	41·023	41·746	42·583	43·509	44·529	45·633
5	44·229	—	44·650	45·440	46·092	46·880	47·796	48·811	49·929	51·143
5½	49·143	—	49·549	50·452	51·161	52·015	53·010	54·113	55·330	56·652
6	54·057	—	54·037	55·464	56·231	57·149	58·223	59·415	60·731	62·161
6½	58·972	—	59·346	60·476	61·300	62·283	63·437	64·718	66·131	67·670
7	63·886	—	64·244	65·488	66·370	67·417	68·650	70·020	71·532	73·179
7½	68·800	—	69·142	70·501	71·439	72·552	73·863	75·322	76·933	78·689
8	73·714	—	74·041	75·513	76·508	77·686	79·077	80·624	82·323	84·198
8½	78·629	—	—	80·525	81·578	82·820	84·290	85·226	87·734	89·707
9	83·543	—	—	85·537	86·648	87·954	89·504	91·228	93·134	95·217
9½	88·457	—	—	—	91·717	93·089	94·717	96·530	98·535	100·726
10	93·372	—	—	—	96·786	98·223	99·931	101·832	103·936	106·235

TABLE V.—THEORETICAL UNIT DISCHARGES, ETC.—*continued*.

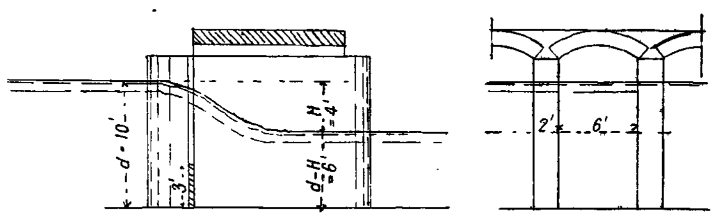
$H = 2.$

d	$\sqrt{2gH} \times \frac{2}{3}(d-H)$.	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5
6	60·528	61·251	61·820	62·534	63·370	64·361	65·461	66·681	67·990
$6\frac{1}{2}$	66·202	66·971	67·580	68·342	69·235	70·295	71·473	72·781	74·181
7	71·877	72·692	73·339	74·149	75·100	76·329	77·485	78·881	80·373
$7\frac{1}{2}$	77·551	78·413	79·098	79·956	80·965	82·163	83·497	84·981	86·565
8	83·226	84·133	84·857	85·763	86·830	88·097	89·509	91·081	92·757
$8\frac{1}{2}$	88·900	—	90·617	91·571	92·696	94·032	95·521	97·181	98·948
9	94·575	—	96·376	97·378	98·561	99·966	101·533	103·280	105·140
$9\frac{1}{2}$	100·249	—	102·135	103·185	104·426	105·900	107·545	109·380	111·332
10	105·924	—	107·894	108·992	110·291	111·834	113·557	115·480	117·524
$H = 2\frac{1}{2}$.									
$7\frac{1}{2}$	84·590	—	86·039	86·846	87·792	88·949	90·177	91·573	93·095
8	90·935	—	92·458	93·310	94·307	95·527	96·825	98·301	99·910
$8\frac{1}{2}$	97·779	—	98·876	99·773	100·822	102·104	103·473	105·029	106·725
9	103·623	—	105·295	106·237	107·337	108·683	110·121	111·756	113·549
$9\frac{1}{2}$	109·968	—	111·713	112·700	113·852	115·261	116·769	118·484	120·355
10	116·312	—	118·132	119·163	120·367	121·839	123·418	125·211	127·171
$H = 3$.									
9	111·198	—	112·775	113·668	114·719	115·969	117·367	118·942	120·618
$9\frac{1}{2}$	118·147	—	119·793	120·727	121·824	123·131	124·594	126·252	127·999
10	125·097	—	126·811	127·784	128·929	130·293	131·821	133·563	135·379
$H = 3\frac{1}{2}$.									
$10\frac{1}{2}$	140·124	—	141·829	142·796	143·987	145·306	146·826	148·532	151·665
11	147·632	—	149·398	150·401	151·686	153·009	154·589	156·364	159·663
12	155·139	—	156·968	158·007	159·385	160·712	162·352	164·196	167·661
$12\frac{1}{2}$	162·646	—	164·537	165·613	167·084	168·416	170·115	172·028	175·659
$H = 4$.									
12	171·200	—	173·011	174·102	175·305	176·654	178·402	180·258	182·251
$12\frac{1}{2}$	179·225	—	181·094	182·225	183·466	184·837	186·667	188·567	190·651
13	187·250	—	189·176	190·348	191·527	193·020	194·933	196·896	199·051

(**23**) We will now proceed to examine the case of a fall through the openings of a head regulating bridge, escape head, or undersluices.

Figs. 5 and 5a are views of a work of this description; the surface of water being free, the case apparently resolves itself into the simple one of a submerged overfall, to which formula (8a) is applicable. The conditions, however, present some differences to those of a fall over a weir wall. The presence of the bottom floor diminishes the friction at the base of the current, and consequently in such cases as noticed in par. 9 the coefficient always rises in value. Against this rise is the diminution in effective width, due to the piers. As seen in par. 11, the reduction in the effective length of the water section would be ·1*nd*, or ·2 × 10 = 2 feet. This, however, applies only to the square and raised flanks of an ordinary weir wall, and not to piers provided with well shaped cutwaters, as should invariably be adopted. In the latter case the effect of the cutwaters is to nullify in a great measure the contraction alluded to.

(24) With regard to the coefficient, the ordinary one of ·623 would clearly be inapplicable. In the case of orifices, proper reference to the list in par. 8 shows that the rise of the coefficient in the case of bottom sluices is roughly about 2 points, *i.e.*, from about ·7 to ·9. This would apply to the under-current portion of the fall only, so that it is considered that the coefficient



(5a)

FIGS. 5, 5a.

should be raised from ·623 to ·66 (the Castel coefficient). The discharge under such conditions would be that given by Table IV. increased by $\frac{66}{62}$ or by $\frac{33}{31}$. For example, in this case $H = 4$, $d = 10$; in Series VIII. the unit discharge due to this combination is 82·2 cubic feet, which, increased by $\frac{33}{31}$, will become 87·5 cubic feet per second, and in a 6-foot bay, 6 × 87·5, or 525 cubic feet per second.

(25) If the arch were depressed below the surface of the water up stream as is often the case in head and undersluices, the surface would not be free and contraction occurring at the top as well as the sides of the opening would cause the latter to be classified as an orifice, when formulas (2a) or (3) would be applicable, with a suitable coefficient selected from the list given in par. 8. In this case ·90 would probably apply.

If a bottom gate or baulk were left in the grooves in Fig. 5 the discharge would be considerably modified. Supposing this obstruction to be 3 feet deep, then on the assumption that H is of the same value, viz., 4 feet, d will be reduced to 7 feet, and the coefficient falls back to ·623; the discharge according to Table IV., Series VIII. for a depth of 7 feet would then be

$$54·3 \times 6 = 325 \text{ cubic feet.}$$

As a matter of fact, H would rise till equilibrium was produced somehow, a matter very difficult to estimate with any precision unless the whole river supply went through the sluices.

The depth of the escape channel, *i.e.*, of the tail water or $d - H$, depends on the slope of the channel and on its sectional area, *i.e.*, its discharging capacity, and has been dealt with in another place.

(**26**) In a manner similar to the case of notch falls, a reduction in discharge per foot run can be effected in order to make allowance for the shape of the weir crest. If the latter is broad crested like that of type C in Chap. VI., a coefficient of ·577 is often adopted in lieu of ·623. The discharges should then be multiplied by $\frac{577}{623}$ or by ·92. It would be near enough to deduct $7\frac{1}{2}$ or 8 per cent.

Discharge of Channels.

(**27**) The formula for discharge of channels is Chezy's with Kutter's coefficient, viz.,

$$Q = AV = 100\, Ac\sqrt{RS} \qquad (13)$$

Sometimes expressed as

$$= 100\, C\sqrt{RS}$$

in which R is the hydraulic mean depth, or $\frac{\text{area of section}}{\text{wetted perimeter}}$, and S is the sine of the slope of water surface which can be assumed as parallel to the average slope of the bed of channel. The coefficient c is obtainable from Kutter's formula, which has superseded all previous ones, as Bazin's. This formula expressed in feet is

$$c = \frac{\sqrt{R}}{100n} \times \frac{(m + 1{\cdot}811)}{(m + \sqrt{R})} \qquad (14),$$

in which $m = n\ (41{\cdot}6 + \frac{\cdot 00281}{S}$ and n, termed the coefficient of rugosity, varies with the nature of the channel and its degree of smoothness. The values of n for earthen channels are as follows:—

For firm trimmed soil . . .	$n = {\cdot}020$
For canals in good condition . .	$n = {\cdot}0225$
For canals in average condition .	$n = {\cdot}025$
For canals below average condition .	$n = {\cdot}0275$
For canals in defective condition, or rivers	$n = {\cdot}030$
For very defective channels . .	$n = {\cdot}035$

In calculations for canal discharges, n is given a value either of ·025 or ·0225, the latter being used in the new Panjab Canals.

(**28**) This formula (14) being extremely cumbersome to work out, tables are invariably made use of in its application. Jackson's "Hydraulic Manual" and Higham's and Colonel Moore's tables are invaluable aids for calculation, as the tables provided enable rapid computation to be effected. First the value of the expression $100\sqrt{RS}$ for various values of R and S is obtained from one table and then the coefficient c for varying values of R, S and of n. These two values thus obtained, multiplied together and by the mean area $A = Q$, *i.e.*, the discharge in cubic feet per second. It is often convenient to substitute for S, the sine of the inclination of the water surface, S in 1,000, termed S°. The formula for velocity can then be expressed as $V = c\sqrt{10RS^\circ}$.

Thus if $R = 4$ and S° is ·2 per 1,000, v will equal $2c\sqrt{2} = 2{\cdot}83c$ cubic feet per second.

For channels other than earth the values of n are as follows :—

Well planed planks	$n = {\cdot}009$
Cement, plaster or enamelled pipes . .	$n = {\cdot}010$
Unplaned timber	$n = {\cdot}012$
Ashlar and brickwork, cast and wrought iron .	$n = {\cdot}013$
Rubble masonry	$n = {\cdot}017$

The value given to n, the coefficient of rugosity, has a great effect on the discharge, so the selection of a suitable value in the case of river discharges is very essential. The coefficient in the case of natural channels will vary from $n = {\cdot}0250$ to ${\cdot}0350$.

(29) The lately published tables by Colonel Moore, R.E. (Batsford), entitled, "New Tables for the Complete Solution of Ganguillet's and Kutter's Formula," are of great use in finding the velocity, though for the reverse processes of finding S, corresponding to certain values of R and of V they are not so suitable as Jackson's or Higham's. In this work Chezy's form of equation, viz., $V = 100\, c\sqrt{RS}$, is not used, but Kutter's long formula, which gives the velocity direct, not the derived coefficient applicable to Chezy's formula given in par. 27, is worked out.

The long formula as modified by Colonel Moore is

$$V = \frac{\frac{l}{n} + \left(a + \frac{m}{S}\right)}{\sqrt{R} + \left(a + \frac{m}{S}\right) n} \quad R\sqrt{S}.$$

This is abbreviated into $V = \dfrac{NR}{\sqrt{R} + D}$

where $$N = \left\{\frac{l}{n} + \left(a + \frac{m}{S}\right)\right\} \sqrt{S}$$

and $$D = \left(a + \frac{m}{S}\right) n.$$

The tables give the values of N, log. N and of D for every possible value of S or surface inclination from 1 over 1 to 1 over 20,000.

The value of R has to be calculated or found in the tables given, it being equal to $\dfrac{A}{WP}$ or the area divided by the wetted perimeter. Thus by putting a few figures together the value of V is obtained. A large diagram is also annexed, in which the value of c, applicable to Chezy's formula, can be obtained. These tables form a valuable labour-saving help in the manipulation of Kutter's formula, and should be used in conjunction with other tables which employ the Chezy formula with Kutter's coefficient, viz., $V = 100\ c\sqrt{RS}$, or $V = C\sqrt{RS}$, as expressed in some works.

(**30**) Methods of calculating flood discharges of rivers.

As already noted, the discharge of any channel is $A \times V$, in which A is the area of the water section and V the mean velocity of the current. The mean velocity can be deduced by actual observation of the surface velocity in feet per second, and the latter multiplied by a proper coefficient, which varies from ·65 to ·8, gives the mean velocity (*vide* par. 31).

In a wide stream several surface velocities can be observed, the water section being divided into any convenient number of parts, and the floats are caused to pass through the centre of each of these divisions as near as possible; then the discharge will be the sum of these areas, each multiplied by the respective observed surface velocity, reduced to mean velocity.

A more accurate method is by means of vertical rod floats of various lengths. These can be either tin tubes weighted at the bottom so as to float nearly immersed, or else wooden rods with a piece of lead gas piping twisted round the bottom. The former are used in India, while the latter method, which is simpler, is recommended by Drs. Tudsberry and Brightmore in "Water Works Engineering." These velocity rods should float just clear of the bed of the stream. If the latter is very uneven, as is often the case in rivers, the system will not work. The rods give the actual mean velocity at once and consequently furnish more accurate results than reduced surface observations.

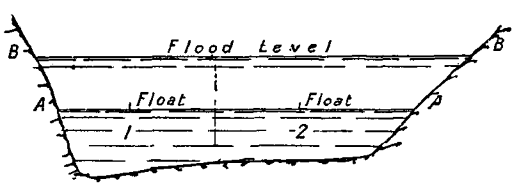


FIG. 6.

Another method of obtaining actual mean velocity observations is by the use of current meters. The meter is immersed at $\frac{3}{5}$ the depth of the water in the middle of each division of the water area section, and the velocity is recorded automatically by an electric current, the meter being connected with a battery.

(**31**) To determine the mean velocity in a cross-section of a current where the maximum surface velocity is known, the following formula by Prof. von Wagner, gives reliable results:—

$$V = ·705 \times ·003\ v^2 \qquad (15)$$

In which V is the mean and v the surface velocity.

Thus supposing the observed maximum surface velocity of a section to be 3 feet per second, the mean velocity V will be $·705 \times 3 + ·003 \times 9 = 2·115 + ·027 = 2·142$ feet per second, *i.e.*, a little over ·7 V. Other authorities, as Jackson, give ·8 as the coefficient. In canal and river discharges in India ·7 or ·75 are the coefficients usually employed, so that the ratio given in formula (15) can well be adopted.

(**32**) Fig. 6 is an imaginary cross-section of a channel. It is divided into two parts, separate surface velocity observations being supposed to have been taken in the centre of each division. The following form illustrates the method of arriving at the total discharge:—

1 Division.	2 Observed Surf. Vel.	3 Calculated Mean Vel.	4 Area Measured.	5 Discharge. Col. 3 × Col. 4.
	Feet.	Feet.	Feet.	Feet.
1	3	2·142	51·5	110·3
2	2·7	1·925	54·0	103·9
			105·5	Total $Q = 214·2$ cubic feet per second.

(33) Supposing it is required to deduce the maximum flood discharge at a level higher than was the case when the velocity observations were taken, then the following formula will approximately determine the discharge required:—

$$Q = \frac{AqC\sqrt{R}}{ac\sqrt{r}} \text{ or } Q = \frac{Aq\sqrt{R}}{a\sqrt{r}} \times \frac{C}{c} \qquad (16)$$

in which A and a are the areas of the two water sections, R and r their respective hydraulic mean radii or $\frac{A}{WP}$, and Q and q the discharges in feet per second; C and c being the proper coefficients.

For example, in Fig. 6, supposing the flood discharge of the channel is required when the water is at the level BB, while the velocity observations have been taken when the water level was at AA. Then by reference to the Table in last paragraph, $q = 214·2$, $a = 105·5$, $wp = 32·5$, and $r = \frac{a}{wp} = \frac{105·5}{32·5} = 3·2$, and of the new section by measurement.

$$A = 228 \text{ cubic feet, } WP = 43 \text{ whence } R = \frac{228}{43} = 5·3, \text{ and}$$

$$\frac{Aq\sqrt{R}}{a\sqrt{r}} = \frac{228 \times 214·2 \times 2·3}{105·5 \times 1·79} = 595 \text{ cubic feet per second, nearly.}$$

Now the value of the coefficient C is largely influenced by that of R, but slightly so by that of S; consequently if S be known only approximately the fraction $\frac{C}{c}$, in which S and also n are common to both the numerator and the nominator, can be obtained with sufficient exactitude without the precise value of S being known. In the above let n be ·0275, and let $S°$ be assumed as ·2 per 1,000, then from Table VII., Part 2, "Hydraulic Manual," p. 74, with $R = 5·3$, $C = ·75$, and with $r = 3·2$, $c = ·654$, the fraction $\frac{C}{c}$ will then be $\frac{75}{65}$, and Q will be $595 \times \frac{15}{13} = 686$ cubic feet per second.

If the bed slope be assumed as ·3 per 1,000 the fraction will become $\frac{725}{655}$ $= \frac{145}{131}$, and Q will become $595 \times \frac{145}{131} = 660$ cubic feet per second. This proves that for a very considerable difference in the value of S or S^o, that of the resulting discharge is not so great proportionately.

In the example just given the whole water section has been dealt with, but greater accuracy would be obtained by having two values of A, a, etc., viz., those belonging to each of the two divisions in which the lower water section was divided.

(**34**) It often happens that actual velocity observations cannot be taken during a freshet, and there is no time available for putting up self-recording white gauge posts which will indicate the flood surface by the discoloration due to muddy water passing. In such cases the slope of the water surface being unknown, it can be assumed as parallel to the average bed slope, for which purpose a longitudinal section of the bed for at least a mile in length should be levelled and plotted, and at least three cross-sections taken of the channel showing the maximum flood level, which can be approximately obtained by observation of floating detritus deposited on the bank or on bushes, and by inquiry. These three cross-sections, taken at different places, should be plotted one over the other, and from this an average representative cross-section should be drawn. Having obtained S, *i.e.*, the sine of the bed slope, by levelling, A and WP are measured from the section and thus R is obtained. The probable value of n, the coefficient of rugosity, has to be fixed with reference to the natural state of the channel, and by application of formula (13), viz., $Q = 100\, Ac\sqrt{RS}$, the discharge is obtained. The value of the expression $100\sqrt{RS}$ can be obtained from Jackson's "Hydraulic Manual" or other tables, in which Kutter's formula is worked out for English feet.

(**35**) For example, again utilising Fig. 6, supposing the value of S obtained by levelling is $\frac{1}{3,000}$ or ·3 per 1,000, the known value of r is 3·2, of a is 105·5, whence the value of the expression $100\sqrt{RS}$ with $R = 3·2$ and $S =$ ·30 per 1,000 is found in Table VII., Part II., Jackson's "Hydraulic Manual" equal to 3·1 nearly. Again assuming n, the coefficient of rugosity, as ·0275; in Table XII., Part IV., the value of c (with $R = 3·2$ and $S =$ ·3 per 1,000) is by interpolation ·654, whence the mean velocity is $(3·1 \times ·654)$ $= 2·03$ feet per second, and the discharge $Q = AV = 2·03 \times 105·5 = 214·1$ cubic feet per second, *i.e.*, identical with previous result. Such close coincidence would not be obtained in actual practice. When velocity observations are taken, the surface slope should usually also be obtained by carefully checked levelling, and noted in the discharge book. From the data thus available, viz., values of S, R, and A, the really correct value of n which is applicable to this particular channel can be deduced, which will be most useful in future similar operations. The deductions can be made by trial and error, using the tables.

(36) The usual procedure of taking velocity discharges is as follows:—A convenient reach being selected, poles are erected in pairs at the starting and finishing points on either side of the channel, their direction being at right angles to that of the channel. The distance apart of these pairs of poles is generally 50 feet for a slow current and 100 feet to 150 feet for a more rapid stream. If more than one velocity observation is required in the cross-section, ropes should be stretched between the two pairs of poles clear of the water, on which the centres of the divisions, in which the water section is divided, should be clearly indicated by attaching a red streamer or otherwise. There should be two observers, one at each pair of poles, the third having the watch and record book. The floats, discs of wood for surface velocities, or the weighted rods as already described for mean velocity observation, are let go by an attendant at the proper place some short distance above the first pair of poles. When the float crosses the line of sight, the first observer calls out the number of the float, the discharge taker then enters the exact second and minute in the form book. The same is repeated when the float crosses the second line of sight. Stop watches are now invariably employed for this purpose, reading to half-seconds. At least three observations should be taken, the mean being adopted as the true velocity. If a float strays from the line it is expected to follow, that observation should be cancelled. Still weather is indispensable for surface observations. Two floats can generally be observed at short intervals, the second being dispatched before the first arrives.

(37) In the case just described the water section is first of all divided into divisions, but the most accurate method, particularly in taking large rivers, is to settle the width of the divisions in accordance with the actual course taken by the floats. The method of taking the discharges of the Nile illustrates this system, and the description obtained from "Egyptian Irrigation" is as follows:—

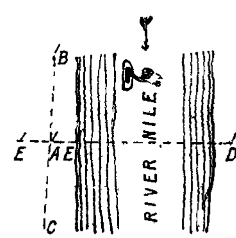


FIG. 7.

"A site was chosen where the river was fairly straight for fully 2 kilometres, the most uniform cross-section of the river was found by taking a large number of rough cross-sections. A peg was fixed in the bank opposite this section.

"In the accompanying plan, A is this fixed point.

"Through A, a line B, A, C, was marked, exactly parallel to the direction of the river, making AB, and AC, each equal to about the observed width of the river. A theodolite or plane table was put up at A, and the point D across the river on the line AD at right angles to BC was fixed. Since AB and AC were both capable of being measured, they were measured, and with the aid of the theodolite or plane table the length AD was obtained.

"In the line AD, or its continuation, flags were put up at E and E_1 to direct the man in the steamer on the Nile. The theodolite or plane table was now put up at B or D, and observations made on a steamer or boat on the Nile, which as it got on the line AD, threw out a signal and took a sounding either with a line or sounding-rod. The boat or steamer traversed the whole section, and if any gaps were left without soundings, the plane table showed where they were, and they could be immediately filled in. While all this was going on, the nearest river gauge was being observed, and also a temporary gauge erected at the site of the discharge site. The two were afterwards connected by levelling. This completed the observations necessary for the cross-section, which was now plotted (*vide* Fig. 8).

FIG. 8.

(38) "The surface velocity observations were now made.

"Two lines, HJ and KL (Fig. 9) were fixed, the former 50 metres up stream of AD and the latter 50 metres down stream. Between these lines the surface floats were observed. A theodolite or plane table was fixed at H and the boat or steamer was sent up stream with the floats. The observer stood at H, and another at A. The boat dropped a float into the stream at a distance of about 50 metres up stream of the line HJ at a convenient point, and the theodolite or plane table at A followed it until the man at H called out, on its crossing the line HJ. It was then observed and recorded. This was repeated as the float crossed the line KL.

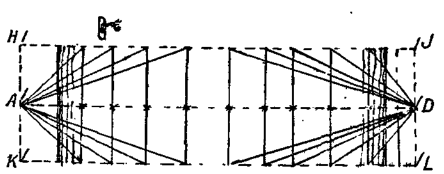


FIG. 9.

"The observers at H and K noted the time. When a sufficient number of well-placed velocities over the half of the river near A were observed, observations for the other half were made from D. The field work was now over. On the cross-section, Fig. 10, the points E, F, G, and C, where the velocities were observed, were plotted.

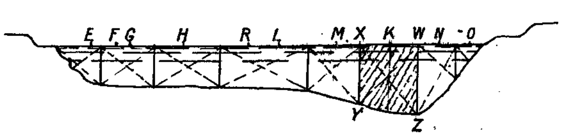


FIG. 10.

"The cross-section was divided into a number of suitable sections, each ruled by one or more observed velocities. Each section was calculated separately, for instance, the section $XYZW$. Its area was $XYZW$, its wetted perimeter was YZ; its hydraulic mean depth was $\frac{XYZW}{YZ}$ and the rest followed from Bazin's coefficients and tables.

(39) "In addition to the discharges obtained from surface velocity observations, others were calculated from surface slope observations. A very

careful cross-section of the river was taken as before, on a straight reach of some three or four kilometres, and at a point where the cross-section was fairly uniform, showing that the river here was flowing normally. A section taken in winter was found more accurate than in flood. There was enough water in the river to preserve its normal slope of water surface, and not enough to render the taking of soundings difficult. The water surface was compared with the nearest fixed gauge of the river. The section was very carefully plotted and the water surface drawn on it, with the corresponding gauge readings written against it. Horizontal lines (*vide* Fig. 11), 1 metre or half a metre apart, were then plotted on this section, corresponding to the different gauge readings. The cross-sections and hydraulic mean depths (*i.e.*, A and R) were now calculated for each gauge reading. For the slope of the river surface a long reach of 50 kilometres was taken, as the chances of error were very much less than they would have been if 3 or 4 kilometres had been taken. This was easily effected on the Nile, as there were carefully levelled gauges 50 kilometres apart.

FIG. 11.

"The mean slope on this reach of 50 kilometres was taken as the slope for calculating the discharges. Of course, on curves the slope varies, but since the cross-section had been taken in a carefully chosen normal site, the mean slope would refer to it. From the calculated velocities and discharges, a discharge and velocity diagram was made. This diagram was checked frequently by surface velocity observed as already described."

From the above the importance of taking the longitudinal section for some miles in length to ensure accuracy in the average surface slope, which is naturally variable, is demonstrated. The same would apply with equal force to a longitudinal section of the bed of a river taken when dry if the water slope cannot be otherwise ascertained.

Afflux.

(40) The calculation for afflux, *i.e.*, the rise of the water surface in a river due to an obstruction in such a weir, is best arrived at by use of Tables II. and IV. when once the value of the velocity of approach or $Q \div A_1$ has been decided on.

In a free overfall the afflux is the depth of film passing, plus that of the tail water surface below crest of weir. In a submerged fall, H is the afflux. To obtain d, the flood discharge, which is a known quantity, should first be divided by the length of the weir, and the quotient will be the discharge per foot run. The next procedure is to find by interpolation in Table II. that value of d which corresponds to the discharge and the velocity of approach.

(41) For example, supposing a weir to be built across the channel the section of which is shown in Fig. 6. Let its length be 30 feet and let the flood discharge of the river be estimated at 600 cubic feet per second. Now the area of the flood waterway is 228 square feet (par. 42), whence the mean

velocity $V = \frac{Q}{A}$ and Va, the velocity of approach $= \frac{Q}{A_1}$. Assuming $A = A_1$ $Va = \frac{600}{228} = 2\cdot63$ feet. The unit weir discharge being 20 feet, by Table II., under $V = 2\frac{1}{2}$, we see that d will lie between $d = 3$ and $d = 3\frac{1}{2}$, where the discharges in that column are 18·064 and 22·631 respectively. Their difference is 4·567, and that between the lower and the given discharge is 1·936 feet. Therefore the addendum to $d = 3$ will be $\frac{1,936}{4,567} \times \frac{1}{2} = \cdot21$ nearly, and d will $= 3\cdot21$. Allowing roughly for the difference between 2·63, the actual velocity, and 2·5 which has been used, d will probably equal 3 feet. The afflux will be 3 feet plus height of crest above tail water.

(**42**) The afflux in the case of a submerged weir is found by the same process, Table IV. being used. Let us assume the crest of the weir to be 3 feet below the flood level of the river below the weir. The discharge, as before, is 20 feet per foot run; d will equal $H + 3$. If H be taken as 1, $d = 4$, and the corresponding discharge in Table IV., Series II., for $V = 2\frac{1}{2}$ is 17·9 cubic feet. H must therefore exceed 1 foot. If taken as 1·5 the corresponding discharge of $d = 4\frac{1}{2}$ in Table IV., Series III., is 22·9 cubic feet, which is too large; hence the value of H must lie between $1\frac{1}{2}$ and 1, and as 20 is a mean between the two values 22·9 and 17·9 the value of H may roughly be taken as $1\frac{1}{4}$. For practical requirements the afflux need only be calculated to the nearest half-foot.

(**43**) The solution of some other problems will exhibit the great use of the Tables in saving calculation.

In a free overfall as in Fig. 3 (par. 12) it is required to find what height of weir will raise the up stream water level to a given height above the base of the weir. The following data are given:—

D the depth of river channel
Q its discharge per foot run
V its velocity ,, ,,
H the afflux is known as $H + D =$ height given.

Then d, the depth of film reciprocal to D, is at once found from Table II., Q and V being known, and x the required height of the dam $= D + H - d$.

(**44**) If the weir is submerged, we have two unknown quantities d and H. First the value of d for free overfall must be found from Table II. Then in Table IV. the nearest value of d to the depth obtained from a free overfall, which will give the same discharge, if found will give the value of H. For example, let $Q = 30$, $D = 10$, $V = 3$. Then from Table II. the depth of overfall d to produce a discharge of 30 feet per second with velocities 3 feet is found as before to be about $4\frac{1}{4}$ feet. Now from Table IV. we want a combination of d and H which will give the same discharge, d being as near a value above $4\frac{1}{4}$ feet as possible. By looking over the Table, Series IV. with $d = 5$ and $H = 2$ is the nearest, and gives the discharge of 30 feet nearly. H being thus found, $x = (D + H) - d = 12 - 5 = 7$ feet.

(45) Two rather complicated problems, connected firstly with the rise of a river or stream up stream of an obstacle, and secondly with the fall of the surface due to an opening, producing a draft on the discharge, have been dealt with by M. Bresse, a French mathematician. For the resulting formulas and tables we are indebted to that excellent work "A Treatise on Hydraulics" by Prof. Merriman (New York).

In that work they are termed the "Backwater surface curve" and the "drop down" curve. The latter appears a rather crude designation; it will be termed the "Falling surface curve."

Fig. 12 is a diagram explanatory of the Backwater surface curve.

BB is the sloping bed of the river assumed uniform and with a uniform width, so that the depth *D* can be taken to represent the area per foot run as well as the hydraulic mean depth. *CC* is the ordinary surface parallel to the bed *BB*. The obstruction of the weir wall causes a rise in the surface which can be found by the method explained in the last paragraphs. The depth of afflux level is designated d_2 and the depth up stream at a

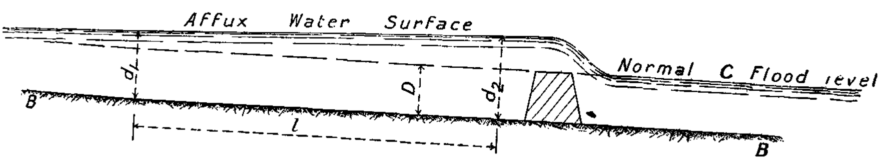


FIG. 12.—Diagram of Backwater Surface Curve.

distance *l*, above *W*, is d_1. *S* is the slope. Now *l* is found, when the other values are given, by the following formula

$$l = \frac{d_2 - d_1}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right]. \qquad (17)$$

Table VI. gives the values of the decimal fraction $\frac{D}{d}$ and the corresponding value of the function of $\frac{d}{D}$.

(46) The greater depth d_2 need not be at the weir itself, but anywhere above it, and d_1 is the depth *l* feet above it.

For example an actual case will be taken. At Narora Weir (Figs. 3 and 4, Chap. VI.) the afflux level is 18 feet above normal river bed and the depth below the weir is 16 feet. Thus $D = 16 = d_2 = 18$ feet, and d_1 will be taken as 17 feet. The slope of the Ganges River is about 1 in 10,000, $\frac{1}{S}$ therefore is 10,000. Required to find at what distance above the weir the depth of the water will be 17 feet, *i.e.*, 1 foot rise above former flood levels. Here $\frac{D}{d_2} = \frac{16}{18} = \cdot 89$, and by reference to Table VI. the corresponding value of $\phi\left(\frac{d_2}{D}\right)$ is $\cdot 6173$; in the same way $\frac{D}{d_1}$ is $\frac{16}{17} = \cdot 941$, and $\phi\left(\frac{d_1}{D}\right)$ is $\cdot 82$ nearly;

c is taken as ·70, and g, the gravity sign, is 32, and $\frac{(100c)^2}{g} = 153$. Then

$$l = 10{,}000\,(18 - 17) + 16\,(10{,}000 - 153)\,(·820 - ·617)$$

$$\therefore l = 10{,}000 + ·157552 \times ·203$$

$$= 41{,}983 \text{ feet or nearly 8 miles.}$$

(**47**) Another example will be given of the same formula in which d_1 is required and l is given. In this case various values must be tried of $\frac{D}{d_1}$ and of $\phi\left(\frac{d_1}{D}\right)$ until the right-hand expression of the equation equals the left.

TABLE VI. VALUES OF THE BACKWATER FUNCTION.

$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$
1·	∞	0·954	0·9073	0·845	0·5037	0·61	0·2058
0·999	2·1834	·952	·8931	·840	·4932	·60	·1980
·998	1·9523	·950	·8795	·835	·4831	·59	·1905
·997	1·8172	·948	·8665	·830	·4733	·58	·1832
·996	1·7213	·946	·8539	·825	·4637	57	·1761
·995	1·6469	·944	·8418	·820	·4544	·56	·1692
·994	1·5861	·942	·8301	·815	·4454	·55	·1625
·993	1·5348	·940	·8188	·810	·4367	·54	·1560
·992	1·4902	·938	·8079	·805	·4281	·53	·1497
·991	1·4510	·936	·7973	·800	·4198	·52	·1435
·990	1·4159	·934	·7871	·795	·4117	·51	·1376
·989	1·3841	·932	·7772	·790	·4039	·50	·1318
·988	1·3551	·930	·7675	·785	·3962	·49	·1262
·987	1·3284	·928	·7581	·780	·3886	·48	·1207
·986	1·3037	·926	·7490	·775	·3813	·47	·1154
·985	1·2807	·924	·7401	·770	·3741	·46	·1102
·984	1·2592	·922	·7315	·765	·3671	·45	·1052
·983	1·2390	·920	·7231	·760	·3603	·44	·1003
·982	1·2199	·918	·7149	·755	·3536	·43	·0995
·981	1·2019	·916	·7069	·750	·3470	·42	·0909
·980	1·1848	·914	·6990	·745	·3406	·41	·0865
·979	1·1686	·912	·6914	·740	·3343	·40	·0821
·978	1·1531	·910	·6839	·735	·3282	·39	·0779
·977	1·1383	·908	·6766	·730	·3221	·38	·0738
·976	1·1241	·906	·6695	·725	·3162	·37	·0699
·975	1·1105	·904	·6625	·720	·3104	·36	·0660
·974	1·0974	·902	·6556	·715	·3047	·35	·0623
·973	1·0848	·900	·6489	·710	·2991	·34	·0587
·972	1·0727	·895	·6327	·705	·2937	·33	·0553
·971	1·0610	·890	·6173	·70	·2883	·32	·0519
·970	1·0497	·885	·6025	·69	·2778	·30	·0455
·968	1·0282	·880	·5884	·68	·2677	·28	·0395
·966	1·0080	·875	·5749	·67	·2580	·25	·0314
·964	0·9890	·870	·5619	·66	·2486	·20	·0201
·962	·9709	·865	·5494	·65	·2395	·15	·0113
·960	·9539	·860	·5374	·64	·2306	·10	·0050
·958	·9376	·855	·5258	·63	·2221	·05	·0015
·956	·9221	·850	·5146	·62	·2138	·00	·0000

This case will be taken in connection with Okhla Weir (Fig. 16, Chap. VI.).

The given values are as follows:—

$$D = 14{\cdot}7,\ d_2 = 17{\cdot}7\ \therefore \frac{D}{d_2} = {\cdot}83 \text{ and } \phi\left(\frac{d_2}{D}\right) = {\cdot}4733 \text{ (from Table VI.)}.$$

l will be taken as 3 miles or 15,840 feet, $100c$ as 170, and $\frac{100c^2}{g} = 153$, $S = {\cdot}001$. The equation will then be $15{,}840 = 177{,}000 - 10{,}000\, d_1 + 14{\cdot}7$ $(9{,}847)\left[\phi\left(\frac{d_1}{D}\right) - {\cdot}4733\right]$.

This worked out

$$10{,}000 d_1 - 144{,}750\left[\phi\left(\frac{d_1}{D}\right)\right] = 177{,}000 - 84{,}348 = 92{,}652.$$

Let a trial value of d_1 be 17 feet, then the equation becomes

$$170{,}000 - 144{,}750 \times {\cdot}5494 = 90{,}415$$

which is as near as it possibly can be to the proper value which is 92,650.

The value ·5494 is $\phi\ \frac{d_1}{D}$ when $\frac{D}{d_1} = \frac{14{\cdot}7}{17} = {\cdot}865$, and is obtained from Table VI. opposite ·865.

Thus 3 miles above the weir the flood afflux level will be $17 - 14{\cdot}7 = 2{\cdot}3$ above the original river flood level.

The Falling Surface Curve.

(**48**) When a sudden fall occurs in a stream, such as a drop in the bed, as in a canal fall which has not a raised or narrowed crest, or from a side opening

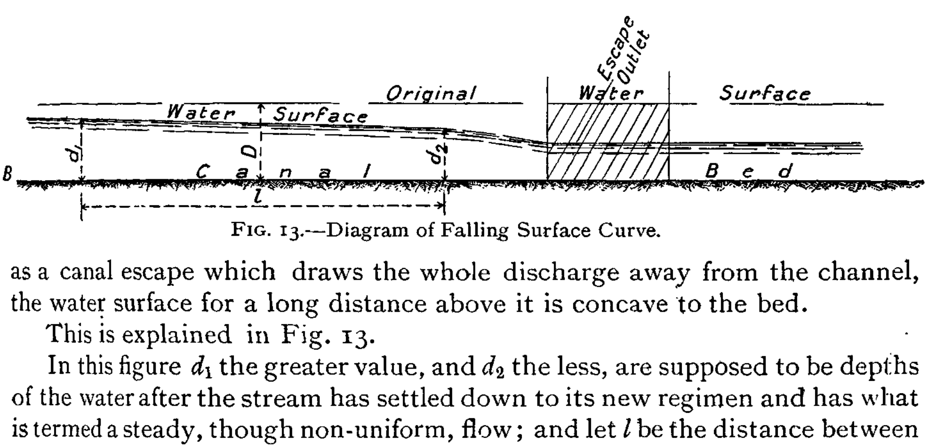


FIG. 13.—Diagram of Falling Surface Curve.

as a canal escape which draws the whole discharge away from the channel, the water surface for a long distance above it is concave to the bed.

This is explained in Fig. 13.

In this figure d_1 the greater value, and d_2 the less, are supposed to be depths of the water after the stream has settled down to its new regimen and has what is termed a steady, though non-uniform, flow; and let l be the distance between them. Then the principle of formula (17) will apply in this case as well, the only difference being due to d_1 having the higher value. It stands as below

$$l = -\frac{d_1 - d_2}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right]. \quad (18)$$

(**49**) An example of the working of this formula will be taken from Merriman's "Hydraulics."

Take a canal 10 feet deep having a co-efficient $100c$ equal to 80, and let the slope of the bed be $1 \div 5{,}000$ and its surface slope be the same as when the water is in uniform flow. Here $D = 10$ feet $(100c)^2 \div g = 200$ and $1 \div S = 5{,}000$. Then

$$l = -\,5{,}000\,(d_1 - d_2) + 48{,}000\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right].$$

It is required to find the distance between two points where $d_1 = 8$ feet and $d_2 = 7$ feet. Here $\frac{d_1}{D} = \cdot 8$, for which $\phi\left(\frac{d_1}{D}\right) = \cdot 3459$ and $\frac{d_2}{D} = \cdot 7$, for which $\phi\left(\frac{d_2}{D}\right) = \cdot 1711$. Inserting these values in the equation, there is found $l = 3{,}390$ feet.

TABLE VII.—VALUES OF THE FALLING SURFACE FUNCTION.

$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$
1·	∞	0·954	0·8916	0·845	0·4478	0·61	0·0454
0·999	2·1831	·952	·8767	·840	·4353	·60	·0325
·998	1·9517	·950	·8624	·835	·4232	·59	·0199
·997	1·8162	·948	·8487	·830	·4114	·58	+ 0·0074
·996	1·7206	·946	·8354	·825	·3988	·57	— 0·0050
·995	1·6452	·944	·9226	·820	·3886	·56	— ·0172
·994	1·5841	·942	·8102	·815	·3776	·55	— ·0293
·993	1·5324	·940	·7982	·810	·3668	·54	— ·0412
·992	1·4876	·938	·7866	·805	·3562	·53	— ·0530
·991	1·4486	·936	·7753	·800	·3459	·52	— ·0647
·990	1·4125	·934	·7643	·795	·3357	·51	— ·0763
·989	1·3804	·932	·7537	·790	·3258	·50	— ·0878
·988	1·3511	·930	·7433	·785	·3160	·49	— ·0991
·987	1·3241	·928	·7332	·780	·3064	·48	— ·1104
·986	1·2990	·926	·7234	·775	·2970	·47	— ·1216
·985	1·2757	·924	·7138	·770	·2877	·46	— ·1327
·984	1·2538	·922	·7045	·765	·2785	·45	— ·1438
·983	1·2323	·920	·6953	·760	·2696	·44	— ·1547
·982	1·2139	·918	·6864	·755	·2607	·43	— ·1656
·981	1·1955	·916	·6776	·750	·2520	·42	— ·1765
·980	1·1781	·914	·6691	·745	·2434	·41	— ·1872
·979	1·1615	·912	·6607	·740	·2350	·40	— ·1980
·978	1·1457	·910	·6525	·735	·2260	·39	— ·2086
·977	1·1305	·908	·6445	·730	·2184	·38	— ·2192
·976	1·1160	·906	·6366	·725	·2102	·37	— ·2298
·975	1·1020	·904	·6289	·720	·2022	·36	— ·2403
·974	1·0886	·902	·6213	·715	·1943	·35	— ·2508
·973	1·0757	·900	·6138	·710	·1864	·34	— ·2612
·972	1·0632	·895	·5958	·705	·1787	·33	— ·2716
·971	1·0512	·890	·5785	·70	·1711	·32	— ·2819
·970	1·0396	·885	·5619	·69	·1560	·30	— ·3025
·968	1·0174	·880	·5459	·68	·1413	·28	— ·3230
·966	0·9965	·875	·5305	·67	·1268	·25	— ·3536
·964	·9767	·870	·5156	·66	·1127	·20	— ·4042
·962	·9580	·865	·5012	·65	·0987	·15	— ·4544
·960	·9402	·860	·4872	·64	·0851	·10	— ·5046
·958	·9233	·855	·4737	·63	·0716	·05	— ·5546
·956	·9071	·850	·4605	·62	·0584	·00	— ·6046

(50) When the whole discharge is not used up, but only a portion, formula (18) must be modified.

Let Q be the discharge per foot run which is taken out, and let D_1 be depth at a section where the surface slope is S_1. Then $Q = 100c\sqrt{RS_1} \times D_1$. As D_1 is substituted for R, being assumed the same value, the expression becomes

$$Q = 100c\sqrt{S_1}\, D_1^{\frac{3}{2}}$$
$$S_1 = (100c)^2\, D_1^3.$$

From this it is evident that the relations of D to D_1 are as $\sqrt[3]{S} : \sqrt[3]{S_1}$.

In the case in question let $Q = 30$ cubic feet, D_1, 10 feet, $S = \frac{1}{10{,}000}$, $100c = 80$, to find S_1, the water surface slope.

$S_1 = \frac{Q^2}{(100c)^2 D_1^3} = \frac{1}{7{,}100}$. Now S is $\frac{1}{10{,}000}$ whence $D = D_1 \left(\frac{7{,}100}{10{,}000}\right)^3 = 11{\cdot}2$ feet.

In employing the formula, $11{\cdot}2$ is the value to be used for D.

Let it be required to find the distance between two points where the depths of water are 10 and 9 feet.

Here $S = \frac{1}{10{,}000}$, $100c = 80$, $D = 11{\cdot}2$, $\frac{d_1}{D} = {\cdot}890$, $\frac{d_2}{D} = {\cdot}804$, $\phi\frac{d_1}{D} = {\cdot}5785$, $\phi\frac{d_2}{D} = {\cdot}3550$.

Whence $l = -\ 10{,}000\ (10 - 9) + 109{,}800\ ({\cdot}578 - {\cdot}355) = 14{,}400$ feet. From this it is gathered that an escape or outlet admitting the given discharge Q, or 30 feet per foot run of width, will not draw the water down in the canal to a depth less than 9 feet, if located 14,400 feet from the section where the depth is 10 feet.

(51) When the bed is level, as in a navigation channel, $S = o$, the following formula will then apply

$$l = \frac{(100c)^2}{4Q^2}\,(d_1^4 - d_2^4) - \frac{(100c)^2}{g}\,(d_1 - d_2). \qquad (19)$$

Using the same data as in the previous example, but with a level bed in the canal

$$l = \frac{6{,}400}{4 \times 900}\,(10{,}000 - 6{,}561) = \frac{6{,}400}{32}\,(10 \times 9) = 5{,}920 \text{ feet.}$$

Accordingly the water level with a horizontal bed is reduced in depth 1 foot in about one-third of the distance as when the inclination is 1 in 10,000 as previously. This proves the impaired capacity of discharge of a channel with a level bed.

Supply from run-off of Rainfall.

(52) In the cases of storage reservoirs it frequently happens that the supply is obtained, not by one large stream which can be gauged and its minimum and maximum supply estimated, but by numerous rills and

streams which cannot well be separately dealt with. Under such conditions the supply has to be estimated from the run-off of the rainfall, *i.e.*, the least and also the greatest supply available from the catchment area. This subject has been well treated in the "Madras Irrigation Manual," and as Southern India has a very extensive system of irrigation from storage as well as from direct canal irrigation, no better authority could be found. The following consists in great part of extracts from the above-mentioned work.

(**53**) *Rainfall.*—Rainfall is the source of all water used for irrigation purposes, and therefore a knowledge of its amount, character, seasons or periods, and of the effects produced by it, is of primary importance to all whose duty it is to design, carry out, improve or maintain irrigation works.

The resulting discharge from rainfall has to be considered in two ways: (1) the water to be utilised; (2) the water to be otherwise disposed of, and in connection with every irrigation work these two points require to be concurrently taken into account.

Rainfall Registers.—Rainfall registers are the foundation of a knowledge of the water resources of a country. It is desirable, therefore, that they should be kept in a convenient form at as many stations as possible; and, indeed, whenever there is someone available either permanently or temporarily to measure and record the rainfall. All old registers should be carefully preserved, abstracted and used as data. It should be remembered that an accurate record of the duration of all very heavy falls of rain is of much importance, and this should be impressed upon all in charge of rain-gauges.

(**54**) *Catchment Basin.*—The ground or country over which the rain falls and then drains off into the line of outfall or watercourse is called the catchment basin; the boundary line of this basin is the watershed.

Every irrigation work is dependent for its supply of water upon the discharge, due to rainfall, from a basin of greater or less extent, varying in Madras from the 115,570 square miles of the Godavari basin above the anicut, to the fraction of a square mile supplying a small tank. The discharge of a catchment basin may be stored in tanks or reservoirs, in which case the proportion of the whole discharge intercepted may be large or small; or it may be utilised by means of irrigation canals or channels, with or without the aid of an anicut or weir, to raise the water-level in the watercourse, which may be the immediate source of supply. The water so drawn off may be used for what is termed direct irrigation, as in the supply of canals, or for that of tanks. In all cases, however, the whole quantity of water discharged by the basin has to be considered, and divided into two parts, the one to be used for irrigation and the other to be safely passed on or discharged to waste.

(**55**) *Maximum Discharge of Catchment Basin.*—Upon a right estimate of the greatest quantity of water liable to be discharged by a catchment basin will depend the safety of the works which may exist, or which it may be proposed to construct, to utilise a part of the water for irrigation purposes. An absolutely correct determination of this quantity is not possible, owing to the ever-varying circumstances under which heavy rain-storms occur; but nevertheless it is quite practicable, if a right use be made of available data, to form a fairly accurate judgment of the quantity of water which may have been disposed of, and then, by allowing a reasonable margin for errors in the estimate, the requisite safety may be secured.

(56) *Flood Data and Flood Discharge Formulas.*—The first data to be ascertained and made use of are the records of the levels and discharges of floods which have occurred in the past. These data require to be very carefully examined and checked by comparison of the evidence obtained at different points; and when it has been decided what, in the case of a particular discharge, the flood levels were, it is necessary to ascertain from the rainfall registers whether that discharge is likely to have approximated to the maximum to be expected.

Experience has shown conclusively that the flood discharge from large areas is proportionately much less than that from small basins. This fact has led to the introduction of formulas to assist in the determination of these discharges, and if judiciously used, they are of considerable value for this purpose and as a check on the results obtained from local observation and evidence.

(57) It should be borne in mind, however, that though such formulas may be confidently used for the practical settlement of questions connected with the design of works, when the conditions of their applicability have been determined on adequate data, they are otherwise to be relied on only as a guide to the reasonableness or the reverse of the conclusions independently arrived at. The formulas which have been in use are the following:—

(1) Ryves' formula	. .	$Q = k_1\ 100\ K^{\frac{2}{3}}$	(20)
(2) Dickens' ,,	. .	$Q = k\ 100\ K^{\frac{3}{4}}$	(21)
(3) Fanning's ,,	. .	$Q = 200\ K^{\frac{5}{6}}$	(22)

in which K represents the area of the catchment basin in square miles, k is a coefficient depending for its value upon rainfall, soil, slope of ground forming the basin, etc., and Q is the resulting discharge, which is usually taken in terms of cubic feet a second. The first of these formulas assumes that the discharge from catchment basins of differing areas varies as the cube root of the square of the area, while in the second the variation is supposed to be as the fourth root of the cube of the area. The general result is, in the former case a much more rapid diminution of proportionate discharge as the area increases than in the latter case. The data for determining which is the more generally applicable formula for Southern India do not as yet exist, and either may be usefully employed under the restrictions above noticed as necessary. The following additional caution should, however, not be lost sight of. No such formula can be strictly applicable with the same coefficient to areas of various sizes even in the same part of the country and within the influence of the same intensity of rainfall, unless the other circumstances, such as slope of the ground, description of the soil, etc., be approximately similar.

(**58**) The values of $K^{\frac{2}{3}}$, $K^{\frac{3}{4}}$ for areas from 1 square mile to 50,000 square miles are given in Table VIII. The chief difficulty will be found in the selection of a suitable coefficient. For the comparatively limited areas in the coast districts, where the country is flat, and the drainage takes a longer time to run off, $k = 4$ or 5 have been found to be suitable coefficients in Ryves' formula, and 6·75 is a suitable coefficient for limited areas near the hills.

(**59**) The two Tables herewith given are (1) the discharges in thousands of cubic feet per second, calculated from the two formulas, Ryves' $Q = k_1 100\, K^{\frac{2}{3}}$ and Dickens' $Q = k 100\, K^{\frac{3}{4}}$. In both the formulas for an area of 1 square mile the discharge is equal to the coefficient employed, *i.e.*, the run-off is equal to the whole rainfall. The first square mile is the base of the formulas; as the area increases, the proportion of run-off becomes less, in one formula as $K^{\frac{2}{3}}$ and the other as $K^{\frac{3}{4}}$. To utilise the Tables properly, *i.e.*, to find the proper coefficient to be used, the method in Madras is to adopt as base an area of 5 square miles, in which area the maximum recorded rainfall is precipitated; this may be anywhere in a catchment basin. It is rightly assumed that heavy cyclonic storms only occur over a very limited area. In Table IX. the run-off from the areas in square miles is converted from Table VIII. into inches deep of run-off. But 5 square miles being taken as the base, this run-off will be equal to the rainfall. The example on p. 155 is taken from the "Madras Manual."

(**60**) "For example, suppose the greatest recorded rainfall within or near a catchment basin under investigation to have been 11 inches. The nearest run-off and rainfall to this in the line of 5 square miles (Table IX.) is 10·86 inches under coefficient 500 for Ryves' formula, and about midway between 400 and 500 for Dickens' formula. Were no other data available, either formula with the coefficient indicated might be used to obtain the approximate discharge from the catchment. Supposing the catchment area to be 500 square miles, the resulting maximum discharge would be

by Ryves' formula . 31,500 cubic feet per second (Table VIII.).
,, Dickens' ,, . 47,500 ,, ,, ,,

a very material difference. Suppose, however, for a part of the basin to be dealt with, say for 250 square miles, there should be a recorded maximum discharge of 20,000 cubic feet per second, and that the rainfall at the time this occurred was the highest on record, viz.; 11 inches. These data would indicate Ryves' formula to be applicable with the coefficient 500, whereas if

TABLE VIII.—(Q) DISCHARGES IN THOUSANDS OF CUBIC FEET A SECOND.

Areas in Square Miles. K.	$Q = k_1\,100\,K^{\frac{2}{3}}$. Coefficients (Ryves') (k_1).							$Q = k\,100\,K^{\frac{3}{4}}$. Coefficients (Dickens') (k).						
1	0.200	0.300	0.400	0.500	0.600	0.700	0.800	0.200	0.300	0.400	0.500	0.600	0.700	0.800
5	0.584	0.876	1.168	1.400	1.752	2.044	2.336	0.668	1.002	1.336	1.670	2.004	2.338	2.672
10	0.928	1.392	1.856	2.320	2.784	3.248	3.712	1.124	1.686	2.248	2.810	3.372	3.934	4.496
25	1.710	2.565	3.420	4.275	5.130	5.985	6.840	2.236	3.354	4.472	5.590	6.708	7.826	8.944
50	2.714	4.071	5.428	6.785	8.142	9.499	10.86	3.760	5.640	7.520	9.400	11.28	13.16	15.04
100	4.308	6.462	8.616	10.77	12.92	15.08	17.23	6.324	9.486	12.65	15.81	18.97	22.14	25.30
250	7.938	11.91	15.88	19.84	23.81	27.79	31.76	12.57	18.86	25.15	31.43	37.72	44.01	50.30
500	12.60	18.90	25.20	31.50	37.80	44.10	50.40	21.15	31.72	42.29	52.86	63.44	74.01	84.58
1,000	20.00	30.00	40.00	50.00	60.00	70.00	80.00	35.57	53.35	71.13	88.91	106.7	124.5	142.3
2,500	36.84	55.26	73.68	92.10	110.5	128.9	147.4	70.71	106.1	141.4	176.8	212.2	247.5	282.8
5,000	58.48	87.72	117.0	146.2	175.4	204.7	234.0	118.9	178.4	237.8	297.3	356.8	416.2	475.6
10.000	92.83	139.2	185.7	232.1	278.4	324.9	371.4	200.0	300.0	400.0	500.0	600.0	700.0	800.0
25,000	171.0	256.5	342.0	427.5	513.0	598.5	684.0	397.6	596.5	795.3	994.1	1,193	1.392	1.591
50,000	271.4	407.1	542.9	678.5	814.2	950.0	1,086	668.7	1,003	1,337	1,672	2,006	2.340	2,674

TABLE IX.—(d) *i.e.*, RUN-OFF IN INCHES DEEP FROM AREAS IN SQUARE MILES IN 24 HOURS.

$$d = \frac{9Q}{242K}\quad K = \text{areas in square miles.}$$

Areas in Square Miles. K.	$Q = k_1\,100\,K^{\frac{2}{3}}$. Coefficients (Ryves') (k_1).							$Q = k\,100\,K^{\frac{3}{4}}$. Coefficients (Dickens') (k).						
1	7.43	11.45	14.87	18.59	22.30	26.02	29.74	7.43	11.15	14.87	18.59	22.30	26.02	29.74
5	4.34	6.52	8.69	10.86	13.03	15.20	17.38	4.96	7.44	9.92	12.40	14.88	17.36	19.84
10	3.35	5.03	6.71	8.39	10.06	11.74	13.42	4.18	6.27	8.36	10.45	12.54	14.63	16.72
25	2.54	3.82	5.09	6.36	7.63	8.90	10.18	3.33	4.49	6.66	8.33	9.98	11.65	13.33
50	2.02	3.03	4.04	5.05	6.06	7.07	8.08	2.79	4.19	5.59	6.99	8.38	9.78	11.18
100	1.60	2.40	3.20	4.00	4.80	5.60	6.40	2.35	3.52	4.60	5.78	7.04	8.12	9.20
250	1.18	1.77	2.36	2.95	3.54	4.13	4.72	1.87	2.81	3.75	4.69	5.62	6.56	7.50
500	0.93	1.40	1.87	2.34	2.80	3.27	3.74	1.57	2.35	3.14	3.93	4.70	5.49	6.28
1,000	0.74	1.11	1.48	1.85	2.22	2.59	2.96	1.32	1.98	2.64	3.30	3.96	4.62	5.28
2,500	0.54	0.81	1.09	1.36	1.63	1.90	2.18	1.05	1.57	2.10	2.63	3.14	3.67	4.20
5,000	0.43	0.65	0.87	1.09	1.30	1.52	1.74	0.88	1.32	1.76	2.21	2.64	3.08	3.52
10,000	0.34	0.51	0.69	0.86	1.03	1.20	1.38	0.74	1.11	1.48	1.86	2.22	2.59	2.96
25,000	0.25	0.38	0.51	0.64	0.76	0.89	1.02	0.59	0.88	1.18	1.47	1.76	2.06	2.36
50,000	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.49	0.74	0.99	1.24	1.48	1.73	1.98

the discharge had been about 28,000 cubic feet per second, Dickens' formula with coefficient 450 would be more likely to give a correct approximation of the discharge of the larger basin."

(**61**) The following Table shows the coefficients arrived at in the Cuddapah district in Madras, and may be useful as a guide. How the coefficients were arrived at is not stated, but they do not apparently agree with the method just described.

TABLE X.

Group.		Av. Rainfall 9 years.	Max. Rainfall in 24 hours.	Coefficients k_1.
1st group	A	33·74	3·50	2 flat.
				2·5 mixed.
	B	32·40	4·25	3 hilly.
2nd group	C	29·45	5·40	
	D	31·82	5·50	2·5 flat.
	E	31·91	5·70	3 mixed.
	F	28·37	6·00	3·5 hilly.
	G	24·04	6·20	
3rd group	H	28·25	7·60	3 flat.
				3·5 mixed.
	J	28·44	8·41	4 hilly.
4th group	K	34·35	10·45	4 flat.
				4·5 mixed.
	L	40·37	13·45	5 hilly.

(**62**) In a large number of tanks in Bombay Presidency, some of very great water spread, the proportion of run-off was taken at one-quarter the average annual rainfall, and the length of the waste weirs calculated accordingly. Colonel Mullins, in the "Madras Irrigation Manual," is of opinion that this proportion is excessive, and that it should more correctly vary somewhat with the slope of bed of stream as well as being proportionate to the area of the catchment basin.

(**63**) The shape of the catchment affects the coefficient, and attempts have been made to modify the coefficient so as to take the latter into account. There is a great deal of literature on this difficult subject, but nothing of a definite character. The formula is $Q = k100 \frac{B}{L} K^{\frac{3}{4}}$ where B and L are breadth and length of the catchment (*vide* Jackson's "Hyd. Manual," Chap. I.).

The minimum discharge from a catchment area may be considered as 10 per cent. of the average rainfall.

When the capacity of a reservoir is well below its supply, it will fill several times during the rainy season, and its useful capacity can be taken to be $1\frac{3}{4}$ of its estimated storage capacity.

(64) The following Table of runs-off is given in Strange's "Storage Reservoirs," and is most useful for calculations of flood discharge based on actual values of run-off in inches per hour.

TABLE XI.—TABLE OF WASTE-WEIR RUNS-OFF.

1	2	3	4	5	6
Increments of Catchment Area.	Run-off from each increment of Catchment Area in col. 1.	Discharge from each increment of Catchment Area in col. 1 due to Run-off.	Discharge from Total Catchment Area due to Run-off.	Average Run-off from Total Catchment Area.	Remarks.
Square miles.	Inches per hour.	Cubic feet per second.	Cubic feet per second.	Inches per hour.	
0—1	3·00	1,936	1,936	3·00	
1—2	2·65	1,710	3,646	2·83	
2—3	2·30	1,484	5,130	2·65	
3—4	2·00	1,291	6,421	2·49	
4—5	1·85	1,194	7,615	2·36	
5—6	1·72	1,110	8,725	2·25	
6—7	1·62	1,045	9,770	2·16	
7—9	1·52	981	10,751	2·08	
8—9	1·45	936	11,687	2·01	
9—10	1·40	903	12,590	1·95	
10—15	1·16	3,743	16,333	1·69	
15—20	1·00	3,227	19,560	1·51	
20—25	0·92	2,968	22,528	1·40	
25—50	0·80	12,907	35,435	1·10	
50—75	0·75	12,100	47,535	0·98	
75—100	0·70	11,293	58,828	0·91	
100—150	0·65	20,973	79,801	0·82	
150—200	0·60	19,360	99,161	0·77	

Col. 3 = Increment of area in Col. 1 × Col. 2 × 645·33 cubic feet.
Col. 4 = Sum of entries in Col. 3.

$$\text{Col. } 5 = \frac{\text{Col. } 4}{\text{Last figure in Col. } 1 \times 645{\cdot}33}.$$

(65) Another rainfall table, with estimated run-off, is based on observations taken near Nagpur, in the central provinces of India, a usually dry tract, by Sir Alexander Binnie, published in the Min. Pro. Inst. C.E.

This gives the run-off and percentage for three catchments, classed as good, average, and bad. The good catchment approximates to that observed near Nagpur. It is based on general ideas, and will be of assistance when no actual recorded stream or river discharges are available.

TABLE XII.—TABLE OF TOTAL MONSOON RAINFALL AND ESTIMATED RUN-OFF AND YIELD PER SQUARE MILE FROM CATCHMENT AREAS. IN NAGPUR (C. P. INDIA).

1	2	3	4	5	6	7	8	9	10
	Good Catchment.			Average Catchment.			Bad Catchment.		
Total Monsoon Rainfall in inches.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. c. ft.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. c. ft.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. c. ft.
1	0.1	0.001	0.002	0.1	0.001	0.001	0.05	0.0005	0.001
2	0.2	0.004	0.009	0.15	0.003	0.006	0.1	0.002	0.004
3	0.4	0.012	0.028	0.3	0.009	0.021	0.2	0.006	0.014
4	0.7	0.028	0.065	0.5	0.021	0.048	0.3	0.014	0.032
5	1.0	0.050	0.116	0.7	0.037	0.087	0.5	0.025	0.058
6	1.5	0.090	0.209	1.1	0.067	0.156	0.7	0.045	0.104
7	2.1	0.147	0.341	1.5	0.110	0.255	1.0	0.073	0.170
8	2.8	0.224	0.520	2.1	0.168	0.390	1.4	0.112	0.260
9	3.5	0.315	0.732	2.6	0.236	0.549	1.7	0.157	0.366
10	4.3	0.430	0.999	3.2	0.322	0.749	2.1	0.215	0.499
11	5.2	0.572	1.329	3.9	0.429	0.996	2.6	0.286	0.664
12	6.2	0.744	1.728	4.6	0.558	1.296	3.1	0.372	0.864
13	7.2	0.936	2.174	5.4	0.702	1.630	3.6	0.468	1.087
14	8.3	1.162	2.699	6.2	0.871	2.024	4.1	0.581	1.349
15	9.4	1.410	3.276	7.0	1.057	2.457	4.7	0.705	1.638
16	10.5	1.680	3.903	7.8	1.260	2.927	5.2	0.840	1.951
17	11.6	1.972	4.581	8.7	1.479	3.435	5.8	0.986	2.290
18	12.8	2.304	5.353	9.6	1.728	4.014	6.4	1.152	2.676
19	13.9	2.641	6.135	10.4	1.980	4.601	6.9	1.320	3.067
20	15.0	3.000	6.970	11.25	2.250	5.227	7.5	1.500	3.485
21	16.1	3.381	7.855	12.0	2.535	5.891	8.0	1.690	3.927
22	17.3	3.806	8.842	12.9	2.854	6.631	8.6	1.903	4.421
23	18.4	4.232	9.832	13.8	3.174	7.374	9.2	2.116	4.916
24	19.5	4.680	10.873	14.6	3.510	8.154	9.7	2.340	5.436
25	20.6	5.150	11.964	15.4	3.862	8.973	10.3	2.575	5.982
26	21.8	5.668	13.168	16.3	4.251	9.876	10.9	2.834	6.584
27	22.9	6.183	14.364	17.1	4.637	10.773	11.4	3.091	7.182
28	24.0	6.720	15.612	18.0	5.040	11.709	12.0	3.360	7.806
29	25.1	7.279	16.911	18.8	5.459	12.683	12.5	3.639	8.455
30	26.3	7.890	18.330	19.7	5.917	13.747	13.1	3.945	9.165
31	27.4	8.494	19.733	20.5	6.370	14.799	13.7	4.247	9.866
32	28.5	9.120	21.188	21.3	6.840	15.891	14.2	4.560	10.594
33	29.6	9.768	22.693	22.2	7.326	17.019	14.8	4.884	11.346
34	30.8	10.472	24.329	23.1	7.854	18.246	15.4	5.236	12.164
35	31.9	11.165	25.939	23.9	8.373	19.454	15.9	5.582	12.969
36	33.0	11.880	27.600	24.7	8.910	20.700	16.5	5.940	13.800
37	34.1	12.617	29.312	25.5	9.462	21.984	17.0	6.308	14.656
38	35.3	13.414	31.163	26.4	10.060	23.372	17.6	6.707	15.581
39	36.4	14.196	32.980	27.3	10.647	24.735	18.2	7.098	16.490
40	37.5	15.000	34.848	28.1	11.250	26.136	18.7	7.500	17.424
41	38.6	15.826	36.767	28.9	11.869	27.575	19.3	7.913	18.383
42	39.8	16.716	38.835	29.8	12.537	29.126	19.9	8.358	19.417
43	40.9	17.587	40.858	30.6	13.190	30.643	20.4	8.793	20.429
44	42.0	18.480	42.933	31.5	13.860	32.199	21.0	9.240	21.466
45	43.1	19.395	45.058	32.3	14.546	33.793	21.5	9.697	22.529
46	44.3	20.378	47.342	33.2	15.283	35.506	22.1	10.189	23.671
47	45.4	21.338	49.572	34.0	16.003	37.179	22.7	10.669	24.786
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

(**66**) The accompanying Table of discharges of waste-weir channels, derived from Strange's "Storage Reservoirs," will assist calculations for depth of water flowing down bye-washes which are so common a feature in many large storage and diversion dams.

TABLE XIII.—TABLE OF THE DISCHARGES OF A WASTE-WAY CHANNEL HAVING A BED-WIDTH OF 200 FEET AND A BED-SLOPE OF 1 IN 100.

1	2	3	4	5	6	7	8	9
Total Depth.	Afflux Height, d_1.	Tail Depth, d_2.	Afflux Co-efficient, c_1.	Channel Co-efficient, c_2.	$\frac{D}{c_1 w \sqrt{2g}}$.	$\sqrt{d_1}\,(d_2 + \frac{2}{3} d_1)$.	Mean Velocity of Tail Channel.	Discharge, D.
Feet.	Feet.	Feet.					Feet per second.	Cubic feet per second.
1	0·30	0·70	0·60	41·00	0·495	0·495	3·40	476
2	0·74	1·26	0·60	52·10	1·510	1·505	5·78	1,457
3	1·24	1·76	0·60	59·00	2·85	2·87	7·78	2,741
4	1·70	2·30	0·62	64·4	4·47	4·46	9·66	4,444
5	2·15	2·85	0·64	68·8	6·34	6·29	11·42	6,510
6	2·63	3·37	0·66	72·0	8·29	8·29	13·03	8,784
7	3·08	3·92	0·68	75 0	10·46	10·45	14·55	11,407
8	3·52	4·48	0·70	77·5	12·80	12·84	16·04	14,374
9	3·95	5·05	0·72	79·5	15·23	15·28	17·41	17,585
10	4·36	5·64	0·74	81·4	17·87	17·87	18·80	21,210
11	4·77	6·23	0·76	83·1	20·56	20·51	20·11	25,057
12	5·17	6·83	0·78	84·5	23·34	23·34	21·38	29,203
13	5·54	7 46	0·80	85·8	26·24	26·20	22·57	33,668
14	6·00	8·00	0·80	86·8	29·44	29·40	23·61	37,775
15	6·47	8·53	0·80	87·7	32·65	32·61	24·56	41,893
16	6·94	9·06	0·80	88·6	36·03	36·00	25·51	46,236
17	7·42	9·58	0·80	89·3	39·47	39·52	26·43	50,645
18	7·86	10·14	0·80	90·0	43·10	43·06	27·28	55,304
19	8·33	10·67	0·80	90·7	46·88	46·81	28·19	60,157
20	8·80	11·20	0·80	91·3	50·67	50·63	29·03	65,035

Discharge through Syphons.

(67) The head or difference of levels above and below a syphon is generally required to be ascertained, the mean velocity through the barrels being a given quantity. This is sometimes taken at 15 feet per second in order to clear out all detritus, 8 feet being usual. The head is found by the following formula

$$H = M\frac{V^2}{2g} = \cdot0155\ MV^2 = hM \qquad (23)$$

h being the head due to the velocity V, or $\cdot0155\ V^2$.

In this expression $M = \left(1 + f_1 + f_2\frac{L}{R}\right)$ in which L = length of barrel, R = hydraulic mean depth, or $\frac{\text{Area}}{WP}$, f_1 is the coefficient for loss of head by entry, and varies from ·08 for an ideal converging entrance to ·505 for an abrupt change from a wide channel into a round pipe or square culvert. This latter value, or some modification of it, is most suitable for masonry syphon barrels; f_2, the second coefficient, is that of friction on the sides of the barrel and $= a\left(1 + \frac{b}{R}\right)$.

Suitable values of a and of b must be selected from the Table below.

Nature of Barrel.	a	b
Iron pipes	·00497	·084
Cement plaster	·00316	·10
Ashlar or brickwork	·00401	·23
Rubble masonry	·00507	·82

If the converse to formula (23) is required

$$V = 8{\cdot}035 \sqrt{\frac{H}{M}}. \qquad (24)$$

If velocity of approach be considered

$$H = M \times \frac{V^2}{2g} - \frac{V_a^2}{2g} \text{ or } = {\cdot}0155 \left(MV^2 - V_a^2\right). \qquad (25)$$

$$\text{Conversely } V = \sqrt{\frac{2g\ H + V_a^2}{M}}. \qquad (26)$$

For example, let the syphon barrels be 6 feet long by 4 feet deep, then $R = \frac{A}{WP} = \frac{24}{14} = 1{\cdot}7$ nearly. Let $L = 170$ feet, then $\frac{L}{R} = 10$. Further, let V be assumed as 15, f_1 as ·5, a as ·004, and b ·23 from Table, then $f_2 = {\cdot}004\left(1 + \frac{{\cdot}23}{1{\cdot}7}\right) = 1{\cdot}545$.

By formula (23), $H = {\cdot}0155 \times 1{\cdot}545 \times 225 = 5{\cdot}38$ feet.

If velocity of approach of 3 feet per second be assumed, the above value of H will be reduced by $\cdot0155 \times 9$, or by ·1395.

H will then $= 5{\cdot}14$ feet. This shows the small effect of velocity of approach and the great effect of the coefficient f_1.

USEFUL MEMORANDA.

(68) 1 cubic foot of water $= 62{\cdot}4$ lbs. $= \frac{1}{36}$ ton.

1 ton of water = 36 cubic feet.

Inches of rainfall = 53·3 acre-feet per square mile.

1 inch run-off per hour per square mile ≐ 645·3 second-feet = 1 cubic foot per acre per second.

1 cubic foot per second ≓ 2 acre-feet.

Feet per second = ·68 miles per hour.

1 acre-foot = 43,560 cubic feet.

Pressure of water per square foot $= \frac{H}{36}$ tons.

VELOCITY OF WATER.

(69) V = Theoretical velocity in feet per second.

g = Force of gravity = 32·2.

$2g = 64{\cdot}4$. $\sqrt{2g} = 8{\cdot}025$. $\frac{1}{2g} = 0{\cdot}0155$.

H = Head of water in feet.

$V = \sqrt{2gH} = 8{\cdot}025\sqrt{H}$.

$H = \frac{V^2}{2g} = 0{\cdot}0155\,V^2$.

TABLE XIV.—THEORETICAL VELOCITY DUE TO DIFFERENT HEADS IN FEET PER SECOND, or $\sqrt{2gH}$.

Head in Feet.	0	1	2	3	4	5	6	7	8	9
0	—	8·025	11·35	13·90	16·05	17·94	19·66	21·23	22·69	24·08
10	25·38	26·62	27·80	28·93	30·00	31·08	32·10	33·09	34·05	34·98
20	35·89	36·77	37·64	38·48	39·31	40·12	40·92	41·70	42·40	43·21
30	43·95	44·68	45·40	46·10	46·79	47·47	48·15	48·82	49·47	50·11
40	50·75	51·39	52·01	52·62	53·23	53·83	54·43	55·02	55·60	56·18
50	56·74	57·31	57·87	58·42	58·97	59·51	60·05	60·59	61·11	61·64
60	62·16	62·68	63·19	63·69	64·20	64·70	65·19	65·69	66·18	66·66
70	67·14	67·62	68·09	68·57	69·03	69·50	69·96	70·42	70·87	71·33
80	71·78	72·23	72·67	73·11	73·55	74·00	74·42	74·85	75·28	75·71
90	76·13	76·55	76·97	77·39	77·81	78·22	78·63	79·04	79·44	79·85

CHAPTER VI

DIVERSION WEIRS ON SAND FOUNDATIONS

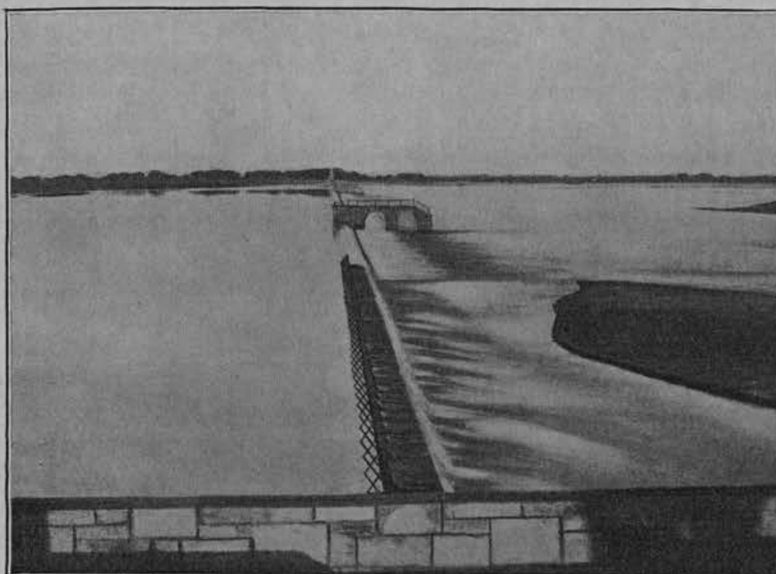


FIG. 1a.—Narora Weir, Lower Ganges Canal.

(1) A CLASS of weir peculiar to India includes those erected across the great rivers of the peninsula, such as the Ganges, the Jumna, the Chenab, the Jhelum, and many others in Upper India, and the Mahanuddee, Sôn, Kistna, Godaveri, and Pennér in Bengal and Southern India. These are naturally exclusively diversion weirs, and are of no great height, 10 or 12 feet above normal river bed level, or low water level being generally the outside limit of their height. What they lack in this respect is, however, made up, not only in length but in width. The weir over the Sôn River at Dehri is 2½ miles long, and those spanning the Great Godaveri River, with its flood discharge of over 1,200,000 second-feet, are nearly as long. The Okhla Weir is 250 feet wide, and several others run this dimension very close. Thus it is that these canal head works rank among the largest and most important in the whole world.

There could hardly be a greater contrast between the narrow but immensely high American dams, built over narrow rocky gorges amid wild and sterile surroundings, and these long, low Indian weirs, which are generally situated among cultivated lands and a teeming population, often amidst historic remains of great antiquity. The main peculiarity of these Indian weirs is the nature of the foundations, which is nothing but pure sand, and that frequently of a very light and friable nature. This sand reaches down to immense depths, so that the foundations of the structure cannot possibly attain to solid clay or rock.

Up to a quite recent period the design of such works had not been based on any definite principles, the profiles being merely more or less close copies of older works of similar nature, and the sections were eventually adjusted by failure and renewal to actual requirements.

Now, however, that the principles regulating the stability of structures founded on a pervious and loose material as sand have been definitely evolved, their design is simply a matter of the practical application of these principles.

A drowned diversion weir on sand, although its height is seldom over 10 feet above the normal river bed, is not only exposed to the destructive influences of a large river in flood, but its foundation, being necessarily the

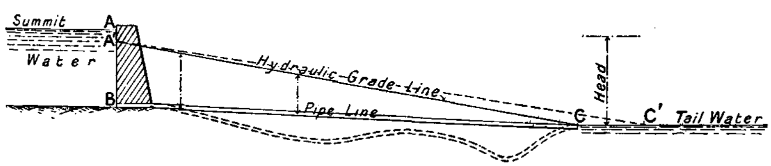


FIG. 1.—Diagram of Pipe under water pressure.

sandy bed of the river, is liable to be undermined and washed out by the hydrostatic pressure of the water upheld in rear of the weir. In spite of these drawbacks, it is, however, quite practicable to design a work of such outline as will successfully resist all these disintegrating forces, and remain a permanent and solid structure.

(2) The principle which underlies the action of a head of water on a porous stratum of sand, over which a heavy impervious weight is imposed, is analogous to that which obtains in a pipe under pressure.

Fig. 1 exemplifies the case with regard to a pipe-line BC leading out of a reservoir. The head H, is the difference of levels between A', a point somewhat lower than A (the actual summit level) and, C, the level of the tail water beyond the outlet of the pipe. The water having a free outlet at C, the line $A'C$ is the hydraulic gradient or grade line. The hydrostatic pressure in the pipe at any point is measured by vertical ordinates drawn from the centre of the pipe to the grade line $A'C$. The uniform velocity of the water in the pipe is dependent directly on the head, and inversely on the frictional resistance of the sides of the pipe, that is on its length. This supposes the pipe to be straight, or nearly so.

(3) We will now consider the case of an earthen embankment thrown across the sandy bed of a stream. The pressure of the impounded water will

naturally cause leakage beneath the impervious earthen base. With a low depth of water impounded, it may well be understood that such leakage might be harmless, that is, the velocity of the percolating undercurrent would be insufficient to wash out the particles of sand composing the foundation of the dam. When, however, the head is increased beyond a safe limit, the so-termed "piping" action will take place and continue until the dam is completely undermined.

(4) The main determinating factor in the stability of the sand foundation is evidently not the superimposed weight of the dam, as the sand is incompressible, although a load in excess of the hydrostatic pressure must exercise a certain, though possibly undefined, salutary effect in opposing disintegration of the substratum. However this may be, it is the enforced length of percolation or the so-called *creep* of the undercurrent that is the real determining influence.

(5) In the case of a pipe, the induced velocity is inversely proportional to its length. In that under consideration, the hydraulic condition being practically identical, it is the enforced percolation through the sand and the resulting friction against its particles as the water forces its way through this medium, that effects the reduction of the velocity of the undercurrent, and this frictional resistance is clearly directly proportional to the length of passage.

In the example under consideration, this length of enforced percolation is evidently that of the impervious base of the dam. The moment this obstruction is passed the water is free to rise out of the sand, and the hydrostatic pressure ceases.

(6) It is evident, therefore, that to ensure safety from undermining, this length of enforced percolation, or creep, which will be symbolised by (l), must be some multiple of the head H; and if a reliably safe value for this factor or coefficient can be found suitable to any particular class of sand, we shall be enabled to design any work on a sand foundation with perfect confidence with regard to stability. If this coefficient be symbolised by c, then l, or the length of enforced percolation, will equal $c \times H$, H being the head of water. The coefficient c will vary in value with the quality of the sand.

(7) Fig. 2 represents a case similar in every respect to the last, only that instead of a dam of earth the obstruction consists of a vertical wall, termed the weir or drop-wall, having a horizontal impervious floor $ACDB$ attached thereto, which appendage is necessary to prevent erosion of the bed by the current of falling water when the weir is overtopped.

The level of the tail water is supposed to be that of the floor level; consequently the hydraulic gradient will be HB, and, as in the previous case of the pipe-line, the ordinates of the triangle HAB will represent the upward hydrostatic pressure exercised on the base of the weir wall and of the floor.

(**8**) The safety of the structure is evidently dependent on the following points:—

First, the weir wall must be dimensioned so as to resist the overturning moment of the horizontal water pressure. This has been dealt with in a previous chapter.

Secondly, the thickness, *i.e.*, the weight, of the apron or floor must be such that it will be safe from being blown up or fractured by the hydrostatic pressure. And thirdly, its base length, or that of the enforced percolation (l), must not be less than $c \times H$, or than the product of the coefficient c with the head H.

It is evident that the value of the coefficient c must vary with the nature of the sand substratum in accordance with its qualities of fineness or coarseness. Fine, light sand will be closer in texture, passing less water under a given head than a coarser variety; but at the same time fine sand will be disintegrated and washed out under less pressure. Reliable values of c, on which the design mainly depends, can only be obtained experimentally, not from artificial experiments, on a small scale, but by deduction from actual examples of weirs, among which the most valuable are the records of failures due to insufficiency in length of percolation. From these statistics a safe value of the relation of l to H, or of the coefficient c, which also defines the inclination of the hydraulic gradient, can be derived.

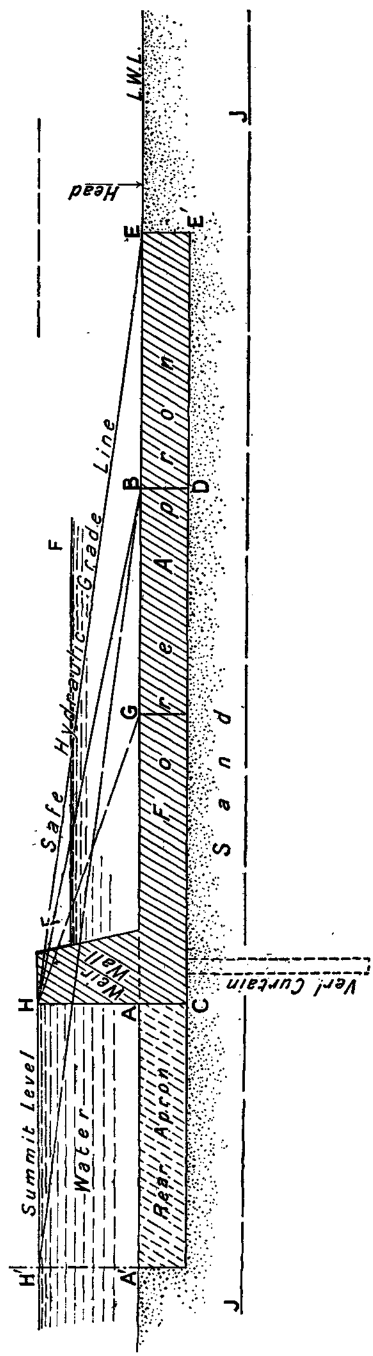


FIG. 2.—Diagram of Weir with floor.

(**9**) The following values of c have been adopted for the classes of sand detailed below, the derivation of which will be explained later:—

Class I. River beds of light silt and mud, as the Nile. $c = 18$.

Class II. Fine micaceous sand, as in Himalayan rivers and such as the Colorado in the United States. $c = 15$.

Class III. Coarse-grained sands, as in 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.

(**10**) In Fig. 2, supposing that the width of the base of the apron and drop wall, or CD, is found just sufficient to produce equilibrium as regards length of percolation, then the hydraulic gradient will be HB. The value of l thus provided will, however, be inadequate. A factor of safety of one-half is required, so that the floor should be extended to E, AE being equal to $1\frac{1}{2}$ AB. The apex of the triangle of pressure will then be moved to the point E, the hydraulic gradient being AE. This extension provides increased safety from the danger from piping, but enlarges the area of hydrostatic upward pressure from the triangle HAB to HAE. Consequently, in addition to the element of cost, there is a distinct disadvantage in lengthening the impervious floor beyond what is requisite to ensure an absolutely sufficient length of creep.

Supposing the width of the floor to be reduced below the minimum CD or AB to AG, the hydraulic gradient will then be HG. This reduces the area of the triangle of pressure to HAG, but failure will take place by piping, the sand substratum being gradually washed out, causing the floor to collapse.

(**11**) The proper profile of the apron to resist the hydrostatic pressure is evidently that of a triangle, its base being at the root or toe of the drop wall. In practice this will be a trapezoid, as the end thickness cannot be made less than about 3 feet. Its weight can be conveniently represented by $t \times \rho$, t being the thickness and ρ the specific gravity of the material.

The value of $t\rho$ must exceed that of the upward hydrostatic pressure by at least one-third. This latter can be represented by the symbol $(H—h)$, H being the head acting on the base and h the neutralisation of pressure, that is, of head, effected by the length of percolation up to this point. Let l^1 be this length, then $h = \frac{l^1}{c}$.

(**12**) At this juncture a few points regarding loss of weight due to displacement of immersed parts of the weir require notice.

With regard to the hydrostatic pressure on the base:

In Fig. 2, if a hole were supposed bored in the floor CE and a pipe inserted, the water would rise up as far as the hydraulic gradient HE. The pressure acting on the base CE would therefore be greater than the triangle HAE—would, in fact, be the trapezoid $HCE'E$. This is accounted for by the addition to the external head, which is HA, of that due to the displacement of the floor.

To avoid confusion, the extraneous or active head of water symbolised by H, which always represents the difference of levels above and below a weir or regulator, will be kept distinct from that due to displacement or immersion, this latter pressure being allowed for by reduction in the effective weight of the immersed body.

(13) A solid material immersed in water loses, as is well known, a proportion of its weight equal to the weight in water of the area immersed; or, if specific gravity be substituted for weight, and if the letter ρ, symbolise the specific gravity of a material when the same is immersed in water, its specific gravity can be considered as if reduced by unity, or to be $(\rho—1)$.

Thus, if the specific gravity of a block of masonry be $2\frac{1}{4}$ when immersed, the reduced weight can be expressed by $(2\frac{1}{4} - 1) = 1\frac{1}{4}$.

(14) In Fig. 2, when the low water level is at E the floor is immersed and its effective weight is as $t\ (\rho—1) : t\rho$, t being the thickness. When the water level rises to FF, the status is in no way altered, as the excess of upward pressure above what it was at the lower level is compensated by an equivalent load of water lying above the floor.

When the low water level is at CDE' the weight is unimpaired, when half-way between E and E' half of the floor will be considered as of lower specific gravity than the other half.

If the water level were to fall below the level of the base of the floor as to JJ, the sand substratum being porous, when the latter is under pressure, as from a head of water held up by the weir wall, the water will rise up to the base of the impervious floor, thus practically reducing the head from HJ to HC. This is conditional on its being contained in the pressure area, *i.e.*, below the hydraulic gradient. The head acting on the apron cannot be measured to any depth lower than the base of the latter.

(15) Another most important point requiring notice is the relative position of the apron. With regard to the drop wall in Fig. 2, supposing the impervious floor were extended backwards to A', and AA' made $= BE$, then the action of the head of water is thrown back from H to H' and the hydraulic gradient will be $H'B$ parallel to $H'E$, and the statical condition in no wise altered.

Thus we see that the position of the drop wall with reference to the floor, or apron, is immaterial as regards foundation stability; but on the other hand a certain length of solid apron in front of the drop wall is necessary to protect the bed from erosion, so that only a proportion of the floor length can well be placed in rear of the drop wall.

(16) This rear projection is termed the rear apron, in contradistinction to the fore apron or floor proper lying in front of the drop wall. The rear apron is in a peculiarly advantageous position. It is almost entirely free from erosive action except from cross currents, and is not subjected to any hydrostatic pressure. The latter, represented by the trapezoidal area of pressure $H'A$, is more than counterbalanced by the superincumbent weight of water, the rectangle $H'A$. For this reason the rear apron can be constructed of less expensive material than the fore apron. On the other hand, it must be impervious and must have a watertight connection with the drop wall, otherwise water pressure may act on the base between it and the rest of

the work, in which case it would become isolated, and its value as a component part of the structure entirely destroyed.

This actually happened in the case of the Narora Weir, to be exhibited later, where the rear apron being cut off, the value of l, the effective length of base, was reduced below the safe limit and failure occurred.

(**17**) From the above, it would seem that the rear apron would be effective if only a thin, impervious layer, but weight is necessary to some extent in order to prevent the possibility of the undercurrent partaking of the nature of a surface current, and so losing the efficacy of the frictional resistance of the sand. For the same reason it is always advisable, as has already been noted, to make the effective weight of the fore apron at least one-third in excess of actual requirements as regards hydrostatic pressure.

Weight in these submerged structures is always a desideratum, and is only limited by the necessities of cost.

(**18**) We have seen that the rear apron has to be limited in width because it performs but one function, viz., neutralisation of statical pressure; whereas the fore apron, or floor, performs a double duty, having, in addition to its functions in a hydrostatical sense, to provide a solid cushion for resistance to the erosive force of the current of falling water, for which purpose width also is a necessity. The width need not be all composed of impervious masonry, but can be formed in the lower portion, or talus, of loose riprap. The total width is dependent on several considerations, and must remain more or less a matter of individual judgment, although certain approximate rules can be deduced from examination of sections of existing works of similar character which will prove reliable guides to the designer. This point will come up again later, in pars. 25 and 27.

(**19**) The line of creep, we have hitherto considered, has been a horizontal one. If vertical depressions are placed below the base of the floor, or apron, the line of percolation, as has been proved by experiment, is forced to follow round these obstructions, and does not, as might be imagined, take the line of least resistance. Thus, if an impervious line of sheet piling or a curtain wall of masonry, as one of undersunk hollow blocks connected together by piling, be inserted below the floor, as near C in Fig. 2, the line of creep will follow down one side of the vertical obstruction and up the other side. The added length of creep will thus be *twice* the depth of the curtain.

The insertion of a curtain wall of any kind, is therefore a most valuable means of increasing the length of the enforced percolation, and of thus immediately reducing the pressure at any part of the base. The effect of vertical depressions in the floor on the hydraulic gradient line, is to break the continuity of the hypothenuse of a triangle of pressure by steps, each step being equal to the depth of the vertical obstruction divided by the coefficient, or if the obstruction is sheet piling, or a very narrow wall, the depth at this point will be twice the above, or $2\,l' \div c$. It is, however, considered that the efficacy of curtain walls in this respect is dependent on their not being spaced nearer to each other than twice their depth.

(20) A practical example of the method of applying the principles already set forth will now be given in Fig. 3 under the conditions actually prevailing in the Narora Weir on the Ganges River. This work is of the direct overfall type, which will be classified as type A. The data on which the design is based are as follows :—Sand, class 2—coefficient $c = 15$; H, or difference of summit and of low water levels—the latter always symbolised by the letters L.W.L., 13 feet; then the required lengths of creep, or of l, must be $c \times H = 15 \times 13 = 195$ feet. We have now to decide what proportions of this length are to be placed in the fore apron, in the rear apron, and in the vertical sheet piling. The minimum allowable width of the fore apron is naturally a leading consideration in this problem. This width of apron is affected in two ways, firstly, by the nature of the river bed, represented by its coefficient c, and secondly, by the obstruction offered by the weir wall, *i.e.*, by the height of the permanent crest, excluding the crest shutters, which latter are only in use during low water. The height of the shutter crest above the floor, *i.e.*, the depth of the drop, will be symbolised by H^a, in order to differentiate it from H the head of water. In some cases H^a and H are identical.

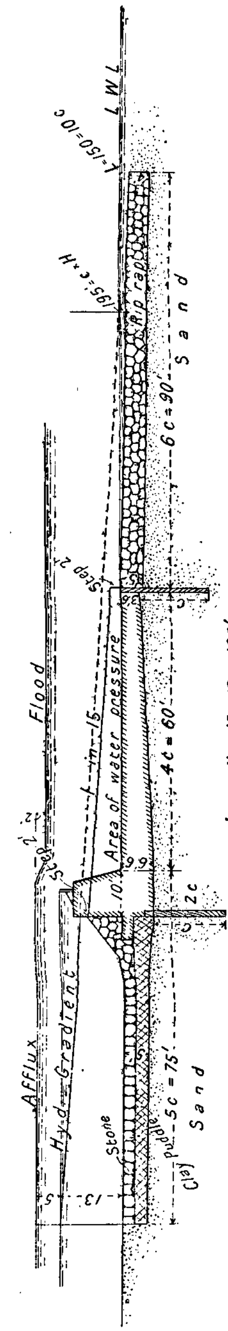


FIG. 3.—Alternative Section for Narora Weir.

(21) Taking Narora as standard, the following formula has been evolved, based on the theory that a proportion of $4c$, is a proper width of a weir of 13 feet in height; and for greater or less heights the length is subject to variation in proportion to the square root of the heights of shutters above floor—or as $\sqrt{H^a} : \sqrt{13}$.

W is the width of floor or apron. Then,

$$W = 4c + \sqrt{\frac{H^a}{13}}. \qquad (1)$$

In the sloping apron, type B (*vide post*), H is taken for H^a.

The use of c as a factor makes the width some function of the quality of the sand, which is admittedly a sound proposition.

(22) In designing a weir, the dimensions relating to l should, for convenience sake, be all multiples of the coefficient c, as by this means the

neutralisation of head at any point is at once ascertained, each length in feet equal to c, representing the neutralisation of 1 foot of head.

(**23**) In the present instance, to revert to the design, Fig. 3, the width of the fore apron will be according to formula (1), $4c\sqrt{\frac{13}{13}} = 4c = 4 \times 15 =$ 60 feet. This will leave $13 - 4 = 9c$ to be disposed of in the rear apron and in the sheet piling. If the latter be given a total depth of $2c = 30$ feet, for two lines of curtains, a further item of $4c$ can be deducted, leaving $5c$ for the width of the rear apron. This width is measured backwards from the toe of the drop wall, and is 75 feet. Out of the 30 feet allowed to the sheet piling, a depth of 15 feet will be placed below the weir wall, equivalent to an effective length of $2c$. The remainder will be situated at the toe of the fore apron, where it will serve another purpose in addition to its ordinary function, viz., of forming a protective barrier at the end of the masonry floor in case the loose stone continuation is washed out.

Thus the full required value of l, or of cH, *i.e.*, 195 feet, is provided. It may be noted that the small vertical drop, due to the thickness of the rear apron, as well as that due to the thickness of the toe of the fore apron, has been neglected. It will be seen that there are two steps in the gradient line (which is 1 in 15), one of 2 feet, at the incidence of the rear sheet piling, and another, also of 2 feet, at the termination of the floor. The designer can, of course, vary this arrangement at pleasure, by reducing the vertical curtains and increasing the rear apron.

(**24**) As regards the thickness of the parts: the rear apron is composed of clay puddle overlaid with stone riprap, a construction which provides a heavy impervious platform, economical in point of cost, and in its protected position as safe as if constructed of masonry. It is given a thickness of 5 feet.

We next come to the drop wall, provided with crest shutters 3 feet deep, which collapse automatically when freshets come down. Its base thickness is made 10 feet or

$$\frac{H^c + d}{\sqrt{\rho}}, \qquad (2)$$

d being that depth of film passing over the crest, which is reciprocal to a down stream water level at the crest line; d in this case is calculated to be $3\frac{1}{2}$ feet. The $\sqrt{\rho}$ in this case is $\frac{3}{4}$, as ρ is taken for the brick wall at a value of 2. This formula of $(H + d) \div \sqrt{\rho}$ applies to all dwarf submerged weirs (*vide* par. 42, Chap. II.). H^a is shutter crest above floor; H is the head; H^b weir wall crest above L.W.L. H^c is the height of the masonry drop wall above floor.

(**25**) We now come to the thickness of the fore apron at its root or commencement. This is really the most critical point in the whole design. This thickness can be varied at pleasure by the values given to the rear apron and to the rear sheet piling, and is a matter always kept in mind when deciding what these values should be. This thickness must not, as a rule, be less than 4 or 5 feet, except in very low weirs, and should not much exceed the higher dimension either, as otherwise the fore apron will become unnecessarily costly. Up to this point the head (h) neutralised by the length of creep is equivalent to that of $7c$, or 7 feet of head; this leaves a value of $(H - h)$, $= 6$ feet of upward water pressure. We have already noted that the effective weight must exceed that of the hydrostatic pressure by at least one-third. The floor is immersed, and its specific gravity considered as reduced by unity; consequently the thickness of the floor (t) at this point can be expressed in the following formula:

$$t = \tfrac{4}{3}\,\frac{H - h}{(\rho - 1) \text{ or } \rho} \qquad (3)$$

$= \tfrac{4}{3}\,\dfrac{6}{1\tfrac{1}{4}} = 6{\cdot}4$ feet. In (3), if the floor is immersed the denominator will be $(\rho - 1)$, if not, it will be ρ. It has been made $6\tfrac{1}{2}$ feet thick, tapering to $3\tfrac{1}{2}$ feet at the end—average 5 feet. The pressure trapezoid is shown in the space enclosed between the surface of the floor and the grade line. The terminal thickness of the floor, $3\tfrac{1}{2}$ feet, may be considered almost as a minimum. The insertion of the 15 feet deep sheet piling here causes 2 feet hydrostatic pressure at the toe of the apron. There is no objection to this, as the apron is capable of withstanding $3\tfrac{1}{2}$ feet pressure. In fact the arrangement may be considered as utilising a part at least of the excess weight at this point. Weight is, however, desirable, solidity being required in the fore apron apart from purely hydrostatical considerations.

(26) There now only remains the packed stone pitching, or riprap, which forms the talus of the weir body. The width of this is dependent on the same kinetic considerations which influence the width of the masonry floor, but with two others in addition. These are, firstly, the nature of the river bed, which will, as before, be represented in the formula by its coefficient; secondly, H^b, the height of the permanent obstruction above L.W.L. Where the floor surface is at L.W.L., H^b will equal H^c; where, however, it is raised, as in Figs. 5 and 6, or where in other types there is no floor but an inclined apron, it will be the height of the obstruction above L.W.L., or normal bed level which may be higher. Thus where no crest shutters are used H^b will equal H. The second matter requiring consideration is the flood discharge per foot run of weir, which is symbolised by the letter q. This item varies considerably from 66 second-feet over the Dehri Weir (Fig. 17), to 244 second-feet over the Madaya (Fig. 22) and must necessarily have considerable influence on the width of the talus. Narora Weir (Fig. 4) will again be taken as a guide in framing a formula generally suitable for all conditions. The end of the talus measured from the drop wall was originally 140 feet, and was subsequently increased to 170 feet after the accident. It is, however, deemed that a width of 150 feet or of $10c$ would fairly represent a proper average value for (L) or the distance of the toe of the talus from the drop wall.

(27) The extreme width of the apron, including talus, is clearly independent of that appropriated to the floor or masonry apron; its width L is

consequently measured from the drop or crest wall. The formula will then become—

$$L = 10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}. \qquad (4)$$

For sloping aprons a somewhat higher factor, viz., $10\frac{1}{2}$ or $11c$, might be adopted.

This formula is grounded on the theory that the width L varies with the square root of the height of the obstruction (H^b) and with that of the unit flood discharge (q), the standard being what these values, viz., 10 and 75 respectively are in Narora Weir. This height H^b, as already stated, is equal to H^c when there are no crest shutters and L.W.L. is at floor level, and is always the depth of L.W.L. below the permanent masonry crest of the weir. This formula, though more or less empirical, gives results remarkably in consonance with actual values, and consequently will, it is believed, form a valuable guide to the designer. The following table will conclusively prove this.

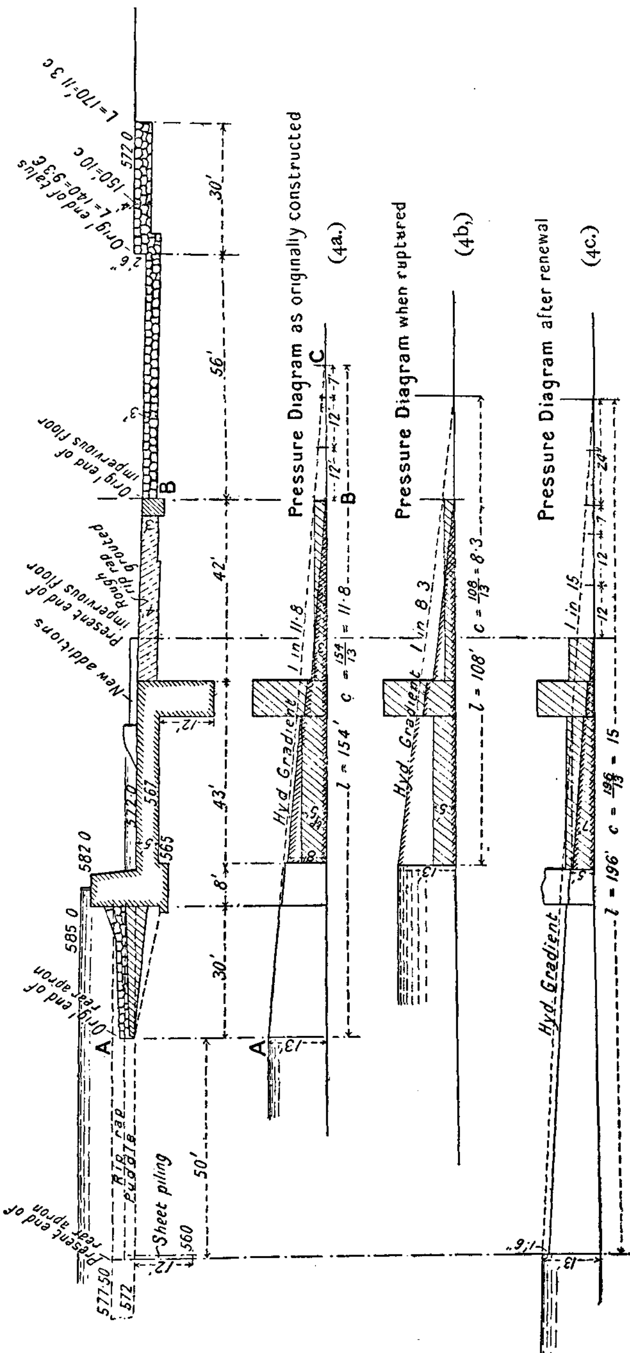
TABLE I.—FORMULA $L = 10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$ (4). SHOWING ACTUAL AND CALCULATED VALUES OF L OR TALUS WIDTH.

River.	Name of Work.	Type.	C.	H^b.	q.	Calculated value of L.	Actual value of L.
Ganges - -	Narora - -	A	15	10	75	150	140 to 170
Colerun -	Colerun - -	A	12	$4\frac{1}{2}$	100	106	72
Vellar - -	Pelandorai -	A	9	11	100	108	101
Tampraparni -	Srivakantham -	A	12	6	90	102	105
Chenab -	Khanki - -	B	15	7	150	182	170
Jhelum - -	Jhelum - -	B	15	6	135	160	135
Pennér - -	Adimapali -	B	12	$8\frac{1}{2}$	184	172	184
,, - -	Vellore - -	B	12	9	300	228	232
,, - -	Sangam - -	B	12	10	147	168	145
Godaveri -	Dauleshwiram -	B	12	13	100	158	217
Jumna - -	Okhla - -	C	15	10	140	210	210
Kistna - -	Beswada - -	C	12	13	223	236	220
Son - -	Dehri - -	C	12	8	66	100	96
Mahanadi -	Jobra - -	C	12	10	140	163	143
Madaya - -	Madaya - -	C	12	8	280	257	235

Type A, Vertical drop, wall with horizontal floor. (B) Sloping masonry apron. (C) Rock fill "anicut," with sloping apron.

(28) The Narora Weir itself forms a most instructive object lesson of the proper base length, or value of l, suitable for sands of class 2.

Fig. 4 (full lines) represents the work as originally constructed. The pressure diagram, Fig. 4a, shows that the hydraulic gradient works out to less than 1 in 12, and in addition to this, the floor is very deficient in weight. The hydrostatic pressure at the toe of the drop wall is 8 feet of water; to meet this the floor has a value of tp of only 5 feet. The specific gravity is taken at 2, which figure the floor would not be likely to exceed, being mostly formed of brick concrete. In consequence of submergence the effective specific gravity is only unity. This work stood as constructed with but little repairs for about twenty years, when a heavy freshet having set up a cross current, the rear apron of puddle protected by stone riprap was washed away. On this occurrence failure at once took place, and the whole floor as well as its continuation in grouted riprap blew up. The diagram, Fig. 4b, clearly shows the excessive pressure in the floor, which is 13 feet of water against only 5 of the floor. This full pressure, probably in actual fact, extended without diminution right through up to the curtain wall, the pressure area being a rectangle instead of a trapezoid. It is otherwise probable that the grouted pitching would not have blown up, as actually occurred. Previous examination of the sand foundation shortly



FIGS. 4, 4a, 4b, 4c.—Narora Weir, Ganges River. Discharge (circ.), 300,000 second-feet. Length, ¾ mile.

before the accident divulged the fact that piping had already removed the greater part of the sand from beneath the floor, which at that time was actually prevented from collapse solely by the water pressure, and had been in that dangerous condition for some time. As soon as ever the rear apron was completely washed out, the increased pressure lifted the floor. We learn from this that the originally given hydraulic gradient of 1 in 11·8 was insufficient, and when this was reduced to 1 in 8, failure took place.

(29) In restoring the work, the rear apron was run out backwards, as shown dotted in Fig. 4, to a distance of 80 feet beyond the drop wall, and was made $5\frac{1}{2}$ feet thick, of puddle covered with riprap and provided near the weir wall with a solid masonry covering. The puddle foundation also was sloped down to the level of the floor base, so as to form a sound connection with the drop wall. At its extreme termination, sheet piling was driven to a depth of 12 feet below floor level, *i.e.*, below R.L. 572. The grouted pitching in the fore apron was relaid dry, the first 10 feet only having been rebuilt in mortar, so as to form a continuation of the impervious floor. This had the effect of reducing the pressure on the floor. Even then it would have been too great, so that a water cushion 2 feet deep was formed over the floor by building a dwarf wall of concrete (shown on the section) right along its edge. This adds 2 feet to the effective value of t_p, and is given credit for in the diagram Fig. 4c. In this diagram it will be seen that the hydraulic gradient now works out at 1 in 15, the value of c which has been adopted for similar light sands, and from which that of others, as classes 1 and 3, have been deduced.

(30) Some brief explanation will now be given of the diagram and the graphical method of construction.

Take Fig. 4. The first step is to ascertain what is the hydraulic gradient of the section as it stands. The first incidence of the water pressure is at the point A. From this starting point the horizontal length or width of the impervious rear and fore aprons up to B is 123 feet. To this must be added the vertical components of l, which are 7 feet at the base of the drop wall and 24 feet, or twice the depth of the curtain wall, at the termination of the floor proper. This makes the total value of l to be $123 + 31 = 154$ feet.

This is then measured out from A to C in Fig. 4a. The line AC then drawn is the hydraulic gradient of $\frac{154}{13}$, or 1 in 11·8. Now from C the vertical components are set out backwards in reverse order; thus the 7 feet depth is first marked, then the two 12 feet lengths, this last measurement coinciding with the point B.

The outline of the area of water pressure is traced by drawing lines parallel to the hydraulic gradient AC from these several points till they intersect the verticals through the positions to which they respectively belong. Thus, from A the hydraulic gradient continues unaffected until the drop wall is reached; here a step occurs. The next line is drawn from the 7-foot point near C, parallel to AC. This line continues till the curtain wall is reached; here another step down occurs, fixed by a parallel drawn up from the end of the 12 feet measured at the other side of the curtain; another step takes place by the intersection of the last parallel line, which continues down, meeting the base at the termination of the impervious apron. The area enclosed between the stepped sloping lines and the horizontal base evidently represents that of the water pressure acting below the base of the fore apron. Over this the area of resistance (hatched closely) is now drawn, the depths being $t\ (\rho - 1)$, or the actual thickness multiplied by the specific gravity reduced by unity, the floor being below L.W.L. and consequently immersed.

In this, the value of ρ for the floor proper is taken at 2, while that of the rough grouted block kunkur pitching, at $1\frac{3}{4}$. The depth $t\rho$ is therefore equal to the actual thickness of the floor in the first case, but is less in the grouted portion.

The unbalanced water pressure can now be seen at a glance.

The other diagrams are all constructed on the same lines. Comparison between them is most instructive, as the weak points in the original section and the means whereby the deficiencies were made good are at once demonstrated to the eye.

We shall proceed to further analyse some other sections, pointing out deficiencies, suggesting improvements, and occasionally producing an alternative design embodying the above.

(31) The section of the Burra Weir, Fig. 5, offers some points of interest. It is on the Mahanuddee system, of which the Jobra Weir, Fig. 15, is the principal head work, the value of c is therefore 12. The floor was originally built shorter than it is at present, but the talus was washed away and had to be renewed, with a prolongation of the floor itself. Even now its stability is barely adequate. This is an example of an overfall weir type A, with the floor surface not at L.W.L. as in the last case, but raised above it to the extent of its thickness so as to avoid wet construction, and is termed type A2. In all such cases the proportionate hydrostatic stress on the floor is greater when the latter is just submerged than it is with the higher pressure.

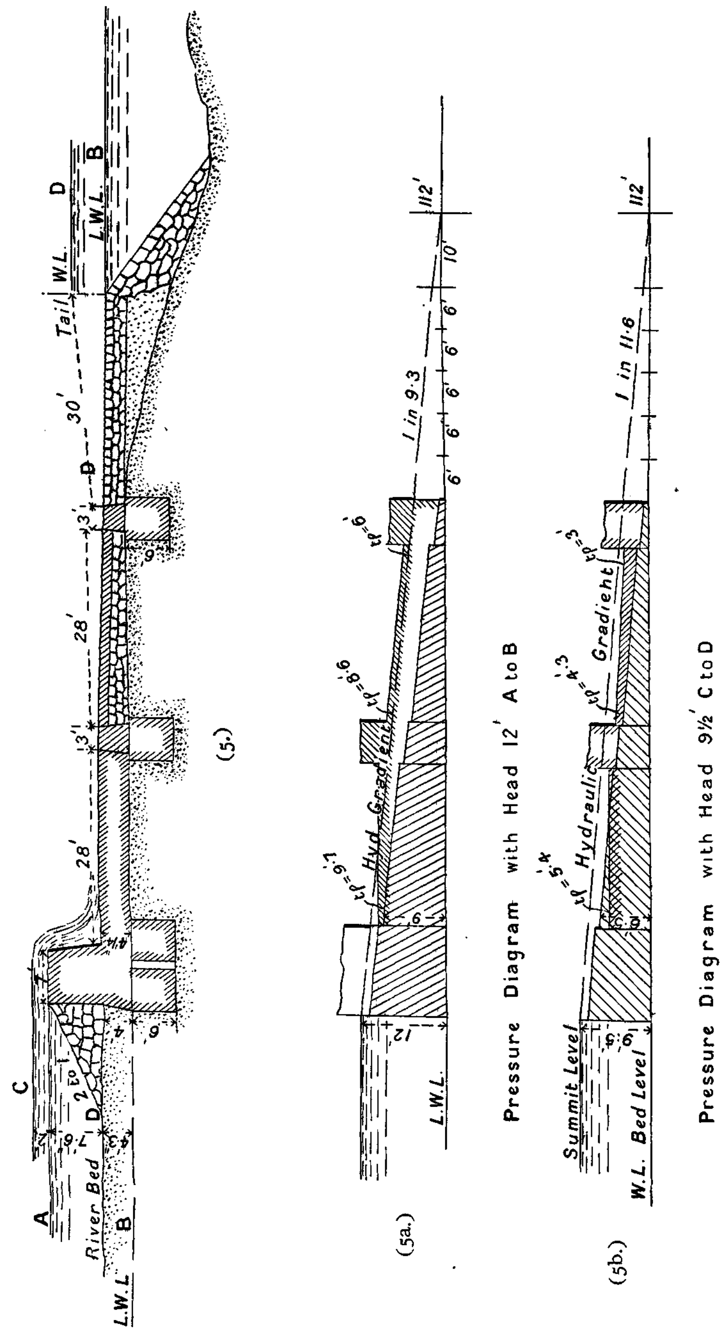
The head measured from summit level A, which is also crest level, to L.W.L. (B) is 12 feet. This will require a value of l, or of cH, of $12 \times 12 = 144$ feet. In the pressure diagram, Fig. 5a, the actual value is only 112 feet, giving an hydraulic gradient of 1 in 9·3 which is insufficient for absolute safety.

When the floor is submerged, the tail water to effect this must rise $4\frac{1}{4}$ feet to D; the reciprocal depth of film over crest, or d, to produce this rise is estimated to be about 2 feet. The summit level will then be at C. The pressure diagrams of both cases are shown in Figs. 5a and 5b. In 5a the balance pressure on the floor near the toe of the weir wall is $t\rho - (H - h) = (9{\cdot}7 - 9) = +{\cdot}7$.

In Fig. 5b the balance pressure at the same point is $5{\cdot}4 - 6 = -{\cdot}6$ ft. At the end of the floor in Fig. 5a it is $6 - (1{\cdot}3) = +\ 4{\cdot}7$ feet, and in 5b is $3 - (1{\cdot}1) = +\ 1{\cdot}9$ feet only.

Thus we see that the floor is strained more with the lower than with the greater head. On the other hand, with the lower head Fig. 5b, the hydraulic gradient is flatter, being 1 in 11·6 to 1 in 9·3 of 5a.

(32) In all cases where the floor is founded above L.W.L., it is necessary



FIGS. 5, 5a, 5b.—Burra Weir.

to test the thickness by formula (3) for another value of H, which occurs when the floor is under water. This may be less than before, but may have a greater effect owing to the submergence of the floor.

The greatest difference of levels always occurs when the summit level is at crest level and the tail channel empty, and this maximum possible statical head is the value of H, from which the length of creep (l) is calculated. In this design the floor requires thickening at its root and widening, or else has to be provided with deep vertical curtain walls. According to formula (1) par. 21, the width with $c = 12$ should be $48\sqrt{\cdot 7} = 40$ feet, nearly; it is actually 56 feet, although the last bay is not all formed of solid masonry. In remodelling a section of this kind, a rear apron should first be provided; concrete sheet piling could also well be substituted for the undersunk blocks.

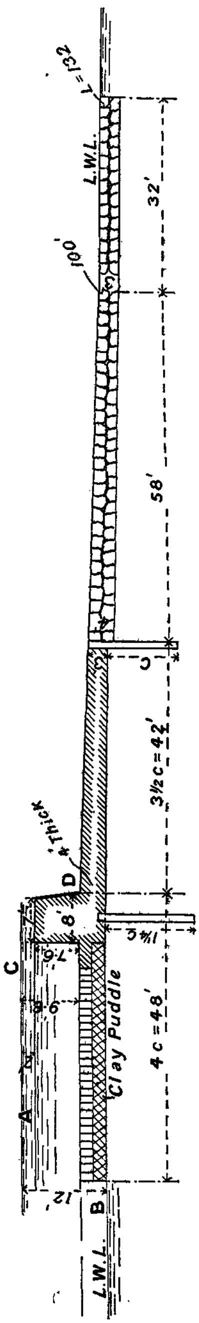


Fig. 6.—Alternative Section, Burra Weir.

(33) Fig. 6 is the profile remodelled. The fore apron is made $3\frac{1}{2}\,c$ wide, or 42 feet. A row of piling of a depth of 12 feet ($= c$) will be placed at the end of the apron. This is equivalent to a length of creep of $2\,c$. Consequently the fore apron takes altogether $5\frac{1}{2}\,c$ in length, leaving $12 - 5\frac{1}{2} = 6\frac{1}{2}\,c$ for the rear apron and rear sheet piling, which will be made $4\,c$ and $2\frac{1}{2}\,c$ respectively. The thickness of the apron at the toe of the weir wall, with the full head of AB will be, according to formula (3) (par. 25), $\frac{4}{3} \times \frac{12 - 6\cdot 5}{2} = 3\cdot 7$ feet. The same, calculated from formula (3) with a reduced head of CD, or of $9\cdot 5$ feet, will be $\frac{4}{3} \times \frac{9\cdot 5 - 6\cdot 5}{1} = 4$ feet. The latter dimensions will thus be adapted. The apron will taper to a thickness of 3 feet at its extremity, and will be horizontal at the base, inclined on the surface.

The talus will slope down to L.W.L., tapering from 4 to 3 feet thickness. The width of the talus calculated by formula (4) (par. 27) will be $10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$, which works to $120 \times 1\cdot 1 = 132$ feet, in which $H^b = 12$ feet, $c = 12$, and q is taken as 75 second-feet.

The revised section would, it is believed, cost less than the original. The raising of the floor above L.W.L. although it undoubtedly usurps the waterway to a certain extent, results in considerable economy. If the floor surface was placed lower it would have to be not only thicker but longer, and the construction being below spring level would be more costly. This type of raised floor will, as already noted, be classed as a sub-type of A, viz., as type A^2.

(**34**) Fig. 7 is of a low weir of type A on the Upper Colerun River in the Madras Presidency. The original work was also built with too short a floor, subsequently lengthened to its present dimensions. This shows that these works were designed haphazard; in fact, as far as is known, rules for guidance in design based on definite principles are now set out for the first time.

The principle of length of percolation influencing design is not the author's invention, but its practical application has never hitherto been rendered available by the promulgation of formulas suitable to the varying conditions met with in practice.

The Colerun Weir has lately been entirely rebuilt in rear of the present site, and is changed to an open regulator with 25 feet spans and roller counterbalanced gates operated by gear, the present floor forming part of the new work. It is believed that the head of water has also been considerably increased. A photograph of the piers and gates is given in "The

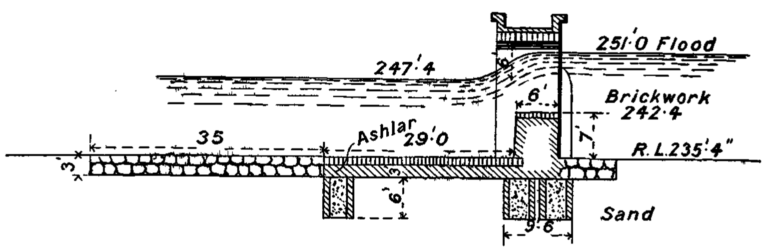


FIG. 7.—Colerun Weir. Discharge, 284,000 second-feet. Length, 2,926 feet.

Irrigation Works of India" (Buckley), which is decidedly useful, but a working drawing, if even a rough diagram, would be much more so.

(**35**) The next three examples will be of another type, designated type B. In these weirs there is no direct vertical drop, the fore apron not being horizontal but sloping down from the crest to the L.W.L. or to a little above it, the talus beyond being either also on a flat slope, or else horizontal.

In the modern examples of this type which will first be examined, the height of the permanent masonry weir wall is greatly reduced, with the object of offering as little obstruction as possible to the passage of flood water.

The canal summit level is attained by means of deep crest shutters. In the Chenab Weir (Fig. 8), the weir, or rather crest wall, is 7 feet high above L.W.L., while the shutters are 6 feet high; it therefore holds up 13 feet of water, the same as was the case with Narora Weir.

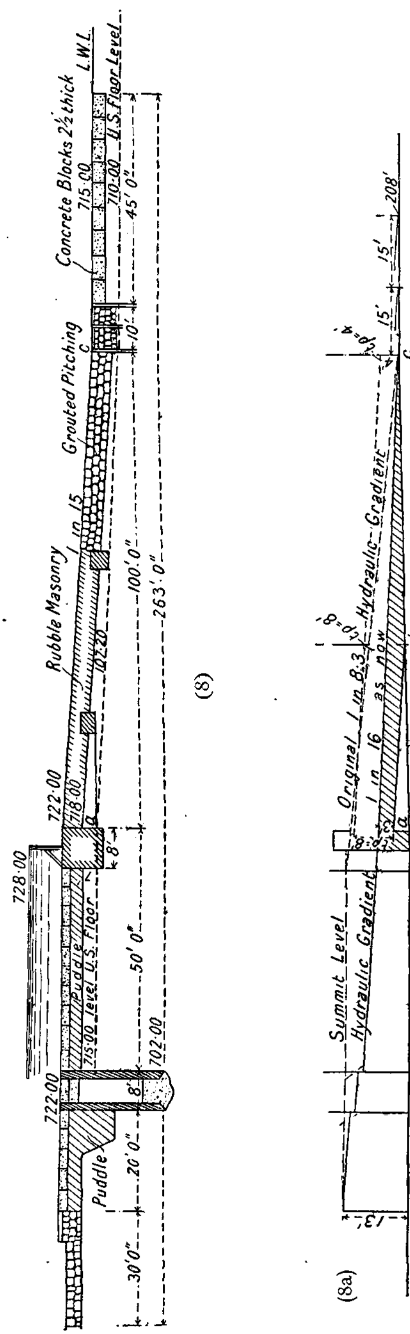
The object of adopting the sloping apron is to avoid construction in wet foundations, as most of it can be built quite in the dry above L.W.L. The disadvantage of this type lies in the contraction of the waterway below the breast wall, which causes the velocity of overfall to be continued well past the crest. With a direct overfall, on the other hand, a depth of 7 feet for water to churn in would be available at this point; this checks the flow, and the increased area of the waterway rendered available must reduce the velocity. For this reason, although the action on the apron is possibly less, that on the talus and river bed beyond must be greater in type B than in the drop wall A type.

(**36**) This work, like Narora, failed for want of sufficient effective base length, and it consequently forms a valuable object lesson.

As originally designed, no rear apron whatever, excepting a small heap of stone behind the breast wall, was provided. The value of l up to the termination of the grouted pitching is but 108 feet, whereas it should have been $c \times H$, or $15 \times 13 = 195$ feet. The hydraulic gradient, as shown in Fig. 8a, is only 1 in 8·3. This neglects the small vertical component at the breast wall. In spite of this deficiency of effective base width, the floor, owing to good workmanship, did not give way for some years, until gradually increased piping beneath the base caused its collapse.

Owing to the raised position of the apron, it is not subject to high hydrostatic pressure. At its commencement it is 10 feet below the summit level, and 9 feet of water acts at this critical point; this is met by 4 feet of masonry insubmerged, of specific gravity 2, which almost balances it. Thus the apron did not blow up, as was the case with Narora, but collapsed.

(**37**) Some explanation of the graphical pressure diagram Fig. 8a is required, as it offers some peculiarities differing from the last examples. The full head, or H, is 13 feet. Owing however, to the raised and sloping position



FIGS. 8, 8a.—Chenab Weir at Khanki. Length, 4,000 feet. Discharge (circ.), 600,000 to 700,000 second-feet.

of the apron, the base line of the pressure area will not be horizontal, coinciding with the L.W.L., but will be an inclined line—from the commencement *a* as far as the point *b*, where the sloping base coincides with L.W.L.; from *b* onward, the base will be horizontal. With a sloping apron the pressure is pretty well uniform, so that the water pressure area is not wedge-shaped, but approximates to a rectangle; the apron therefore is also properly rectangular in profile, whereas in the overfall type the profile is, or should be, that of a truncated wedge.

(**38**) After the failure of this work, the restoration was on very similar lines to that at Narora. An impervious rear apron 70 feet long was

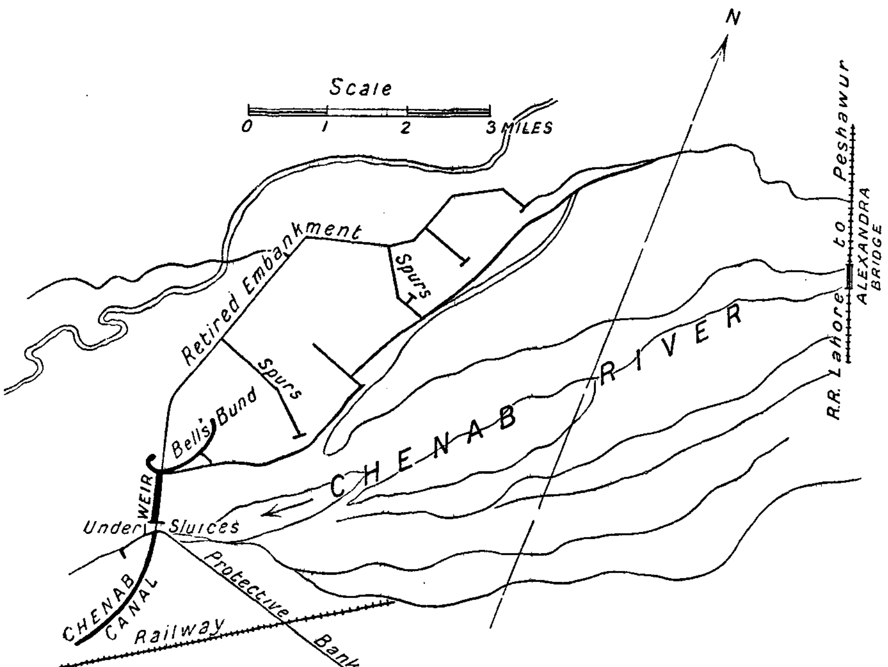
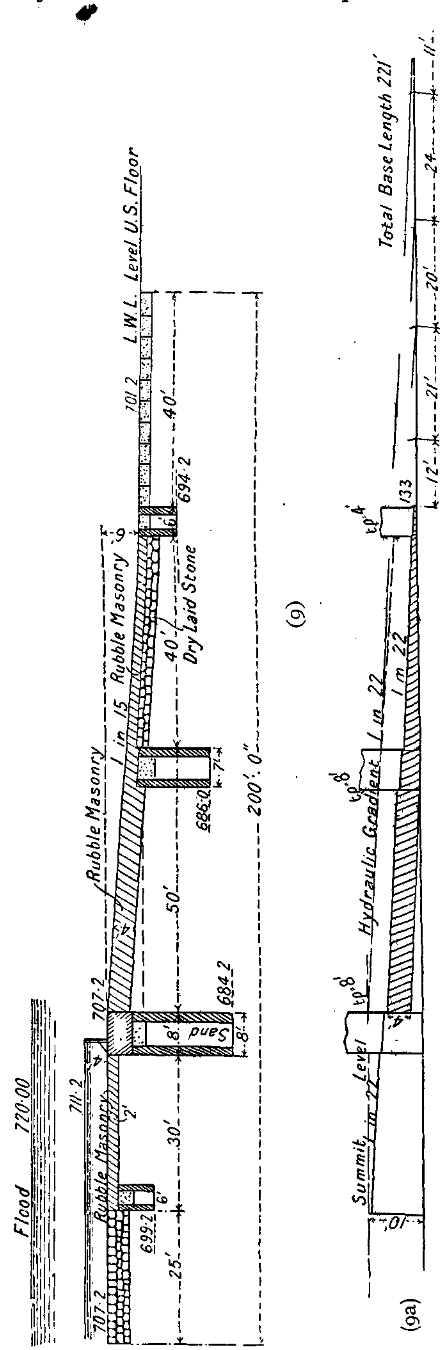


FIG. 8b.—Sketch Plan of Khanki Head Works.

constructed of puddle covered with concrete slabs, grouted in the joints; a rear curtain consisting of a line of rectangular undersunk blocks 20 feet deep was provided. These additions reduce the gradient to 1 in 16. The masonry curtain, considering its great cost, is considered as of doubtful utility; a further prolongation of the rear apron or a line of sheet piling would have been equally effective. Now that reinforced concrete sheet piling is available, this type will probably in time supplant the ponderous and expensive block curtains which are such a marked feature in Indian works. A site plan of the weir is given in Fig. 8b.

(**39**) The Jhelum Weir (Figs. 9 and 9a) is another example of type B, the section of which as originally constructed proved deficient and had to be enlarged. In the eighties when these works were carried out the principles involved in their design were not properly understood, and consequently the profiles were the result of nothing better than guesswork. In this case the hydraulic gradient is now 1 in 22, affording large superfluity of stability. If all the lines of curtain walls were abolished and one line of sheet piling below the breast wall of a depth of 15 feet substituted, the hydraulic gradient would be reduced to a more reasonable ratio of 1 in 16.

The value of L, or distance of toe of talus from the crest wall, measures 135 feet. According to formula (1) $c = 15$, $H^b = 6$, and $q = 135$. It should be therefore $10 \times 15 \times \sqrt{\frac{6}{10}} \times \sqrt{\frac{135}{75}} =$ 160 feet, which is not a very close correspondence. A site plan of the head works is given in Fig. 9b which is most instructive.



FIGS. 9, 9a.—Jhelum Weir. Length, 4,000 feet. Discharge, 600,000 second-feet.

(**40**) Another instance of the same type is that of the Jamrao Weir (Fig. 10). The head here is 8 feet, requiring a base length, or value of l, of $8 \times 15 = 120$ feet; it is actually 137 feet, excluding the wooden sheet piling in the rear. This gives a gradient of 1 in 17, which is somewhat in excess of requirements. The diagram in Fig. 10a shows the effective floor weight at the critical point of the apron, which is here beyond the second inset, to be 6 feet, being 3 feet of brickwork at specific gravity 2 insubmerged. The water pressure is 4·75, requiring one-third excess, viz., $6\frac{1}{3}$ feet. From the point where the surface slope of the apron intersects the L.W.L., the whole floor is submerged, and consequently tp is only 3 feet, at which value it remains until the fore curtain is reached. If the tail water rose to weir crest level and the shutters were still up (a possible contingency), the head acting below the apron would be $4 - 3\frac{1}{4} = \frac{3}{4}$ foot only. With the crest shutters down, the depth d above crest would not exceed 2 feet.

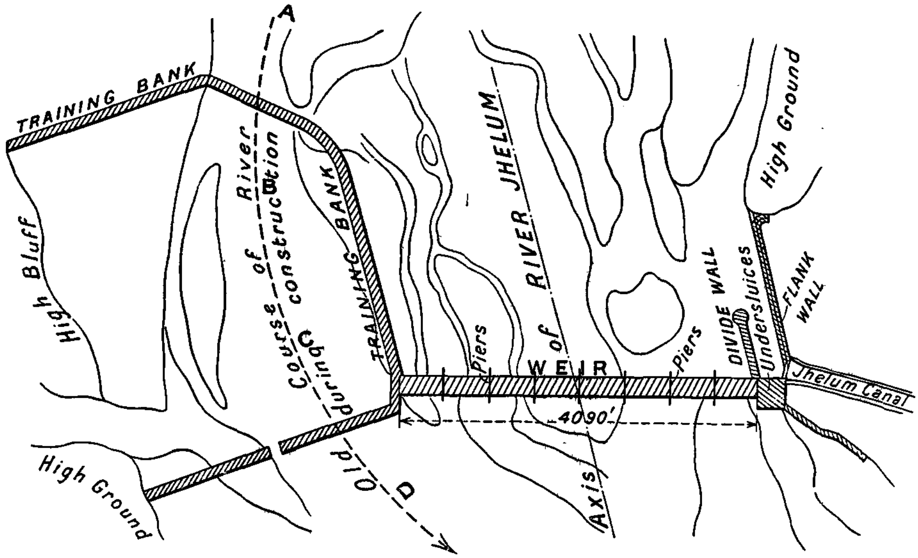
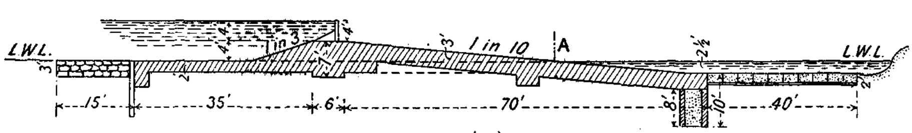


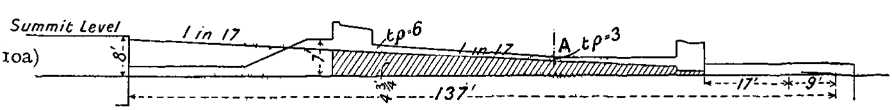
FIG. 9b.—Head Works of Jhelum Canal.

The height of this weir being so low, renders the construction of a solid apron a necessity, in order that the requisite weight be afforded without having to greatly increase the depth lying below L.W.L. Material for loose stone



(10)

(10a)



FIGS. 10, 10a.—Jamrao Weir.

filling was not obtainable at this site, so that the rear apron had also to be built in the same way, and the talus constructed of concrete blocks.

The width of the masonry apron according to formula (1) should be $4c\sqrt{\frac{H^a}{13}}$. Here $H^a = H = 8$, and W works out to $4 \times 12 \times \cdot 9 = 43$ feet. It is actually 70 feet, but this is an exceptional case where masonry has to take the place of the usual riprap. The width of talus from crest wall (formula (4)) should be $11c \times \sqrt{\frac{4}{10}} = 104$ feet; it is actually 110 feet.

(41) Another example of the same type is furnished in Fig. 11 of the Adimapali Anicut. This work, which is of an older pattern, is not provided with the usual collapsible crest shutters. Under such conditions the hydrostatic stress on the underside of the apron has generally a greater effect when the down-stream water level is at crest level than when the channel is empty, although the head is less. This is accounted for by the reduction of effective weight owing to submergence. The hydraulic gradient works out to 1 in 12·6, if half value only is accorded to the circular well curtains, which are not perfectly watertight. When the tail channel is empty the head (H) on which the value of c must always be calculated is $8\frac{1}{2}$ feet, but as the root of the sloping apron is situated at 4 feet above L.W.L., the head acting on the base there is but 4·5 feet. This amount is further diminished by the loss of head (H) due to percolation, or by the length of creep up to this point divided by c, which amounts to $\frac{14}{12\cdot6} = 1\cdot1$ foot leaving a balance of 3·4 feet. This pressure is met by the resistance of $t\rho$, or by $4\cdot5 \times 2 = 9$ feet, the difference being $9 - 3\cdot4 = 5\cdot6$ feet in favour of stability.

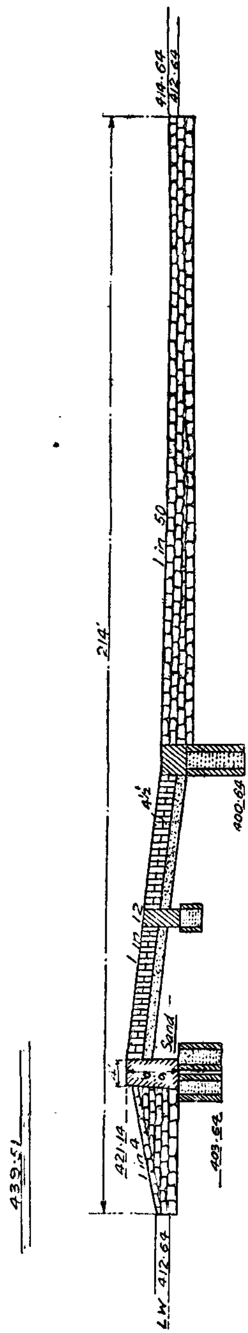


FIG. 11.—Adimapali Weir, Pennér River. Length, 2,243 feet. Discharge, 413,500 second-feet.

Supposing the tail channel filled up to crest level, calculation has to be made as to what depth of water will be passing over the weir to produce a discharge capable of raising the water in the tail channel up to this level. If we assume the bed slope as steeper than 1 foot in 5,000, a reciprocal ratio of $\frac{d}{D}$ exceeding ·5 will be found in the last column of Table II., Chap. II., par. 49. The value to be assigned to d will then be somewhat in excess of $\frac{H}{2}$ or of $\frac{8\cdot5}{2}$, which will be about 4·5 feet. This then is the head acting on the work under these new conditions. It happens to be the same as that previously obtained, which is a coincidence. The pressure at the root of the apron will therefore also be 3·4 feet, but the resistance is less owing to the submergence of the apron, the value of $t\rho$ being only $4\cdot5 \times 1 = 4\cdot5$, leaving a balance in favour of stability of ·9 foot only, against one of 5·6 feet previously arrived at. The thickness of the apron therefore is only just sufficient.

The extensive well foundations could be dispensed with, and one, or probably two rows of reinforced concrete sheet piles substituted. These and other modern substitutes were, however, unknown at the time these works were executed; the sheeting piles which would be more effective in forming a watertight barrier could be made up *in situ* and be much cheaper than well or block curtains, being easily sunk in sand by a water jet.

This work is situated high up on the Pennér River, and is subject to very heavy floods, the unit foot discharge of which amounts to 184 second-feet, a quantity $2\frac{1}{2}$ times greater than that of the Ganges at Narora. These abnormal

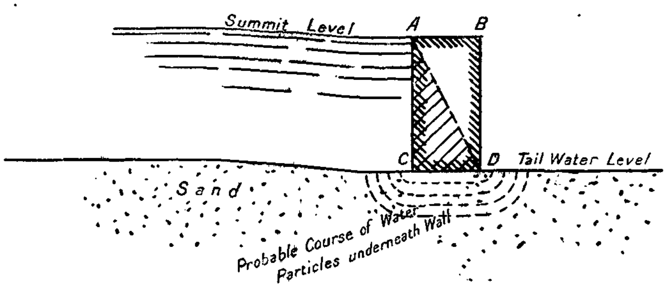


FIG. 12.

conditions will necessarily affect the design in the increased width of apron and talus.

The width of the fore apron is 60 feet. According to formula (1) it comes to 45 feet. This discrepancy is easily accounted for by the abnormal conditions prevailing. With regard to the talus, formula (4) makes allowance for high unit flood discharge. According to this formula, taking $10\frac{1}{2}$ and not 10

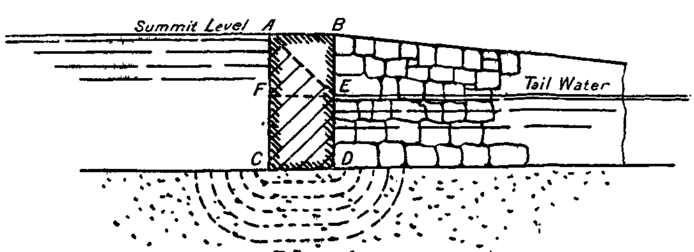


FIG. 13.

as the multiplier of c (par. 27), $L = 10\frac{1}{2}c \times \sqrt{\frac{8 \cdot 5}{10}} \times \sqrt{\frac{184}{75}} = 180$ feet; it is actually 184 feet.

It is evident that the design could be improved by adopting a direct overfall with horizontal apron of type A^2, and providing a rear apron as well. The work could then be constructed entirely above L.W.L. This change of type would lessen the violence of the flood action by affording a deeper waterway just below the crest of the obstruction.

(42) The previous examples of types A and B have all been cases where

the weir has an appendage, an impervious floor which is subject to hydrostatic pressure. There is another very common type which will be termed C, in which there is no impervious floor and the material which composes the body of the weir is not solid masonry, but a porous mass of loose stone filling, the only impervious parts being vertical, not horizontal walls. In spite of this apparent contrariety it will be found that the same principle, viz., that of length of enforced percolation, influences the design in this type in the same way as in the others we have been considering.

Fig. 12 represents a wall upholding water to its crest, and resting on a pervious substratum as sand, gravel, or boulders, or a mixture of all three materials. The hydraulic gradient is *AD*, the upward pressure area, *ACD* and the base *CD*, is the line of percolation. Unless this base length is equal to *AC* multiplied by the coefficient obtained by experiment, piping will set in and the wall will be undermined.

Now, as shown in Fig: 13, let a mass of loose stone be deposited below the wall. The weight of this stone will evidently have an appreciable effect in checking the disintegration and removal of the sand foundation. The water will not have a free untrammelled egress at *D*; it will, on the contrary, be compelled to rise in the interstices of the mass to a certain height, *EE*, which is determined by the extent of the obstruction caused to the flow.

The resulting hydraulic gradient will now be *AE*, flatter than *AD*, but still too steep for permanency.

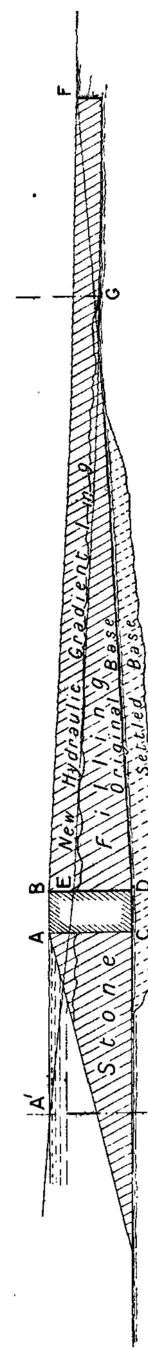


FIG. 14.

(43) In Fig. 14 the wall is shown backed by a rear apron of loose stone, and the body extending to *F*. The water has now to filter through the rear apron underneath the wall and up through the stone filling in the fore apron. During this process a certain amount of sand will be washed up into the porous weir body, and the loose stone will sink, until the combined stone and sand forms a compact mass offering as great, or a greater obstruction to the passage of the percolating water than exists in the sand itself, and possessed of far greater resisting power to disintegration. This will cause the level of water at *E* to rise until equilibrium results. Where this is the case, the hydraulic gradient is flattened out to some point *G* near *F*. If a sufficiently long body is provided the resulting gradient will be equal to that found by experiment to produce permanent equilibrium. The mass, after the sinking process has finished, is then made good up to the original profile, by fresh stone filling. Near *F*, the toe

of the slope, the stone offers but little resistance either by its weight or depth, so it is evident that the slope of the prism should be flatter than that of the hydraulic gradient.

The same action takes place with the rear apron, which soon becomes silt-charged so as to be impervious, or nearly so, to the passage of water; but until silt is deposited in the river bed behind, as will eventually occur right up to crest level, the thin portion of the rear slope as well as the similar portion of the fore slope near the toe cannot be counted as effective. Consequently, of the whole base length CF, roughly about one-quarter can be deemed inefficient as regards length of percolation. As the consolidated lower part of the body of the weir gains in consistency, it can well be subject to hydrostatic pressure. Consequently the value of tp of the mass should be in excess of that of $(H-h)$, just as was the case with an impervious floor.

(**44**) In Fig. 15 a further development is effected by the introduction of vertical body walls of masonry in the pervious mass of the fore apron. These impervious obstructions materially assist the stability of the foundation; so much so that the process of underscour and settlement, which must precede

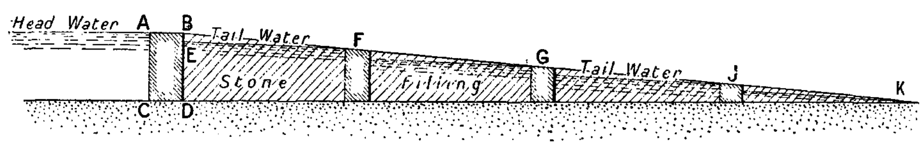


FIG. 15.

the equilibrating of the opposing forces in the purely loose stone mass, need not occur at all, or nothing like to the same extent. If the party walls are properly spaced, the surface slope can be that of the hydraulic gradient itself, and thus ensure equilibrium. This is clearly illustrated in Fig. 15. The water passing underneath the wall base CD, will rise up to the level F; the point E being somewhat higher, and similarly percolating under the other walls through the sand substratum, will fill up all the partitions with water. The head AC will therefore be split up into four steps.

The value of the stanchness to percolation of the rear apron is so marked that it should be rendered impervious by a thick underlayer of clay, and not left to more or less imperfect surface silt stanching as has hitherto been the case. The specific gravity of stone filling with 50 per cent. void is about 1·3. When immersed, this will be reduced to ·8, for the following reason: the specific gravity of the solid stone is 2·6; when immersed, 1·6. Divide this by 2, as half is void: the result is ·8. In cases where it is convenient to assume the full head acting on the whole work, the weight of material, plus that of the water filling the interstices, will represent the actual overlying weight. This can be considered as having a specific gravity of $\cdot 8 + \cdot 5 = 1\cdot 3$. This happens to be identical with that of the unsubmerged mass, which is but a coincidence.

(45) Fig. 16 is the section of the Okhla Weir over the Jumna River situated eight miles below the historic city of Delhi. It is remarkable as being the first rock fill weir not provided with any lines of curtain walls projecting below the base line, which adjunct had hitherto invariably been adopted. The stability of its sand foundation is consequently entirely dependent on its weight and effective base length. As will be seen, the section is provided with three body walls in addition to the breast wall. The slope of the fore apron is 1 in 20. It is believed that a slope of 1 in 15 would be equally effective, a horizontal talus making up the continuation, as has been done in the Madaya Weir (Fig. 22, *post*).

The head, when the shutters are up and the weir body empty of water, a condition that could hardly exist, is 13 feet. This would require an effective base length, or l, of 195 feet; the actual is 250 feet; but, as noticed previously, the end parts of the slopes cannot be included as effective, consequently the hydraulic gradient will not be far off from 1 in 15. The weight of the stone, or $t\rho$, exactly balances this head at the beginning of the fore apron, as it is $10 \times 1{\cdot}3 = 13$ feet. If the water were at crest level and the weir full of water, $t\rho$ would $= 8$ feet, or, rather, a trifle less, owing to the lower level of the crest of first body wall. This head of 13 feet is broken up into four steps, the first, 3 feet deep, acting on a part of the rear apron together with 30 feet of the fore apron, say, 1 in 15; the rest are 1 in 20. A slope of 1 in 15 from the first party wall would cut the base at a point 40 feet short of the toe. Theoretically a fourth party wall is required at this point, but practically the riprap below the third dwarf wall is so stanched with sand as not to afford a free egress for the percolation; consequently the hydraulic slope may be assumed to continue on to its intersection with the base. As already noted, material would be saved in the section by adopting a reliably stanch rear apron and reducing the fore slope to 1 in 15, with a horizontal continuation.

This type (C) is only economical where stone is abundant; it requires little skilled labour, or masonry work. On the other hand the mass of the material used is very great, much greater, in fact, than is shown by the section. This is owing to the constant sinking and renewal of the talus, which goes on for many years after the first construction of the weir.

The action of the flood on the talus is undoubtedly accentuated by the contraction of the waterway due to the high sloping apron. The flood velocity 20 feet below

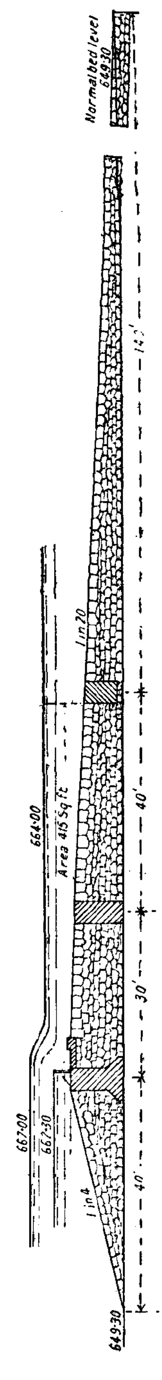


Fig. 16.—Okhla Weir, Jumna River, Agra Canal. Discharge (c.r.c.), 150,000 second-feet.

he crest of Okhla Weir has been gauged as high as 18 feet per second. This would be very materially reduced if the A type of overfall were adopted, as the area of waterway at this point would be more than doubled.

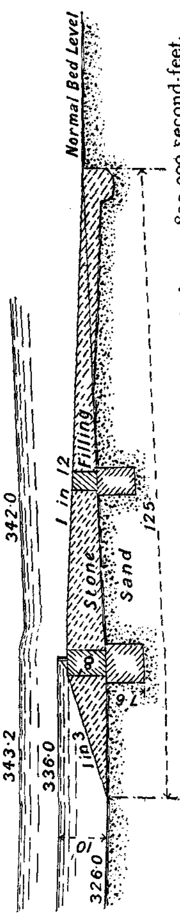
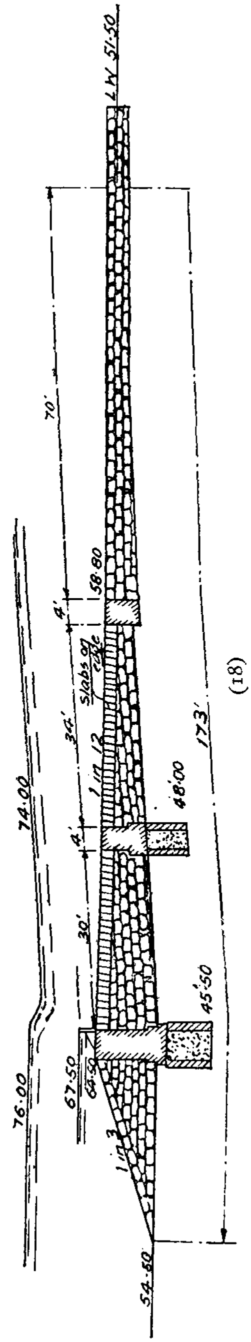
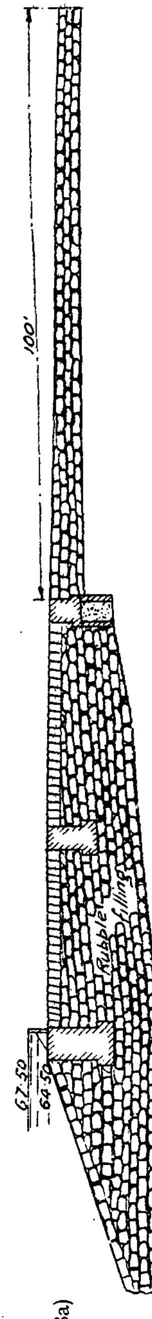


FIG. 17.—Dehri Weir, Sôn River. Length, 12,500 feet. Discharge, 830,000 second-feet.



(18)



(18a)

FIGS. 18, 18a.—Jobra Weir, Mahanadi River. Length, 6,400 feet. Discharge, 900,000 second-feet.

(46) Another typical example of this class is the Dehri Weir over the Sôn River (Fig. 17). The value of l, if the apices of the two triangles of stone filling are deducted and the curtain walls included, comes to about 12 H, 12 being the coefficient adopted for this class of sand.

The curtain walls, each 12,500 feet long, must have been enormously costly. From the experience of Okhla, a contemporary work, on a much worse class of sand curtain protection is quite unnecessary if sufficient horizontal base width is provided. The head on this weir is 10 feet and the height of breast wall 8 feet; $t\rho$ is, therefore, $1{\cdot}3 \times 8 = 10{\cdot}4$, which is

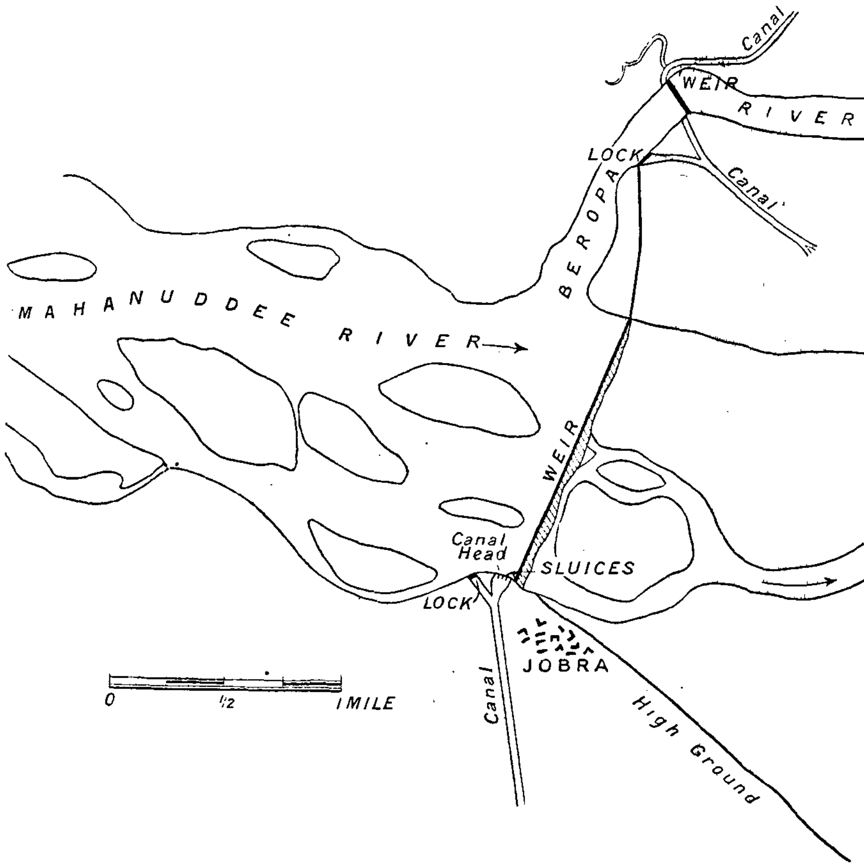


FIG. 18b.—Site Plan of Jobra Head Works.

sufficient, considering that the full head will not act here. The lines of curtains could be safely dispensed with if the following alterations were made:—(1) Rear apron to be reliably stanched in order to throw back the incidence of pressure and increase the effective base length; (2) three more body walls to be introduced; slope 1 in 12 retained but base to be dredged out towards apex to admit of no thickness under 5 feet. This probably would not increase quantities, except in masonry, above what they are now, and would entirely obviate nearly 5 miles of undersunk block curtain walls.

(**47**) Figs. 18 and 18a are of the weir at Jobra, over the Mahanadi River, also in Bengal. The site plan is Fig. 18b. The head measured up from normal bed level, viz., R.L. 54·50, is 13 feet, identically the same as in Okhla Weir. The total base width is, however, 173 feet to 250 feet of the latter. This great difference is due to the nature of the sand in the river. The flood discharge per foot run is $\frac{900,000}{6,400} = 140$ second-feet, which is the same as that over Okhla. The weir has two lines of curtain walls which add 34 feet to whatever is the effective base width. With $c = 12$, the correct value of l will be $12 \times 13 = 156$; the actual is $173 + 34 = 207$ feet. The base length is therefore just sufficient.

If the section were altered on the same lines as was proposed for the Dehri Weir, the curtain walls could also be dispensed with. These are, stanching the rear apron and increasing its length by 10 or 20 feet, and adding two more party walls in the fore apron, reducing their spacing accordingly. By this means an efficient base length or value of l of 156 feet could easily be afforded. The site plan of the head works is very instructive.

(**48**) Another example of a rock fill crib weir, designed by the author for some intermittent torrents with deep sandy beds in upper Burma, is given in Fig. 19. This type was necessitated by the exigencies of the situation, neither skilled labour nor lime for mortar being available, and the time of construction was limited to a few months. The object of the framing of posts and laggings is to act as a support for the covering boards. These are essential to prevent the stone-filling, which was not packed on the surface, from being washed out. The foreslope is 1 in 15, formed by steps of 1 foot at 15-foot intervals. An improvement would have been to have carried the longitudinal vertical planking at each step, right down to the sand foundation instead of stopping short, as was done. They would then act the same way as the body walls shown in Figs. 15, 16, etc., and be of great value in affording stability to the sand foundation. Even without them these weirs answered well, the settlement not being great. This was due to the rapid silting up of the river bed, the first freshet having piled sand up to crest level, thus rendering the weir watertight. One of these weirs, whose foreslope was altered to 1 in 10, settled considerably and required renewal of the sunk stone-filling, which proved that the hydraulic gradient was too steep. The body had to be lengthened in consequence. A photograph of one of these weirs is given in Chap. VIII., Fig. 27.

(**49**) In Figs. 20 and 20a is an improved alternative section on the same lines, in which clay is used as a stratum below the stone-filling. The base is therefore rendered impervious, and the correct value of l can be estimated with accuracy, and any subsequent settlement is rendered impossible. It will probably also cost less. The scantlings used are of the lightest possible dimensions, the laggings being 6 inches by 3 inches, the post 8 inches diameter round hard wood, and the planks were only 1 inch thick; these were spaced 6 inches apart and were screwed down with coach screws. Heavy scantlings of light wood are unsuitable for crib work as they become buoyant when immersed, thus reducing the weight of the weir body, so as often to require anchoring down, to prevent the whole being floated off.

(50) Fig. 21 is a section of the recently constructed Damietta and

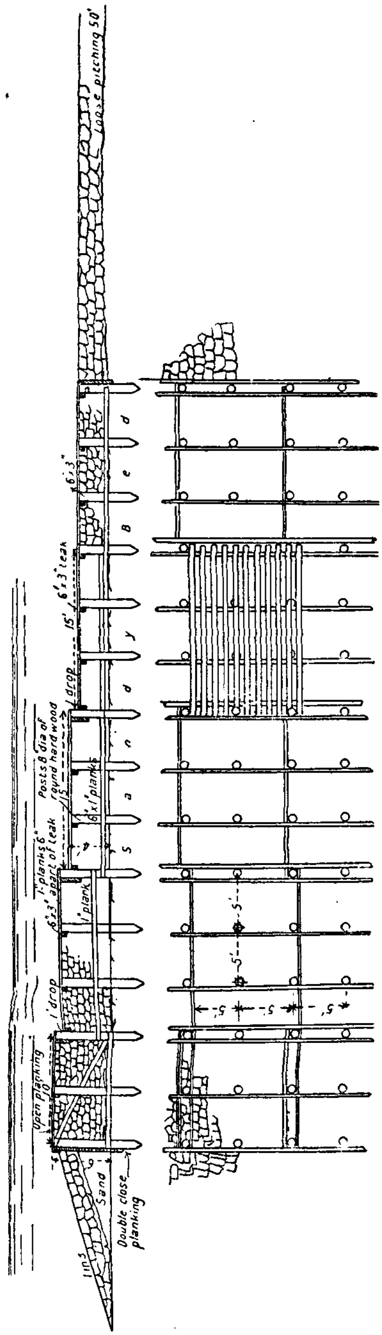


Fig. 19.—Crib Weir in Burma.

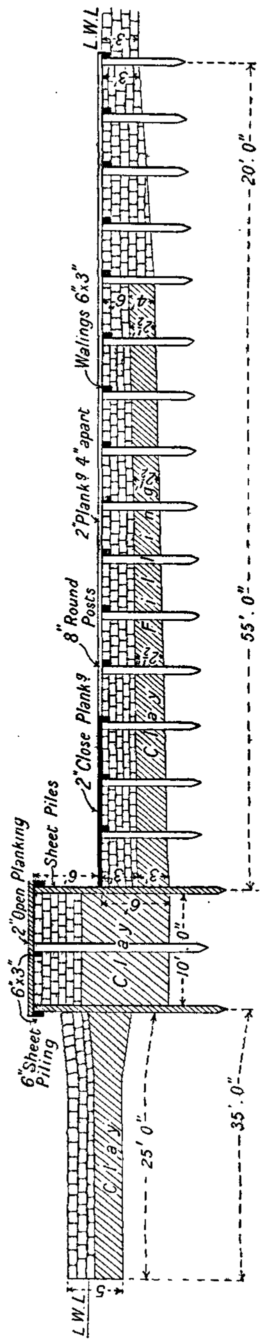


Fig. 20.—Alternative Section for Crib Weir.

Rosetta subsidiary weirs across the Nile River below the Grand Barrage. Fig. 21a is a site plan.

The profile of this work is of type C, but the method of construction is quite novel, and it is this that renders this work a most valuable object-lesson. The work was carried out without any pumping, all material having been deposited in the water of the Nile River. First the profile of the base was dredged out, as shown in the section. Then the core wall was constructed by first depositing loose stone in a temporary box or enclosure secured by a few piles from barges floated alongside. The whole was then grouted with cement grout poured through pipes let into the mass. On the completion of one section all the appurtenances were moved forward and another section built, and so on until the whole wall was completed. Clay was deposited at the base of the core wall and the profile then made up by loose stone-filling.

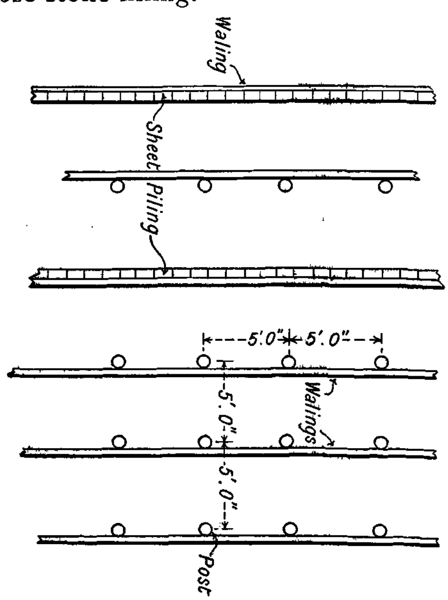


FIG. 20a.—Part Plan.

This novel system of subaqueous construction proved so eminently satisfactory that in many cases it is bound to quite supersede older methods. Details of the construction of these two weirs are to be found in Sir Hanbury Brown's "Irrigation" (Constable), a most valuable addition to irrigation literature. Notwithstanding these excellent innovations in methods of rapid construction, the profile of the weir itself is open to the objection of being extravagantly bulky even for the type adopted, the base having been dredged out so deep as to greatly increase the mean depth of the stone-filling.

It is open to question whether a row, or two rows of concrete sheet piles would not have been just as efficacious as the deep breast wall, and would certainly have been much less costly. The pure cement grouting was naturally very expensive, but the admixture of sand proved unsatisfactory, as the two materials of different specific gravity formed layers, and so pure cement had perforce to be used. It may be noted that the value of L here is much less than would be expected. At Narora Weir L is $11c$, or 165 feet long; here with a value of c of 18, L is but 150 feet $= 8\frac{1}{2}\,c$. The values of H^b are the same in both weirs, consequently this discrepancy can only be accounted for by the low unit discharge and flood velocity at the Nile weirs in comparison with the Ganges.

It is a great advantage when time is short to have the mass of a large work like this constructed by unskilled labour which can be crowded into the work, collection of stone having been made a year previous. The provision of mortar and the building of concrete or masonry by skilled labour also involves a great deal of arrangement and supervision which is avoided when loose rock construction is adopted. A site plan is given in Fig. 21a.

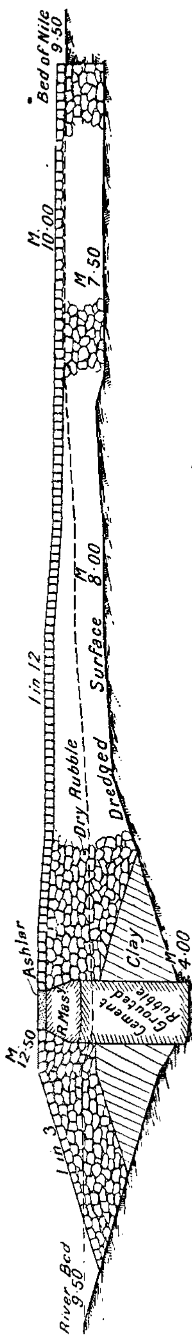


FIG. 21.—Damietta Subsidiary Weir (Egypt).

(51) The section of the recently constructed Madaya Weir in Burma is shown in Fig. 22. On account of the exceptional heavy unit flood discharge, the conditions are quite abnormal, so that although

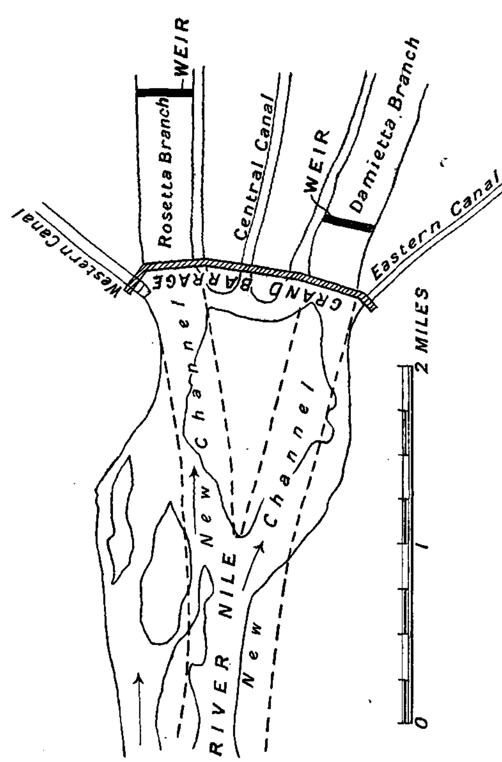


FIG. 21a.—Site Plan of Damietta and Rosetta Weirs.

the material of the bed of the river is a shingle and coarse sand with a value of c of about 10, yet the width of the talus exceeds even that of Okhla. It is considered that a masonry weir of type A would have given better results. By means of the increased area of waterway thus provided, the scour would be materially reduced, thus allowing of economy in repairs and renewals, if not in reduction in the width of the talus.

In the Madaya Weir the rear apron has been properly designed of impervious clay puddle, well protected on the surface and at its junction with the crest wall. This weir is only 200 feet long.

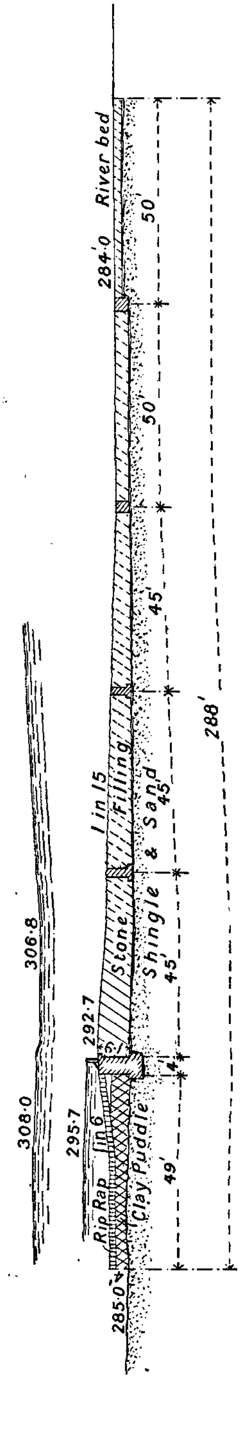


FIG. 22.—Madaya Weir (Burma).

(**52**) As an instructive example of what can be accomplished with simple clay filling in the body of a weir, with a mere covering of bricks laid on edge, the Sidnai Weir section (Fig. 23) is herewith given.

The weir is founded partly on deep sand and partly on clay. The profile, where on clay, is similar to the figure, but with the base cut off at the top of the piles. Clay is heavier than stone filling, having a specific gravity of 1·8 to 2, against 1·3 of the stone (with 50 per cent. voids), consequently if its protection can be ensured it is a valuable material, having further the advantage of being impervious. It is considered that its use should not be confined to the rear apron of weirs, but be also employed below rock filling, or concrete blocks, or rubble masonry in the fore apron. There is no reason why this should not be done.

(**53**) In Fig. 24 is an example of a 20 feet high overfall weir of type A on boulders and gravel. It is of the Granite Reef diversion weir, Salt River project, of which the Roosevelt Dam is the main feature. The profile, which has stood the test of actual trial (though not yet for long), is extremely valuable as an object-lesson wherefrom a reliable value for the coefficient c for river beds formed of boulders and gravel can be deduced. The estimated length of creep or (l) amounts to 84 feet, which when divided by the head of 20 feet, the quotient c is 4·2. The peculiarity of the section consists in the floor, being almost entirely relieved of hydrostatic pressure by spaces having been purposely left between the 10 feet square concrete blocks which form the surfacing of the floor. This arrangement, by shortening the base length, effects considerable reduction in the pressure area of the drop wall. Such a device, however, would not be practicable when pure sand forms the foundation, as it would inevitably be blown up through the interstices and washed away, causing the floor to collapse. The fore curtain is also pierced

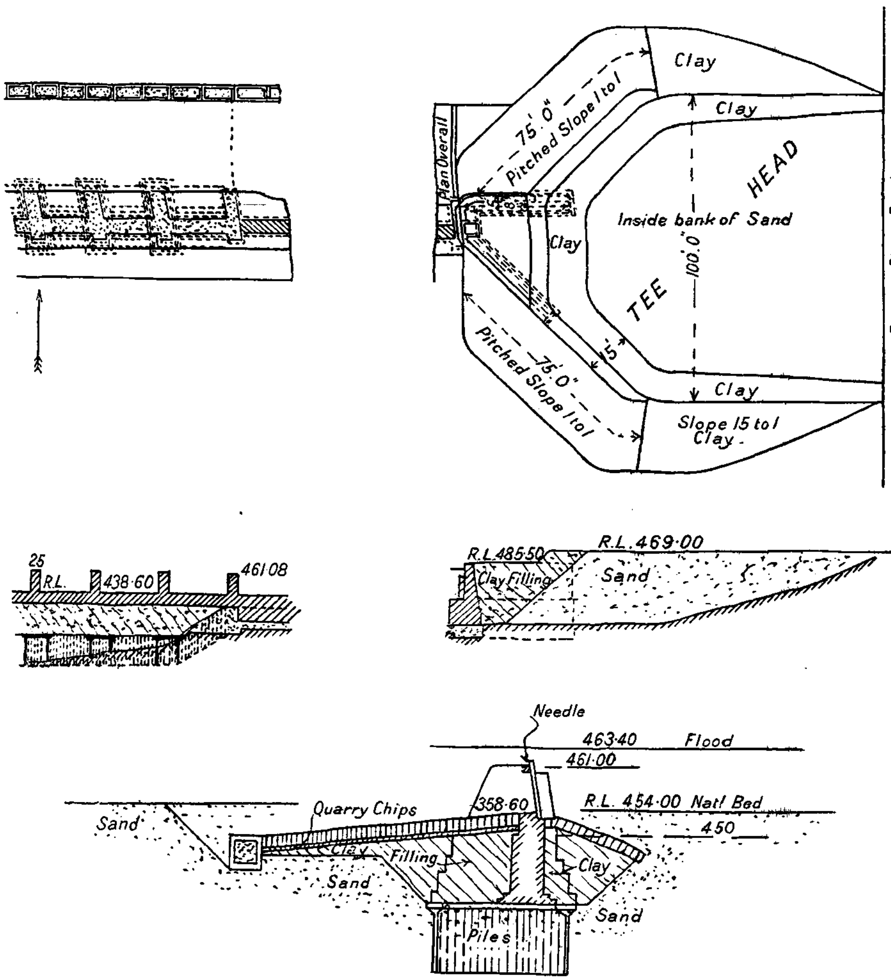


FIG. 23.—Sidnai Canal Needle Weir, Sutlej River.

by vents with the evident object of reducing the hydrostatic pressure below the weir by allowing the percolating water a free passage. This idea is purely chimerical; the only possible effect of the vents will be to nullify the utility of the fore curtain, as providing an additional length of creep. In the diagram the outlet not being quite free, a reduced pressure of $1\frac{1}{2}$ feet is allowed to exist at the commencement of the floor tapering to *nil* at its extremity.

(**54**) In Fig. 25, while retaining the general characteristics of the original profile, the following modifications have been introduced :—

An impervious rear apron has been provided, and the floor widened

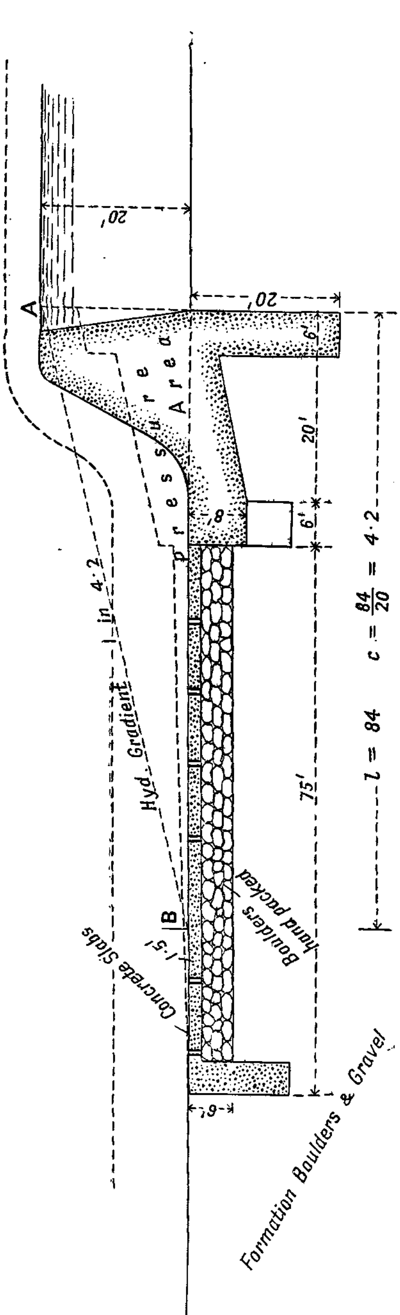


FIG. 24.—Granite Reel Weir, Salt River, Arizona.

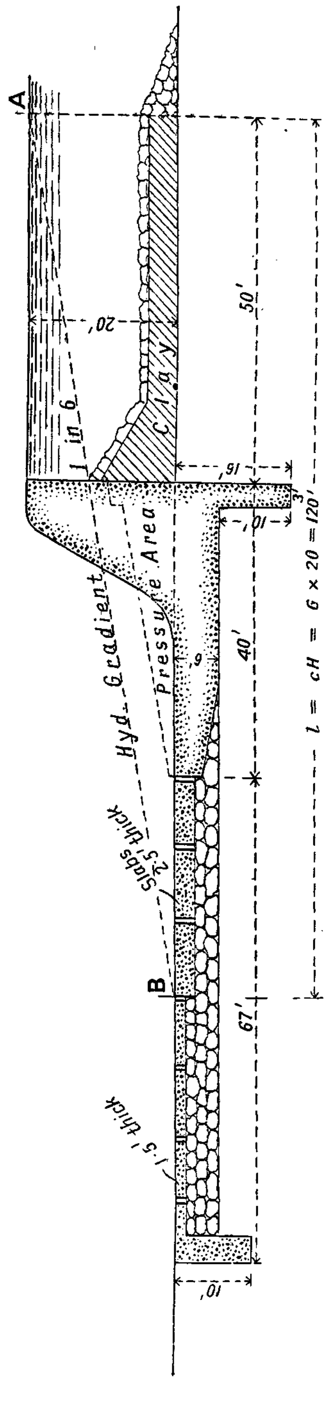


FIG. 25.—Alternative Section.

considerably. The head of water acting on the back of the weir wall is reduced from 20 to 16 feet; the section can, therefore, be reduced. The hydraulic gradient provided is 1 in 6, the previous one of 1 in 4 not being deemed flat enough. The floor-covering slabs have been increased in thickness from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet, the previous thickness not providing sufficient weight, as they are practically submerged and lose weight from displacement. As thus altered, the work will not cost more, and be very much more stable. The weir is 1,000 feet long. The section over rock is given in Fig. 31, par. 58, Chap. II.

(55) A section of the Grand Anicut at Dauleshwiram over the Godaveri River is given in Fig. 26.

This work is remarkable as differing from the ordinary anicut type in many respects. It was constructed some time in the 'forties, when erroneous ideas regarding the destructive effect of water in a direct overfall were entertained, and it was deemed essential in the case of an overfall to resolve the current in a horizontal direction. The same error was perpetuated in the ogee profiles of the falls in the old Bari Doab and Ganges canals, and the practice is still prevalent in the United States.

This weir consists of two breast walls founded on rows of circular 9 inch thick wells, sunk 6 feet in the sand and filled with rubble stone. These breast walls are 36 feet apart, and are connected by a wide horizontal crest and curved apron, bringing the level down to 8 feet below crest and to 2 feet above that of the normal river bed. Beyond this masonry apron is packed pitching, 30 feet in length, where a shallow masonry wall finishes the made apron. The talus of more or less hazard stone pitching extends for another 150 feet. The apron and horizontal crest are founded on sand filling, a system at once economical, and with this description of coarse sand, quite safe. This making up by sand filling is quite a common practice in Madras works, which might well be followed with advantage in many cases where rubble filling has been resorted to.

It is evident that this section contains the disadvantages or weak points of both types A and C. The expensive masonry apron is quite equal in sectional area to that necessary in the floor of a direct overfall, while the contour of the profile must

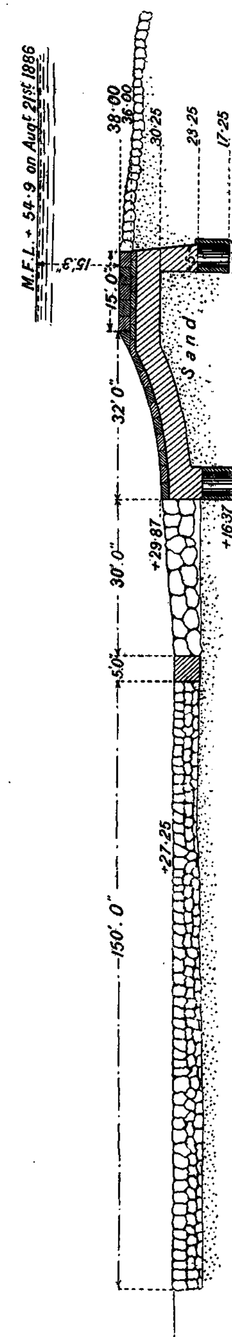


Fig. 26.—Dauleshwiram Anicut, Godaveri River. Length, 7,734 feet. Discharge, 1,200,000 second-feet.

tend to increase the erosive action on the talus, which is a point it is desirable to avoid as much as possible.

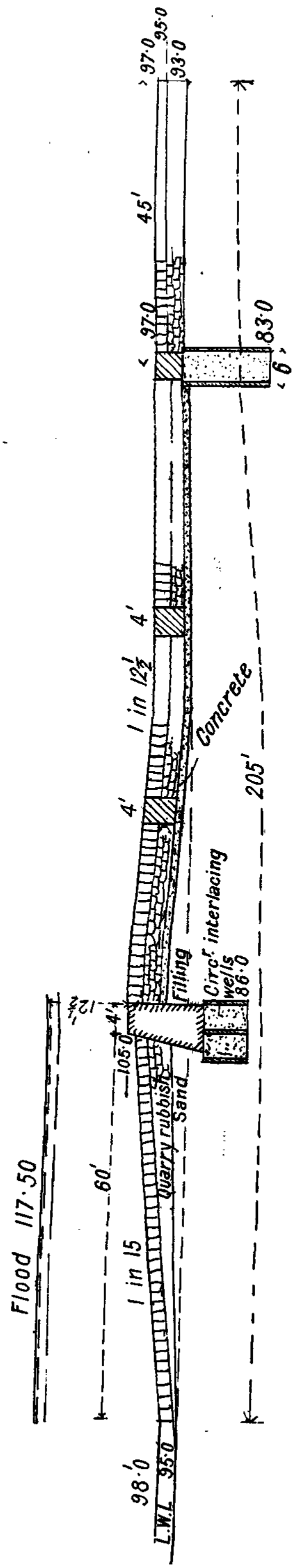
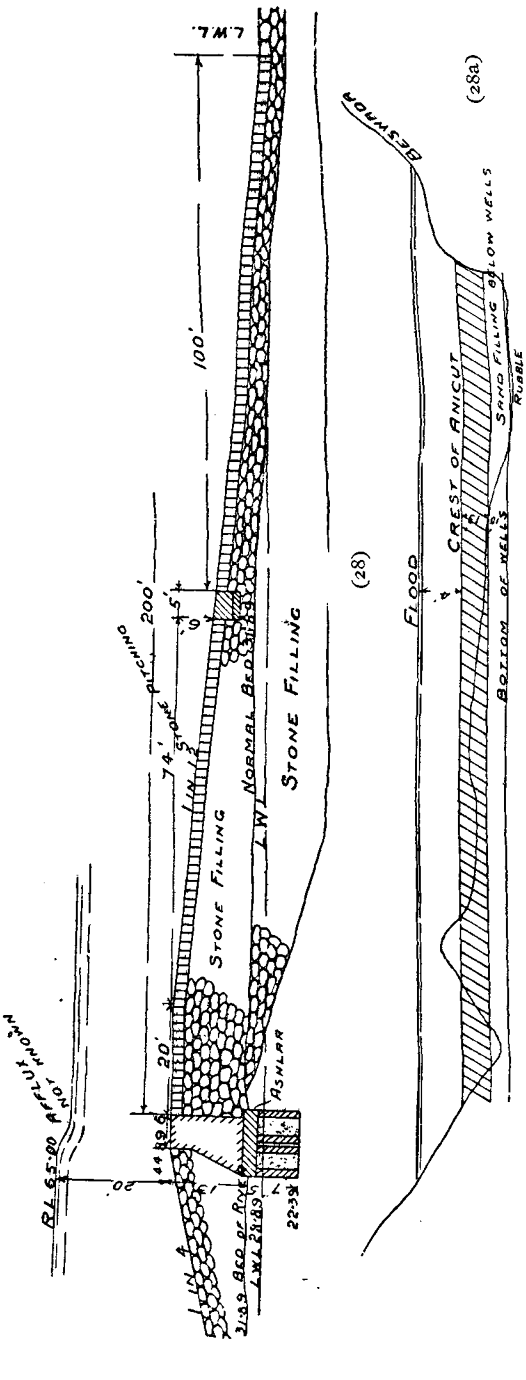


FIG. 27.—Sangam Anicut, Pennér River. Length, 4,076 feet. Discharge, 600,000 cubic feet.



FIGS. 28, 28a.—Beswada Anicut, Kistna River. Length, 4,000 feet. Discharge, 736,000 second-feet.

The width of the weir is 230 feet, which is exceptionally great, being only exceeded by the 250 feet of the Okhla Weir, which work, however, is founded on sand of a very inferior description to that in the Godaveri River. The crest of this weir is 11 feet above normal bed level.

(56) The Sangam Anicut (Fig. 27) is a work on the same river as the Adimapali, but lower down the stream; the whole discharge is greater, but the unit discharge is less, being 147 to 184 of the latter (*vide* Table I., par. 27). Its fore apron, which is impervious, is 100 feet long; this much exceeds what would be obtained from formula (1), while L, the width of talus, is less by 20 feet. The rear apron is exceptionally long, which in this case is unnecessary unless required to protect the crest wall from being undermined by cross currents. The value of l much exceeds strict requirements.

(57) The Beswada Anicut, Figs. 28, 28a, and 28b, at the head of the Kistna River canal system, which was constructed 1854–55, is a notable work.

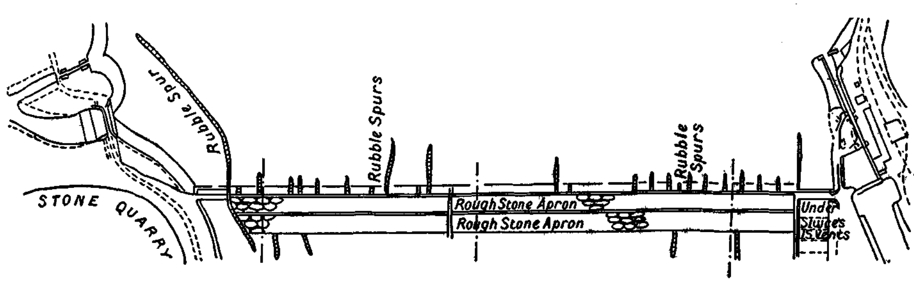


FIG. 28b.—Beswada Anicut. Site Plan.

The river at the site of the flood had actually a flood velocity of 10 second-feet. The weir obstructs half the water section, so that it is strained to the utmost.

In the list given in Table I, par. 27, the unit discharge exceeds that of any other work, excepting only the Madaya.

A row of circular wells of the usual Madras type form the foundation of the breast wall. Where the bed was deep in the cold weather channel it was first filled up with sand above L.W.L., and the wells were then built on this bank and undersunk in the dry, the current having been diverted elsewhere. The remaining part of the low channel was then filled up with stone. This is explained in Figs. 28 and 28a.

In the case of the Mahanuddee River Weir, at Jobra, a different procedure was adopted, as is shown in Fig. 18a. Where the deep bed existed it was filled up to above L.W.L. with stone, the line of blocks being entirely omitted at a part where, apparently, they are most wanted. The breast wall was then built on the rubble mass.

This forms a further proof of the absence of necessity in the use of these expensive curtain walls. Until Okhla Weir established a precedent, at least one curtain wall of blocks and wells was considered absolutely necessary for the safety of these works, although the designers would have been hard put to find a reason for their belief.

(**58**) The late General Rundall deserves the gratitude of posterity for having had the foresight to conceive and the boldness to execute a large, loose rock weir without any foundations below the bed of the river.. At the time Okhla Weir was under construction, most engineers, as the author well recollects, prophesied its utter failure.

These old weirs were not provided with crest shutters, as is now universally the case; hence their obstruction to floods was very great and considerable trouble was experienced from the river-bed silting up in rear of the weir and forming large islands, to be covered soon with vegetation. These caused dangerous cross-currents, to prevent which rubble spars had to be run out up stream. These are shown in Fig. 28b. A modern weir of ype C would have a crest some 4 to 6 feet lower, that is, about half as high as in Fig. 28, the required summit level being attained by collapsible weir shutters of some kind.

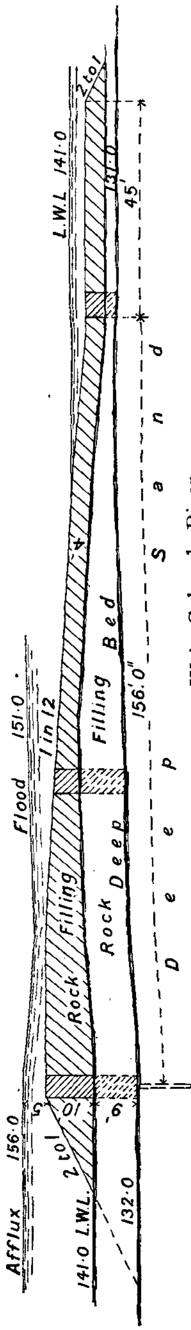


FIG. 29.—Laguna Weir, Colorado River.

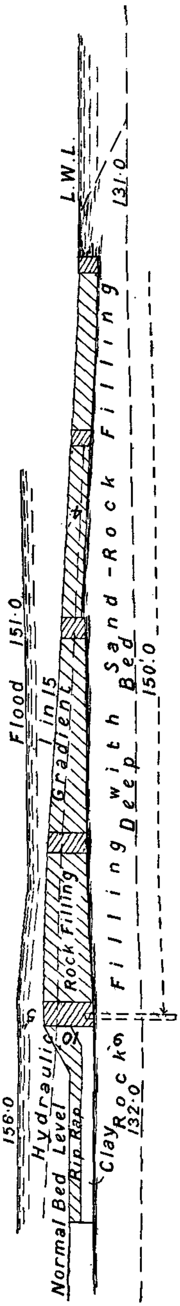


FIG. 30.—Alternative Section.

(**59**) The Laguna Weir (Fig. 29) has the distinction of being the only weir of type C in America. It is consequently termed the "Indian" weir.

The Colorado River has all the attributes of one of the deltaic rivers of India, the main difference lying in its low flood level and comparatively small discharge, which dwindles down to a very small amount in the dry season.

From the description of the sand, the coefficient will be at least 15. The weir is 4,500 feet long, as measured on the plan, and the depth of afflux is 5 feet. The crest of this weir is placed at high flood level, consequently it blocks the whole of the water section, a procedure which is quite unique.

The unit discharge is from 35 to 40 second-feet only, the overfall being free. No crest shutters are used, and the rear apron is not stanched with a clay layer. The object of crest shutters is to reduce the obstruction to floods, and consequently the scouring action of the overfall, in addition to which they prevent the formation of high silt deposit in the rear of the work, which often proves a great inconvenience. The flood unit discharge being so extremely moderate, the first given reason may be said to be non-existent; with regard to the second, however, it is an open question, whether in this particular case the adoption of crest shutters would be worth the expense and annual cost of manipulation.

(60) In the profile the fore slope is 1 in 12; this for coefficient 15 is deemed not sufficiently flat, and 1 in 15 would be better, and be more in accordance with the hydraulic gradient. It would also insure a heavier mass, which is a desideratum.

The breast and body walls are carried right down to the deep bed level, and the former has sheet piling below that again. These precautions, in face of the experience gained in Okhla Weir and also in the Mahanuddee Weir at Jobra (Fig. 18a), appear unnecessary.

If body walls were successfully founded on top of the rock filling at Jobra and at Okhla, the same could be done here. The rock filling in the deep channel will silt up immediately and form a mass of natural uncemented concrete of a very solid description. Below the weir, as the low supply will all be carried through the weir sluices, the deep water channel will soon cease to exist as such. The body walls are not considered to be sufficiently near to each other to be properly effective. These suggested improvements are embodied in the alternative profile shown in Fig. 30. The wooden sheeting is retained below the breast wall, but solely with the object of blocking the deep channel, which during construction should be diverted through the sluice channel.

If this is done there is no reason whatever why the deep channel should not be filled up with nothing but sand, the latter being kept in place by a heap of rubble deposited above and below it.

Sand filling is commonly practised in Madras weirs and, provided there is no current which will work it away, or its removal by scour is otherwise prevented, no adequate reason, beyond that of rank timidity, can be adduced to show why this material, which is always at hand in unlimited quantities, should not be made use of.

It is believed that the most satisfactory profile for this work would be type A^2. That is, a direct overfall with a raised horizontal floor, but this is entirely a question of cost. Considering the great discharging capacity of the weir-scouring sluices provided for this work and the low unit flood discharge, it is an open question whether it would not be an economical move to reduce the length of the crest several hundred feet by running out an embankment as continuation from the left side, the same terminating with a T head similar to what is shown in Fig. 23 of the Sidnai Canal Weir.

The plan of these head works is given in Fig. 13, Chap. VII., par. 28.

(**61**) Fig. 31 is a section of the Srivakantham Anicut over the Tampraparni River. As their unpronounceable names suggest, these works are situated in the Madras Presidency. Although the section seems insignificant, its length is a quarter of a mile, and the flood water of the river is 15 feet deep; the unit discharge is but 90 second-feet.

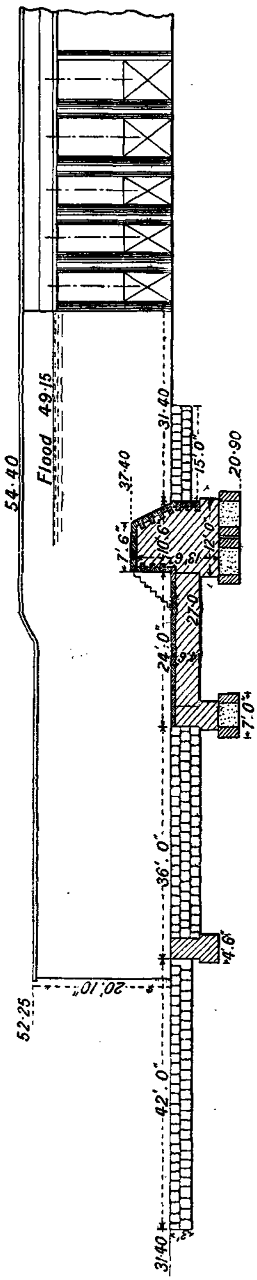


Fig. 31.—Srivakantham Anicut, Tampraparni River. Length, 1,380 feet. Discharge, 123,000 second-feet.

This work is practically founded on hard clay. The sand being quite shallow, the curtains fore and aft of the floor penetrate into the clay substratum. The work is, therefore, free from hydrostatic pressure. If it were not for this the length of the floor would be quite inadequate.

(**62**) Fig. 32 is the Pelandorai Anicut, likewise a Madras work. The head is 9 feet, consequently a base length or a value l of 108 feet is required (the coefficient being 12). The actual value is about half this, proving that the work, like the previous one, is founded on clay.

For falls on clay or rock the length of floor or apron should $= 3H$, or, if there are shutters, $3H^1$. The riprap or pitching beyond should, it is considered, extend at least as far again.

(**63**) We have hitherto dealt with the section of the weir itself, but canal head works comprise, in addition to the weir itself, the following distinct items, which may be enumerated as :—

First : The weir across the river.

Second : The weir-scouring sluices or undersluices.

Third : The canal head or intake.

With regard to the position of the weir relative to the river, the first point to be determined, in deciding on the position of a canal take-off from a river, is the most suitable site for the purpose.

The following remarks, excerpted from Mr. Buckley's valuable work "The Irrigation Works of India," will explain the points to which attention must be directed in forming an opinion on this matter :—

"The selection of the site of the head works is often a matter of considerable difficulty. One of the most essential points to be first considered is the nature of the silt carried by the river, whether it is fertilising or the reverse, what proportion of it is desirable to carry down the canal, and whether the soil in which the canal is to be cut can stand the velocity which can carry the silt. When the slope of the canal is determined, an idea can be formed from the levels of the country at what point the water can be delivered on the surface of the ground, and the area under command of different sites for the head works can be ascertained. The approximate length and depth of cutting of the unprofitable portion of the canal which lies between the head works and the first point of irrigation can also be roughly worked out, and it is necessary to do this, as the cost of this portion may often be very great. The height of the weir above the bed of the river should next be determined. This will depend on the depth of water required in the canal and the level of the canal bed with reference to it. Then the effect produced by weirs of various heights on the flood level of the river should be worked out, the maximum flood discharge must be ascertained from gauge readings, from cross-sections of the channel and known surface slopes, and from these data the afflux, or height by which the flood level will be increased by the obstruction caused by the weir, can be calculated. This point is of great importance, as upon it depends the necessity for

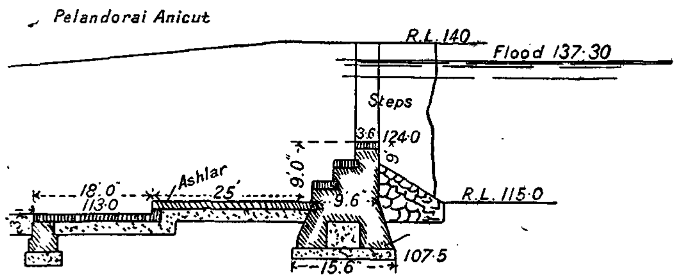


FIG. 32.—Pelandorai Anicut. Length, 860 feet. Discharge, 850,000 second-feet.

constructing embankments to control the river above the weir, so as to prevent inundation and the possibility of the river outflanking the weir. The most suitable position as a rule for a weir and head works of a canal is on a portion of the river where the channel is straight, the velocity uniform, and the sectional area of the stream fairly constant. A narrow gorge of a river appears to have the advantage of cheapness, but it may prove the most expensive owing to the greater velocity and depth of the current necessitating a heavier section than usual. A particularly wide reach of a river has, on the other hand, the disadvantage that the average velocity being decreased the deposit of silt and sand is encouraged, and the bed of the river above the weir is likely to become so raised that difficulty will be experienced in keeping a channel open to the take-off of the canal. Another point of importance is to select a site as near as possible to a stone quarry, so as to diminish the lead of building materials."

(64) A canal, as a rule, should take off on the "cutting" side of river. The channel here is generally deep and open in times of low supply, besides which the bank on the cutting side often consists of a high bluff of clay soil, so that the canal Head as well as part of the weir sluices can be founded on

solid material, which will prove a great saving in the cost of the foundations of these expensive works. In many cases canals take off at both sides of the weir, as in that of the Dauleshwiram head works of the Godaveri canals; the best site would then be where high grounds exist on both banks.

If such a site is not obtainable, the weir sluices belonging to the head work on the low side should be designed of more powerful draft than those on the natural cutting side of the river, and further supplemented by training works above the weir so as to form and retain an open channel leading past the canal head.

The usually best alignment of a weir in connection with a river channel is at right angles to a long straight reach, but there are occasions when a skew weir is desirable, an example of which is given in the block plan of the Rupar head works, Fig. 5b, Chap. VII.; also in those of the Chenab and Jhelum canal head works shown in Figs. 8b and 9b of this Chapter.

Where the river is of abnormal width, consisting of wide sand banks overflowed in flood time only, the channel can be narrowed with advantage, care being taken not to reduce the length of the weir below that of the average width of the river at sites where high friable banks occur.

This reduction in width will generally take place on one side only, viz., on the opposite bank to where the canal off-take is situated. This is effected by the construction of a T head composed of earth or even sand of considerable dimensions and pitched with stone; an example of this is given in the plan of the head works of the Sidnai Canal, in which the T head is shown (*vide* Fig. 23). Up stream, training banks or spurs will also be necessary as a support, as inspection of Figs. 8b and 9b will show.

(**65**) A difficulty experienced with weirs over very wide, sandy river beds is the formation of islands of sand in the bed above the weir. These islands get soon overgrown with weeds, and if not removed form a serious menace to the safety of the weir by causing cross currents which, if they run parallel to the work, may undermine it from the rear. The best preventive of this danger is the adoption of collapsible iron shutters fixed on the crest of the weir; these are generally about 3 feet in depth and some 20 feet in length. The supports consist of iron rods hinged to the crest of the weir and also to the shutter at the height of the centre of pressure. Thus, when the river water tops the shutter crest, they turn over automatically and fall prone on the weir crest till raised again by hand. The leakage through them is insignificant. The weir then need not be built as high as would have been necessary prior to the introduction of shutters, and so does not offer near so much obstruction to the water section of the river when in flood. Any tendency to accumulation of sand in the bed above the weir crest level would also be swept away as freshets come down. In the Beswada Weir (a block plan of which is produced in Fig. 28b), which is not provided with shutters, extensive rubble spurs had to be run out and maintained up stream in order to guide the current straight on to the weir. These expensive training works would probably not be necessary at all, if collapsible shutters had been originally adopted in this work.

In the older works, such as the Okhla, Narora, and Dehri Weirs, the depth of the crest shutters did not exceed 3 feet. In more recent works, and also since fitted to older weirs, much deeper shutters are used. In the Rupar and Chenab weirs 6 feet deep shutters are used, as also on the Betwa and Dhukwa weirs.

The Jhelum shutters are 4 feet deep and 6 feet wide; the Chenab shutters are 6 feet deep and 4 feet wide. They fall automatically when topped, or can be dropped as occasion demands. They are in some cases tripped up by a chain with discs at intervals, worked by a winch on the abutments.

CHAPTER VII

WEIR SLUICES

(1) In all weirs constructed across wide rivers having sandy beds, and also in others which carry much silt in suspension, the provision of weir sluices is a necessity.

The function of these works is two-fold. First, to train the deep channel of the river, the natural course of which is obliterated by the weir, past the canal head, and to retain it in this position; otherwise in a wide river the low water channel might take a course parallel to the weir crest itself, or else one distant from the canal head, and thus cause the approach channel to be blocked with deposit. Secondly, by manipulating the sluice gates, silt is allowed to deposit in the slack water in the deep channel thus preserved, and when this accumulation becomes excessive, it can be scoured out by opening the gates.

(2) The sill of the weir sluice is placed as low as can conveniently be managed, being generally, either at L.W.L. itself or somewhat higher, its level generally corresponding with the base of the drop or breast wall. Thus the maximum statical head to which the work is subjected is the height of the weir crest plus that of the weir shutters, or $(H + d)$.

(3) The ventage provided is regulated by the low water discharge of the river, and should be capable of taking more than the average dry season discharge. In one case, that of the Laguna Weir, Fig. 13a (*post*), where the river low supply is deficient, the weir sluices are designed to take the whole ordinary discharge of the river excepting high floods. This is with the object of maintaining a wide, deep channel which may be drawn upon as a reservoir. This case is, however, exceptional and its practical utility is questionable.

(4) The efficiency of weir sluices in the prevention of silt deposit in the neighbouring canal head might be said to be ineffective without the co-operation of the latter work. Of late years it has become recognised that the sill of the canal head, or intake, must be located several feet higher than that of the weir sluices, in order to provide a deep basin outside in which silt can be permitted to deposit, while the supply is tapped from the surface. This deposit can be periodically removed as before stated by opening the weir sluice gates.

Another important innovation is the provision of a so-termed "divide";

that is, a high partition wall running out up stream as a continuation of the face of that abutment of the weir sluices which adjoins the weir itself. This wall should be advanced well beyond the canal head, and terminate in a rounded end.

The divide wall cuts off the deep channel, maintained in front of the canal intake, from the rest of the river bed, and is found to be of great utility in the matter of hindering silt deposit in the canal, which for years has been such a bugbear to irrigation engineers.

(5) As the object of a weir sluice is to pass water at a high velocity in order to scour out deposit for some distance to the rear of the work, it is evident that the openings should be wide, with as few obstructions as possible in the way of piers, and should be open at the surface, the arches and platform being built clear of the flood level. Further, in order to take full advantage of the scouring power of the current, which is at a maximum at the sluice itself, diminishing in velocity with the distance to the rear of the work, it is absolutely necessary not only to place the canal head as close as possible to the weir sluices, but to recess the head as little as practicable behind the face line of the abutment of the end sluice vent.

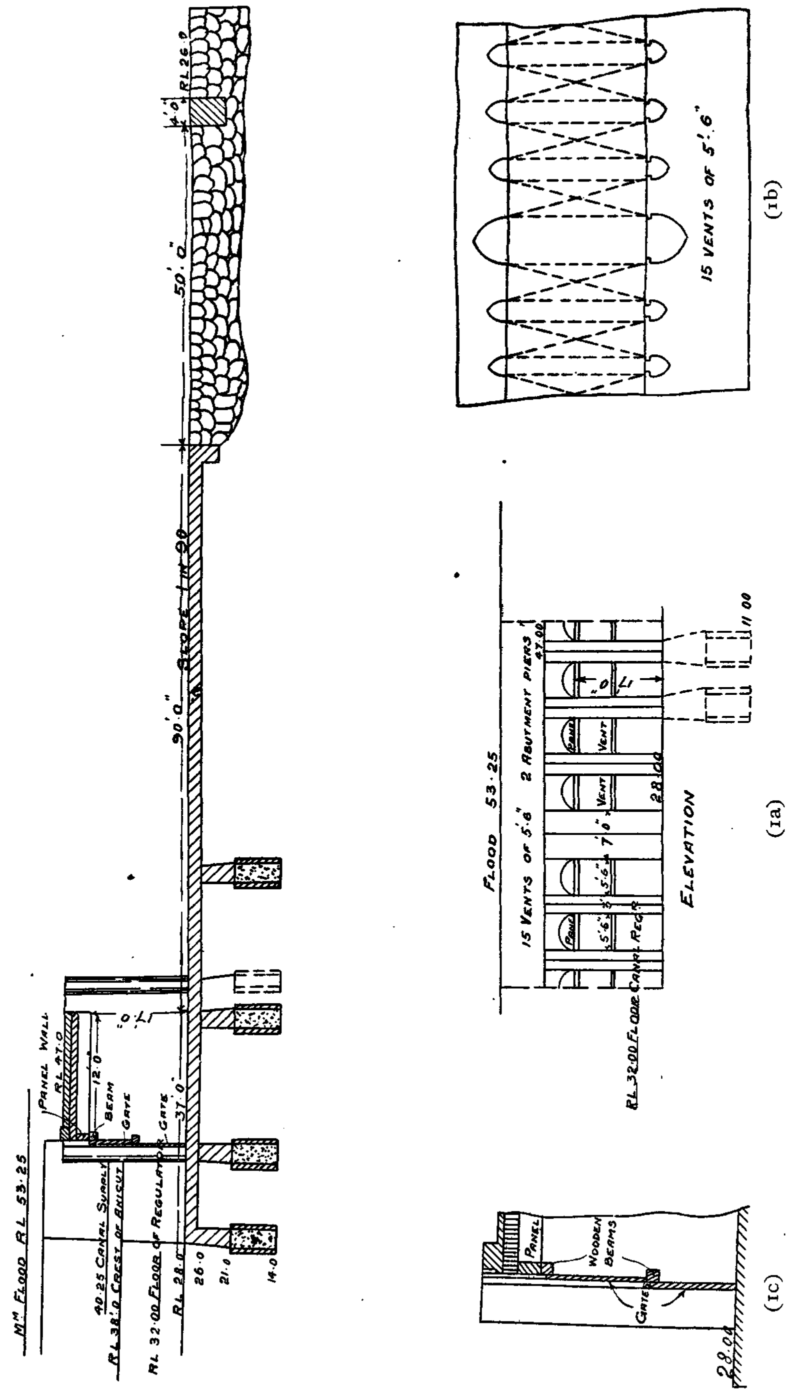
In the examples of old Indian works, which will now be given, we shall find that these conditions have often been violated with great detriment to efficiency. This in part was unavoidable, as prior to the introduction of iron drop gates fitted with anti-friction rollers, the openings of weir sluices and regulators generally had perforce to be designed of narrow width, in order to admit of the gates being worked under a pressure of water. Thus we see that the width of waterway in each opening was limited to 4, 5, or 6 feet only, whereas now such openings vary from 20 feet to 25 feet in width. Many old works have been remodelled on this principle.

(6) With regard to water pressure, weir sluices perform a *rôle* identical with that of river regulators, or, as they are also termed, open dams or barrages; that is to say, they only require to be designed for partial or limited regulation. They are not intended to hold up the full flood depth of water in the river, but as a rule only when the river is low are the gates closed, in order to dam up the water to a certain level, sufficient to force supply down the adjacent canal head. In some cases, where the velocity of the current at full flood is so great as to overstrain the flooring, the lower sluice gate or both gates are kept down, acting then as a submerged weir.

When the river rises much above canal full supply level, the gates can be lifted clear of the flood to afford a free passage to the current.

Canal head regulators or intakes, on the other hand, have occasion to be entirely closed during the highest floods in the river; consequently the regulation they have to perform is entire, not partial, so that these works are subjected to a much greater statical stress than weir sluices, and consequently, for convenience of manipulation, are usually designed with narrower openings than are necessary or desirable in the latter.

Some examples of works with critical and emendatory remarks will now



Figs. 1, 1a, 1b, 1c.—Dauleshwiram Weir Sluices, Godaveri Canals.

be given, as by this method the principles governing design can be clearly set forth.

(7) Fig. 1 is a representation of the Dauleshwiram weir sluices of the weir of that name on the Godaveri River in the Madras Presidency.

This old work consists of fifteen vents of $5\frac{1}{2}$ feet width, which are arched over above canal supply level, presenting a level platform 19 feet high above sill. The openings are completely closed by wooden draw gates in grooves, reaching above the spring line, while the arch segments are closed by narrow panel walls which are supported on wooden beams. The upper gates rest on another line of wooden sills, against which the top side of the lower gates bears. The maximum flood line is $25\frac{1}{4}$ feet above the sluice sills, and so the whole work is submerged during high floods, a depth of $6\frac{1}{4}$ feet passing unobstructed over the platform.

The R.L. of the sill is	28·00
,, ,, of the sill of head regulator. .	32·00
,, ,, of the crest of the anicut . .	38·00
Full supply level in canal	40·25
Level of submerged platform . . .	47·00
Level of maximum flood	53·25

On the occasion of floods the gates are either suspended at the top of the grooves, in which position they will still somewhat obstruct the waterway through the sluice openings, or else are taken out bodily and removed.

The obstruction offered in this old work to the free passage of floods is very great. Regulation is only required up to a depth of 12 or 13 feet, so that if the work were reconstructed on modern lines it would be less expensive and much more effective. The remodelled design would roughly consist of the following:—Four spans of 20 feet with piers 5 or 6 feet thick, double or treble iron gates 13 feet high, running in separate grooves with anti-friction rollers. Spring of arches to be at R.L. 53, or 28 feet above sill; the platform at top 7 feet higher, or at R.L. 60·00. This will allow the gates to be drawn up and left hanging clear of the flood line. The gate grooves will be situated outside the main arches, as in the original work. To facilitate the lifting of the gates, another arch and narrow platform is desirable outside the grooves. Its function is to carry the outer rail of the travelling winch, the inner resting on the edge of the main platform.

This latter need not be over 10 or 12 feet wide, the pier ends battering outwards, in order to spread the weight of the superstructure and to afford length of base sufficient to bring the resultant line of pressure within its middle third. The upper gate can, if so required, be lowered behind the under gate.

The work thus remodelled is shown in Fig. 2. The design is very similar in appearance to that of the Assiût Regulator, plan of which is given in Fig. 10.

(8) The next plan to be examined is that of Fig. 3, of the Sangam weir sluices on the Pennér River, Madras Presidency. This is a much more modern work than that at Dauleshwiram, having been constructed in 1870–80.

This design is on the principle of a reservoir escape sluice, or canal head, for which purpose, with certain modifications, it would answer well; but it cannot be said to be in the least adapted for the *rôle* of a scouring sluice. As will be seen by reference to the plan, the design consists of a series of small vents 6 feet wide and 5 feet high, which are topped by a heavy breast wall carried up to above flood level, which barrier effectually prevents any passage of water except through the sluice vents. This system is the very reverse of what is now deemed correct, which latter principle, as has already been explained, is to afford as free a passage as possible through the work, so as to cause effectual scour of deposit in front of the adjacent canal head.

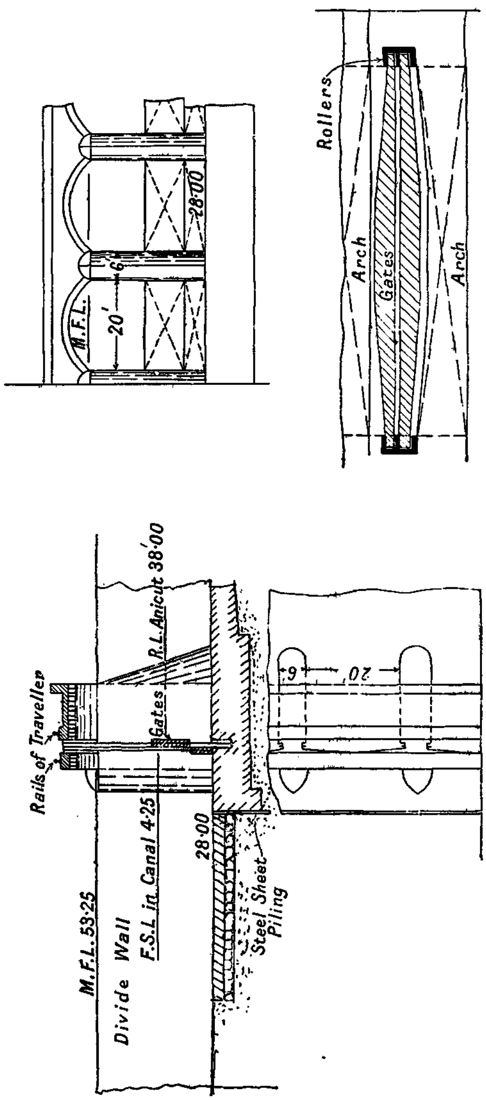


FIG. 2.—Alternative Design.

The design, if remodelled on these lines, would be very similar in appearance to Fig. 2, or to the Assiût Regulator and Rupar weir sluices, viz., a simple bridge of a few large spans of 15 feet or 20 feet with arches and platform above M.F.L.

A noticeable peculiarity in Fig. 3 is the roadway provided behind the breast wall, which is carried by a separate set of arches springing from every alternate pier, which pier is lengthened to receive them. This roadway slopes down to the anicut crest, and is probably used for cart traffic.

The defences of the floor against erosion or upward statical pressure are very considerable. There are triple lines of interlacing curtain wells, one at either end of the masonry apron and the third at the termination of a length of very deep rubble pitching. One minor point deserves mention, and that is the disposition of the dwarf partition wall dividing the weir sluice floor from that of the weir itself. This wall is given an outward trend,

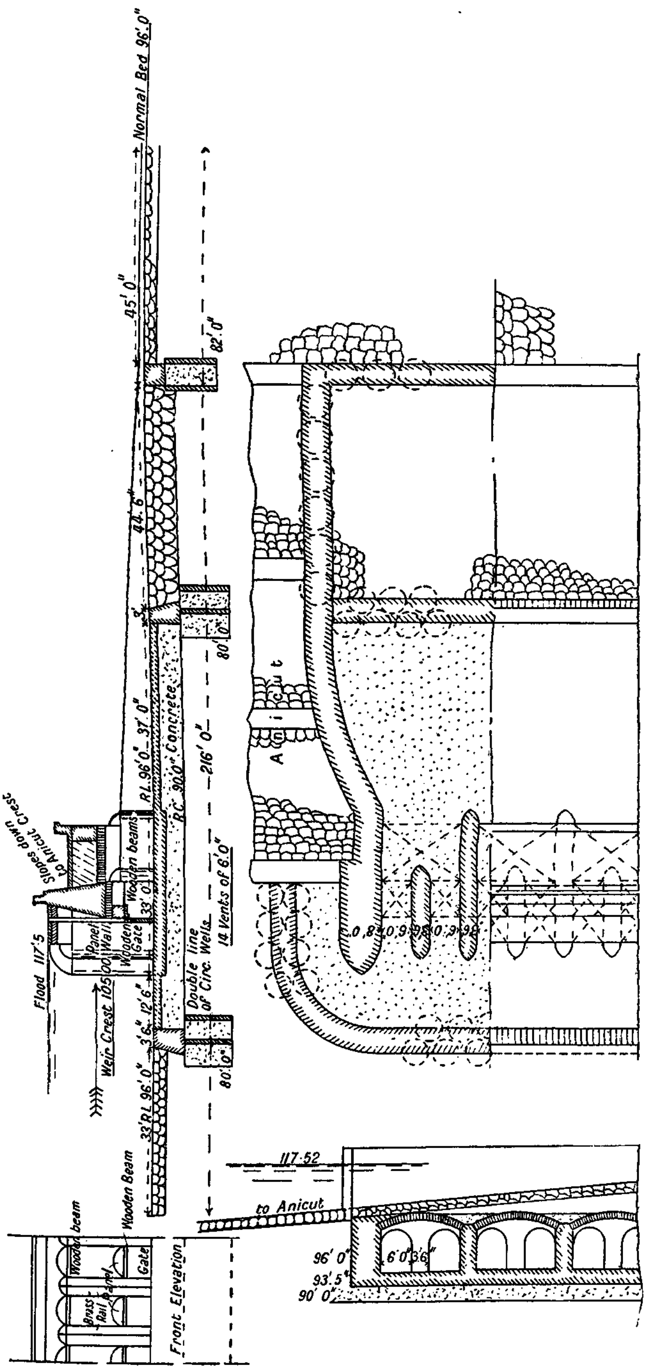
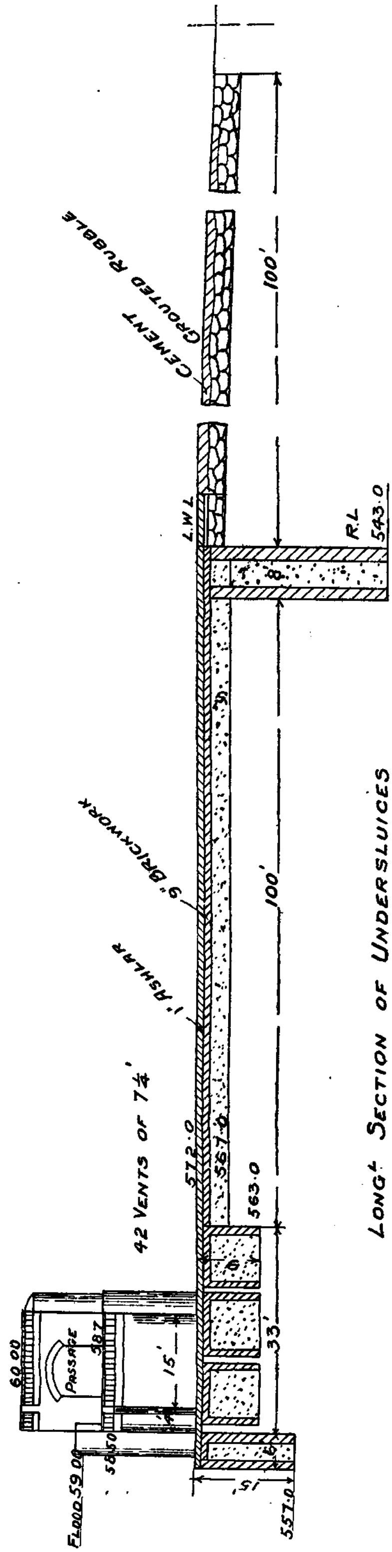


FIG. 3.—Sangam Weir Sluice, Pennér River.

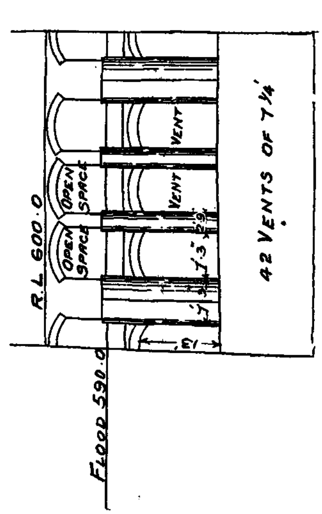
so it encroaches on the weir apron and talus. This cannot be considered a good arrangement; the partition wall should be perfectly straight and normal to the weir crest. If widening of the exit waterway is deemed requisite, this should be effected on the opposite side by throwing the flank wall inwards.

(9) The following remarks will serve to clear up a somewhat obscure point:—

The maximum horizontal stress on the superstructure of a partial regulator may possibly be in excess of what is produced by the maximum statical head, which means that a greater effect can be produced by a less head. This is owing to the influence of the increased depth of the water fore and aft of the work. We have already seen in Chaps. II. and VI. that the same applies to weirs, particularly when subject to flotation. The opposing horizontal overturning moments are proportionate to the cubes of the depths of water on either side of the dividing wall or structure, if the division is complete, which, however, is not quite the case. Thus, supposing the maximum statical head on a weir sluice headwork



FIGS. 4, 4a.—Narora Weir Sluices.



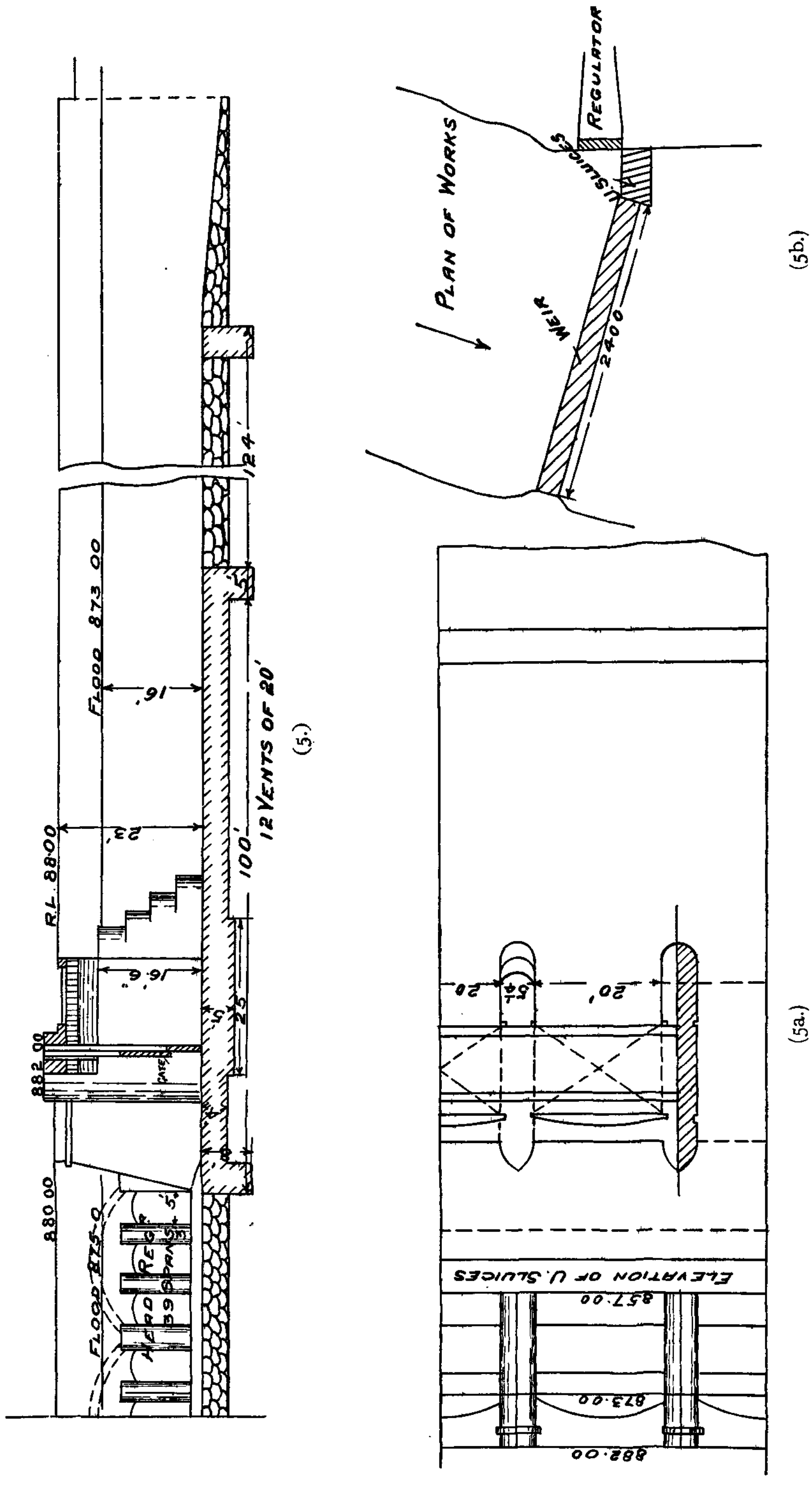
FRONT ELEVATION OF U.S. (4a.)

to be 10 feet; this must occur when there is no tail water, *i.e.*, when the whole river low supply is taken into the canal. The overturning moment M can then be represented by $10^3 - 0^3 = 1{,}000$. Now with 2 feet passing over the weir and gates, the depth of water down stream can be calculated with sufficient accuracy for the purpose by using Table II., Chap. II. Assuming the river bed slope to be 1 in 10,000, the ratio of rise will be about ·35. Then $D = \frac{d}{r^0} = \frac{2}{35} = 5{\cdot}7$ feet. The depth up stream $(H + d)$ will then be 12 feet, and down stream 5·7, and M will be $100 (10 + 6) - (5{\cdot}7)^3 = 1{,}600 - 185 = 1{,}415$ (formula (16), Chap. II). Thus we see that a much greater horizontal stress is induced by the lesser head of 6·3 feet than by the maximum head, which is 10 feet. This takes into account that the greater area of pressure is that of a truncated triangle, with $H = 10$ and $d = 2$ feet.

(10) The Narora weir sluices are given in Fig. 4. These were built about 1875-80, and present a marked advance on the last example. The vents are $2\frac{1}{4}$ feet wide in groups of three spans, separated by piers $2\frac{1}{4}$ feet thick. The intermediate piers are 2·9 inches thick, or ·37S nearly. There are two tiers of arches, through both of which an open slit is provided for the passage of the draw gates. The travelling winch runs on the top platform, which is 28 feet above floor level, a height sufficient to allow the gates, when raised up, to hang clear of the flood line. The lower tier of vaulting forms a lower platform, 3 feet below high flood level, and the space between the piers, which continues up to the upper tier, is open, allowing the passage of water above the lower platform. The upper parts of the piers are pierced by wide openings at right angles (*vide* Fig. 4a), thus forming a vaulted passage, which is of great convenience for the manipulation and stacking of the gates when drawn up.

It is quite evident that this design could be greatly improved by simplification. The spans should be widened, say to 15 feet or 20 feet, with piers 4 feet to 5 feet thick, the abutment piers being abolished together with the middle platform. The profile would then closely resemble Fig. 2 or the Assiût Regulator (Fig. 10).

(11) The maximum statical head on the floor is 13 feet; the coefficient c being 15, l will equal $c \times H$, or 195 feet (*vide* Chap. VI.). Up to and inclusive of the curtain wall it is 180 feet, and if the grouted rubble beyond be esteemed as an impervious continuation, the total value of l will be increased to 280 feet. The floor thickness of 5 feet, as was also the case with the weir apron (par. 28, Chap. VI.), is deficient. It should be by formula (1), par. 21, Chap. VI. $= \frac{4}{3} \frac{H - h}{\rho - 1}$. Here h, or the loss of head due to percolation, is $\frac{53}{15} = 3{\cdot}5$ feet; $(H - h)$ then will be $(13 - 3{\cdot}5) = 9{\cdot}5$ feet. The correct value of t will then be $\frac{4}{3} \times \frac{9{\cdot}5}{1\frac{1}{4}} = 10$ feet; thus the floor is in a high state of tension, being only half as thick as it should be. The value



FIGS. 5, 5a, 5b.—Rupar Weir Sluices, Sirhind Canal, Sutlej River.

of L is 220 feet, that is, 50 feet longer than that of the weir. The correct proportion will be determined later, this proving a valuable guide (*vide* par. 13 (*post*)).

(12) The weir sluices of the Rupar Weir (Fig. 5), on the Sutlej River, at the head of the Sirhind Canal, are remarkable as being among the best of any Indian works of this description of which we possess records. Their characteristics are boldness and simplicity. No doubt this design was the precursor of the Assiût Regulators, which are built on closely similar lines. The vents are 20 feet wide bridge openings, the arches springing at maximum

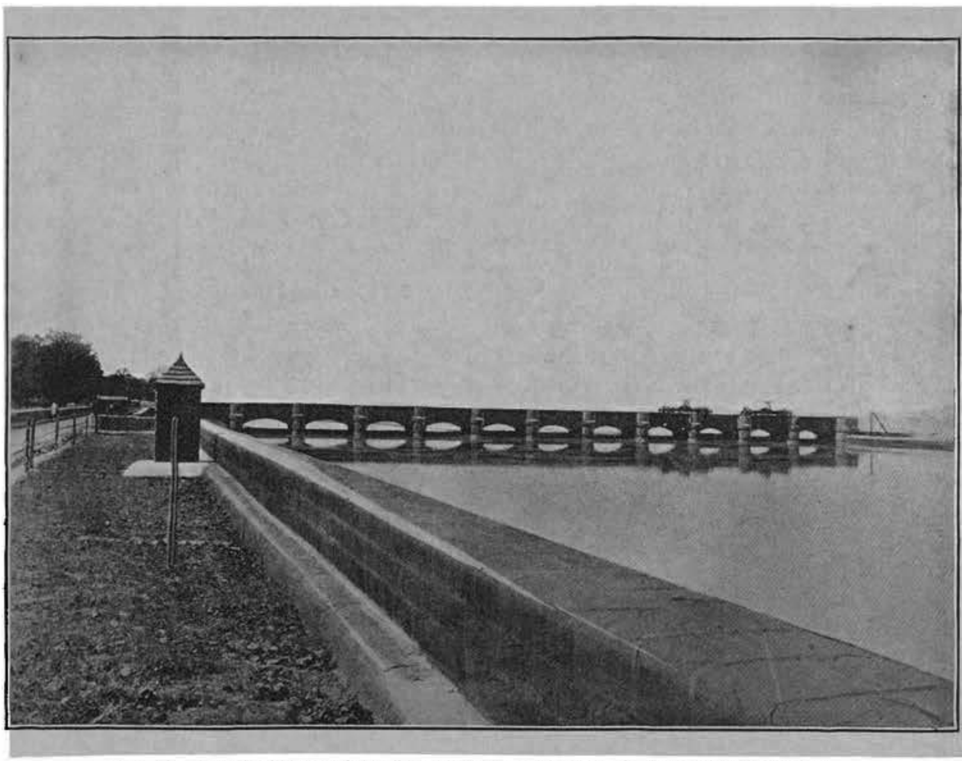


FIG. 5C.—Weir Sluices of the Rupar Weir at the Head of the Sirhind Canal.

flood level. The piers are $5\frac{1}{2}$ feet thick; the floor of the bridge is a solid mass; being founded on clay, or clay and boulders, the thickness can thus be reduced below what would be requisite on a sand foundation.

The superstructure is simple and effective, and consists of a platform carried by a tier of arches springing at flood level, and divided as usual into two parts by the slit for the gates and grooves.

The plans in Fig. 5 show the work as it stood before remodelling. The weir has since been provided with 6 feet deep collapsible shutters raising supply level from 865·5 to 872·0.

The canal head sill has also been raised from a height of 2 feet above weir sluice floor to 9 feet, and all the small jack arches and piers shown in elevation have been cleared out, leaving 21-foot openings. The remodelled head with the weir sluice in elevation is shown in Fig. 1, Chap. VIII. The present weir sluice gates now close to above the arch spring line, so that the sluice head can be entirely closed if necessary. This work is on a boulder bed with coefficient about 9. Fig. 5c is from a photograph of the work.

(**13**) A section and part elevation of the Chenab Weir Sluices at Khanki are given in Fig. 6. These are very similar to the Rupar Weir Sluices. The depth upheld by the gates is 1 foot less, being 13 feet against 14 feet at Rupar. Here, three roller gates are employed, raised by a travelling winch. The outside arch, which has so good an appearance, is here replaced by iron or steel girders which carry the outer rail of the traveller. This work being on a river bed whose sand is of class 2 with coefficient 15, we are able to compare it with the Narora weir sluices; the unit flood discharge of the Chenab River over the weir is, however, double that of Narora. This latter item influences the value of L, the width of the talus, as has previously been shown in par. 27, Chap. VI. Whereas the width of the floor apron is a function of the head, both are influenced by the coefficient representing the nature of the bed; consequently the formulas applicable to weirs will only require multiplying by some enlarging factor to render them suitable for the weir sluices.

If in formula (4), par. 27, Chap. VI., the multiplier of c be increased from 10 to 15, the results will be satisfactory, as the following comparative table will show.

$$(L)\text{, or width of talus for weir sluices} = 15c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}. \qquad (1)$$

The following is a comparison between the values of L of Narora and Khanki, worked out by this formula with the actual dimensions:—

Name.	C.	H^{b}.	q.	Equation.	Amount.	Actual Value of L.
Narora - -	15	10	75	$225 \times 1 \times 1$	225	216
Khanki - -	15	7	210	$225 \times \sqrt{\cdot 7} \times \sqrt{2 \cdot 9}$	319	300

With regard to (W) the length of the apron or floor beyond the head work, in both Narora and Khanki the length given is the same. This would tally with formula (1), Chap. VI.; if $4c$ be increased to $7c$, the formula will then become

$$W = 7c\sqrt{\frac{H^a}{13}}. \qquad (2)$$

This for Narora and Khanki works out to 105, H^a being 13 in either case.

(**14**) The section in Fig. 6 will now be examined as regards hydrostatic pressure.

First, as regards length of percolation. Here the value of l should be

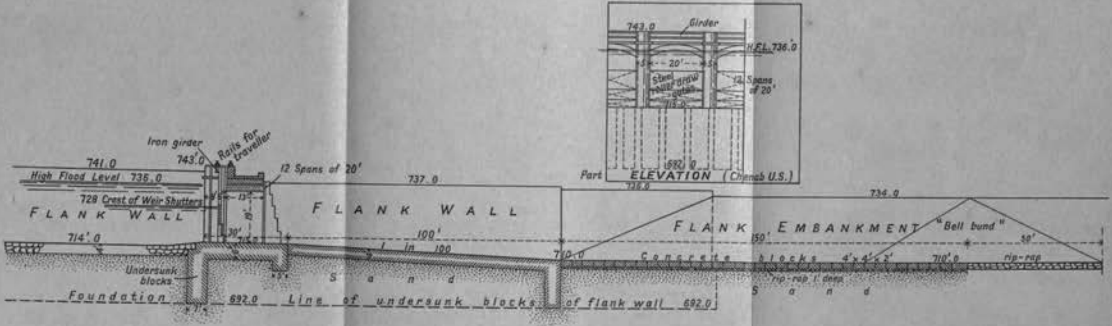


Fig. 6.—Chenab Canal Weir Sluices at Khanki.

[To face p. 216.

$15 \times 13 = 195$ the same as at Narora; it actually measures 227 feet. The thickness of the floor at the critical point just beyond the head work is only 4 feet; according to formula (3) par. 25, Chap. VI., t works out to 7·5 feet. The floor is, therefore, in high tension. This could be avoided by making it thicker at the root, tapering to 4 feet at the end, which is the scientific profile for all floors under diminishing hydrostatic pressure. Further, a rear extension of the floor up stream of the head work in the deep channel would be a great improvement, and should invariably be adopted in all weir sluices founded on sand. As we shall see, this precautionary measure has been carried out in the case of the Jhelum weir sluices (Fig. 7), where however it is not so much required.

One peculiarity of this work is the slope given to the apron, which drops 1 in 20 down to R.L. 710, which is 5 feet below the level of the weir talus as also that of the sill of the bridge floor. This would appear to be a good arrangement if it were worth the extra cost of putting in a large part of the apron and the whole of the talus below L.W.L., which is doubtful. A long flank division wall separates as usual the weir sluice way from the weir itself. This also extends up stream to beyond the canal intake, forming the "divide" wall mentioned as a desirable adjunct in par. 4 (*vide* Fig. 8b, Chap. VI.).

The concrete blocks forming the talus are a common feature in weir sluices of Punjab canals. They can easily be laid in water. They are 4 feet square and 2 thick, and consequently weigh 2 tons each. The concrete is not cement concrete, Portland cement being used very little in India on account of its cost; besides excellent hydraulic limestone is obtainable everywhere in Northern India in a nodular form, close to the surface, where it is quarried just as gravel would be.

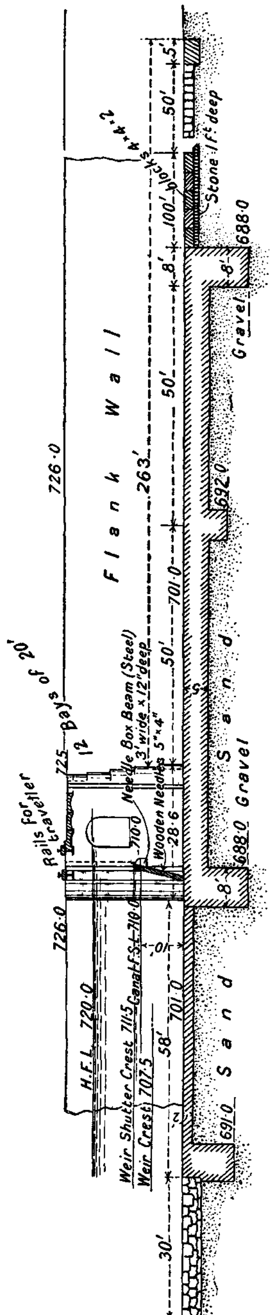
(15) The Jhelum Canal weir sluices at Rasūl are illustrated in Fig. 7. The permanent weir crest is at R.L. 707·5. The value of H^b is consequently 6·5 feet. According to formula (1), par. 13, the value of L works out as follows: $225 \times \sqrt{\cdot 65} \times \sqrt{2} = 256$ feet.

The actual value is 255 feet, so that the theoretical value agrees closely with the actual. W, the width of the masonry floor, according to formula (2), par. 13 $= 7c\sqrt{\frac{H^a}{13}} = 105 \times \sqrt{\frac{11 \cdot 5}{13}} = 99$ feet; it is actually 108 feet. Owing to the provision of a rear apron, the hydrostatic pressure at the critical point is reduced from 11·5 to 3·5 feet, for which the 5 foot thickness is more than sufficient.

This weir is provided with a long divide wall up stream, shown on the site plan, Fig. 9b, Chap. VI. This wall runs parallel to the axis line of the river and to the left flank retaining wall.

The down-stream outer flank wall is normal to the common axis of the weir and of the weir sluices, which are on one line.

The superstructure is a girder bridge with separate beams carrying the travelling winch which lifts the needle-box girder. The closure is effected not by draw gates, but by wooden needles, a cheap but slow and inefficient system which requires a large gang of men to operate. The needle system is also used on the Sidnai weir on the Sutlej River (Fig. 23, Chap. VI.), and has very unwisely been perpetuated in the new Madaya weir sluices in Burma.



Grooves are very properly provided in the piers for roller gates, though the latter are not adopted. The needle closure has this advantage, that it admits of instant opening of the sluice way. When the box girder is lifted, the needles are swept off and float in the current, hanging on by a chain attachment reeved through their heads until recovered. Where quick opening is required, the author considers that his pattern of balance gate, as shown in rough sketch in Fig. 20, Chap. XII., would answer admirably; it could be released instantaneously to a horizontal position, and could then either be hoisted clear of the water or else lowered into a recess below the floor or left open, the water flowing above or below it.

(16) We now come to a different type of weir sluice from any of the foregoing, which has only been adopted in the Province of Bengal. The governing principle of this design is to afford a free passage of flood water between and over piers, which are only of the necessary height for regulation purposes, and have no arched superstructure whatever. The water is held up by collapsible gates of large size, working between the piers.

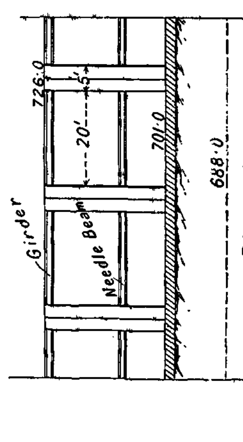
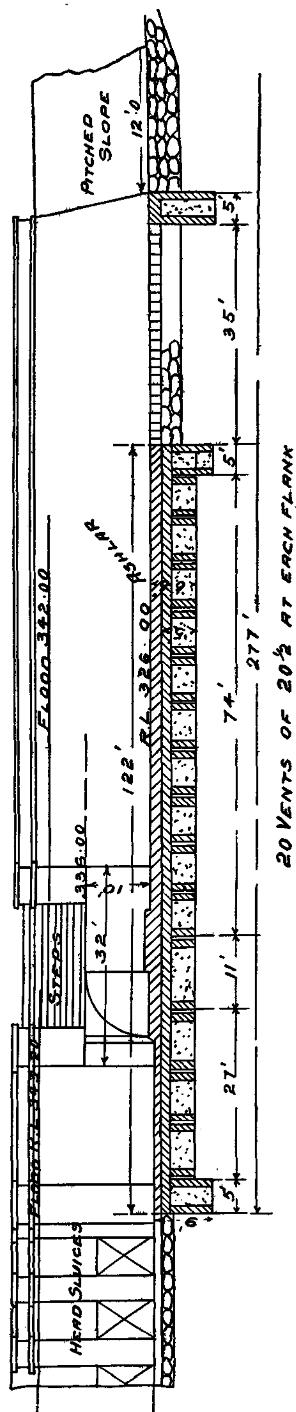
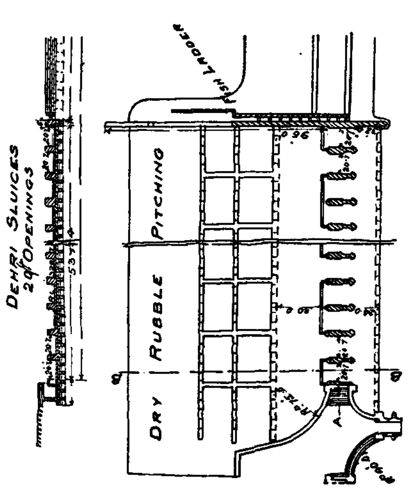


Fig. 7.—Jhelum Canal Weir Sluices at Rasil.

The designers of these open sluice ways were evidently of opinion that, having provided a free passage so different to that hitherto usually adopted, it was unnecessary to have the weir sluice floors at a much, if any lower level than that of the canal head regulator. This has proved to have been a fatal mistake, for as matters now stand, the effect of the weir sluices is to accentuate silt deposit in the canal, which is still further encouraged by the recessed position of the head regulator (*vide* Fig. 8a). It is absolutely necessary that the difference of floor level of these two adjacent works should be considerable. With the heavy drift sand found in these rivers a difference of 4 feet would be none too much. With the assistance of the lower gate of the head sluice, or a dwarf weir wall, this arrangement would admit of a deposit of 6 feet of sand in front of the regulator and behind the undersluices, which accumulation could be cleared off in a very short time by opening the undersluices, and the scoured channel would then be left ready for a further deposit, which will again be swept away by a similar process. By this means, and also by drawing the water over one or two of the gates of the canal head, and not underneath, heavy deposit can be successfully kept out of the canal. But on the other hand, if the sill levels of the two works are identical, there will be no spare depth available for deposit, and consequently it must be swept into the canal, where its periodical removal forms a heavy annual charge.

(17) Fig. 8 represents the Dehri weir sluices on the Sôn River. This work consists of twenty openings, $20\frac{1}{2}$ feet wide, divided by piers 5 feet thick and 33 feet long. These piers are only 10 feet high. The openings are closed by double collapsible wooden gates 20 feet long and 10 feet high. The second gate is for





(8a.)

FIGS. 8, 8a.—Dehri Weir Sluices, Sôn River, Bengal.

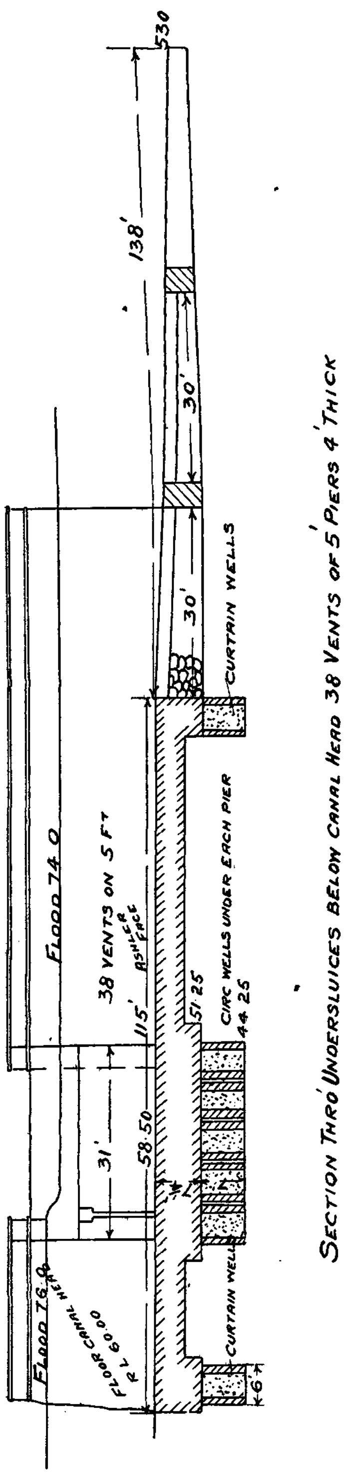


Fig. 9.—Jobra Weir Sluices, Mahanuddee River, Bengal.

temporary use, and is first raised by hand, which enables the real self-falling gate to be hauled upright by tackle, when the temporary first gate is lowered out of the way.

The falling gate is on the same principle as weir crest shutters, being pivoted at the end of inclined rods. On the gate being overtopped it doubles over and falls automatically, the shock being lessened by a hydraulic brake.

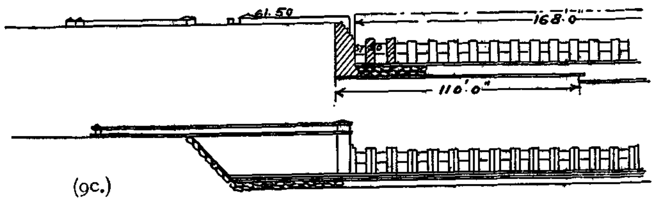
(**18**) As the flood level is only 7 feet above the top of the piers, it would have been a much simpler plan to have adopted an overbridge, as at Rupar, with roller draw gates, raised by a traveller. This weir is provided with sixteen central sluices of $20\frac{1}{2}$ feet width. The object in view was to prevent cross-currents and to train another channel straight on to the weir. These expensive works have, however, proved quite useless and have since been built up. The only practical method of preventing silt islands forming above the weir is to keep the weir crest low and make use of collapsible crest shutters.

On the Jamrao Weir, recently constructed, these latter have been made 4 feet high, and a portion of the weir adjoining the weir sluices is made 4 feet lower than the rest, having 8 feet high collapsible shutters. The old Betwa Weir is provided now with 6 feet, and the new with 8 feet shutters.

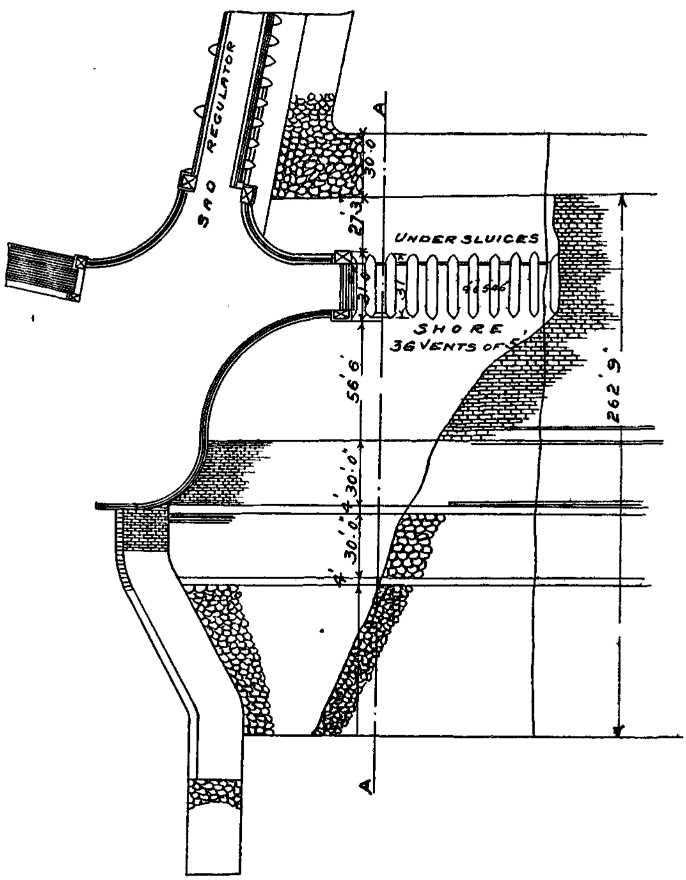
The block plan (Fig. 8a) shows the position of the canal head, which is recessed much too far behind the weir sluice abutment, forming a veritable silt trap. The alignment of the dwarf wall dividing the weir sluice and the weir is shown on the block plan. This arrangement of widening the escape channel towards the river bank and not on the side of the weir is correct.

As regards the remodelling of the adjacent canal head, see next chapter.

(19) The Jobra Weir central sluices on the Mahanadi River are constructed on the same principle as the ones just described, but they are not self-falling.



ELEVATION OF UNDERSLUICE



BLOCK PLAN.

FIGS. 9b, 9c.—Jobra Right Head Works.

These gates are no less than 45 feet long and 11 feet high. The weir sluices (Fig. 9) are formed of isolated piers $4\frac{1}{2}$ feet thick, and only 5 feet apart. They are apparently closed by wooden baulks or shallow draw shutters superimposed. There is no superstructure, so that a temporary plank platform has to be erected and the shutters or baulks removed before the flood season. This arrangement is decidedly primitive.

The masonry floors of both sets of sluices are generally $4\frac{1}{2}$ feet thick, with an additional thickness of 1 foot in the central and 3 feet in the shore sluices; beneath this again is a solid mass of undersunk blocks, 7 feet and 9 feet deep respectively.

The central sluice floor failed on one occasion altogether, and its repair cost a quarter of a million rupees, and has since been closed altogether.

It is considered that the down stream or fore-curtain wall should be much deeper, as an insurance against disaster in case of failure of the pitched talus. The sill of the head regulator is raised only 1 foot above that of the weir sluices, which is far too little, and has probably since been corrected by remodelling. See also Fig. 8b, Chap. VI., for plan of the whole head works.

(**20**) The Assiût and Zifta works, to which reference has more than once been already made, are the latest exponents of partial regulators.

The plans of this work are given in Fig. 10.

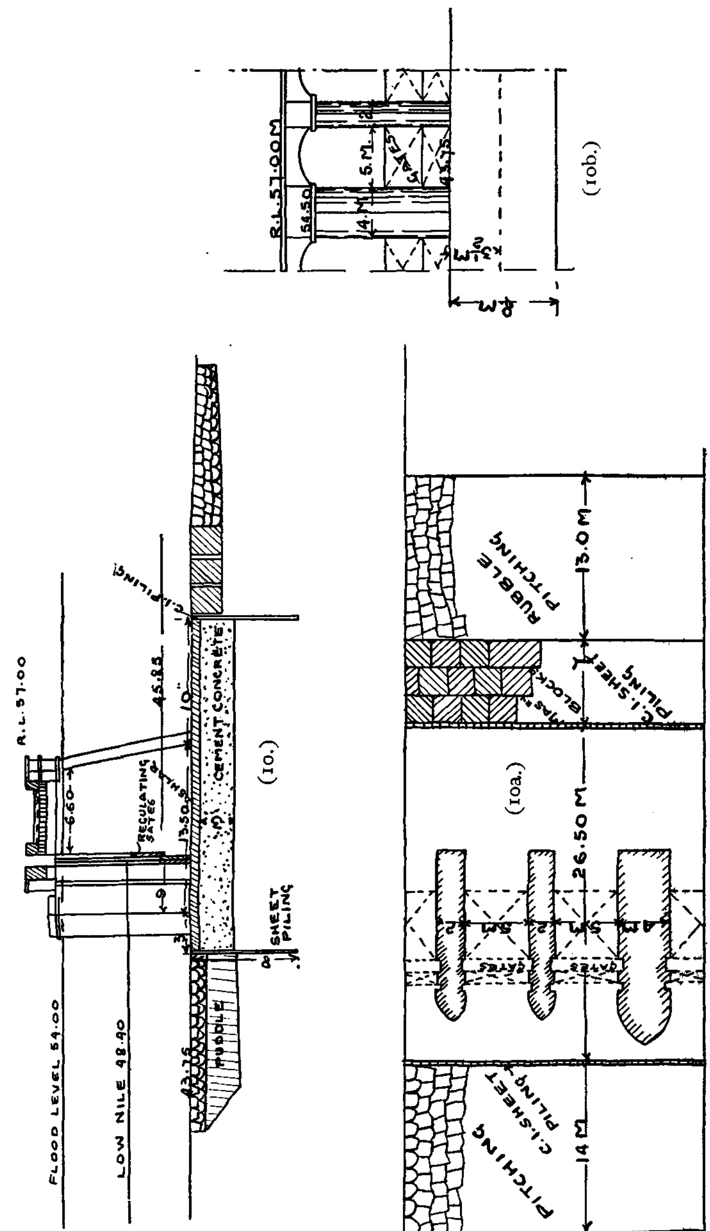
The Assiût Regulator was built across the Nile to regulate the supply in the Ibramiya Canal.

This work will never be entirely closed, and the outside difference of level that will ever be provided against is that between R.L. 48·40 and 45·85, viz., 2·55 metres (*vide* Fig. 10). This is the head, on the extent of which the design of the floor and also the superstructure depends. The value of l will then be (taking c as 18): $18 \times 2{\cdot}55 = 45{\cdot}90$ metres. It measures actually, 40 metres horizontally together with 10 vertically, total 50 metres. If the low supply were all held up, the head would be 4·7 metres, requiring 85 metres as length of percolation.

As this work is not a weir sluice, the length of floor is dependent more on the required value of l, than on any other consideration, scouring action being of minor importance.

(**21**) With regard to the superstructure, the resultant line of maximum pressure should pass through the outer boundary of the middle third of the base of the piers. This test is best arrived at by graphical process, and will be worked out for the Ibramiya Regulator, and so need not be repeated here. This determines the requisite base length of the piers.

The spans are 5 metres, *i.e.*, 16 feet, and the piers are 2 metres thick, a too high proportion of $\cdot 4S$; every third pier is an abutment pier. The general arrangement closely resembles that of the Rupar weir sluice head, but is even more simple. The working platform is carried on arches springing at high flood level. These arches are divided at the grooves, the open space being wider than usual. A travelling winch straddles this opening. The draw gates are of steel, fitted with anti-friction rollers, stanched by loose stanching rods as employed in Stoney's patent gates. The battered back to the piers is a decided improvement on the stepped back so common in old works in Upper India, although the rear face of the piers should be rounded, not square. For plans of gates, grooves, and travelling winch, *vide* "Min. Pro. Inst. C.E.," Vol. CLVIII.



Figs. 10, 10a, 10b.—Assiût Regulator, Nile River.

The foundations were originally designed as rows of sunk blocks under each pier, connected by a concrete floor. Owing to the successful introduction of cast-iron tongued and grooved piling, the use of blocks was abandoned as being slow and expensive, and rows of sheet piling each side of the floor 25 feet deep were substituted, the depth of the floor being increased to a uniform thickness of 10 feet. This depth is necessary to properly distribute the weight of the superstructure to reduce the pressure to 1 ton per square foot.

(**22**) The detached masonry blocks shown in Figs. 10 and 10a are a comparatively new form of construction originally employed in the Punjab. These blocks are made of rubble masonry in cement mortar, and are laid in position when set, on the wet sand or in water by a heavy travelling crane. When *in situ* the interstices are filled up with cement concrete : it thus forms a solid mass of heavy masonry. This arrangement is admirably suited for a continuation of the masonry floor of a regulator. The advantage of the system lies in the fact that the blocks can be deposited in water, the interstices likewise being filled up by depositing cement concrete in skips, or else the work can be partially pumped dry to facilitate this operation.

(**23**) The plans of the Ibramiya Canal Head are given in Fig. 11.

This work, although a canal head, is but a partial regulator, and is consequently in a similar case to a river regulator or a weir sluice head.

The general arrangement of this work is identical with that of the Assiût Barrage.

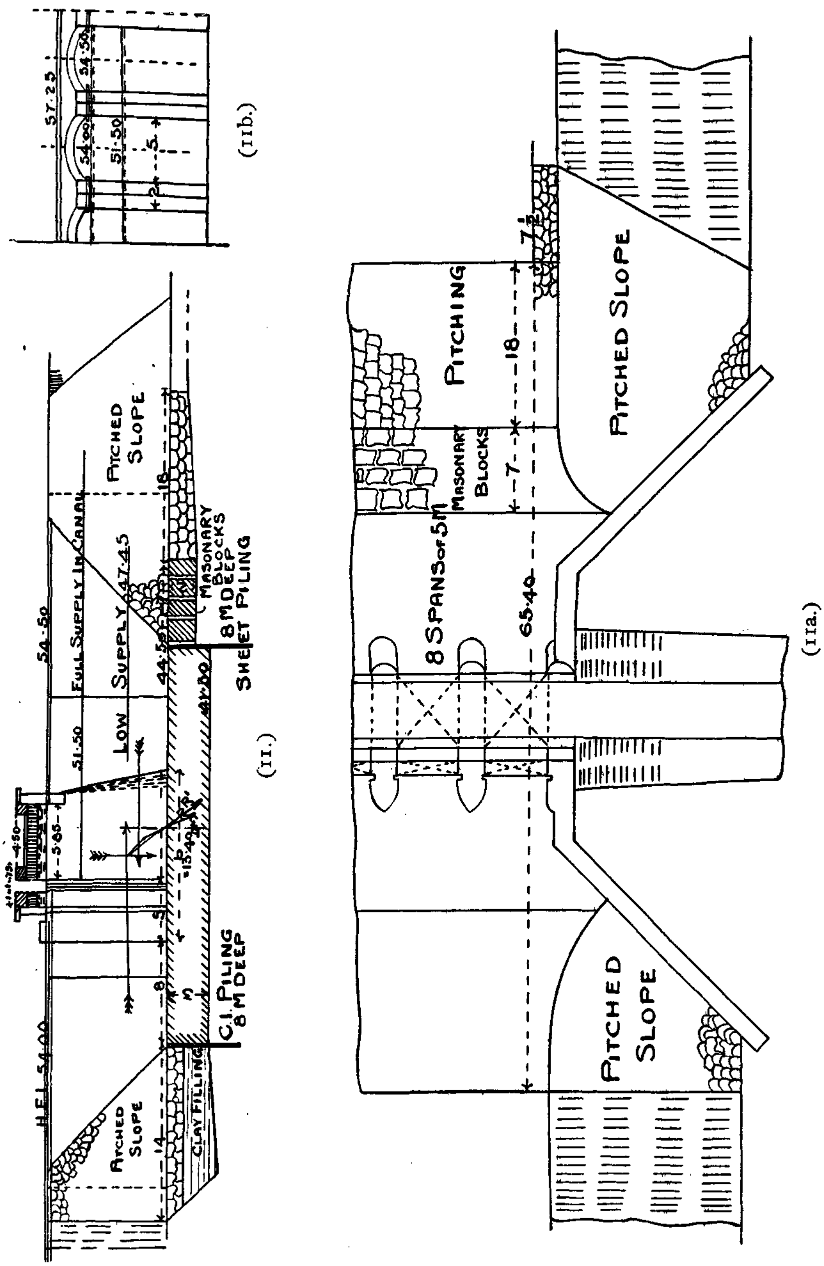
The Ibramiya Canal Head is, however, subject to a greater head of water than the latter work.

The maximum head of water to which the floor is subjected is the difference between low Nile level and the canal bed, *i.e.*, $47·75 - 44·50 = 3·25$ metres; so that this floor is under greater statical pressure than the Nile regulator. The superstructure is also subjected to a much greater pressure than the Nile regulator, owing to the great depth of water on both sides of the gates, which pressure is a maximum in Nile flood with full supply in the canal.

The transverse sections of the piers in either of the regulators are much the same, the base being 13·50 metres in each case ; the top width is somewhat less in the Ibramiya work. The spans and thickness of the piers are the same in both cases, but the Ibramiya Canal Head has no abutment piers. The masonry floor is made wider in the canal head, being 31·40 metres against 26·50, a difference of 5 metres. This is owing to the greater head to which it is subjected. These two are models of good design.

(**24**) The value of l in this case should be $3·25 \times 18 = 58$ metres. The horizontal component is 45 metres ; add the vertical of 10 metres, the sum is 55 metres. If the floor thicknesses at each end be added in it would increase the amount to 61 metres, so that in this instance the actual closely corresponds with the calculated, forming a corroboration of the correctness of the value assigned to c, viz., 18. In these calculations the lower line of sheet piling has not been included, as it is doubtful if it was ever taken into account.

(25) The previous examples are all of works founded on sand. At the heads of rivers, before the stream has left the hilly, rocky country near its

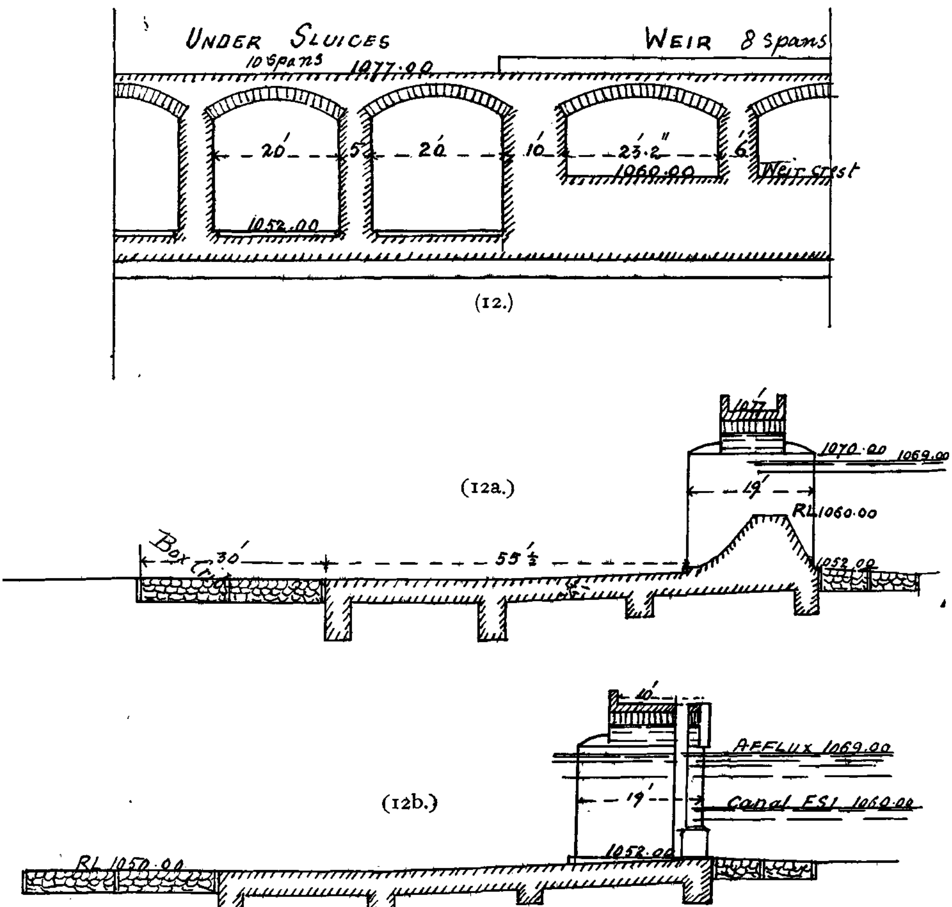


FIGS. 11, 11a, 11b.—Ibramiya Canal Head, Egypt.

source and debouches into the plain, the river bed is generally composed of rock or boulders, the latter being more or less mixed with sand or shingle. It will be as well to give an example of a head work thus circumstanced.

Fig. 12 are reproductions of the record plans of the Western Jumna canal weir and weir sluices.

Of these, Fig. 12 is a longitudinal section through part of the work. From this it will be seen that the high level bridge over the weir sluices is continued right along across the river as a road bridge, the weir being built inside the spans.



FIGS. 12, 12a, 12b.—Western Jumna Canal Weir and Weir Sluices.

This cross communication is a necessary work, as canals take off on either flank of the river. The spans of the weir portion are 23 feet wide with 6 feet wide piers, whereas in the weir sluices they are reduced to 20 feet spans with 5 feet piers.

The weir sluices are partial regulators, and closure is effected by double iron draw gates without rollers, which bear against the sides of recesses formed in the piers.

The superstructure is provided with supplementary arches which are only

18 inches wide outside of the recess or grooves of the piers. The object of these external arches is the usual one of affording a narrow platform up stream of the grooves, which facilitates the manipulation of the draw gates. The afflux level is 17 feet above the floor of the weir sluices, while full supply level in the canal corresponds to the crest of the weir, and is 8 feet above same level. The actual statical head on the weir and weir sluice floor is 10 feet.

(26) The sections of the floor of the sluices and of the weir proper are identical in length as well as thickness. In the previous examples the sluice floor is made much longer than that of the weir, and necessarily so, owing to hydrodynamical considerations. In this case, however, the boulder bed is capable of infinitely greater resistance to erosion than one of pure sand; consequently the talus can be curtailed; at the same time, owing to the great abundance of building material available at the actual site, the cost of a boulder masonry floor will be comparatively so inexpensive that the masonry apron can be carried wider than would otherwise be advisable.

(27) A work built on boulder formation cannot be considered free from hydrostatic pressure below the floor. The coefficient suitable for boulder beds varies from 5 to 9. In this case the actual value of l is about 90 feet; this would make the coefficient 9, which has been adopted (par. 9, Chap. VI.). The floor is too thin for the hydrostatic pressure. The value of $(H - h)$ at its commencement is 6·5 feet; to meet this, the floor, half of which lies below the L.W.L., has a mean specific gravity of 1·75, and is $3\frac{1}{2}$ feet thick; tp is then 6·1, but it should be $\frac{4}{3} \times \frac{6·5}{1·75} = 5$ feet thick.

What the work requires is an impervious rear apron to reduce the hydrostatic pressure at this point, and a trapezoidal wedge-shaped floor, additional thickness being provided underneath the heavy piers in order to distribute the pressure.

The canal head or intake connected with the weir sluices is illustrated in the next chapter. According to formula (1), Chap. VI., the least length of apron of the weir should be $4c\sqrt{\frac{H^a}{13}}$. H^a in this case being 8 feet, W will equal 25 feet. The extra 30 feet provided is, however, necessary for the purpose of affording the requisite creep of percolation, there being no rear apron. With regard to the weir sluices, by formula (2), par. 13, of this chapter W, should have a minimum value of $7c\sqrt{\frac{10}{13}} = 7 \times 9 \times ·88 = 55$ feet, which it actually measures.

(28) In the United States a large number of canals have been constructed in recent years, the head works of which display considerable ingenuity in general design; some are diversion weirs, as dealt with in the last chapter, of low height, but the majority are constructed over the rocky beds of

mountain torrents of considerable height, intended to form storage reservoirs similar in lines to the Betwa, Periyar, and Assuan works.

One example of low weir across a wide sandy river has very similar conditions to Indian works. This is the Yuma irrigation project about to be undertaken, a description of which is given in "Irrigation Engineering," an excellent American work, from which the plans have been obtained.

Fig. 13 is a longitudinal section of the Colorado River. From this it will be seen that the general level of the river bed is, roughly, about R.L. 143·0, the deepest part of the channel being about R.L. 131·00. The weir crest has been fixed at 10 feet above L.W.L., *i.e.*, at 151·00. The design of the weir is a close copy of the Madras anicut type; the section has been given

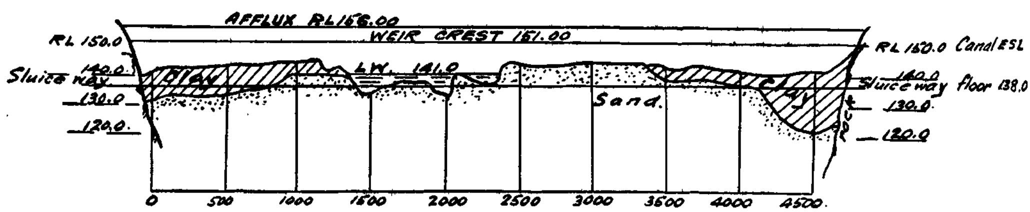


FIG. 13.—Section of Colorado River.

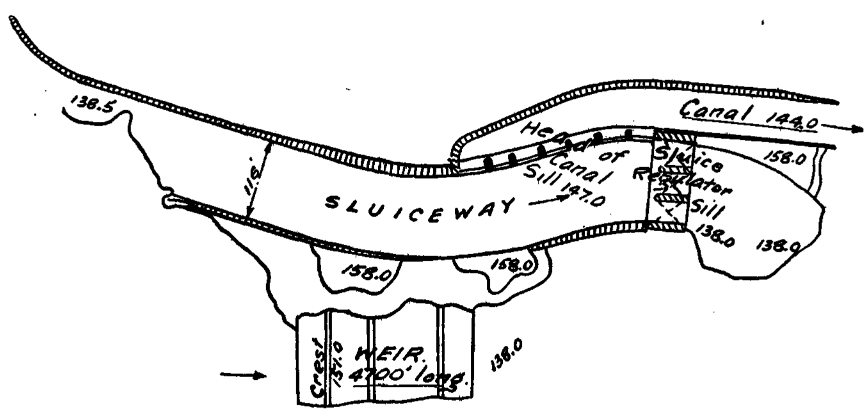


FIG. 13a.—Laguna Weir and Left Head Works.

in Fig. 29, Chap. VI. The level of the horizontal talus where the slope ends is R.L. 138·00, *i.e.*, 3 feet below L.W. The sluice way floors on each side of the weir are at the same level, *i.e.*, 13 feet below crest. This level and that of the talus are deemed to be unnecessarily depressed.

The arrangement of the rest of the head works, viz., the weir sluices and canal head, is so remarkable as to be deserving of considerable attention. The weir ends at both flanks on the existing rocky banks of the river. Beyond these, and separated from the weir, sluice way channels are cut in the solid rock on either flank. These being quite independent of the weir, it is evidently a convenience to fix the position of the weir sluice head *below* the weir so that the junction of the scouring channel with the river bed will take place well clear of the latter work.

Another peculiar arrangement is the alignment of the two canals, which, instead of being, as is almost invariably the case in Indian canals, parallel to the axis of the weir, until the river banks are well cleared, are aligned parallel to the river and at right angles to the weir axis. This arrangement is necessitated by the level of the rocky ground in the vicinity of the head works. The canal head vents, therefore, which are situated just above the weir sluices in the side of the sluice way channel, discharge through the right flank of the canal, *i.e.*, at right angles to its direction.

The sluice way is kept at a very low relative level, viz., R.L. 138, in order to form a silt settling basin, to be periodically scoured out by raising the regulator gates. The sill level of the canal head is at R.L. 147·00; thus a depth of no less than 9 feet of sand can accumulate in front of the canal head, and even more, if double draw gates were adopted.

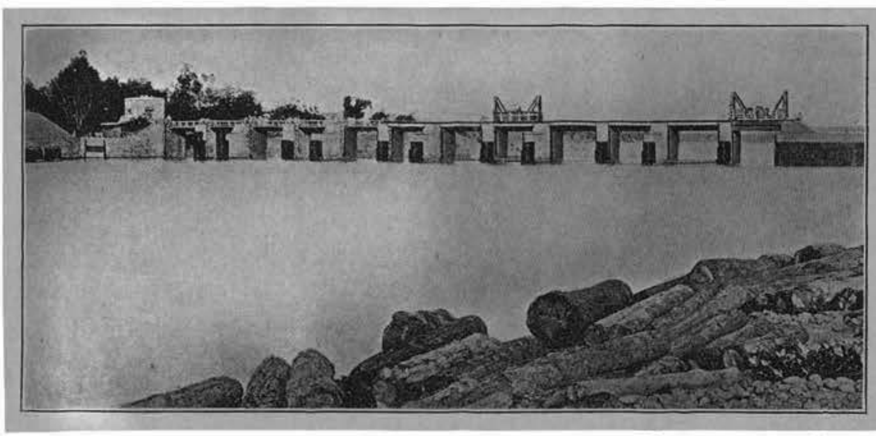


FIG. 14.—Weir Sluices, Madhopur, Bari Doab Canal, Up stream. 20 feet spans. Double gates.

The afflux level above the weir is at R.L. 156·00, and, as we have seen, the weir sluice sill is at 138, consequently 18 feet of water can be held up and released at once. These immense scouring sluices are evidently intended to do more than to keep the canal head clear of deposit, viz., to prevent sand accumulation at the rear of the weir and thus provide a reservoir of water to be drawn on.

In a wide river like this, however, no scouring sluices, however powerful will have any appreciable effect in keeping down the silt deposit in rear of the weir below crest level. The silting up of the whole river channel to the weir crest level, excepting a narrow strip at each flank, is inevitable, so that the success of the arrangement is doubtful.

(29) The Madhopur weir sluices of the Bari Doab Canal, a print from a photograph of which is given as Fig. 14, form an instructive example of remodelling an old work on modern lines. The sluices originally consisted of a series of 5-foot vents. These have all been demolished, and, it is believed, entirely new piers built 5 feet thick, 20 feet apart, the whole roofed in by

girders and jack arches, similar to the Jhelum weir sluices (Fig. 7). The pier noses and the abutments are faced with timber as a protection against damage by boulders which, during floods, are swept along by the strong current. The 20-foot vents are supplied with double roller gates similar to those in the Chenab and Rupar weir sluices.

The view shows the two large travelling winches used in lifting the gates, standing on the tram line.

(**30**) The thickness of the floor of a weir sluice head founded on sand is subject to kinetic as well as to hydrostatic considerations, and with regard to the latter, formula (3) of Chap. VI., viz., $t = \frac{4}{3}\,\frac{H - h}{\rho - 1}$, will apply, whereas for the former, a heavy floor being a desideratum, the following empirical rule will be found in accordance with practice, viz. :—

$$t = \sqrt{\frac{3H}{2}} \qquad (3)$$

H being the statical head.

(**31**) The term "Undersluice," familiar in Northern India, is in reality a misnomer, as in modern weir designs, "undersluices," literally as such, do not exist. Weir sluices, or weir scouring sluices, is a more correct nomenclature.

In the United States, the term *undersluice* applies only to weir or dam *body* sluices—as those in the Assuan or Bhatgarh dams.

CHAPTER VIII

CANAL HEAD REGULATORS

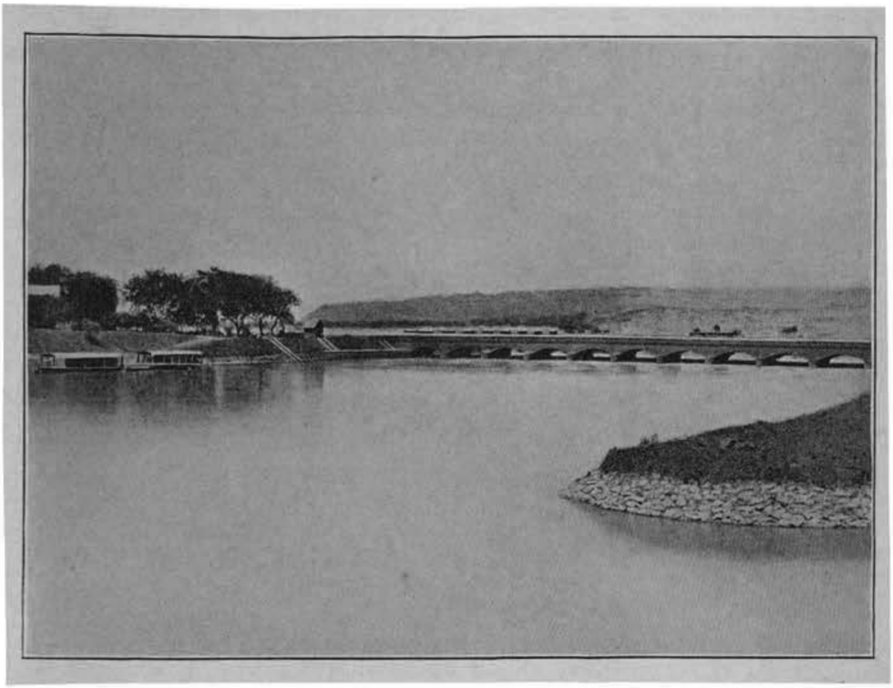


FIG. 1a.—Head Regulator of the Sirhind Canal at Rupar.

(1) THE function of canal head regulators or intakes is to adjust the admission of water into the canal from the river of supply, according to the requirements of the canal.

In almost all cases this regulation is entire, not partial: that is to say, on the one hand, the full supply level in the canal cannot be allowed to be exceeded, and on the other hand, the head work should be capable of completely shutting off all supply, even during the highest floods in the river. From the above it will be evident that the pressure of the statical head of water to be provided against is much higher in this class of work than in weir sluices, while the dynamical forces which are such potent factors in the latter are practically absent in head regulators.

The admissible mean velocity of water in a canal rarely exceeds 3 cubic feet per second, and the waterway of the head is generally made ample in width in order not to cause increased velocity of entry, so that the only occasion when the velocity of entry exceeds the normal is when it passes over the lower of the pair of draw gates, as in a submerged fall, or over a raised sill.

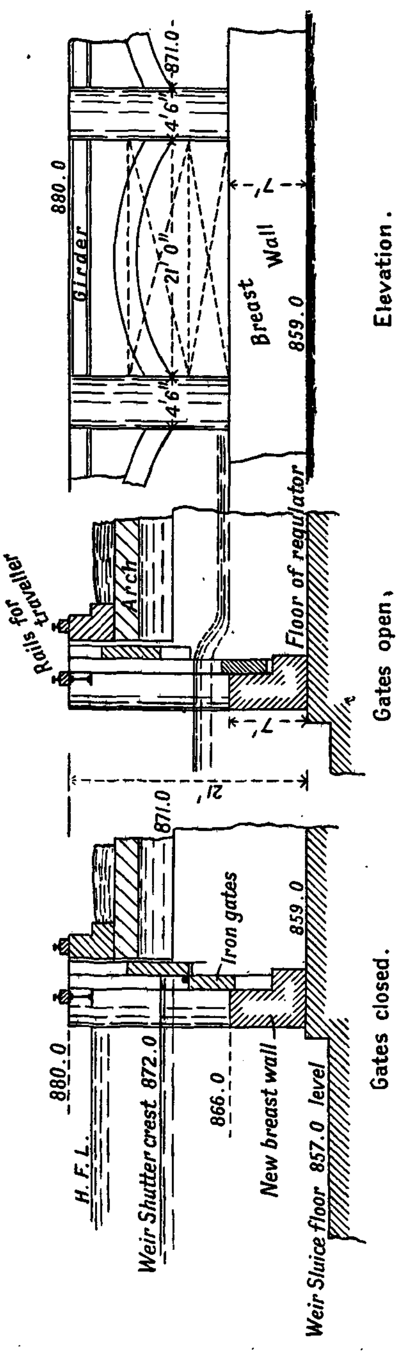


FIG. 1.—Sirhind Canal Head, Rupar, as remodelled.

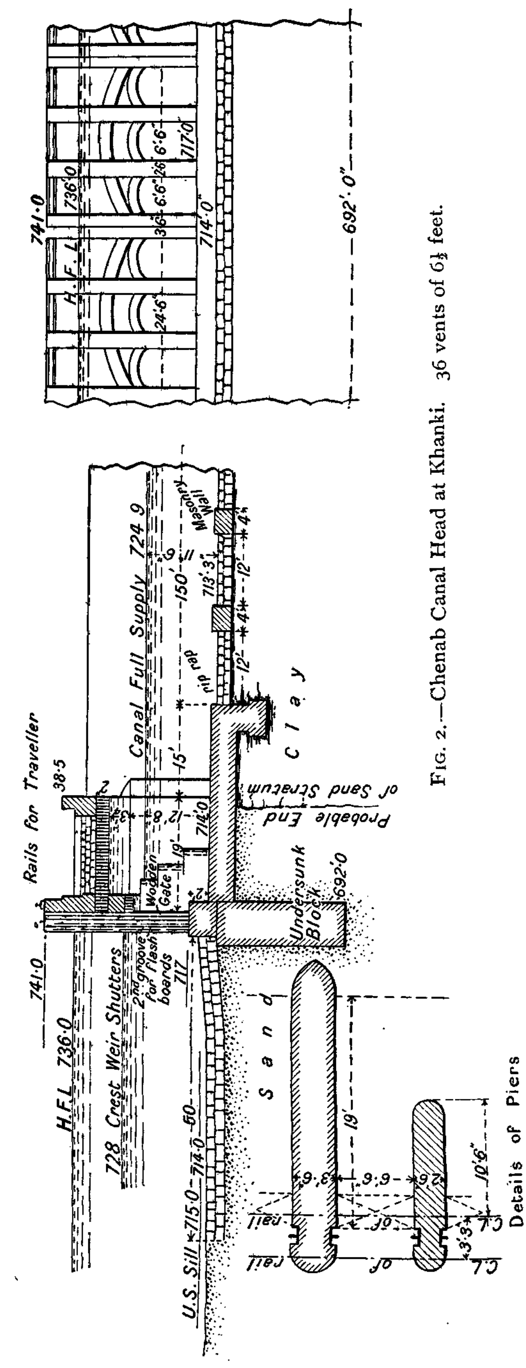


FIG. 2.—Chenab Canal Head at Khanki. 36 vents of 6½ feet.

(2) The following principles regulate the design of canal heads:—

First: The width of the openings should be as large as is consistent with easy manipulation of the draw gates, which are liable to be subjected to much greater water pressure than can be the case with limited regulation.

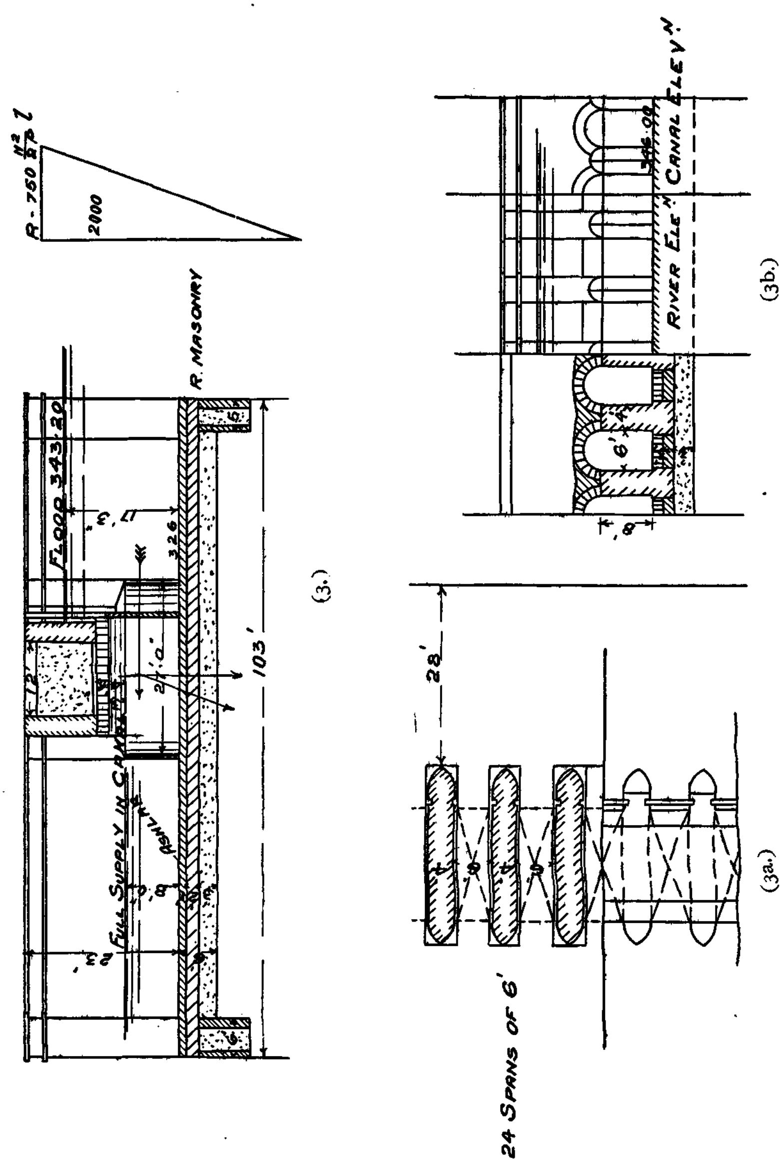
Secondly: In order to prevent the entrance of sand into the canal, which has caused much trouble and immense annual expense in clearance, the sill of the intake must be raised several feet above that of the adjoining weir sluices. This is arranged for by constructing a narrow breast wall across the entrance of the vents, on top of which the lower gate rests, or else, by a still better arrangement, the lower gate can be passed down behind the breast wall. The upper gate also can be lowered behind the under gate; by this device, only surface water is tapped from the river. The deep channel in front of the canal head is allowed to silt-up, and is then periodically scoured out by opening the gates of the weir sluices. In all the older Indian works these conditions were not known or observed; consequently many canal heads have had to be remodelled, wider spans being introduced, and the sill together with the weir shutters in some cases had to be raised.

(3) Perhaps the most instructive example is that of the Sirhind Canal Head at Rupar (Fig. 1), to which reference has already been made in par. 12, Chap. VII. The old intake consisted of thirty-nine spans of 5 feet, and are shown in elevation in Fig. 5, Chap. VII. These fortunately consisted of short piers spanned by jack arches built within the large bridge spans of 21 feet; the bridge piers were 4½ feet thick, consequently it was an easy matter to make a clean sweep of all the jack piers. The bridge piers were then fitted with grooves, an overhead outside girder inserted for the outer rail of a travelling winch, and the transformation was complete. The floor of the old work was 2 feet above the sill of the weir sluices. On this a breast wall no less than 7 feet high was built. The deep channel in front was thus increased in depth from 2 to 9 feet.

The new gates were arranged to slide one behind the other, and were mounted on anti-friction rollers to enable them to be lowered against a head of water by their own weight. A photograph of this work is given as Fig 1a.

(4) The Chenab Canal Head at Khanki, in Fig. 2, was in much the same plight, but in this case the thickness of the bridge piers, only 3½ feet, was insufficient to stand the whole water pressure of a 25 feet bay, 22 feet deep, consequently the old arrangement had to be put up with. From inspection of the section and plan it will be noticed that the outside girder to carry one of the rails for the traveller was an afterthought; so is, probably, the provision of double grooves. The gates are single, the outer groove being used to deposit sleepers or flashboards which answer the purpose of a raised sill; the gate, however, can only be raised. The arrangement is very inferior to that of the last example. The slight projection given to the pier noses has this advantage that it reduces the silt "pocket" formed by them just in front of the gates. The jack pier noses are carried right up to the parapet of the bridge, which is a somewhat unusual feature.

Canal intakes are generally built more or less on a solid clay foundation, so that although subjected to a much greater hydrostatic pressure than is the case with weir sluice floors, the pressure generally stops short of the beginning of the floor. This is evidently the case here. The head is 22 feet,



FIGS. 3, 3a, 3b.—Sôn Canal Head at Dehri.

and with $c = 15$, l will be $15 \times 22 = 330$ feet. The actual length of percolation provided is but 94 feet, consequently the head work must rest partly on a clay substratum. In such cases the hydraulic gradient will be horizontal, and the pressure area a rectangle, as there is no outlet for percolation.

(5) The Sôn Canal Head at Dehri (Fig. 3) is another valuable object-lesson. It is situated on a river which carries an immense quantity of heavy sand in suspension.

The head work was located at the bottom of a deep quadrant bay, quite a long distance from the undersluices, and the pier noses are recessed 30 feet behind the weir sluice land abutment. This is shown on the block plan, Fig. 8b, Chap. VII.

The sill of the canal head is flush with that of the weir sluice. This arrangement, as might well be imagined, was a complete success if regarded from the point of view of a silt trap. Great masses of sand were yearly washed into the canals at both sides of the weir, necessitating the maintenance of a fleet of steam dredgers in the attempt to keep the canal open at all. This nuisance continued for some five-and-twenty years, costing untold sums of money, until at last the remedy was discovered, viz., a raised sill to the intake, and the closure of the gates of the weir sluice as much as possible, so as to allow of still water in the deep channel, whereby deposit outside the canal head is encouraged.

(6) On account of the immense length of the weir ($2\frac{1}{2}$ miles), any raising of the weir shutters must have been deemed impracticable on the score of expense, and the massive canal head did not lend itself to any possible alteration short of complete demolition; consequently the only alternative was to build a supplementary head, to be used only at times of low supply in the river in conjunction with the old head, a temporary raised sill, $4\frac{1}{2}$ feet high, being formed in the latter by keeping planks in the grooves below the gates, which are single and of antiquated pattern.

These arrangements have resulted in complete success, and will prove of immense value financially.

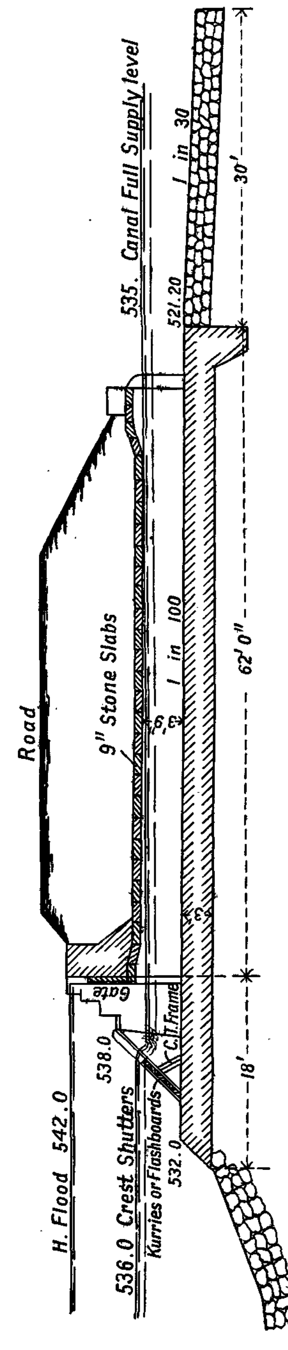


FIG. 4.—Surface Supply Inlet, Sôn Canal.

Fig. 4 is the section of the supplementary surface supply inlet. It consists of twenty small openings roofed by slabs, outside which are cast-iron inclined frames, on the sloping sides of which "kurries" or flashboards are placed at any height which may be desired; and at time of floods the vents are closed by drop gates, and the regulating planks removed. The raised sills or crests of the two works are kept at the same level.

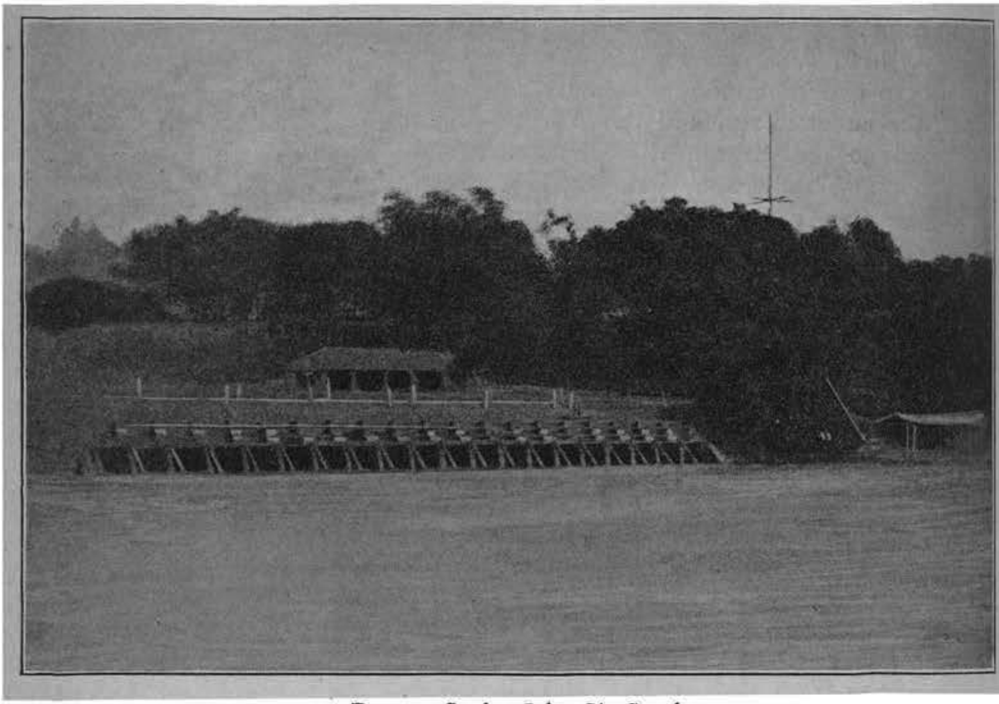
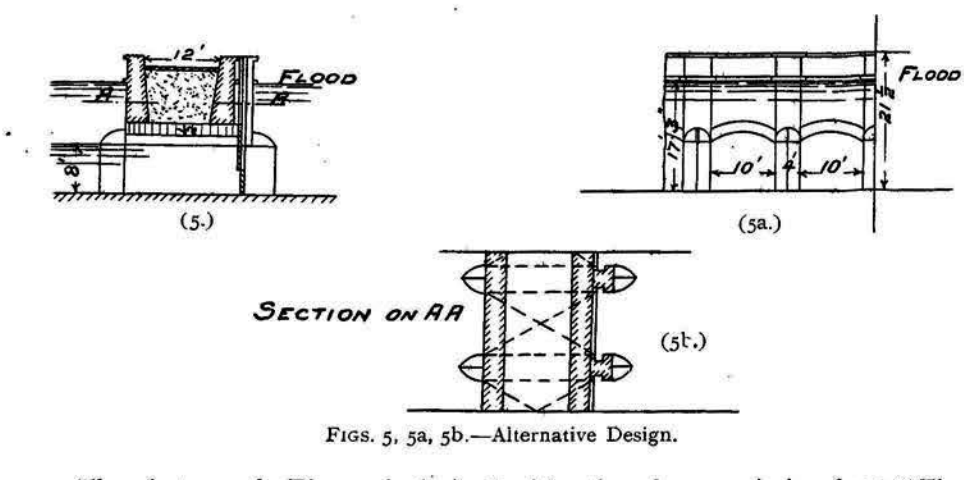


FIG. 4a.—Surface Inlet, Sôn Canal.



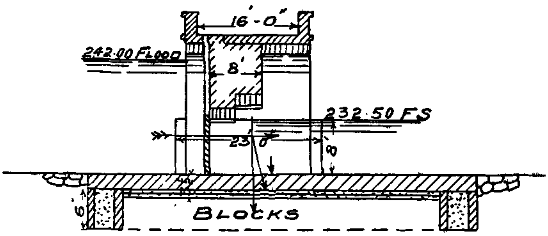
FIGS. 5, 5a, 5b.—Alternative Design.

The photograph, Fig. 4a, is derived with others by permission from "The Irrigation Works of India."

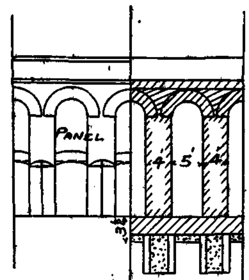
(7) The regulator itself is of the so-called U pattern, having a depressed arch springing at canal full supply level, over which are two spandrel walls, the space between being filled up with rubble stone or concrete.

This work is also partly, if not wholly, on a clay foundation ; if not, it would blow up from the hydrostatic pressure.

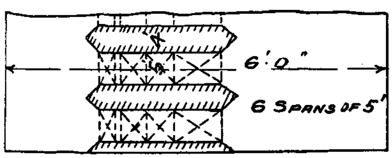
The piers in this work are much too thick, being nearly ·7S. Such a structure, if remodelled on the same lines, would consist of 10 or 12-foot



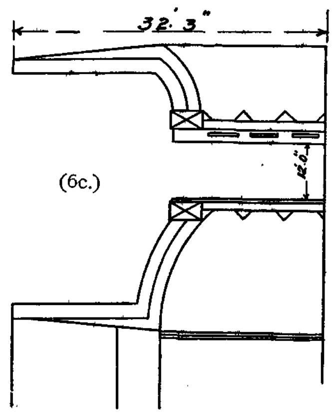
(6.)



(6a.)

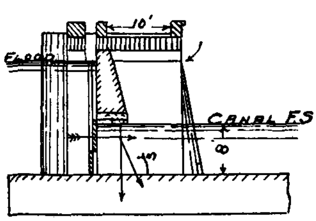


(6b.)

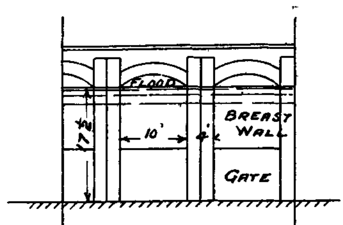


(6c.)

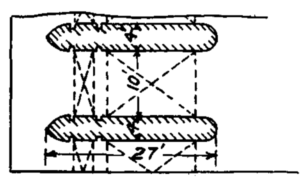
FIGS. 6, 6a, 6b, 6c.—Saran Canal Head.



(7.)



(7a.)



(7b.)

FIGS. 7, 7a, 7b.

spans, with double gates in grooves, *i.e.*, if a permanently raised sill was impracticable. An arrangement like the Sirhind Head (Fig. 1), would of course be preferable. The former is shown in Fig. 5. The gates would be raised by a travelling winch, running partly on the roadway, and fitted with a projecting arm furnished with another wheel which runs on the rail fixed on the parapet. Pulleys are attached to the extremity of the arms over which the chains passing in the grooves are carried. Such an arrangement is in use on other canal heads, but is not to be commended.

(8) In Fig. 6 is the Saran Canal Head in Bengal. It is put forward as an example of a style different to the U section. In this, two tiers of arches are used, the lower one carrying a breast wall. The work is absurdly heavy. The head is only 18 feet, and the piers are ·85 thick. This work has a split archway, but the outer arch only carries a parapet. As we have seen, the split arch is a common feature in weir sluices and canal heads, being utilised to carry the outer rail of the traveller. It is a good arrangement.

(9) Figs. 7 and 8 are alternative sections showing a different disposition of the breast wall. In Fig. 8 the projection of the curved pier noses is reduced to a minimum, the upper part being corbelled out square, the object being to reduce the silt pocket. These alternative designs are all on the same lines as the originals. In modern works provided with a raised sill, the breast wall would be raised higher than shown, to allow headway for the overfall.

To afford comparison of relative cost, a statement of the quantities in all five sections, Figs. 3 and 5 to 8, is given below. These quantities are the superficial area per foot run :—

Design.	Quantities of Masonry in One Span.	Divide by Length.	Quotient.	Reduced Number of Comparison.
Dehri, Fig. 3	2,161 cubic feet	10 feet	216	110
Revised, „ 5	2,460 „ „	14 „	176	90
Saran, „ 6	2,682 „ „	9 „	300	150
„ 7	2,772 „ „	14 „	198	100
„ 8	2,750 „ „	14 „	196	100
„ 8b	2,600 „ „	14 „	185	95

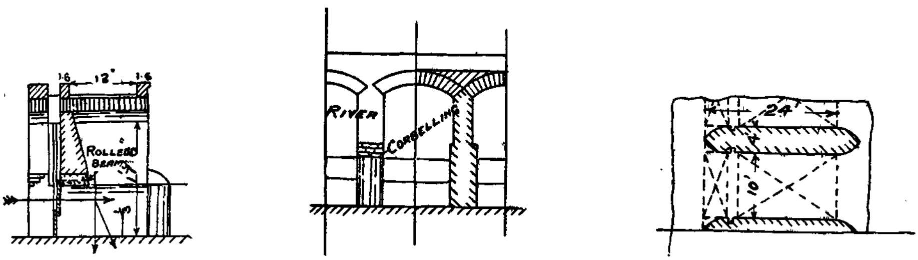
(10) The Narora Head Work (Fig. 9) was the first work supplied with gates fitted with rollers, or rather wheels, the axles of which work in bearings fixed to the gate. The stanching of these gates is effected by cutting the table of the vertical groove, on which the wheels run, to a certain batter at the lower end, so that when the gate reaches the sill its sides and top are in close contact with the iron frame.

The adoption of the usual round or flat stanching rods would be a simpler and equally effective arrangement. The gates are raised by a travelling winch, outside arches being provided for carrying one of the traveller rails, the other resting on the parapet of the bridge. These gates are double.

The design of this work is excellent, excepting the width of the openings, which, under so moderate a head of water, could well have been enlarged to 10 feet spans, with piers 3 feet thick. The sill is 3 feet above weir sluice floor.

(11) An interesting example of a canal head of exceptional design is given in Fig. 10 of the Betwa Canal Head.

This work takes out from a reservoir formed by a high weir across the River Betwa in Upper India, and is subjected to a head of no less than 38 feet of water above the sluice floor. As anti-friction roller gates were

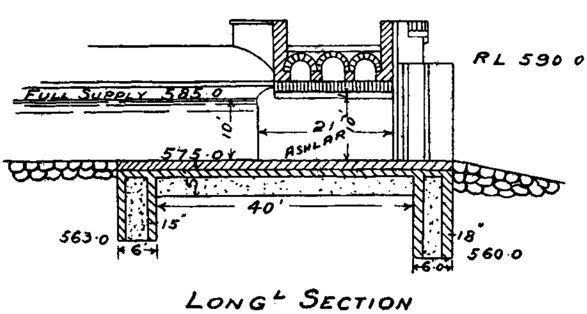


(8.) (8a.) (8b.) (8c.)

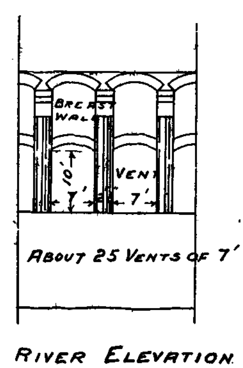
Figs. 8, 8a, 8b, 8c.

hardly known at the time of its construction, special devices had to be employed to modify the pressure of water against the sluice gates, so as to enable them to be manipulated under such severe conditions. This was effected by building a panel screen wall in front of the face of the head work, which was perforated by two sets of vents at different levels, closed by draw gates, one in each bay of the regulator.

These panel walls are arched in plan, somewhat similar to those in the



(9.)



(9a.)

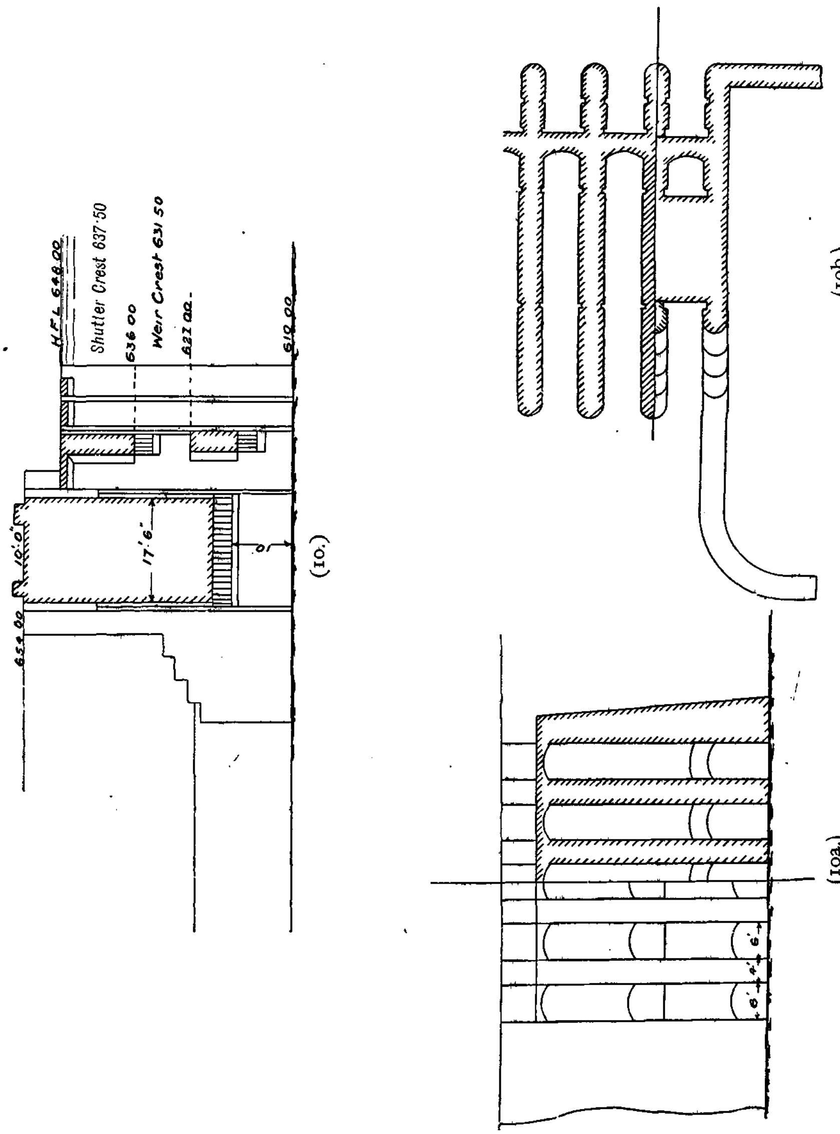
Figs. 9, 9a.—Narora Head Regulator, L. G. Canal.

Assuan Dam, and are carried by the vertical arches over the vents. The upper portion is recessed somewhat in rear of the face of the lower to afford a ledge for the upper drop gate to rest on.

When the reservoir is sufficiently full, the lower outer vents are closed, and water for the canal is taken through the upper outer vents, which falls into the space between the outer screen wall and the face of the regulator.

The arrangement is clearly illustrated in Figs. 10, 10a, and 10b.

This device, though ingenious, is expensive, and with the universal adoption of anti-friction rollers to draw gates, or Stoney's apparatus as employed in the Assuan Dam sluices, which reduce the power required to



FIGS. 10, 10a, 10b.—Betwa Canal Head Regulator.

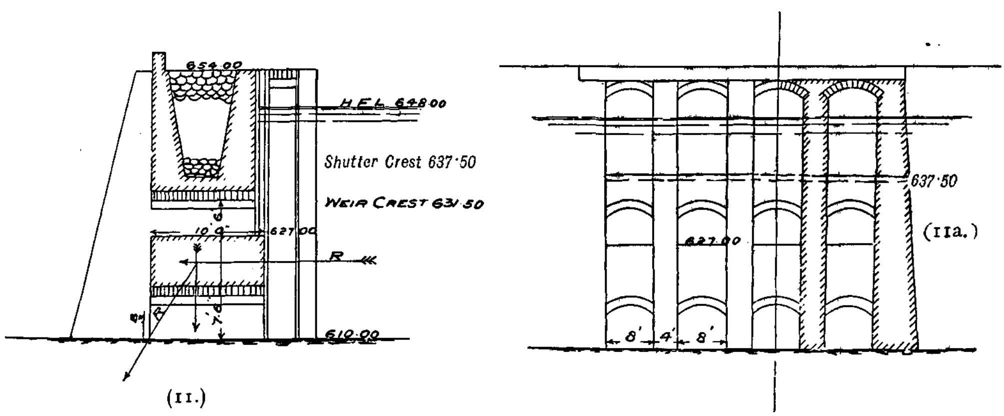
lift a gate by $\frac{1}{80}$ to $\frac{1}{250}$, is not likely to be repeated. The water level in the reservoir has recently been raised 6 feet by crest shutters to R.L. 637·50.

(12) As, however, it is often more convenient, in order to avoid silt depositing in the canal, to tap the river water from a high level, there is no

reason why, even under like conditions, the head work should not be pierced by a double set of sluice vents at different levels.

This arrangement would be just as effective as the last, and would certainly be much cheaper. It is exemplified in Figs. 11 and 11a. The immense solid mass in Fig. 10 is unnecessary for purposes of stability, and can be lightened with advantage by the upper sluice openings, and further by adopting the U section above them, the central space being filled with loose stone or rubbish.

In this design, when the upper vents are open and the lower closed, the water will fall directly into the canal, down stream of the work. There is no disadvantage in this, particularly at this site, where the canal is in rock cutting, a small extension of the floor and wings being all that can be necessary. It is suggested that this arrangement of double sluices, which, as far as is known, has hitherto not been tried, is worthy of adoption in all head

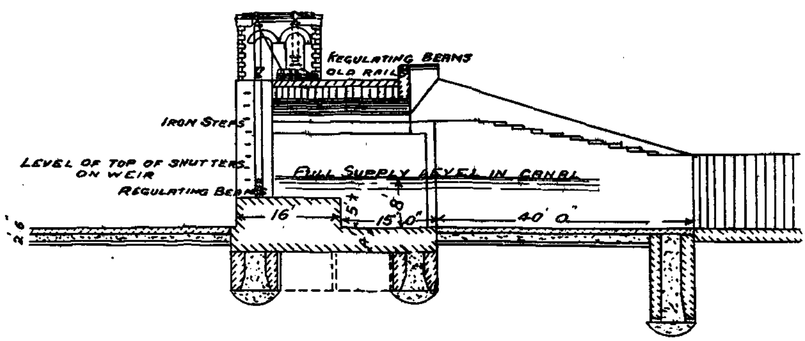


FIGS. 11, 11a.—Alternative Design.

works where the river in flood is very deep and where silt deposit gives trouble. The silt in suspension could be allowed to accumulate in front of the head work to a considerable depth, and then by opening the weir sluice gates it could be swept out, leaving a clear channel ready for the time when the water level has fallen, and supply is drawn from the lower gates.

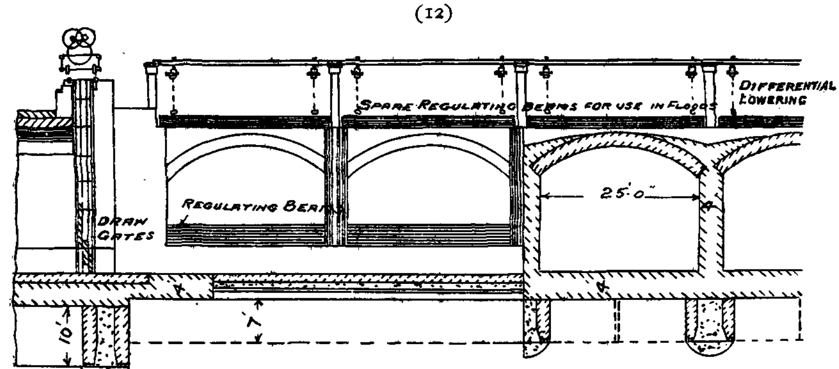
With the adoption of roller gates, the lifting apparatus can consist of a travelling winch spanning the groove openings, the external arching being provided for that purpose. In the existing work, screw lifting gear is used, and that of a primitive pattern, the female screw, held in collars, having the motive power applied, while the male screw rod attached to the gate rises through it. These are worked with great difficulty. The absence of roller attachment necessitates the use of screws to force the gates down against the water pressure, whereas where rollers are provided the gates drop into place of their own weight and only require gear for raising.

(13) It often happens that a canal must take off from the river at a low level, either in order to save a masonry fall in its bed or to economise the height of the weir. To repeat what has been already stated, in order to effect this and yet at the same time retain the floor of the head at a sufficient height above that of the weir sluice to allow of the alternate deposit and scour of silt, the sill of the canal head is raised above its general floor level. On this breast wall the gates rest. This raised sill diminishes the waterway, but affects the discharge of entry in a very moderate degree, as the increased velocity of entry due to the submerged fall nearly compensates for the diminished waterway. If the waterway of the vents were proportionately increased laterally, there would be no appreciable diminution of discharging



LONGL SECTION THRO HEAD

(12)



(12b.) (12a.) PART CROSS SECTION

LONG.L SECTION THRO' UNDERSLUICES OF CANAL HEAD.

FIGS. 12, 12a, 12b.—Jamrao Canal Head.

capacity, and consequently the height of the weir and weir sluice gates need not be raised much above what would be requisite if the raised sill did not exist.

An excellent example of modern design is given in Fig. 12 of the Jamrao Canal Head lately constructed on the Naza, an outlet channel of the Indus River.

In this design the canal bed is at the same level as the floor of the weir sluices. Across the head a breast wall is built 5 feet high. The canal full supply is 8 feet deep, and the crest of the weir shutters is 10 feet above floor; consequently 5 feet of water can pass over the raised sill crest, which will afford the supply required in the canal.

For the purpose of diminishing end contraction in the submerged film passing the dwarf weir, the openings have been made very wide, viz., 25 feet spans, of which there are six. The springing of the arches is at maximum flood level, *i.e.*, at 15 feet above the canal bed, consequently the closure of the 10 feet space above the sill has to be effected by baulks or regulating beams lowered into grooves and raised by means of differential tackle. The weir sluices shown in section in Fig. 12b, consist of seven spans of 20 feet; total ventage, 140 feet against 150 feet of the canal head. The partial regulation of the weir sluice is up to a depth of 10 feet, and is effected by three draw gates manipulated apparently by fixed winches at each pier head.

(14) The fault of the design of the canal head lies in the slowness of regulation necessitated by the width and height of the openings. Complete closure has to be entirely effected by the regulating beams. It is believed that the design would be improved if the spans were reduced to 15 feet and correspondingly increased in number, with flat arches of a versed sine of 2 feet springing at 10 feet above floor. Two rolled beams to be built in at spring line in each span for the top gate to abut on, and for the purpose of carrying panel walls, closing the open arch segments. This will leave 5 feet depth for regulation, which could easily be effected by single or double roller gates raised by a travelling winch.

The abutment spans of both regulator and weir sluices should be tied by iron rods.

This work is evidently founded on sand, and will require a length of l of 225 feet, which it probably has; the section being incomplete, the test cannot be applied. It is provided with a long masonry pavement, forming a rear apron which extends for 140 feet right across the weir sluice approach channel. This is as it should be whenever the canal head is on sand, otherwise the floor beyond the head work would have to be of great thickness to withstand the hydrostatic pressure.

(15) In Fig. 13 are represented plans of the Trebeni Canal Head, a quite recent work. The Trebeni Canal, which is situated in the province of Bengal, takes out of the Gunduk River in its upper reaches. The design of the work is of the usual heavy Bengal pattern of the U-shaped type, added to which, however, is a novel device for the purpose of tapping surface water at different levels of the river with the object of preventing, or at least ameliorating, the entry of silt into the canal. This is effected by the addition of two tiers of arches built towards the river. These vaults are closed by baulks let into grooves, so that the water can be admitted over them at three different levels.

The waterway is designed so that full canal supply can be obtained with only 2 feet depth passing over the platforms. The water, when admitted, falls into the cistern formed in front of the sluice proper, which latter can be closed by a gate operated by a screw worked by a capstan head. In place of the breast wall, now usually adopted to raise the sill of a canal head, the whole floor of the regulator is raised 3 feet above the canal bed, which level is reached by a pitched slope succeeding the masonry floor. This is shown in the transverse section of Fig. 13.

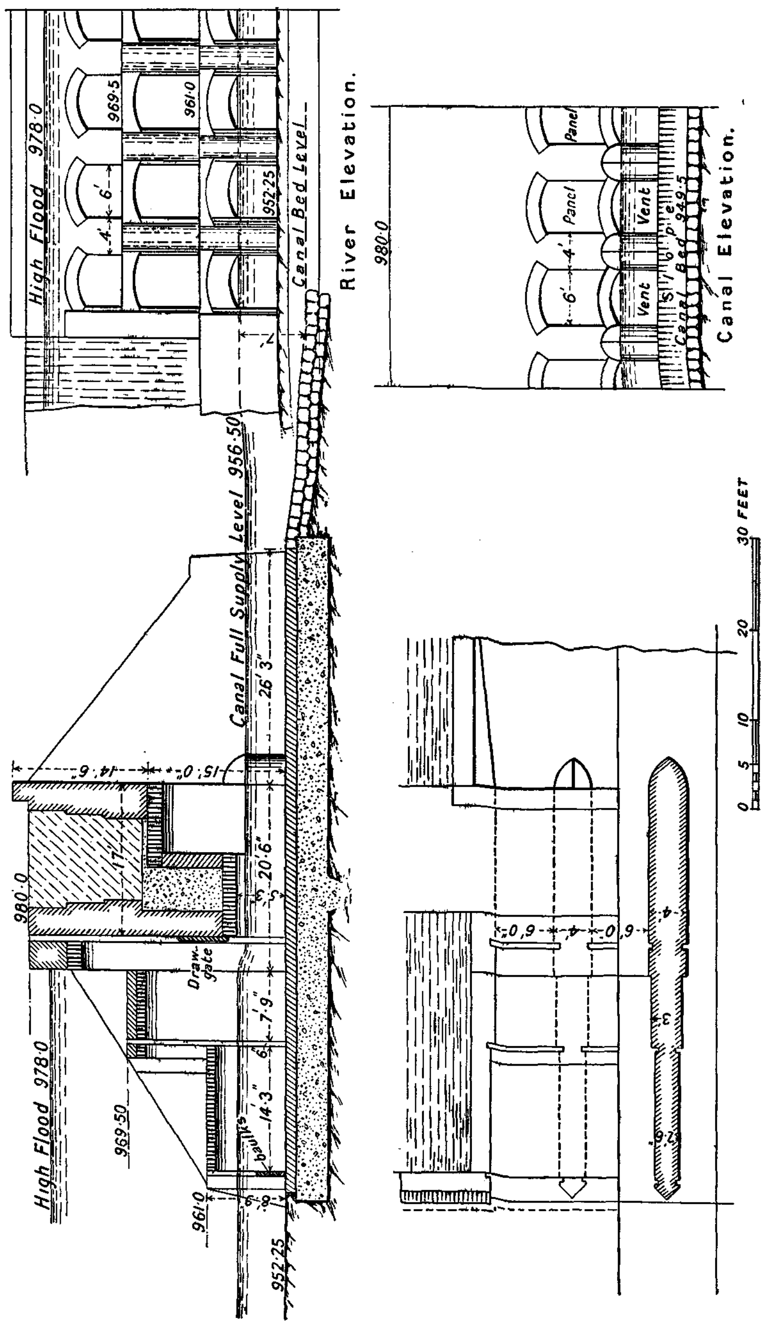


FIG. 13.—Trebeni Canal Head on Gunduk River.

(16) The principle of the design is much the same as that of the Bętwa Canal Head (Fig. 10). It no doubt answers its purpose very well, but the question naturally arises whether the equally good results could not be

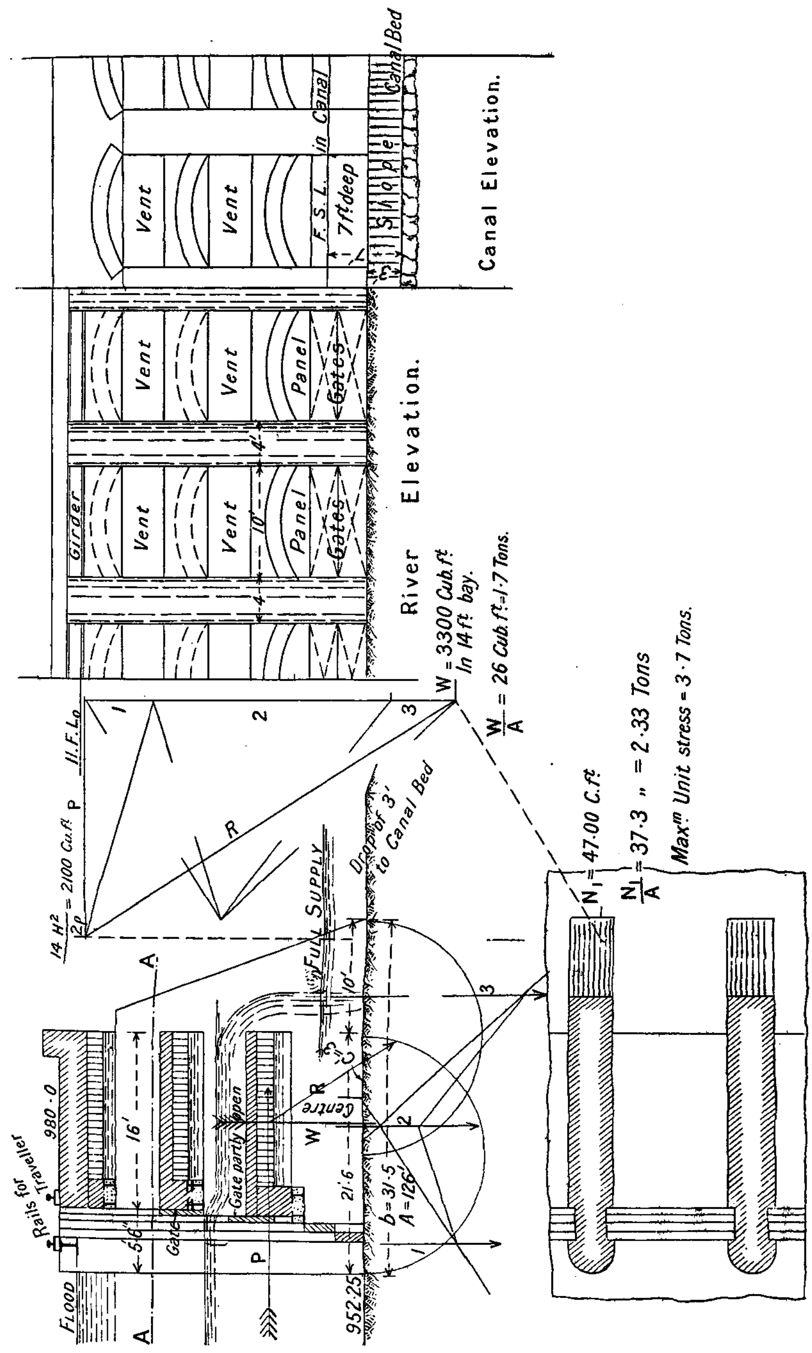


FIG. 14.—Trebeni Canal Head. Alternative Design.

obtained at a much less expenditure. The section of the work is to the eye exceptionally heavy, even for the great head of 26 feet of water which it has to withstand. The only way to settle this question is by an alternative design, which is furnished in Fig. 14. In this, the alternative design (Fig. 11), has been followed, but with the provision of 3 vents, one above the other, in each bay. The spans are widened to 10 feet, the thickness of the piers being retained at 4 feet. The platforms situated above the arches are made deep enough to accommodate the drop gates, of which there are four, each working in a separate groove. Two are provided in the bottom sluice way, so as to enable the upper gate to be either lowered behind the bottom one or raised above it, without blocking the vent. In the section the bottom gates are shown closed, the second one is partially open, admitting water, while the third, which the water has not yet reached, reposes in the space provided for it below its vent. This arrangement permits of easy regulation of the water at any level—only surface water being drawn. The water falls over on to the floor of the regulator beyond the arches. There can be no reasonable objection to this; the floor, if properly built, will stand a vertical fall without the least cause for anxiety, although the firmly-rooted objection to vertical overfalls without the provision of water cushions is an old long-exploded prejudice which still lingers on. The stress diagram and the reciprocal lines of pressure on the section prove that the centre of pressure falls 2 feet within the middle-third of the base.

The maximum unit pressure in the masonry of the pier works out to 3·7 tons—a quite moderate amount. The following comparative statements of the cubical contents per foot run show a saving of 40 per cent. in the alternative over the original design :—

	Contents in one bay.	Length.	Per foot run.
Original section - -	3856	10′	385
Revised " - -	3300	14′	236

(17) Almost all canal regulators take off at right angles to the axis of the stream, *i.e.*, generally parallel to the direction of the weir. As the course of the canal must be more or less parallel to the river, this arrangement involves a large curve in the canal excavation close to the head work. This is objectionable as causing a diminution of the velocity of the current with consequent increased liability to the deposit of silt. The Kistna East Canal, Madras Presidency (Fig. 28c, Chap. VI.), takes off at a very sharp angle to the axis of the stream, probably in order to avoid a rocky hill or some such impediment. The difficulty in that case is got over by the head regulator (which is square) discharging into a large basin from which the canal takes out. A similar difficulty is provided for by the arrangement shown in Fig. 13a, Chap. VII., of the Laguna Head Works. The Jhelum Canal Head may also be instanced (Fig. 9b, Chap. VI.).

Without doubt it would be decidedly advantageous to construct skew heads at many canal head sites. The disadvantage which is inherent in a skew head is the widening of the opening involved and consequent increase in the length of the gates. Now, however, that anti-frictional rollers can be fitted to any gates, be they iron or wood, thus immensely reducing the power required to manipulate them, this objection vanishes altogether. There remains only the question of cost, the skew head being naturally somewhat more expensive.

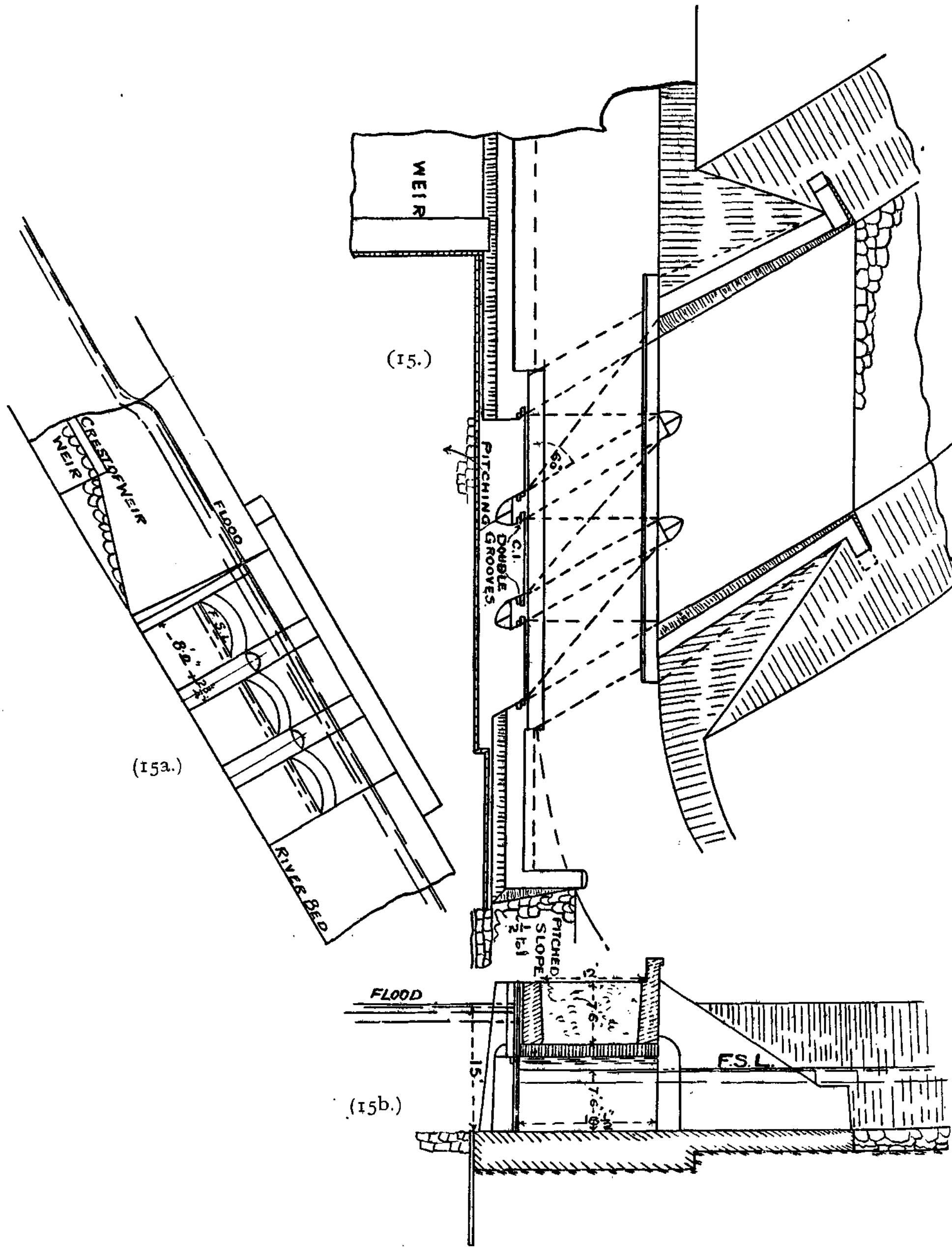
In Fig. 15 a design for a skew head is exhibited. The angle of a skew is 60°.

The canal is assumed to take off at river bed level; the weir is 6 feet high and the afflux 9 feet deep over the weir. A dwarf weir, 3 feet high (erroneously omitted in the figure), is built across the floor at the entrance. No weir sluices are adopted, as the river is supposed to be a small one of intermittent flow, and a scouring escape is provided lower down.

The raised sill wall in the canal head admits of silt deposit 3 feet deep in the canal without interference with the discharge of entry. The material thus deposited will form a steep bed slope. This can be either removed periodically or an escape scouring head can be constructed in solid ground lower down the canal, as has been done in this case. To make this escape effective, a regulator should be built across the canal just below the escape head. By this means, at intervals when the supply is available, the whole discharge can be passed down the escape channel, which will effectually scour out the silt deposited near the head work. The canal bed and banks will require protection by pitching up to the escape head.

(18) If the floods in the stream or river are not very high, a regulator with gates at the canal head can be dispensed with altogether, and the canal intake will then consist simply of an opening protected by masonry flank walls and floor. This arrangement is common in American head works.

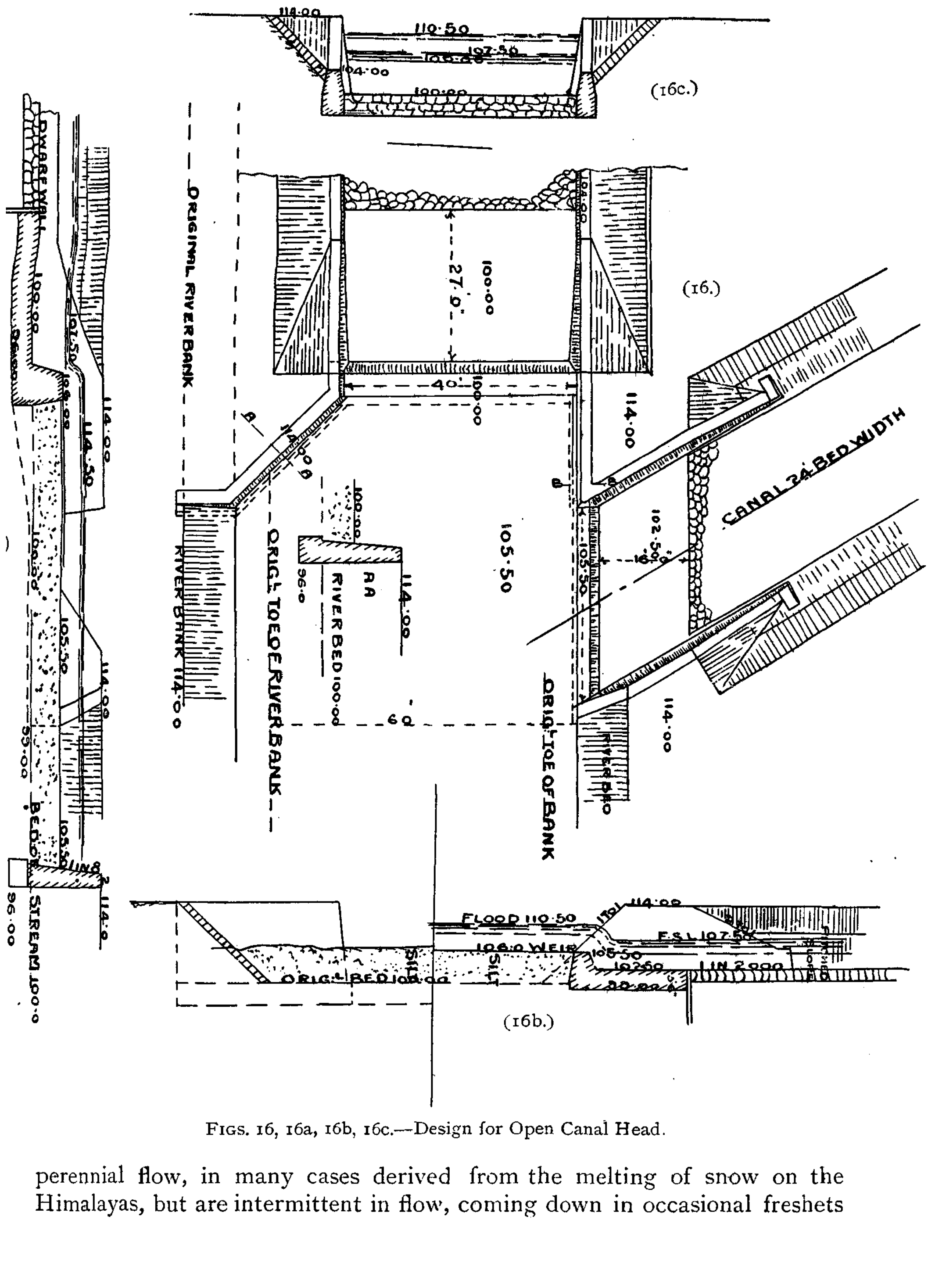
A design on these lines was made for a canal leading out of an intermittently running stream in Burma (*vide* Fig. 16). Assuming the R.L. of the bed of the river to be 100·00, the crest of the river weir was fixed at 106, that of the head canal weir at 105·50, or 6 inches lower, crest shutters 2 feet high being provided. The canal head work took off at an angle of 58° and consisted merely of the weir wall, with a masonry floor flanked by two battering wings. The floor of the head work and bed of the canal were at R.L. 102·50, or 3 feet below the canal weir crest and 4 feet below the river weir crest. The flood discharge of the stream was about 1,200 cubic feet per second, and the maximum discharge of the canal was 450 cubic feet with a depth of 5 feet. With 1 foot depth of water passing over the river weir, the canal would run about 2 feet deep, with a discharge of 120 cubic feet. With 2 feet over the river weir, the canal would run 4 feet deep with a discharge of 300 cubic feet, and with 3 feet over the weir 5 feet deep with a full discharge of 450 cubic feet. At about 300 yards below the canal head, an escape was designed leading back into the river, with a skew regulating bridge across the canal just below it. This was for use when water was not required in the canal; the whole supply could then be



FIGS. 15, 15a, 15b.—Design of Skew Head.

turned down the escape, causing the silt deposited to be scoured out, the canal bed and banks being pitched for this length. The canal bed slope was 1 in 2,000.

(19) The majority of rivers and streams from which water for irrigation can be taken are not, as in the case of Upper India, large rivers with



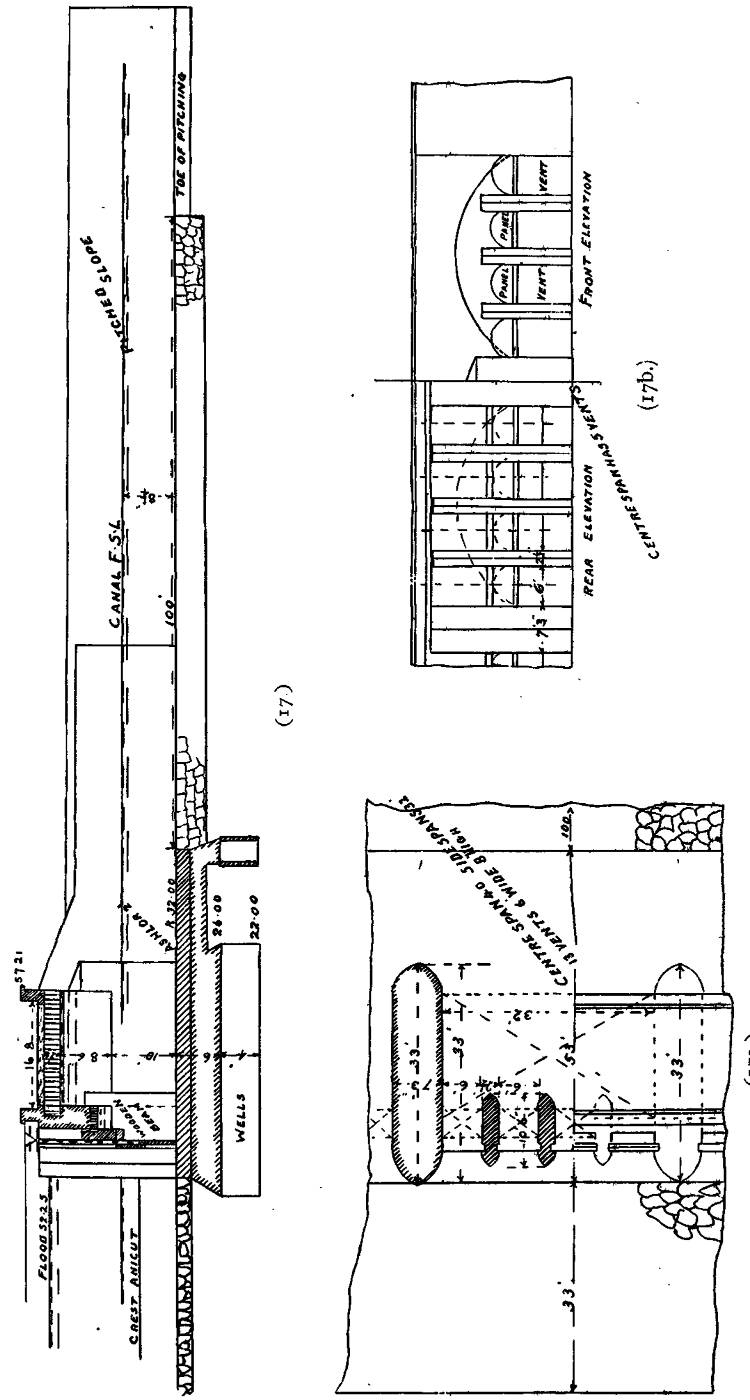
FIGS. 16, 16a, 16b, 16c.—Design for Open Canal Head.

perennial flow, in many cases derived from the melting of snow on the Himalayas, but are intermittent in flow, coming down in occasional freshets during the rainy season, and running perfectly dry during most of the rest of the year. Such streams are numerous in most sub-tropical countries. It is a common error to suppose that small rivers that only bring down water to local rainfall are useless for irrigation purposes; such is by no means the case. Where the local rainfall is insufficient to raise valuable crops such as rice or sugar cane, it can be supplemented with great advantage by irrigation from an intermittent stream. Besides which, these rivers often come down in flood when there is no local rainfall, a rainburst in the hills where their source lies causing a freshet. This water can be utilised either by direct irrigation from a canal or a combination of direct irrigation and storage. The author has constructed several works of this description in Upper Burma. First, the stream was dammed and water taken off by a canal; from this direct irrigation on fields took place, and surplus water was allowed to pass through the fields, which were under a rice crop, and tail into another drainage line. This again was dammed, and another small canal taken off. In this way several drainage systems were practically connected, the main stream tailing into a large tank, whence the surplus water over the waste weir found its way into other tanks and eventually into the original watercourse, which again was dammed and the procedure repeated. The fields themselves, particularly rice fields, which are banked by mud walls 12 inches high, will hold an immense deal of water, and consequently do not require a continuous supply; a freshet once a week or ten days would keep them well supplied with water. The same applies to other crops, but in a less degree.

(20) The thickness and length of the floor in the examples given are designed on the assumption that the foundation is sand. As canals, if possible, invariably take off on the cutting side of a river, the head regulator is often entirely, and the weir sluices partially, founded on solid clay. In such cases no calculation is required for the stability of the floor, and the thickness of the latter beyond the head work is given by the following formula: $t = \sqrt{H_1}$ (H being full head). Thus in Fig. 7, $H_1 = 17\frac{1}{2}$; the thickness of the floor will be $\sqrt{17{\cdot}5} = 4{\cdot}2$ feet. The length will be $2H_1$ or $2 \times 17\frac{1}{2} = 35$ feet, counting from the grooves.

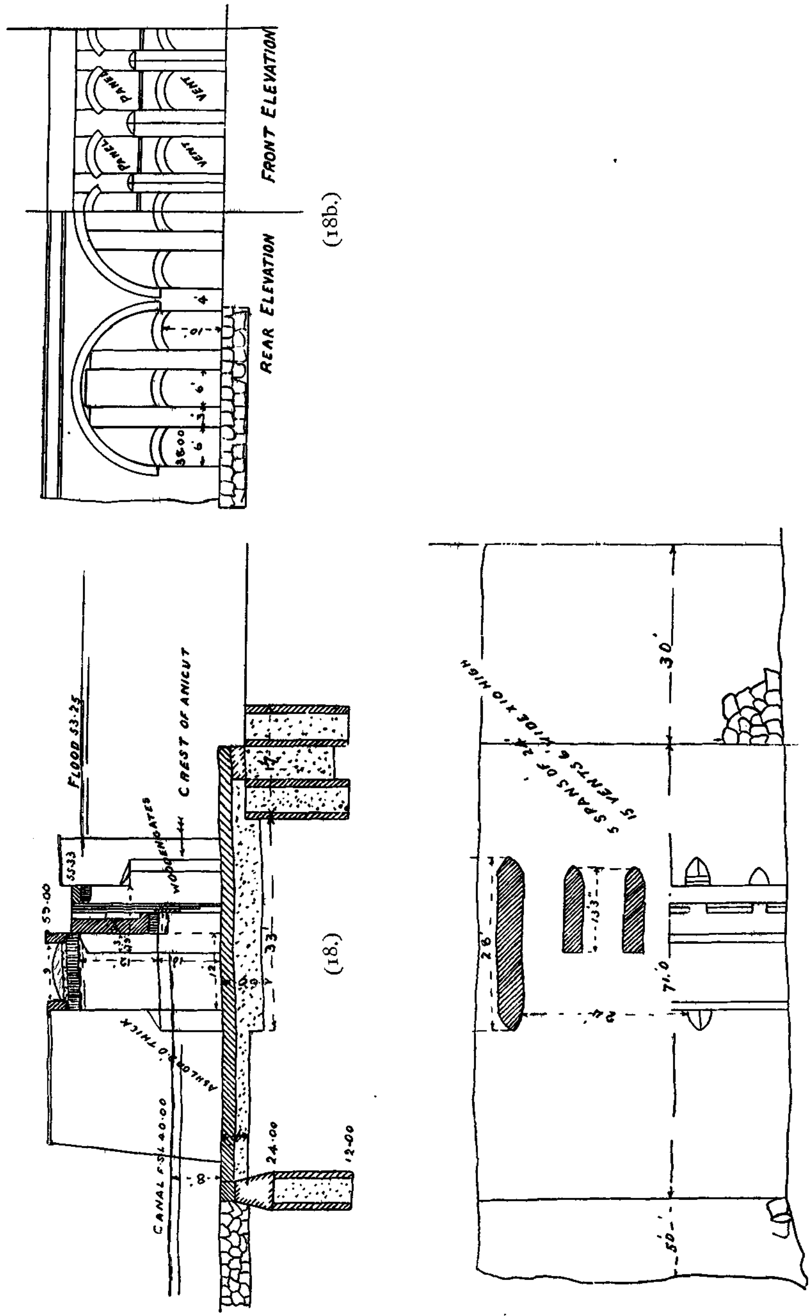
The floor hardly requires thickening under the head work, and could be made, say, 6 feet thick here to distribute the weight. The thickness of piers in these works should be not under $\cdot 4S$, as a general rule (*vide* table in Chap. IV.).

(21) We will now give two examples of the old style of canal head sluice, of which there are many examples in Upper India, as well as in Madras. The design consists of a large span bridge with the interior filled up with smaller piers and vaulting. The system, now that larger vents can be adopted, is quite obsolete. Fig. 17 represents the Dauleshwiram, and Fig. 18 the Babarlanka Canal Head. The works are situated on either side of the Godaveri Anicut. These plans are instructive mainly as regards the foundations, which must be partly on clay. The probability is that both works are now remodelled on the lines of the Rupar Head.



FIGS. 17, 17a, 17b.—Dauleshwiram Head Regulator, Godaveri Canals.

(22) An example of a canal head work built on boulder formation, viz., that of the Western Jumna Canal Head, is given in Fig. 19. In this case



(18b.)

(18.)

(18a.)

FIGS. 18, 18a, 18b.—Babarlanka Head Regulator.

the sill is raised 4 feet above that of the weir sluice, the floor being built on a slope forming a rapid. On this sloping floor the piers of the work are built; a rapid of this description discharges just as much as a vertical drop. The object of raising the sill was to keep boulders from being carried into the canal, the river passing them down over the weir and through the under-sluice at every freshet. The regulating bridge is of the usual U pattern, one peculiarity being that the piers are not provided with cut-water noses in front, but end flush with the face of the rest of the work. This arrangement is to obviate any trap for the accumulation of boulders or detritus in front of or in the vents, and where there is a drop in the head, as in this case, the absence of cut-waters is not so much felt, so that the arrangement, under the circumstances, is decidedly a good one. The vents, which are only 6 feet wide, are closed by wooden draw gates, a recess being formed by an upright beam built into the sill, behind which the gates work. This system of numerous narrow spans is now quite obsolete, the slow regulation being now superseded by wider spans, closed by double roller gates raised by a travelling winch.

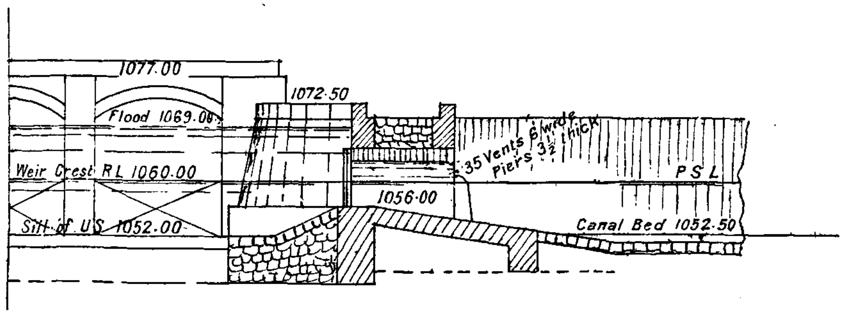


FIG. 19.—Western Jumna Canal Head.

Plans of the weir and undersluice of this head work are given in Fig. 12, Chap. VII.

(23) Fig. 20 illustrates a remarkable and novel design for canal regulating head and gates, employed in the Goulburn Canal, Australia, which is given in "Irrigation Engineering" (Wilson). As will be seen, the gate, when closed, is inclined at 45°, and is attached at the centre to the end of a screw rod which moves it in a vertical direction. The other end rests loosely near the centre of pressure on four rollers carried on a shaft, which is supported by four fixed plummer blocks. The upper ends of the gate are carried on wheels which just fit in vertical grooves in the framework of the iron piers. The gate is thus forced to work vertically; the bottom end is consequently pushed downwards and outwards, eventually assuming a horizontal position when the sluice way is completely open. By this ingenious disposition regulation can be effected with a minimum of applied force, and, in addition, the water is only drawn from the surface. This arrangement, however, would not work where silt is allowed to accumulate at the foot of the gate, and consequently would not answer for head regulators in ordinary cases.

The piers, as well as the abutments, are built of cast-iron, and are 10 feet apart.

(24) The screw-lifting apparatus consists of a cast-iron standard provided with a separate head, carried on an oblong vertical frame of two arms joined at the head. In the space thus formed, the female screw head fits, to which is attached a horizontal bevel wheel. This bevel wheel is worked by a vertical pinion attached to handle shaft, which is carried by a third independent arm. The hollow screw head bears upwards on the frame head and downwards on the top surface of the lower part of the frame, hence does not require a separate thrust plate.

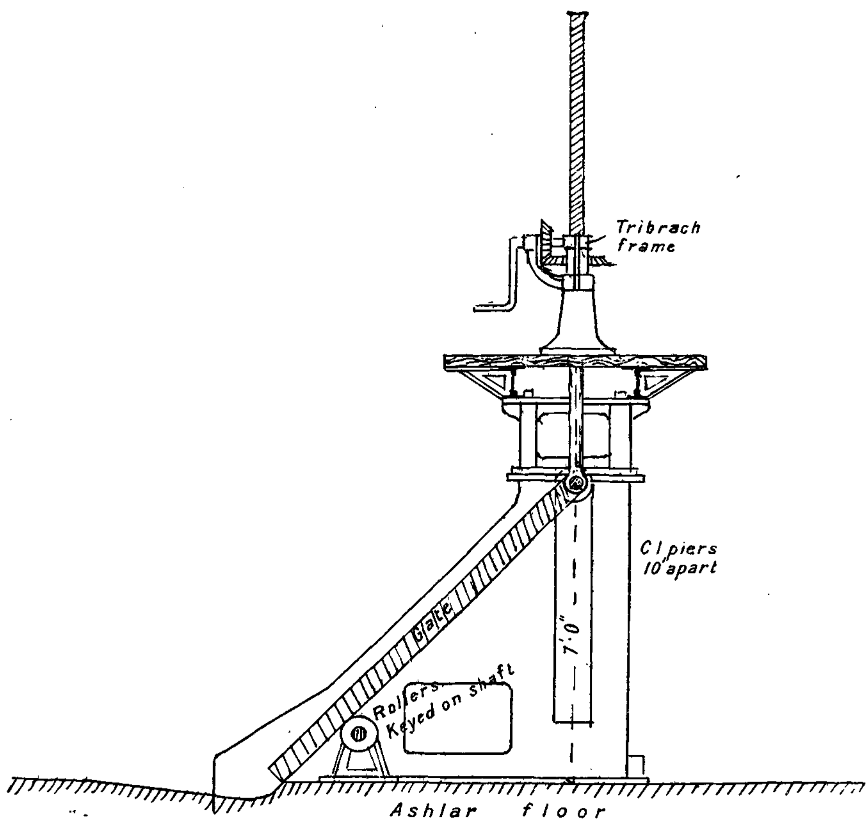


FIG. 20.—Goulburn Canal Regulating Gate.

This arrangement, in common with all others where the female screw has the motive power, is subject to the great disadvantage of the screw rod being in one piece and rising above the platform and frame, and being thus exposed to dust or rain, besides being liable to be bent or damaged. As fully explained in Chap. XIV., where several examples of screw gearing are given, this fault can be remedied by having the motive power applied to the solid, not to the hollow screw. The latter, which should be fixed to the head of a pipe, is prevented from revolving by being attached to the gate, as also to guides, and thus is forced upward or downward by the male screw, which travels inside the pipe. Thus the solid screw is effectually protected from dust and water, and nothing appears above the head of the standard.

This arrangement is obviously superior to any other, and markedly so where the lift is short, as the expensive solid steel screw need only be as long as the lift of the gate, whereas the long connection is formed by

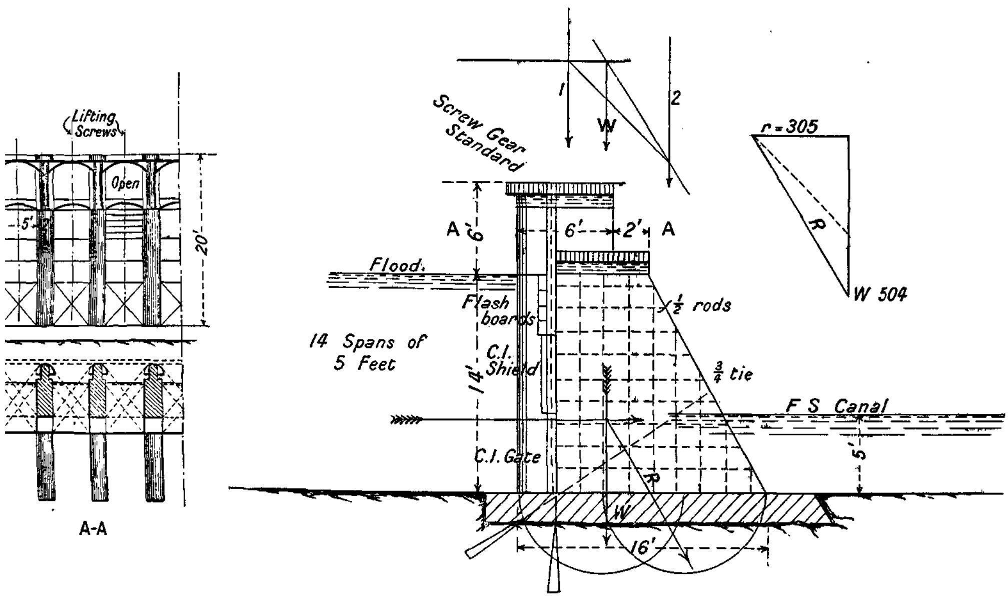


FIG. 21.—Minidoka Canal Head

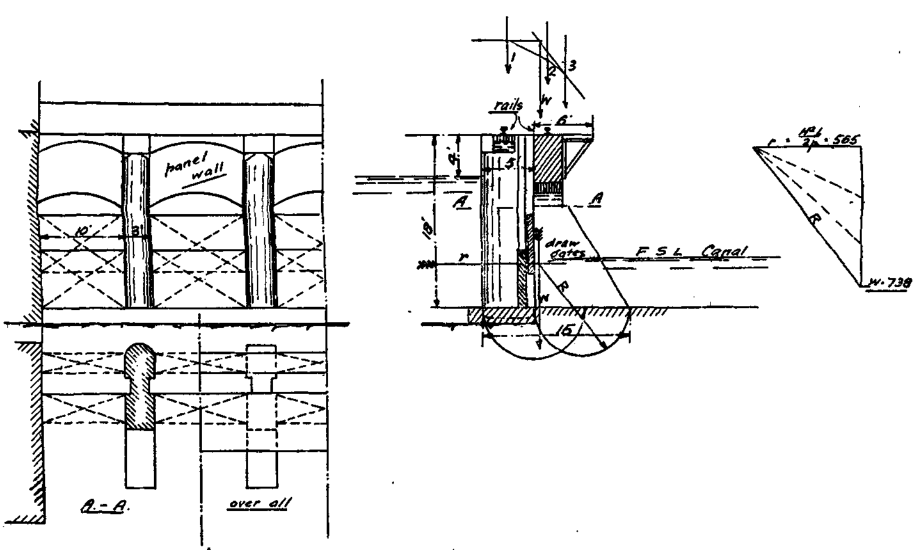


FIG. 22.—Alternative Design.

the pipe, which is better suited for compressive and tortional strain, and need not be more than an ordinary gas or water pipe having a hollow screw head fitted at one end. Ball bearings should also be fitted to the thrust collars.

The propriety of using cast-iron frames for piers and abutments of regulators in lieu of masonry is entirely one of comparative cost, which varies immensely in different countries, and so cannot be settled by any hard and fast rules.

(25) In Fig. 21 we have an instructive example of recent American practice, viz., that of the Minidoka Canal Head in Idaho. Most irrigation works in the United States have hitherto been constructed, in a large measure, of timber, so that the institution of permanent works of masonry is a comparatively new departure. Under these circumstances it is disappointing not to find in these latest productions no improvement over the oriental

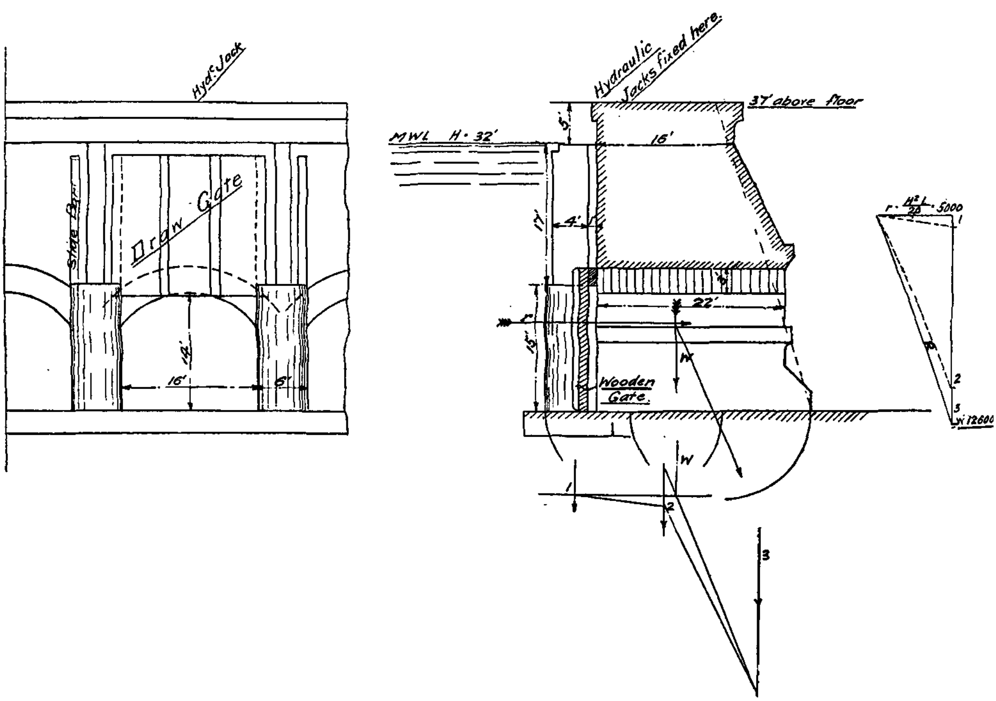


FIG. 23.—Folsom Canal Head.

types common to India and Egypt, but on the contrary discarded forms are reproduced. The principal of modern improvements, largely influencing design, is the adoption of anti-friction rollers to draw gates, whereby larger vents can be used than were formerly practicable, which arrangement tends to economy of material, as well as to the possible avoidance of expensive screw gear for manipulating the gates, and also the practice of admitting surface water only.

In this work, the head of water to which the superstructure is exposed is very moderate, being only 14 feet, *i.e.*, much below what is usual in Indian canal heads; consequently a light superstructure is all that is required to provide the weight necessary to oppose the overturning moment of the water pressure.

The closure of the vents is effected by a cast-iron draw gate working in the grooves of a high frame similar to that used in reservoir sluices. The upper part of the frame is provided with a cast-iron shield, behind which the gate is drawn when lifted. This shield takes the place of the breast wall or depressed arch usual in Indian works. Above the shield closure is effected by baulks, or flash boards as they are termed in America. This arrangement enables water to be drawn from the surface during high floods.

In the design the piers are 2 feet thick to spans of 5 feet, a proportion of ·4S. This proportion is suitable in cases where the head of water equals or exceeds 20 feet, but with a low head, as in the present instance, the proportional thickness undoubtedly errs on the side of solidity.

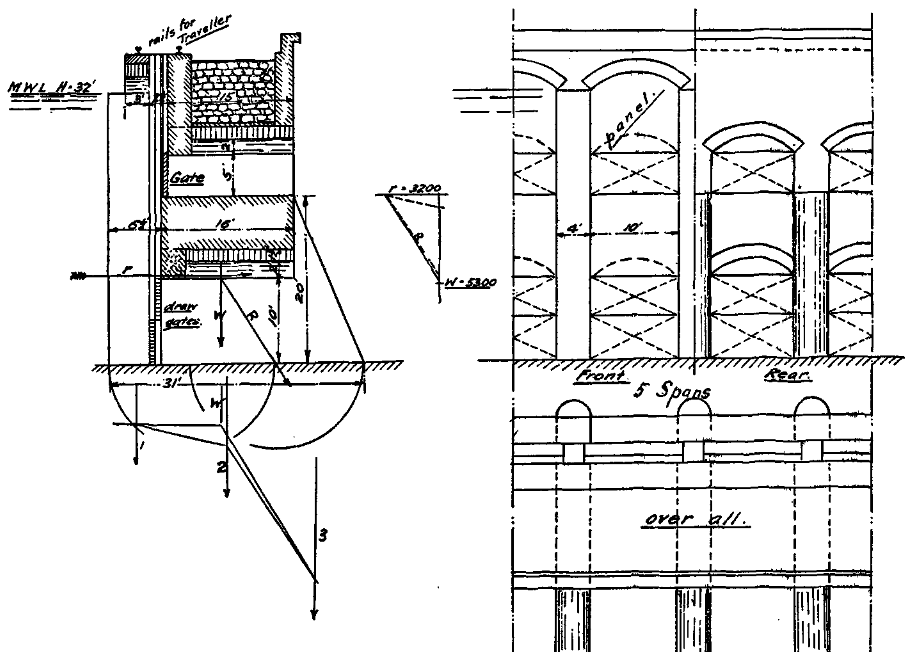


FIG. 24.—Alternative Design.

To prove the unnecessary solidity of the construction, the resultant line of pressure, together with the concomitant graphical diagrams, has been drawn on the profile in Fig. 21, with the result that its incidence on the base line of the piers lies too far within the middle-third with regard to strict economical considerations. The pier bases therefore are subjected to a very moderate maximum pressure, estimated at under $1\frac{1}{2}$ tons per square foot. In spite of this and the fact that the overturning moment is neutralised more than is at all requisite, the piers are reinforced with a network of steel bars, all of which, as well as the anchor bars let into the rock, is absolutely superfluous and unnecessary.

The adoption of a single lifting gate is open to the serious objection that water can only be admitted at the bottom, except on occasions when the river level is over 10 feet above the floor, *i.e.*, on the comparatively few occasions when it passes over the waste weir. Double or treble draw gates running in separate grooves enable water to be admitted at the very top by lowering the upper gate behind the lower gate; in this way the lowest stratum of water is never disturbed, until absolutely necessary. Silt is thus allowed to deposit, instead of entering the canal head, and can be scoured away by undersluices, sand sluices or other expedients.

The screws working the gates are operated by a worm and bevel gearing which is attached to a hollow or female screw head, held in the standards. This revolves and draws the long solid screw bar to which the gate is attached up and down as may be required. When the gates are raised these

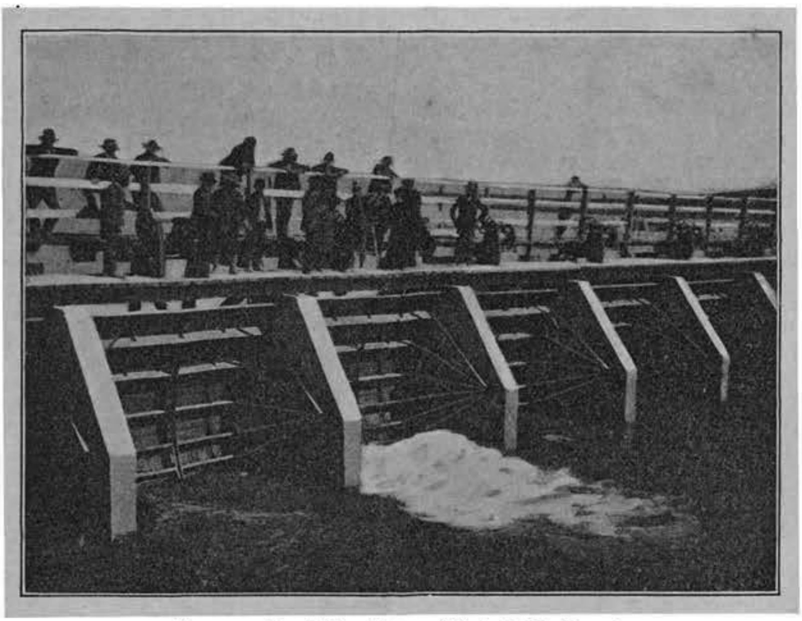


FIG. 25.—Head Regulator of Twin Falls Canal.

fourteen solid screws will project 10 feet above their standards into the air. This objectionable and primitive arrangement is easily avoidable, by the simple expedient of applying the motive power to the solid rod, while the female screw head is attached to the end of a pipe fixed to the gate, which pipe does not revolve, and inside which the solid screw is protected from wet or rust, nothing appearing above the standard. The plans are derived from "Irrigation Engineering" (Wilson).

(26) In Fig. 22 an attempt has been made to provide a more economical and more efficient design. The spans are increased to 10 feet and the piers made ·3*S* or 3 feet thick, which is ample under the conditions of head. A pair of roller gates closes the entrance to the vents, which are 10 feet high. This considerable excess over the F.S.L. in the canal is adopted in order to

draw water always from as high a level as possible. Above this level a simple breast wall carried by narrow arches closes the upper part, the piers stopping short at spring level. A 6 feet way is formed by iron brackets planked over, at the rear of the breast wall. An outer arch is provided for the outer rail of a travelling winch.

These travelling winches are all provided with double winding drums, one over each groove, so that the gates are hauled up or lowered by chains attached to hooked bars projecting upwards and sheltered in recesses in the grooves, so that they can easily be fished for and engaged from above, in case the chains themselves are not drawn to the surface and hooked there ready for attachment to the winch drums. For details, *vide* Min. Pro. Inst. C.E., Vol. CLVIII.

On the section, Fig. 22, the upper gate is shown partly lowered, admitting water.

The comparative quantities of masonry or concrete in either work are as follows:—

Fig. 21: Fourteen spans of 5 feet, 2 feet piers, with reinforcement.
Quantity in 7 feet lengths, 480 cubic feet.
Per foot run, 69 cubic feet.

Fig. 22: Seven spans of 10 feet, 3 feet piers, without reinforcement.
Quantity in 13 feet lengths, 790 cubic feet.
Per foot run, 61 cubic feet.

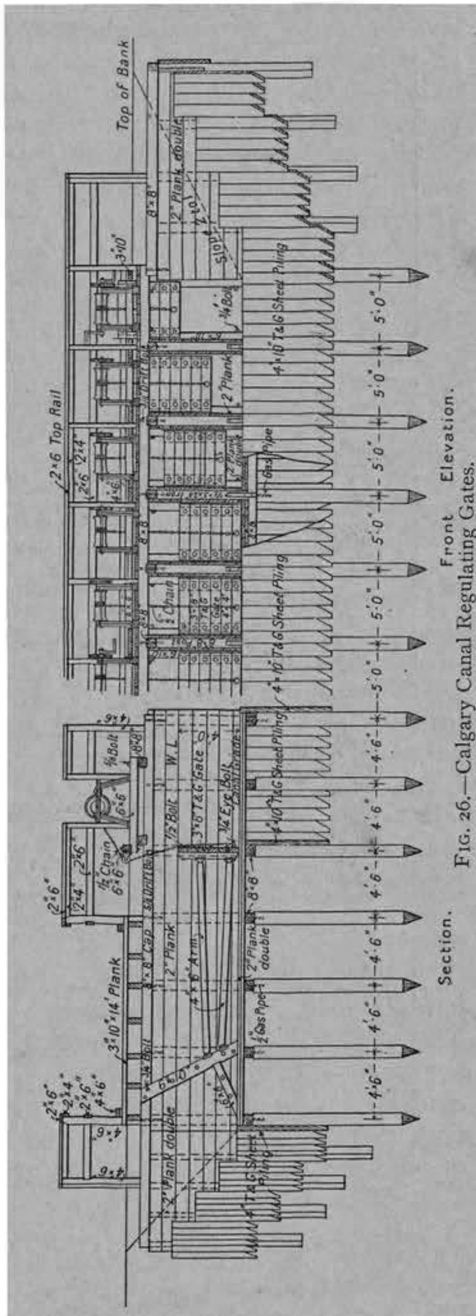


FIG. 26.—Calgary Canal Regulating Gates.

(27) In Fig. 23 is another example of a recent work in America, viz., the Folsom Canal Head Regulator. This work consists of three spans of

16 feet, the piers being 6 feet thick, or a proportion of $\cdot 4S$ nearly. The head is much greater than in the former example, being 32 feet. In this design the depressed arch is very properly made use of, its springing being

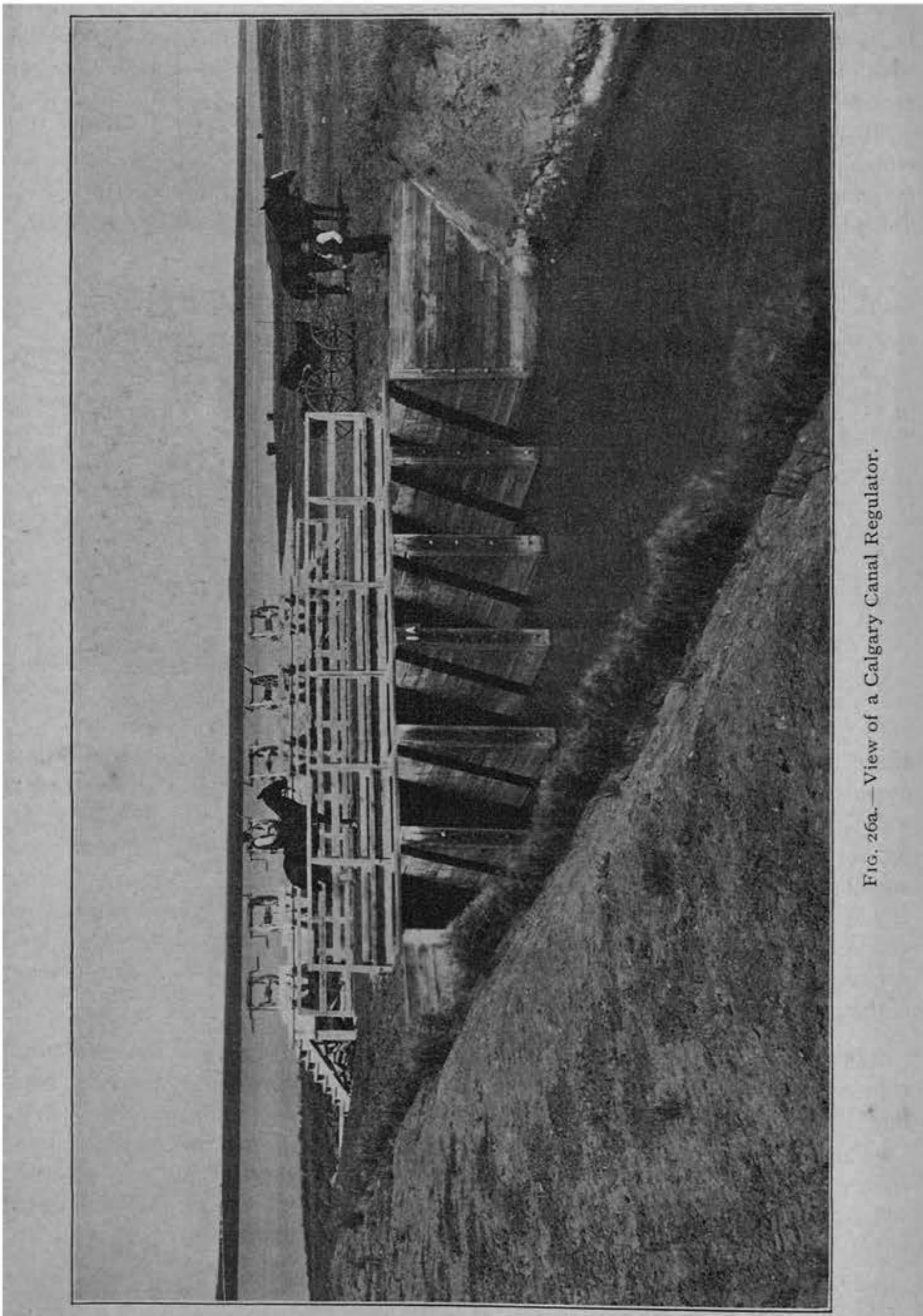


FIG. 26a.—View of a Calgary Canal Regulator.

10 feet above floor. The rest of the work is built up solid for an average width of 20 feet. The area of each draw gate is 16 × 14 feet, which is enormous considering the great head of water. The gates are of wood, not provided with anti-friction rollers, so that very exceptional arrangements have to be made to manipulate them. This is effected by the adoption of hydraulic jacks, one to each gate. These jacks have plungers 14 feet in length fastened to the rear of each gate, and are operated by a head of water provided from a power house.

It is considered that the section, which is very massive, could be economised with advantage, retaining existing conditions of span, etc., by reducing the weight of the solid mass filling and lengthening the piers. The adoption of a U section with side breast walls filled between with any

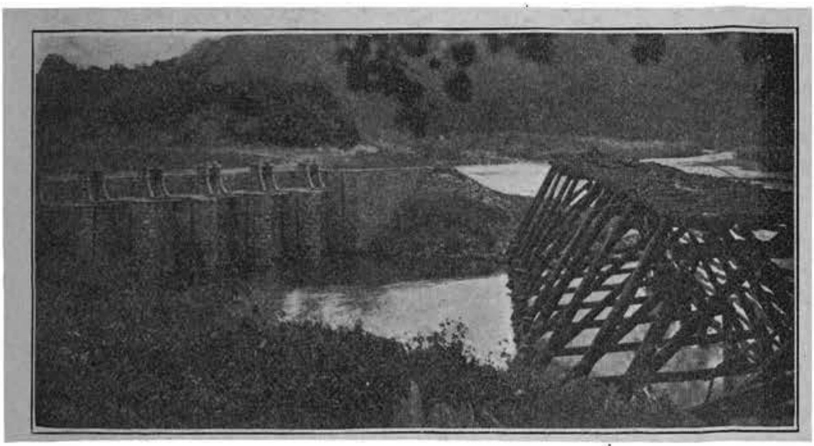


FIG. 27.—Zawgyee Canal Head (Burma).

heavy material, would effect great economy in masonry, without much diminishing the weight; a narrower top width with a battered inner breast wall being also adopted, as shown by a dotted line on the section of Fig. 21. As, however, in this canal in particular, the prevention of silt deposit is an urgent matter, it is deemed that a complete remodelling of the design allowing of water being drawn at different levels, much as was done in the alternative design for the Betwa Head (Fig. 10), would be more suitable. The exceptional size of the vents will also well bear reduction, in view of easy manipulation of the gates by ordinary means.

(28) In Fig. 24 the design is remodelled on these lines; the spans are reduced to 10 feet with piers 4 feet thick, the number being increased from three spans to five.

Three tiers of gates are provided in three sets of grooves with outside arches to accommodate the travelling winch. The U section is adopted with arches depressed to 10 feet above springing. These alterations result in very substantial economy, as below:—

Fig. 23: three spans of 16 feet, 6 feet piers.

Quantity in 22 feet lengths, 12,600 cubic feet. Per foot run, 573 cubic feet.

Fig. 24: five spans of 10 feet, 4 feet piers.
Quantity in 14 feet lengths, 4,200 cubic feet. Per foot run, 300 cubic feet.
This is much to the advantage of the alternative design in point of view

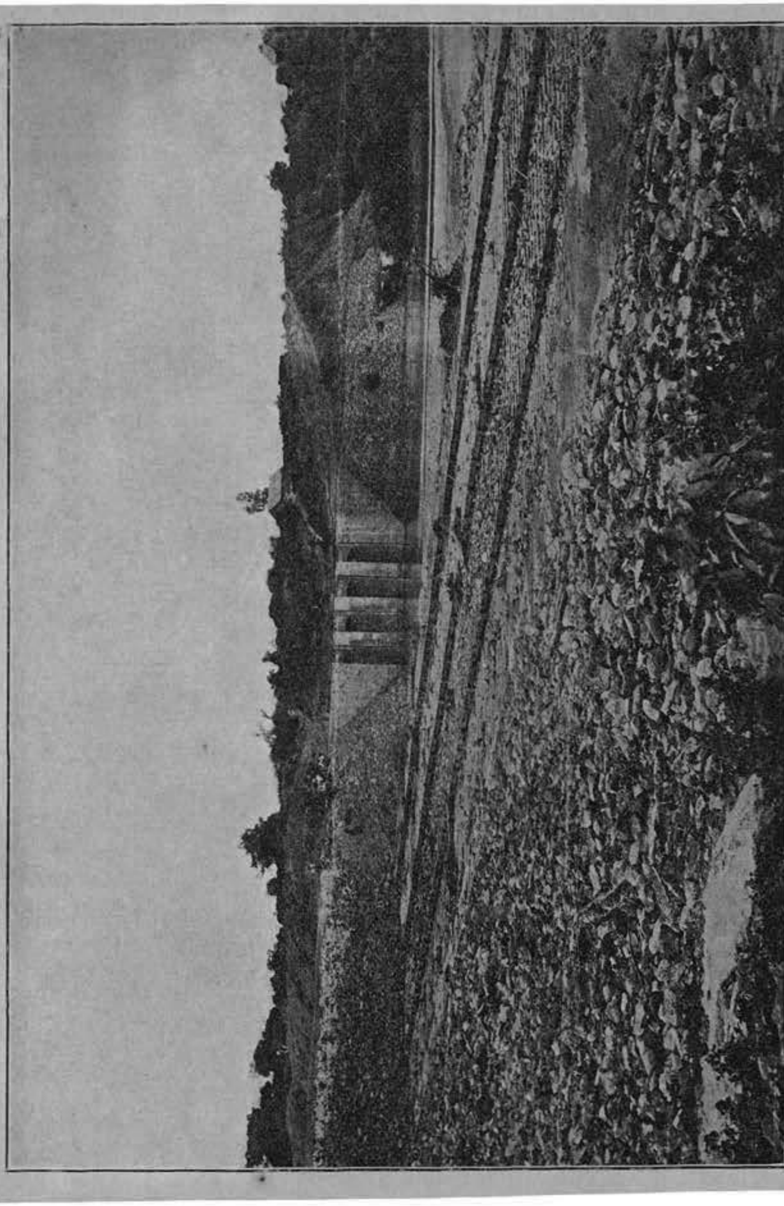


FIG. 28.—Tennyetkon Canal Skew Head and Crib Weir (Burma).

of economy, while the efficiency as regards regulation and prevention of silt entry is undoubtedly enhanced to a considerable degree.

In both these works, viz., Figs. 21 and 23, the same arrangement is adopted for the gates, which slide on flat iron bars let into the face of a recess in the piers, the single or double U-shaped cast-iron grooves, usually adopted in modern practice in India and Egypt, not being employed. The rounded cut-waters also do not extend above the height of the gates, so that there is no semblance of groove above this level. This arrangement would not suit where gates are lifted at each end, not at the centre, the groove being necessary to protect the lifting chains from the current.

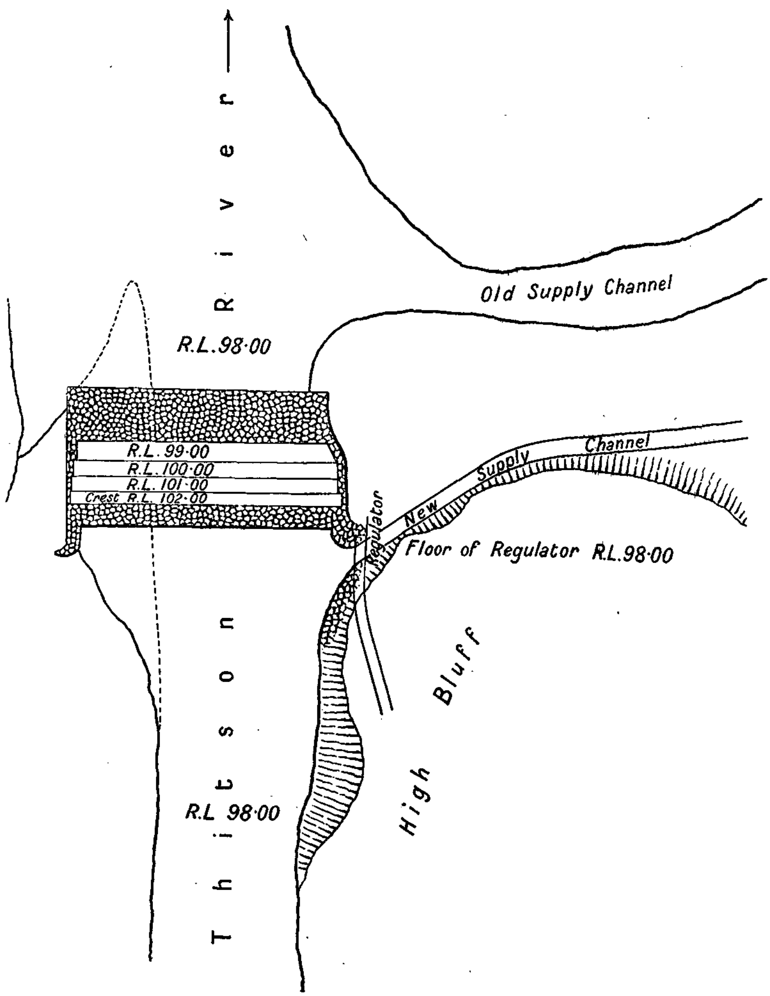


FIG. 28a.—Site Plan, Tennyetkon Crib Weir.

A site plan of the Folsom Canal Head Works is given in Chap. XII. In the section, Fig. 24, a set-back should have been shown in the masonry platform below the uppermost gate, with the object of admitting the grooves being lengthened. This will enable the gate to be lowered as well as raised above its present position ; this has been done in Fig. 14 (*ante*).

(29) It is unfortunate that very few drawings of American canal head

works are available. The pernicious modern system of substituting photographs for working drawings is responsible for this. From these photographs a good idea of the style of work is obtained, and that is all.

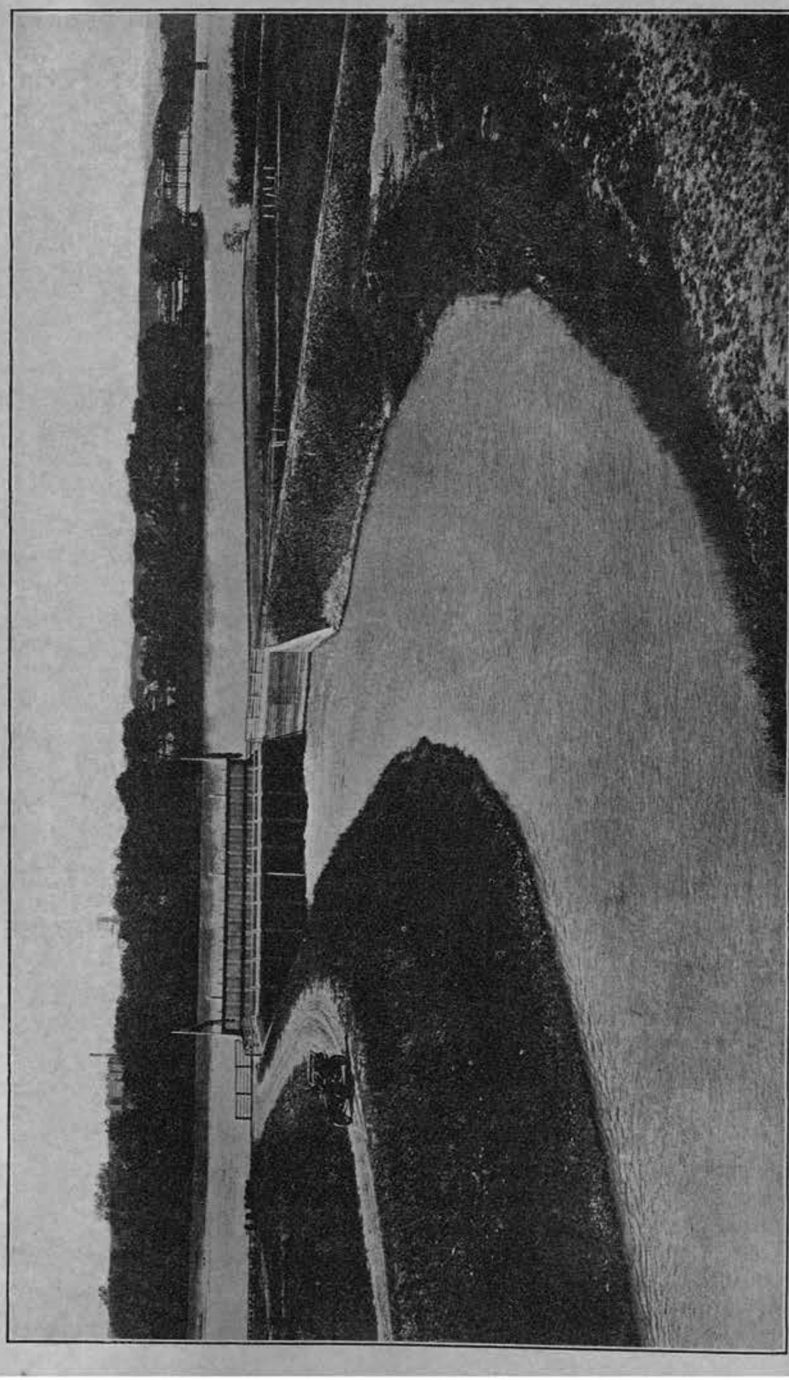


FIG. 29.—View of Head Regulator, Calgary Canal, Alberta, Canada. Discharge, 2,000 second-feet.

One of the Twin Falls Canal Head is reproduced by permission in Fig. 25. It is taken from Schuyler's "Reservoirs," page 74, a technical book of the very highest class.

From this, two main points of divergence from Indian head works is discernible. First, the head of water is very low, the gates are 11 feet in height, and there is not much in any way above them. Secondly, there is no road bridge for cross communication for wheeled traffic. This, which is indispensable in a thickly-populated country like India, can be dispensed with here. The superstructure in this particular case consists solely of a wooden footbridge just sufficient to accommodate the man working the gates. These latter gates consist of fan-shaped segments pivoted in the concrete piers at some distance back. This arrangement admits of very easy manipulation. The gates are evidently of iron, and are probably rendered watertight by stanching strips of rubber.

Each gate has a fixed lifting apparatus, which probably consists of a drum revolved by gear which winds up a chain attached to the bottom of the segment. Segmental gates of similar design were used in the Grand Barrage near Cairo. On the remodelling of this work they were all removed, and double drop gates in grooves with roller bearings, raised by a travelling winch, were substituted. As regards Oriental irrigation works, this style of water gate may be considered to be quite obsolete, the reason being that it has the fatal defect of only admitting water at the bottom. When the gate is revolved the aperture below acts as a scouring sluice, the water entering under pressure as in a submerged orifice, and this must inevitably carry silt and sand into the canal. This action in some cases, where levels admit of it, as in the Folsom Canal Head, is ameliorated by the adoption of under-floor sand traps. But undoubtedly the better plan is to make the water enter over the lower of double gates or flash boards. Where there is not much silt the Australian regulating gate (Fig. 20) would undoubtedly give good results and be just as easy to work by one man as the segmental gate. These gates are 12 feet wide.

The regulators on the new C. P. R. Canal at Calgary, Alberta (Figs. 26 and 26a), are fitted with wooden balance lever gates on the same principle. These are centred very far back, so that the power to lift them is very small. They have a very clumsy appearance.

(30) In Upper Burma, south of Mandalay, an extensive system of irrigation from the Zawgyee and another river, or rather hill torrent, has been in operation for 900 years. These canals have recently been entirely remodelled on modern lines, and are now provided with head regulators, escapes or spillways, falls, and all the various works incidental to proper regulation. The head work of the Zawgyee Canal is illustrated in the photograph in Fig. 27. It consists apparently of small vents some 6 feet in width, closed by single wooden draw gates operated by separate fixed rack gear lifting apparatus. Spare grooves are provided in front to allow of closure by baulks or flash boards, whenever necessary.

Although silt deposit is not troublesome in these canals, still double draw gates would have been a better arrangement and more in accordance with the best modern practice.

The weir was an old Burmese work of loose stone of type C, held in place by rows of small stakes. These works required constant annual repairs and even renewal; they are now probably sheathed with a covering of rubble masonry, or slabs on edge, and so rendered permanent. This treatment was proposed when the author was in charge of the Kyanksè Irrigation Division in 1893, although at that time the remodelling project was but in embryo. This site is most picturesque; the engineer's inspection house close by used, however, to be jocularly termed a "promotion bungalow" on account of the extreme unhealthiness of the locality. The trestle work in front of the regulator prevents the entry of logs or detritus—it hides the weir crest from view.

(**31**) In Fig. 28 is a photograph of the Tennyetkon Canal Skew Head in Burma. This small work is situated above a crib weir, which has been illustrated in Fig. 19, Chap. VI. It is believed to be the only skew head in existence. The head work is built high to admit of the draw gates being hauled up clear of the flood line, as, being an intermittent stream, its full flood depth has occasionally to be passed down. 28a is the site plan.

Fig. 29 is a photographic view of the Calgary Canal Head, on the Bow River, near Calgary, Alberta, Canada.

CHAPTER IX

CANAL FALLS

(1) WHEN the slope of the ground surface exceeds the bed grade of a canal, vertical falls have necessarily to be constructed.

In many cases these drops in the bed are of frequent occurrence, and thus form a by no means inconsiderable item in the cost of the whole work. It is, consequently, very desirable that, in their design, strict economy should be combined with efficiency.

Fifty or sixty years ago, when the first great irrigation works, such as the Ganges, Jumna, Bari Doab, Godaveri and Kistna Canals, were constructed, hydraulic science was in a less advanced state than at the present time and, in addition, successful precedent for guidance in design hardly existed. Consequently, owing to the prevalence of erroneous ideas regarding the destructive effect of a mass of falling water, it was deemed essential to avoid a direct overfall; and thus the drop walls were designed on the profile of an ogee curve. The object sought to be attained was that of diverting the falling current into a horizontal direction, and by this means obviating any direct vertical impact in the floor. Experience, however, soon proved that although the supposed destructive action was successfully diverted from the floor of the fall, the effect of the increased velocity engendered below the work was most injurious to the banks and bed of the canal.

Up to a comparatively recent date the crests of the drop walls of canal falls were built raised above the canal bed level of the upper reach. This effected a vertical contraction of the area of the waterway at the fall itself, proportionate to the increased velocity of the current, and thus the velocity of the overfall was restrained to that admissible in the earthen channel above; that is to say, the velocity of approach was restrained to that of the supply channel.

For example: Supposing the water in the upper reach to be 6 feet deep, with a mean velocity of 3 feet per second, the mean unit, or foot run discharge would be 18 cubic feet per second.

If the crest of the weir were built level with the canal bed up stream, the length of the crest being that of the average width of the channel, then the depth of film passing over will be such as would suffice to give a discharge of 18 cubic feet per second. Now the formula for discharge over a weir (free overfall) usually adopted is $3{\cdot}33d^{\frac{3}{2}}$; if this be equated with 18 cubic feet, or else the result obtained from tables, the value of d will be found to be 3 feet. The velocity of approach will then be $\frac{18}{3} = 6$ cubic feet per second. Such a velocity would be most destructive to the canal bed and banks, and interfere with the water level and regimen of the canal for a mile or more above the fall.

To obviate this, if the crest of the drop wall be raised 3 feet above the canal bed, a depth of film of 3 feet will, as we have seen, exactly pass the discharge. Hence the surface slope of the water of the canal up stream will in no way be affected, nor the mean velocity of the channel interfered with. Similar effect can also be produced by lateral contraction.

This simple matter of raising the crest of the canal weir would answer perfectly were the water level in the canal constant. Such, however, is not the case.

This difficulty of varying supply is practically overcome by the device of so-called "notch" falls, which have proved successful to such a degree as to be now universally adopted in recently constructed Indian canals.

In notch falls, the crest of the drop wall is raised up to Full Supply Level, and the part thus raised above the canal bed grade is pierced by a series of trapezoidal openings, termed notches, through which the water passes. The reduction in sectional area is thus effected almost entirely by lateral, not by vertical, contraction, as the base of the notch is at the level of the canal bed up stream. The subject of the discharge of canal notch falls has already been noticed in par. 21, Chap. V. The Tables numbered II. and IV. in the same chapter supply lists of discharges per foot run, under various conditions of depth and velocity of approach, whether free or submerged, which are applicable to notch falls subject to a deduction of 10 per cent.

(2) It now remains to undertake the practical design of the works in detail.

In Fig. 1 is given an elevation, a plan, and a section of a notch opening said to be used in the Chenab canal, while in Figs. 1a and 1b are similar views of a notch fall actually constructed on the same canal. These drawings are derived from "The Irrigation Works of India." The extreme elaboration of detail shown in the diagrams in Fig. 1 does not seem to be carried out in practice in Figs. 1a and 1b; in the latter case the profile of the notch sides on plan is simply a segment of a circle.

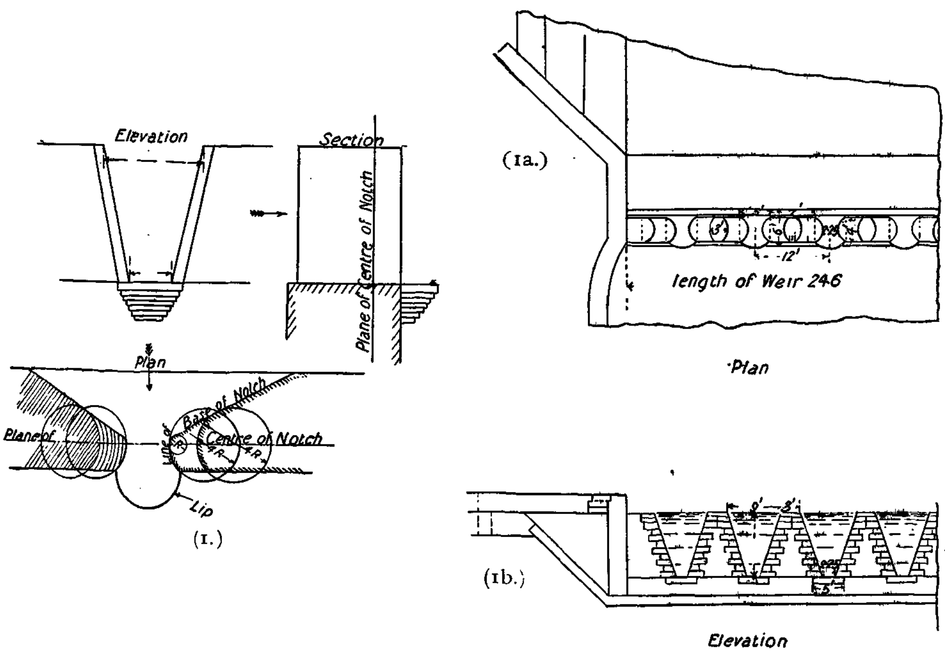
The plan (Fig. 1) would not answer practically, on account of the space required between each notch opening to admit of the great splay given to the sides of the inner face. Such elaboration would undoubtedly conduce to raising the value of the coefficient of discharge; but considering that the trapezoidal notch is after all exactly suitable to the discharge at two levels only, and the coefficient adopted is but approximate, such extreme refinement in the design of the outline appears out of place.

(3) The first point to be determined is the length to be given to the crest of the fall between the abutments, and the number of notches it is advisable to adopt.

This length should not as a general rule exceed the bed width of the canal up stream, and may well be less, up to a limit of, say, $\frac{7}{8}$ of the bed width.

With regard to the number of notches, this point depends mainly upon the top width given to the trapezoidal openings. This top width should clearly bear some proportion to the depth of the notch, *i.e.*, to the depth of full supply. This proportion will be greater in a submerged than in a free overfall, and likewise be in the inverse ratio of the velocity of approach. A safe rule to lay down is that the top width should never exceed the depth, and may vary from $\frac{3}{4}\ d$ to d.

For example: In Figs. 1a and 1b the fall is submerged, the drop being only 3 feet, with a depth of film of 10 feet, and a slow velocity of approach. In this case the top width is $\cdot 9d$.



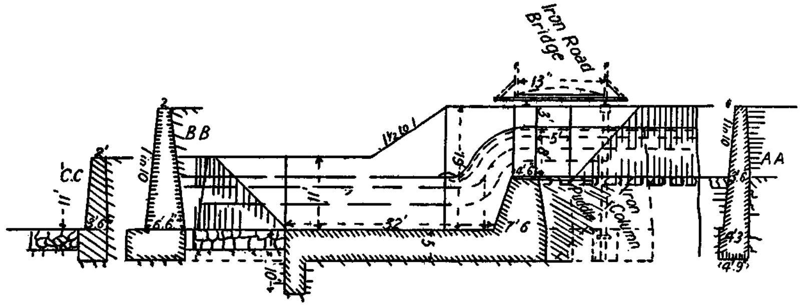
FIGS. 1, 1a, 1b.

(4) The thickness of notch piers should not be less than half the depth and may be made more if they have to carry a superstructure, as is sometimes the case.

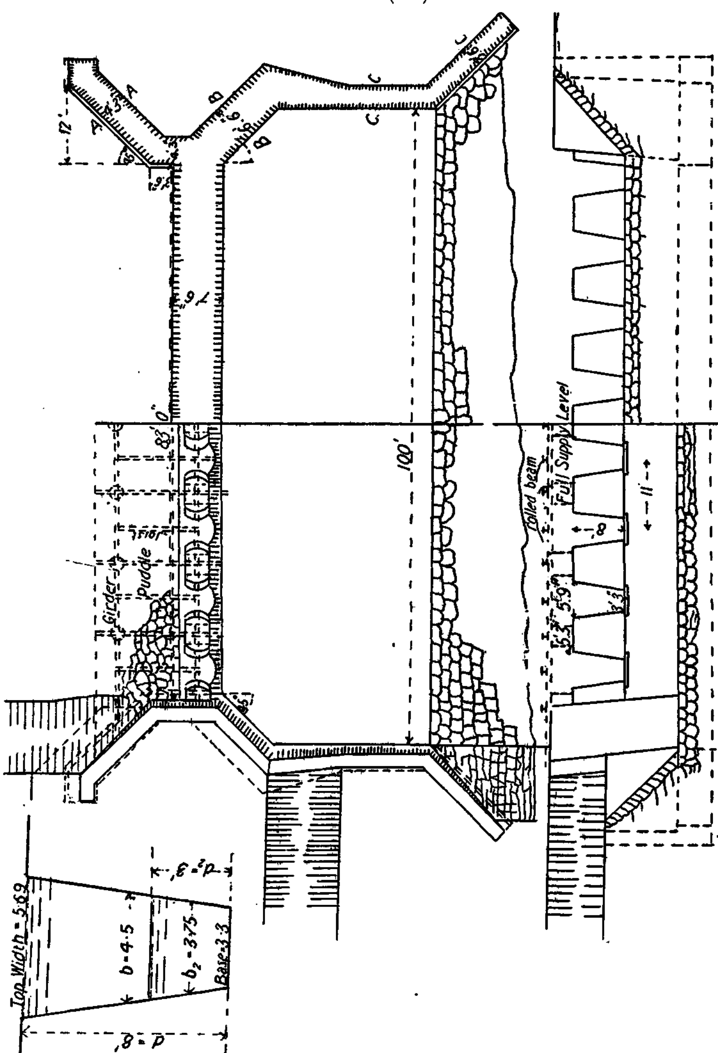
The top length of the piers should not, for purposes of stability, be made less than their thickness, particularly in cases where the piers are carried higher than F.S.L., so that the fall can be crossed by planks laid on their summits, clear of the water.

(5) A practical example of the method of calculating the number and dimensions of notches will now be given.

The design in Fig. 2 is based on an assumed canal discharge of 2,500 second-feet with a mean velocity of 3 feet per second, and a full supply depth of 8 feet (free overfall). To produce these conditions the grade of the canal bed will be about ·18 per 1,000 feet.



(2b.)



(2a.)

FIGS. 2, 2a, 2b.—Design of Notch Fall.

The area of the waterway will be $\frac{Q}{V}$ or $\frac{2,500}{3} = 833$ square feet.

The width of the canal bed, with side slopes one to one will also be $\frac{A}{d} - d = \frac{833}{8} - 8 = 96$ feet. The value of R is $\frac{A}{WP} = \frac{A}{b + 2d\sqrt{2}} = \frac{833}{96 + 22}$ $= 7{\cdot}06$ nearly.

Now with regard to the number of notches. The overfall being free, and the assumed velocity of approach above the average, it can be safely assumed that the top width of each notch will not exceed $\frac{3}{4}d$, or 6 feet. We have seen that the length of the pier crests should not be less than half the depth of water, or 4 feet, and as the piers are intended to carry a light bridge, this length might well be increased to 5 feet.

The spacing of each notch will thus be 11 feet. This will allow of eight notches, or a length of 88 feet between abutments, which is 8 feet less than the bed width of the canal. As, however, the abutments will be designed with battering faces, the actual length at base of the notch piers has been made 85 feet.

(6) Having tentatively decided on the number of notches, the discharge which each has to accommodate will be $\frac{2,500}{8} = 312$ second-feet nearly.

If reference is made to Table II., Chap. V., it will be seen that the discharge per foot run with $d = 8$ and $V = 3$, less 10 per cent., is $69{\cdot}44$ cubic feet per second. The required half width of each notch opening (b) will then be $\frac{312}{69{\cdot}44} = 4{\cdot}49$ feet. This is the value of the half width (b), whatever may be the slope of the sides of the notch opening.

(7) The half width of the whole opening having been thus ascertained, we have now to fix the same for another lower depth of water.

If a notch were designed to discharge precisely the proper amount at every change of level in the surface of the canal water in the upper reach, it would take an ovoidal outline similar to the profile of an egg with both ends truncated, the thick end uppermost. As, however, it has been proved that a trapezoidal-shaped opening correctly calculated to discharge the full supply, and that due to one lower depth, will answer all practical needs, this refinement need not be entertained.

Orifices of this ovoidal section could easily be provided, were cast-iron piers adopted.

(8) To revert to the lower depth, it is found convenient to place this level (d_2) below what is usually the lower supply in the canal (d_1).

In Upper India most canals run at two levels, the higher when water is most in demand, *i.e.*, in the hot season, when the rivers rise owing to the melting of the snows in the Himalayas, and again a lower supply in the winter months; and this double supply level is what obtains in most canals.

Owing, however, to a variety of causes, the fluctuations of level are often considerable, so that it is advisable to adopt a value for d_2 below that of the lower supply level d_1.

This value should be a little over half the lower depth or $\frac{d_1}{2}$, or under half the full depth, *i.e.*, $\frac{d}{2}$.

In the case we are considering, the lower supply level d_1 is assumed at 5 feet in depth, but the value of d_2 will be taken as 3 feet.

(**9**) It now remains to calculate the discharge of the canal at this depth of 3 feet.

In this case $A = 99 \times 3 = 297$ square feet, $R = \frac{A}{WP} = \frac{297}{104 \cdot 4} = 2 \cdot 84$, S is given as $\cdot 18$ per 1,000, or $\frac{1}{5,555}$.

We have now to estimate the mean velocity from the formula $100c\sqrt{RS} = V$.

Omitting c, the expression will become

$$V = \sqrt{2 \cdot 84 \times \frac{10,000}{5,555}} = \sqrt{\frac{5,540}{1,111}} = \sqrt{5 \cdot 11} = 2 \cdot 26.$$

If S per 1,000 be considered, the equation will be $V = \sqrt{2 \cdot 84 \times 10 \times \cdot 18} = \sqrt{5 \cdot 11} = 2 \cdot 26$ (*vide* par. 28, Chap. V.).

The value of the coefficient c, suitable to one of $R = 2 \cdot 84$ and of $S = \cdot 18$ per 1,000, obtained from Table XII., Part IV., of the "Hydraulic Manual," is $\cdot 7$ nearly. The actual mean velocity will then be the previously obtained value of $2 \cdot 26 \times \cdot 7 = 1 \cdot 582$ feet per second, and the discharge of the canal at the lower depth of 3 feet will be AV, or $297 \times 1 \cdot 582 = 475 \cdot 2$ second-feet.

(**10**) Again dividing by the number of notches, the discharge through each must be $\frac{475 \cdot 2}{8} = 59 \cdot 4$ second-feet. The half width b_2 is obtained, as previously, by reference to Table II., Chap. V. In this we find that the discharge per foot run, less 10 per cent. with $d = 3$ and $V = 1\frac{1}{2}$, is $15 \cdot 853$ second-feet; b_2 will therefore $= \frac{59 \cdot 4}{15 \cdot 85} = 3 \cdot 75$ feet.

(**11**) Having thus obtained the widths of the notch openings at two known levels, viz., at $\frac{d}{2}$ and $\frac{d_2}{2}$, we are now in a position to calculate the top and bottom widths.

The vertical distance apart of the two widths b and b_2, are $\frac{d}{2} - \frac{d_2}{2}$. The ratio of widening will then be $2\,\frac{b - b_2}{d - d_2}$. Let these horizontal and vertical differences be designated m and n. Then the expression becomes $2\frac{m}{n}$. Now the top of the opening is distant $\frac{d}{2}$ above the position of b, and the base $\frac{d_2}{2}$ below that of b_2, consequently the top width will be $\left(\frac{d}{2} \times 2\frac{m}{n}\right) + b$, or $(b + d)\left(\frac{m}{n}\right)$ (1). Similarly the base width will be $(b_2 - d_2)\left(\frac{m}{n}\right)$ (2).

The value of m is $4{\cdot}49 - 3{\cdot}75 = {\cdot}74$.
That of n is $8 - 3 = 5{\cdot}0$.

Therefore $\frac{m}{n} = \frac{{\cdot}74}{5} = {\cdot}15$, and transposing these values in the expressions (1) and (2) we have

$$\text{top width} = 4{\cdot}49 + (8 \times {\cdot}15) = 5{\cdot}69 \text{ feet and}$$
$$\text{base } ,, \quad = 3{\cdot}75 - (3 \times {\cdot}15) = 3{\cdot}3 \text{ feet.}$$

The average of both widths should naturally equal b, *i.e.*, $\frac{8{\cdot}99}{2} = 4{\cdot}49$.

The size of the notch is made 5 feet 9 inches at top and 3 feet 3 inches at base. With 11 feet centres, the top length of the piers will be 11 — 5 feet 9 inches = 5 feet 3 inches.

(12) Supposing the fall to be not free, but submerged, *i.e.*, the drop in the canal surface to be 2 feet only.

In such case, with full supply, the discharge (less 10 per cent.), with $d = 8$, $V = 3$ and $H = 2$, as in Table IV., Series IV., is found to be $45{\cdot}0$ second-feet per foot run; the value of b will then be $\frac{312}{45} = 7{\cdot}0$ feet instead of the $4{\cdot}49$ in the free overfall. The lower depth d_2 will be submerged 1 foot, consequently its discharge by Table IV., Series II., deducting 10 per cent. will be $10{\cdot}6$ and $b_2 = \frac{59{\cdot}4}{10{\cdot}6} = 5{\cdot}6$.

$$\text{Then } m = 7{\cdot}0 - 5{\cdot}6 = 1{\cdot}4$$
$$\text{and } n \text{ as before} = 5{\cdot}0$$
$$\text{Whence } \frac{m}{n} = \frac{1{\cdot}4}{5} = {\cdot}28 \text{ and by (1) par. 11}$$
$$\text{the top width} = 7 + (8 \times {\cdot}28) = 9{\cdot}24$$
$$\text{by (2) the base } ,, \quad = 5{\cdot}6 - (3 \times {\cdot}28) = 4{\cdot}76$$

Total . . . $14{\cdot}00$. Mean, $7{\cdot}0 = b$.

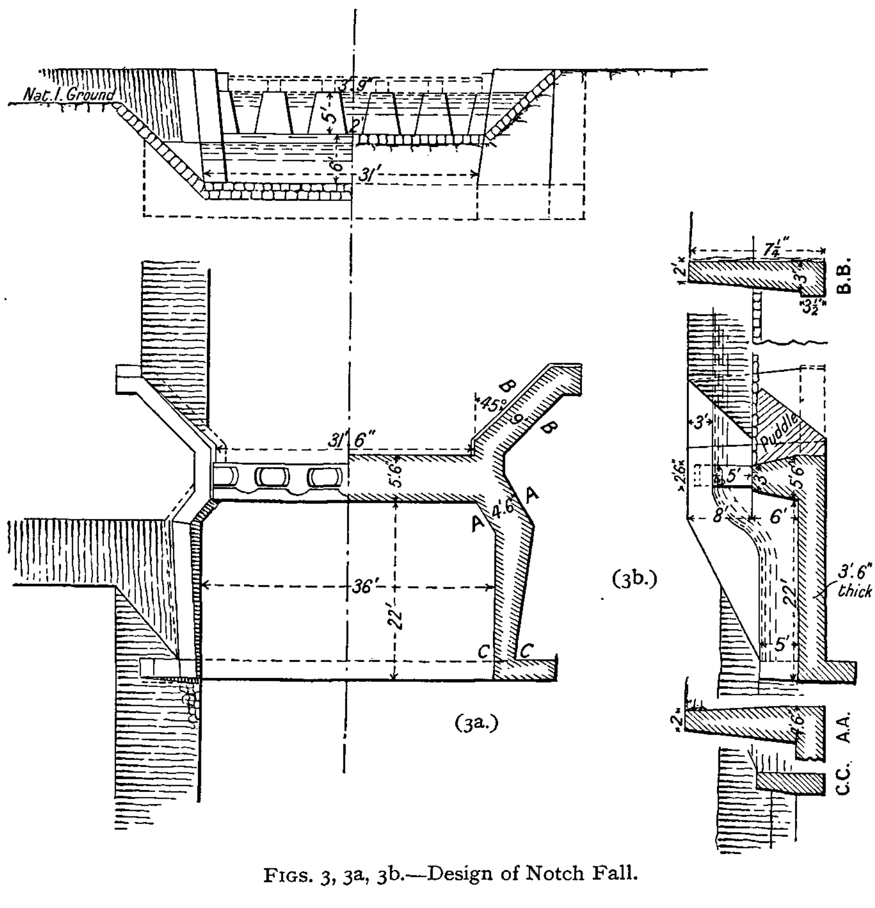
With these dimensions, the yaw of the opening will be noticeably greater, and the spacing of the centres of the notches will have to be made 13½ feet, the weir crest being lengthened accordingly.

(13) Fig. 3 is another design on similar lines to the last, but under differing conditions. Here Q is 500 feet and V 2½ feet, $d = 5$ feet and the canal bed width 35 feet, and $A = 200$ and $R = 4$. There being five notches (*vide post*), the full discharge through each will be $\frac{500}{5} = 100$ second-feet. By reference to Table II., Chap. V., the discharge per foot run, less 10 per

cent. due to $d = 5$ and $V = 2\frac{1}{2}$, is 34·4 cubic feet per second; therefore b will $= \frac{100}{34·4} = 2·9$.

(**14**) The lower level will be taken at a depth of 2 feet. Here $A = (35 + 2) \times 2 = 74$, $R = \frac{74}{40·6} = 1·8$.

The discharge can be obtained with close approximation without having



FIGS. 3, 3a, 3b.—Design of Notch Fall.

to calculate S, by using the proportional ratio of V in either case. Thus $v : V : : 100c\sqrt{r} : : 100C\sqrt{R}$, *i.e.*, $: : c\sqrt{r} : C\sqrt{R}$. In this case $r = 1·8$, $R = 4$ and $V = 2·5$, whence $v = \frac{2·5 \times 1·34}{2} \times \frac{c}{C}$.

If $S°$ in 1,000 be assumed anywhere near its right value, the true values of C and of c can be obtained with close approximation from Table XII., Part IV., "Hydraulic Manual." Let the grade be assumed at ·2 per 1,000. Then with $R = 4$, C will be ·755, and with $r = 1·8$, c will be ·627; b then will be $\frac{2·5 \times 1·34 \times ·627}{2 \times ·755} = \frac{2·1}{1·51} = 1·4$ nearly.

To check this, the actual bed grade will now be found, using the coefficient $\cdot 755$ as before, $V = 100c\sqrt{RS}$.

Transposing and substituting values,

$\sqrt{S} = \dfrac{2\cdot 5}{200 \times \cdot 755} \therefore S = \dfrac{6\cdot 25}{40,000 \times (\cdot 755)^2} = \dfrac{1}{6,400 \times \cdot 57} = \dfrac{1}{3,648}$, whence S° in $1,000 = \cdot 28$ nearly.

Now a slope of $\cdot 2$ per 1,000 was assumed, but if the grade be taken as $\cdot 3$, the corresponding value of C, with $R = 4$, is $\cdot 754$ against the previously adopted value of $\cdot 755$. In the case of c, the coefficient is as $\cdot 631$ to $\cdot 627$. So that any alteration in the already obtained value of V is unnecessary. If the divergence in the ratio of the two old coefficients with that of the two new, *i.e.*, $\dfrac{C}{c}$, were sensibly different, the calculation would have to be gone over again, assuming a more approximately correct value for S.

(15) We have already seen that for the lower depth of d_2 the value of A is 74 feet; the discharge will then be $V \times A$, or $1\cdot 4 \times 74 = 104$ second-feet nearly. This divided by the number of notches, viz., 5, the quotient $20\cdot 8$ is the required discharge at the lower level. By reference to Table II., Chap. V., the discharge per foot run, less 10 per cent. for a value of $d = 2$ and $V = 1\frac{1}{2}$, is $8\cdot 69$ second-feet. The lower width b_2 will then be $\dfrac{20\cdot 8}{8\cdot 69} = 2\cdot 4$ nearly.

The corresponding width (b) at half the depth of the notch has already been ascertained to be $2\cdot 9$, whence $m = \cdot 5$ and $n = 3$ and the ratio $\dfrac{m}{n}$ will be $\dfrac{\cdot 5}{3} = \cdot 17$.

The top width will then be by (1) $(b + d)\left(\dfrac{m}{n}\right) = (2\cdot 9 + 5) \times \cdot 17 = 3\cdot 75$,

and the base width ,, ,, by (2) $(b_2 - d_2)\left(\dfrac{m}{n}\right) = (2\cdot 4 - 2) \times \cdot 17 = 2\cdot 06$

Total . .	$5\cdot 81$
Mean . .	$2\cdot 9 = b$

(16) In the design Fig. 3, the length between abutments has been taken as $31\frac{1}{2}$ feet, *i.e.*, $3\frac{1}{2}$ feet less than the bed width, which is 35 feet. The depth of water being 5 feet, 4 feet was estimated for the top width of the notch opening and $\dfrac{d}{2}$ or $2\frac{1}{2}$ feet for that of the piers, making the notches, five in number, $6\frac{1}{2}$ feet apart. The total length measured at notch crest is $32\frac{1}{2}$ feet. The batter of the abutments, which is 1 in 10, will reduce this at base of notch piers by 1 foot, being 5 feet of height at 1 in 10.

(**17**) The designs in Figs. 2 and 3 are similar in all respects as regards arrangement of parts. The weir wall is of the "hybrid" type of section (*vide* par. 63, Chap. II.). The base being $\frac{H + d}{\rho}$ or, in Fig. 2, $\frac{4}{9} \times 16 = 7{\cdot}1$ feet (made $7\frac{1}{2}$ feet); in Fig. 3, $\frac{4}{9} \times 11 = 5$ feet (made 5 feet 6 inches).

The top width is made 4 feet 6 inches wide in Fig. 2, and in Fig. 3, 3·6. The extra width is required for the accommodation of the notch piers, the thickness of which in both cases is $\frac{d}{2}$.

The length of the floor or apron in both cases is 2 $(H + d)$, according to the rule given in Chap. IV. par. 20.

The thickness of the floor is made $\sqrt{H + d}$, considered sufficient with a clay foundation.

With regard to the width of the floor, it is made in both cases somewhat greater than the canal bed width.

The floor is terminated by a shallow curtain wall, beyond which it is protected by pitching.

(**18**) The abutments, as well as the wings, are all battered at 1 in 10. The abutments are continued up stream for a short distance parallel to the axis of the work, forming the commencements of a pair of level crested splay walls with end returns, of the type mentioned in pars. 64—69, Chap. I.

These not only form an excellent guide to the current on its contraction at the fall, but with the assistance of the level crested portion of the dog-legged down-stream wings, afford a wide level connecting bank and an approach to the bridge way across the fall.

(**19**) The down-stream wings on plan are of the so-called "dog-legged" pattern, having a re-entering angle (*vide* par. 39, Chap. I.). They start splaying outwards from the weir wall until the floor half-widths are reached. This portion is level crested. When the floor widening is thus effected, the direction of the base of the wings runs parallel to the axis of the work. In Fig. 2 the first portion has a sloping crest of 2 to 1, corresponding with the drop in the canal bank; when this is overcome, it continues in the same direction at the level of the down stream canal bank to the termination of the floor, when another splayed return forms a junction with the canal bank side slopes.

(**20**) In Fig. 3 the arrangement is somewhat different. Here the wall crests, beyond the re-entering angles, slope down at 2 to 1 till they reach the lower water level, when they make direct returns into the canal side slopes. The commencement of the sloped crests, and with them the re-entering angles on plan, are fixed with regard to space being left for the return walls within the length of the floor, which latter, as we have seen, is 2 $(H + d)$. H is height of drop wall, d depth of water falling over.

(21) The sections of the wings are strictly in accordance with the rules formulated in Chap. I. The sections at AA in Fig. 2 and at BB in Fig. 3 of the up-stream wings are of walls under partial earth pressure. The earth pressure exists only as far as the up stream canal bed ; below this level no additional external force is applied, so that the base at the upper canal level is made $\cdot 4H - 1$ wide, and below that point the back is carried down vertical, but the face continues battered, as above ground ; in addition a footing is provided near the base; the 1 in 10 batter having at this depth to be thus supplemented to ensure the line of pressure falling within the base.

This important matter, which is so often neglected, has received considerable attention in Chap. I. (*vide* pars. 45 – 53).

(22) In Fig. 2, arrangements are shown for a light bridge across the fall. In this the notch piers are utilised to support square pillars of brick in cement, which carry an **I** beam. A second longitudinal is bolted to iron columns, which are founded on masonry blocks situated at the general foundation level, and these two are spanned by cross joists projecting well beyond the longitudinal beams, which in turn are covered with planking. Parapet rail standards are bolted to the projecting part of the joists, and thus a 13 feet wide roadway is provided.

The level of the platform is a little above that of the banks on either side, and the connection can be formed by a short slope.

An arrangement of this kind is equally effective and vastly more economical than the heavy masonry over-bridges which are such a common feature in old canal falls, and which are perpetuated to the present day, as will be noticed later with reference to the Jamrao Canal Fall in Fig. 5.

In Fig. 3, a simple narrow footbridge is provided by planks laid on pillars built on the notch piers.

(23) We now come to an altogether different type of construction for a fall, which is illustrated in Fig. 4. The conditions are identical with those in Fig. 3, and the weir wall and notch piers are exactly similar. The principle of the construction is to effect economy in the flank defences of the work by causing the earth to support the protective walls instead of *vice versâ*, so that masonry pitched slopes take the place of the down stream flank retaining walls. Up stream there are no water wings whatever. Their place is taken by a horizontal and vertical prolongation of the weir wall, forming single direct return flank walls on each side of the weir, the up stream canal side slopes running right up to them.

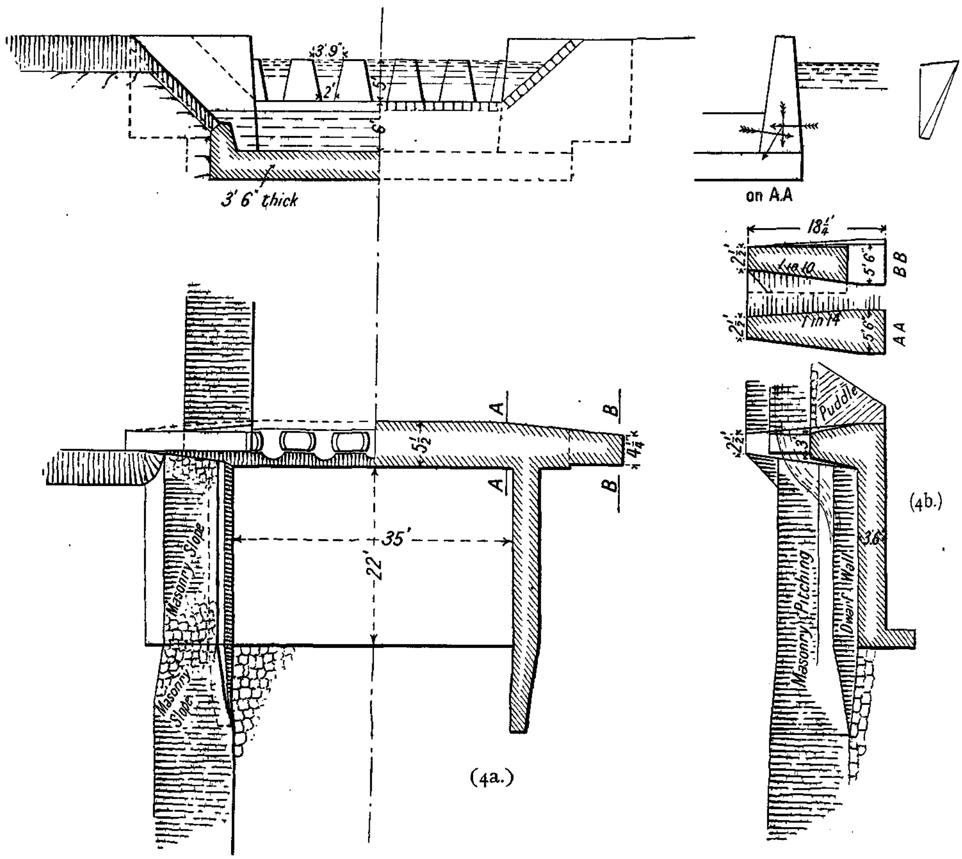
Down stream, the width of the floor is terminated on each side by a dwarf flank wall 3 feet high, which runs out with level crest to the end of the floor, whence the crest dips down at a slope of 2 to 1 to the down stream canal bed level.

Above this dwarf wall, a masonry slope rests on the side of the earth cutting at a slope of 1 to 1, and is carried on thus to the toe of the dwarf wall, where it merges with the dry-pitched canal slopes.

The raised flank continuations of the weir wall have their foundations stepped up.

These walls at their commencement are subjected to lateral water, as well as earth pressure, and so the section gradually diminishes in width till, at their ends, the earth pressure from the up stream side becomes very much reduced.

This is explained by the half-elevations and plans and the cross-sections on *AA* and *BB*.



FIGS. 4, 4a, 4b.—Notch Fall of "Wingless" Type.

(**24**) Excluding the weir wall and the visible portion of the floor, which is common to both Figs. 3 and 4, the quantities of masonry in the flank defences are as follows:—

Fig. 3 . . .	4,450 cubic feet.
Fig. 4 . . .	2,210 ,,

This shows the great economy effected in the wingless design. The deeper the fall the greater would be the saving effected.

In a very wide fall, however, of 200 to over 400 feet in length, the cost of the flank defences form so small a proportion of the total quantity of work, that the economy effected in percentage of the whole, would not be very considerable.

Where falls are narrow, deep and numerous, this type of design is eminently suitable, not only on canals, but in reservoir bye-washes, canal escapes, distributary falls, etc. The author has constructed several across rivers. One

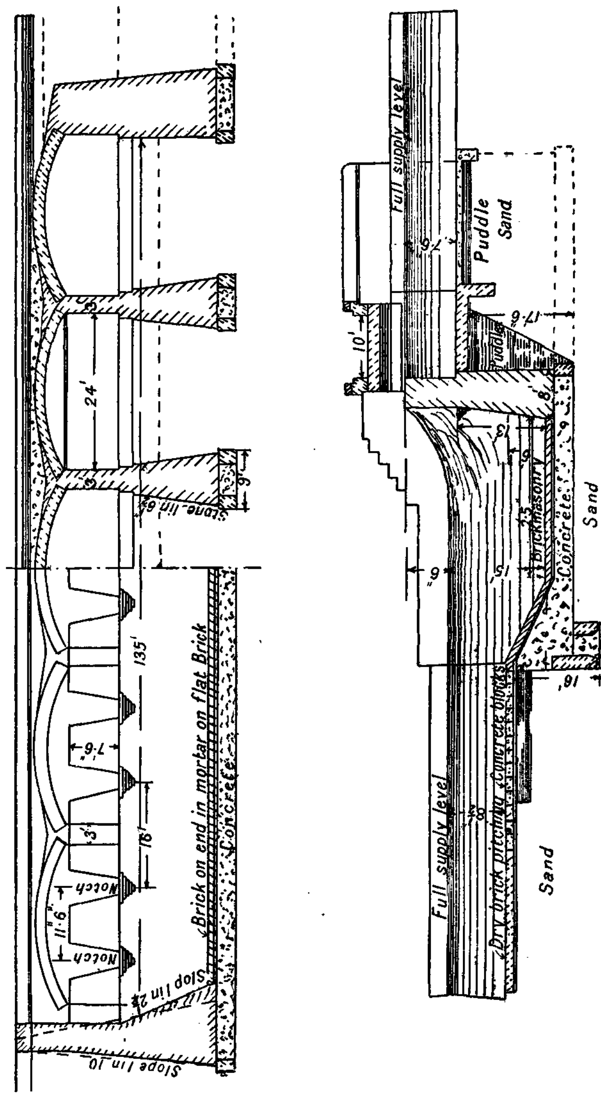


FIG. 5.—Fall on Jamrao Canal.

of these is illustrated in Fig. 24, Chap. I., and described in par. 69 of the same chapter. A photograph of it is given as Fig. 13 at end of this chapter.

(25) Fig. 5 is an example of a recently constructed canal fall on the Jamrao Canal, and is instructive as showing how a too close adherence to obsolete types results in a quite considerable waste of money.

The plans are taken from the Min. Pro. Inst. C.E., Vol. CLVII., p. 278.

The drop in the water surface is 6 feet, whereas that in the canal bed is 7 feet, the depth of full supply up stream being $7\frac{1}{2}$ feet, while that down stream is $8\frac{1}{2}$ feet.

This shows that a change of bed slope takes place at the fall, the slope down stream being flatter than that obtaining above the work.

(26) With the sole exception of the notches, the fall is a more or less close copy of the old and now obsolete Ganges Canal types.

In the first place, a water cushion of no less than 6 feet in depth is provided for a drop of nearly the same amount, and that not of a concentrated waterfall, but one split into jets 11 feet apart. The provision of any water cushion to a notch fall is deemed quite unnecessary, if the floor is properly constructed.

(27) A second point in the design, which is very open to objection, is the combination of an overbridge with the fall. This is also a relict of old times. Except for reasons of economy, there can be no possible object in combining these two works. It is exceedingly unlikely that a road-crossing for traffic purposes is required at the exact site of a fall, and even if it were so, a very much more economical plan would be to build the bridge clear of the other work.

From inspection of the drawing it is evident that this combination is productive, not of economy, but of the very reverse.

The bridge consists of wide spans of 24 feet. This in itself is an uneconomical arrangement, as high wide spans are only of practical use in a navigation canal. The foundations of the piers have to be carried right down to the base of that of the water cushion, *i.e.*, to a depth of $18\frac{1}{2}$ feet below canal bed level. Not more than a foot or two in width of the whole bridge receives any support from the weir wall, so that the combination is clearly only productive of waste. Were the bridge built as a separate work, it would, or should, consist of moderate spans of, say, 10 feet, with short piers 8 feet high only, resting on a floor platform which need not be more than $3\frac{1}{2}$ feet thick, enclosed between two lines of sheet piles. This construction would cost about half that of the present structure. In addition to all this, the presence of the bridge piers on its crest necessitates the lengthening of the weir by at least 40 feet, a further wholly gratuitous waste of material.

Another point involving a further unnecessary lengthening of the weir is the excessive length of the notch piers, which measure 6 feet long at the crest, whereas $4\frac{1}{2}$ feet or 5 feet would have been ample. As there are ten notches, this would add another 15 feet to the length of the weir wall, making it altogether about 55 feet longer than is necessary.

(28) The analysis of the section (Fig. 5) with regard to l, the length of enforced creep, and to t, the thickness of the floor, is as follows:—The

maximum head occurs when the canal is nearly empty, at which time only a small depth of water is passing down, just sufficient to cause hydrostatic pressure, but which will also fill the cistern. The head will therefore equal the drop, *i.e.*, 7 feet. The coefficient being 15, the value of l will be $7 \times 15 = 105$ feet. The actual length of creep is about 110 feet, so it is amply provided for in this respect.

With regard to the floor, supposing the cistern were empty, a possible contingency, the head acting on the floor will then be 13 feet, less some 4 feet absorbed, leaving 9 feet upward pressure. This is met by 4 feet of concrete of specific gravity $\rho - 1$, or unity. Consequently, if the cistern were empty it would be subject to the pressure of 5 feet of water unbalanced. The contingency of the cistern being empty is not a likely one, but the risk is

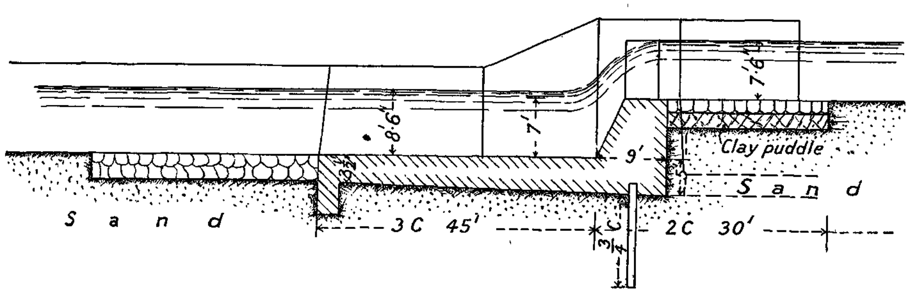


FIG. 6.—Alternative Design of Jamrao Canal Fall.

there, and this shows another of the disadvantages of the water cushion system. With a head of 7 feet, t should $= \frac{4}{3}\frac{H - h}{\rho - 1} = 4$ feet (par. 25, Chap. VI).

(29) An alternative section is given in Fig. 6. Here H as before is 7 feet, and a length of creep of $7c$, or of 105 feet, is required (par. 6, Chap. VI.). The minimum length of floor will be $2 (H + d) = 30$ feet. It will have, however, to be considerably lengthened beyond this dimension to afford a sufficient value for l. It will be made $3c$ or 45 feet long and be provided in addition with a curtain of sheet piling at the drop wall $11\frac{1}{2}$ feet deep, and another shallow concrete wall at the end of the apron. The rear sheet piling will have a value of $1\frac{1}{2}c$, and the fore of $\frac{1}{2}c$; this will make up altogether $5c$ leaving $2c$ for the rear apron. The thickness of the floor $= \frac{4}{3}\frac{H - h}{\rho - 1} = \frac{4}{3}\frac{7 - 3\frac{1}{2}}{1} = 4·6$ feet (par. 25, Chap. VI.). It will be made 5 feet thick at the heel of the drop wall, tapering to $3\frac{1}{2}$ feet at its extremity. By par. 20, Chap. IV., the approximate thickness would be $\sqrt{H + d}$, or 4·0 feet nearly. A heavy floor is always a desideratum in waterfalls of any kind.

The hybrid section should not be adopted for a weir wall on a pervious base, its effective weight being diminished by hydrostatic pressure. The base width will therefore be made 9 feet wide, *i.e.*, a little less than $= \frac{H + d}{\sqrt{\rho}}$ (ρ being $= 2$). The crest width is made 5 feet wide.

(**30**) The advantage of possessing definite rules of guidance applicable to the dimensions of every part of a structure, as also a method of testing the design with reference to foundation stability, if on sand, will be made apparent by the foregoing paragraphs.

Facility of method combined with certainty and precision are indispensable aids to successful design, and the author's endeavour throughout this work is to smooth the rough paths, and render the designing of works an easy as well as a certain matter.

The question often arises, in designing canal works, as to whether it is more economical to provide one deep fall or several small ones. As regards cost of masonry alone, ignoring that of the earthwork in the cutting, the advantage does not lie decidedly either way; but the wider the fall the greater will be the advantage, if any, of the deeper fall, the reason being that the cost of the flank defences bears a greater portion to the whole, in a narrow, than in a wide work. In a long weir the main expense is in the weir wall and floor. So, ignoring the flank defences, the sectional areas of the weir wall and floor alone for different depths of fall are given below. In every case the value of d or depth of film is taken as 5 feet; $H + d$, therefore, will equal $H + 5$ in every case:—

	Heights of Weir above Floor.					
	Feet. 5	Feet. 7½	Feet. 10	Feet. 12½	Feet. 15	Feet. 20
Sectional areas—						
Weir . .	19	45	53	72	101	155
Floor . .	64	88	117	147	180	250
Totals, sup. ft.	83	133	170	219	281	405

	Sect. area in sup. ft.		Sect. area in sup. ft.
Two 5 feet deep falls . .	166	Two 7½ feet falls	266
One 10 " " " . .	170	One 15 " "	281
Three 5 " " " . .	249	Two 10 " "	340
One 15 " " " . .	281	One 20 " "	403
Four 5 " " " . .	332		
One 20 " " " . .	403		

From the above it will be seen that the difference in quantities of weir plus floor is much the same, in the case of several small falls, or of one larger one, the advantage generally lying with the smaller falls.

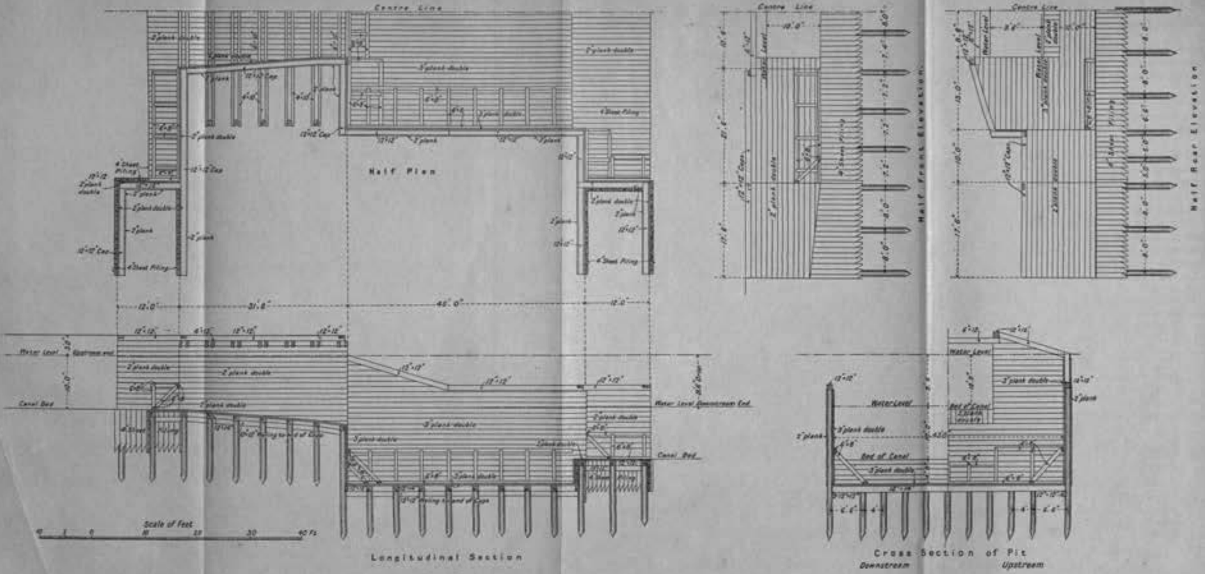


FIG. 7.—10-Foot Fall on Calgary Canal

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The comparative areas of the flank walls will throw the balance further in favour of the smaller falls. In a very wide weir, with adoption of the wingless type, the difference either way will not be very great.

In estimating the sectional areas of the weirs the base width is taken as that of the "Hybrid" section, viz., $\frac{H+d}{\rho}$ not $\frac{H+d}{\sqrt{\rho}}$, the thickness of the floor as $\sqrt{H+d}$, and the length $2(H+d)$.

(31) In the dry zone of Western Canada in the Province of Alberta, large canal systems are in process of formation, the principal of which is the Calgary Canal undertaken by that enterprising corporation the Canadian Pacific Railway. This canal, a short description of which is given in Chap. XIII., will, when fully developed, be by far the largest on the American continent; it is thus fit that the premier railway company of the whole world, should also lead the record in irrigation enterprise. The author has been privileged to obtain the blue prints of some of the principal works, and it will be interesting and instructive to reproduce some plans, particularly with regard to engineers in countries, possessing timber forests in abundance, where irrigation is practised. In India itself timber is scarce and expensive, but in Burma, which has an extensive dry zone, excellent timber, hard wood as well as teak, is available at moderate prices.

The first illustration, in Fig. 7, is of a 10 feet drop situated 15,000 feet from the canal head. The bed width here is 44 feet, side slopes 2 to 1; the average width of the water section will therefore be 64 feet, with a full supply depth of 10 feet.

In these falls the crest is not raised, as is commonly the practice wherever notch falls are not adopted, but on the contrary it is lowered 3 feet below the up stream canal bed grade, the approach being in the nature of a rapid; the crest itself is contracted to a width of 17 feet.

The width of the floor below the drop, 43 feet, is nearly equal to the bed width. A water cushion is provided 4 feet 8 inches deep.

By the arrangement above sketched, the overfall is concentrated in a narrow width and the velocity of approach largely augmented.

The scouring action of the falling water must therefore be accentuated much above what it would be, were the usual wide raised crest adopted.

Modern practice in canal falls is in favour of notch falls, in which type the current, instead of being concentrated in one powerful volume, is split up into several jets independent of each other. The entire crest width of the overfall, including the piers, does not usually exceed that of the normal canal bed width, if the latter has slopes of 1 to 1. However, with a water section having an exceptionally low ratio of $\frac{\text{base}}{\text{depth}}$ as 4·4, together with a high velocity of approach of 3·2 feet per second, the crest width of the fall should be greater than the bed width of the canal, say, more nearly equal to the average width of the water section, which is 64 feet. A photo of this work showing the cross bracing is given in Fig. 7a.

(32) The difficulty inherent in the case of wide canal works built of a so extremely unsuitable material as light pine wood, is that the post and plank wing walls, which support the earthen sides above and below the fall, require strutting across, for mutual support. In the small Western American canals, which are often not much larger than Indian distributaries, this can easily be effected. When, however, the same design is copied on a larger scale, the cross ties have to be made of built beams, and the expense is considerable. This must be one reason why the wide portion is limited to the part below the drop wall.

In this design the revetment walls of the water cushion, well, or pit are not shown as cross braced, but this had eventually to be done, as the pressure of

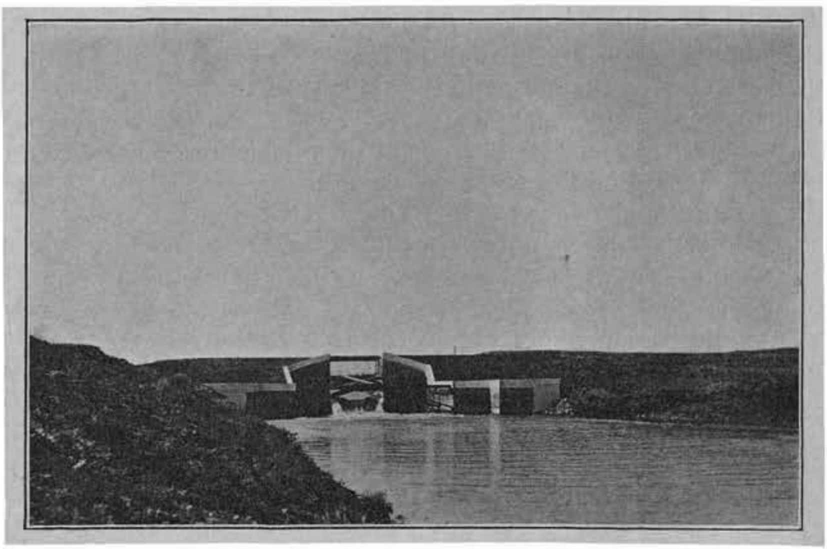

FIG. 7a.—10-Foot Fall on Calgary Canal, Alberta.

19 feet of earth would otherwise have soon overcome the slight resistance offered by the upright poles 5 feet apart.

This difficulty in connection with wooden revetment walls is easily solved by simply stepping outside the trammels of precedent, which seems to have such a hold on some engineers, not only here but in India, that is by abandoning the vertical walled lock, or box style of construction of falls, for an open one, and of substituting the much more reasonable system of allowing the earth sides to stand on a slope, instead of being held up vertically. These slopes, with the assistance of a little overlay of riprap, will stand permanently at a much steeper inclination than the angle of repose of the material. The matter in question has already been gone into in Chap. I.

This method of construction, which is the same as that exemplified in Fig. 4, will be found much cheaper, the saving in woodwork being very considerable; as a set-off against which is the cost of stone filling and of pitching banks and of a puddled clay wall in rear of the main cross drop wall, which latter with wooden structures is deemed an essential feature. This canal has the great advantage of a subsoil consisting of glacial detritus in which large quantities of stones and gravel are mixed up with the clay. This valuable material could easily have been separated from the earth whenever found in cuttings, and stacked ready for use. Safeguarded from erosion by light crib work, stone forms an excellent and effective protection to earthen beds, or banks against water scour.

(33) An alternative design on these lines is given in Fig. 8. The width of crest adopted is 60 feet, which dimension, if anything, errs in excess. The up-stream slopes are steepened from 2 to 1 to $1\frac{1}{2}$ to 1, at which they will easily stand with the possible addition of a 3-inch layer of loose stone. The down-stream side slopes are made such that they will intersect the wing crests at the same point as the up-stream slopes; they will thus stand at ·87 to 1, *i.e.*, a little steeper than 1 to 1. Considering that revetted slopes stand well at $\frac{1}{2}$ to 1, this inclination is by no means impracticable. This slope is revetted with stone $2\frac{1}{2}$ feet thick at the base, tapering to 1 foot at the crest, and the pitching is continued up to the end of the down-stream floor. From this point down, it will widen out to the normal 2 to 1. The drop wall itself consists simply of a single wooden double-planked core wall.

The planks are placed inside the posts, which are spaced 6 feet apart. Another row of posts or piles is placed 15 feet in rear of the face wall, connected to it by sills at the top. No other cross connection is permitted, as otherwise leakage through the puddle wall, contained between the rows of posts, would be encouraged, and the proper consolidation of the clay interfered with. With regard to the wings, both sides are planked, enclosing the puddle core. This latter will be carried down in a trench to a depth of 2 feet below the top of the sheet piling, which extends all along below floor level, stepping up in the wings. The planking is double 3-inch, except in the rear of the wings where it is double 2-inch.

The water cushion is abolished altogether; it adds greatly to the expense of the work and is not considered to be at all necessary with a notch fall. When full supply, or 10 feet of water, passes over the drop it will fall into an equal depth of water. The greatest strain on the floor will be produced if 2 or 3 feet in depth were let down with the channel below empty, which eventuality is amply provided against by the cribbed riprap or filling. The floor is composed of $4\frac{1}{2}$ feet of loose stone or rock, partly covered in by 3-inch planks spiked or screwed down to walings, which in turn are spiked sideways to the piles. These latter are spaced in 6 feet intervals in either sense.

The walings are not secured on top of the posts as sills, which is generally the best arrangement, but to the sides in order that they should form easy connection with the outside piles or posts of the drop wall.

The last two rows of piles are raised 4 feet above the floor, to form a stop wall. This will be filled up with loose rock, so as to allow free percolation; the stop wall has the effect of retaining a greater depth of water over the floor than is below it, and has been found to work well in practice. The revetted side slopes are protected by planking spiked to walings, connecting the inner row of piles in the pit with a further row driven into the slope at a higher level. This protection is only for a short distance, within the range of the falling water.

Considering that canal works constructed of so perishable a material as soft

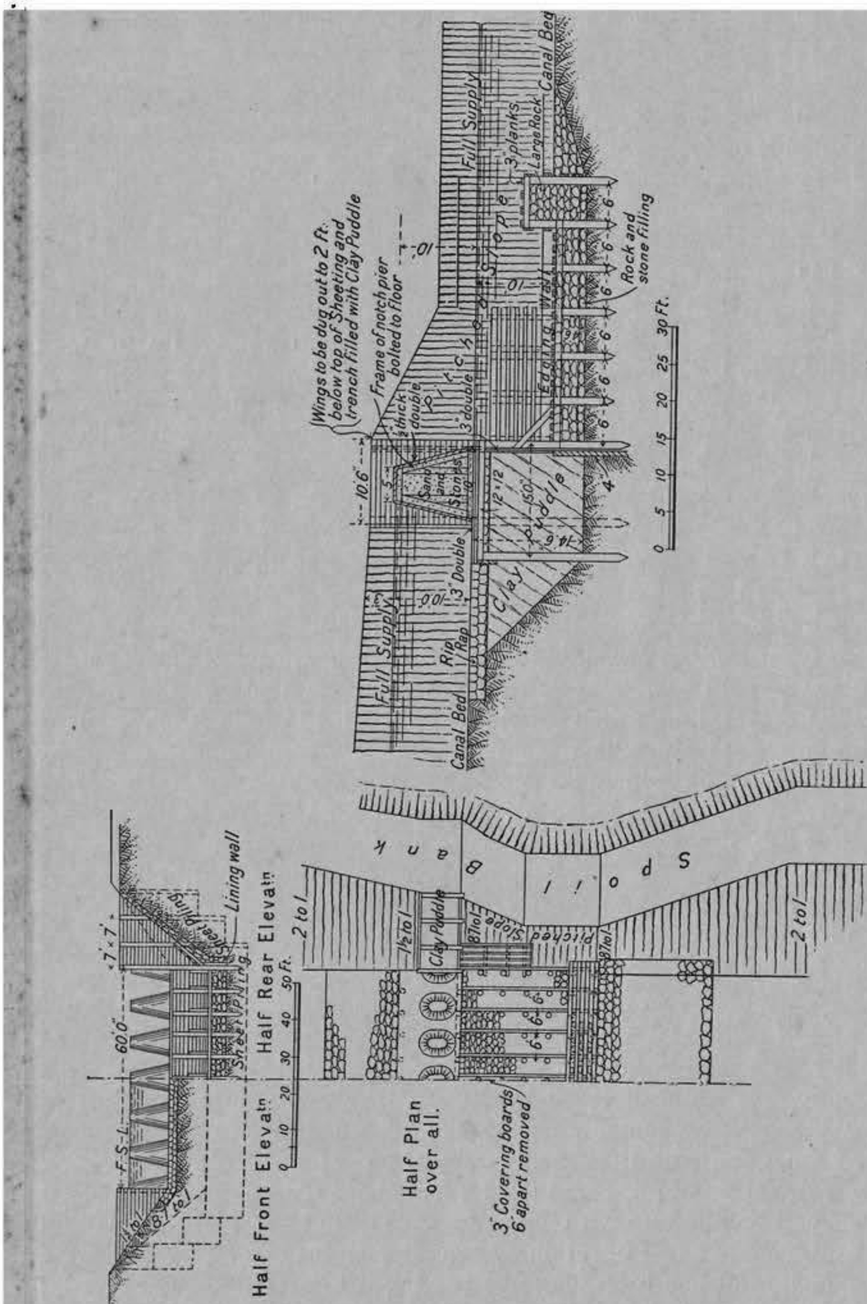

FIG. 8.—Alternative Design (Wingless Type).

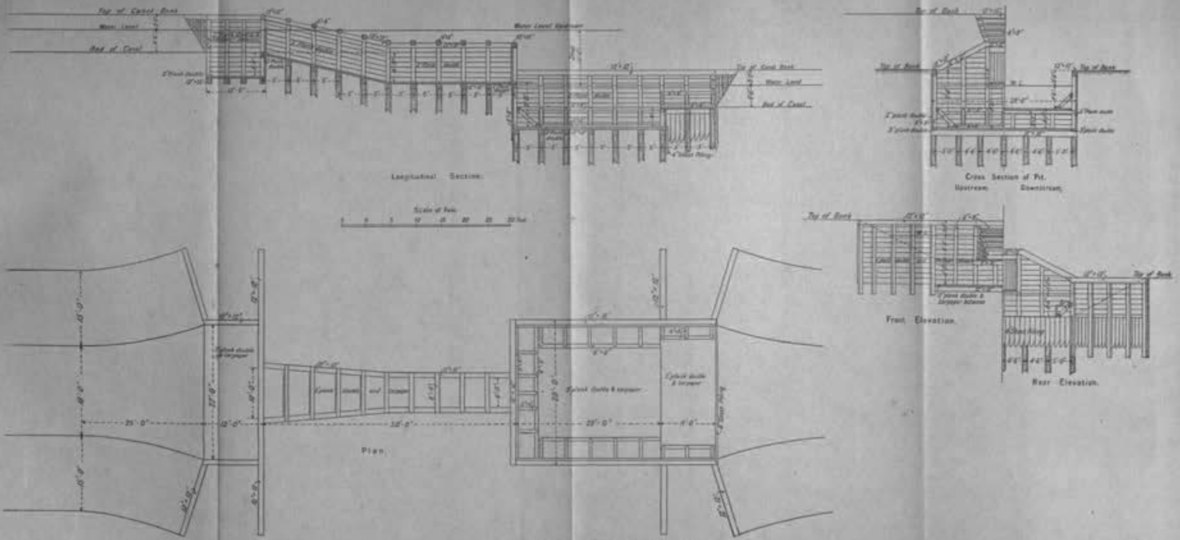

FIG. 9.—Combined Rapid and Fall of 11 Feet, Calgary Canal.

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wood can only be temporary, it is essential that they should be as economical as is consistent with safety. The amount of material in Fig. 7 (44 feet wide) is, piling 4,070 lineal feet, woodwork generally, 116,756 feet (board measure). (Board measure is cubic feet multiplied by 12.)

In Fig. 8, with a 60 feet width of crest, the corresponding quantities are: piling 1,380, woodwork generally, 46,400 B.M., plus stone filling, rip-rap and puddle. Two drops of 5 feet would probably be cheaper than one of 10 feet, and certainly much safer. The notches, it may be added, are wooden frames covered with bent inch boarding, caulked, the hollow filled up with sand or small stones, and the frame bolted down to the upper floor crest of the fall.

The cost of the stonework cannot be estimated; it will however, not nearly equal that of the woodwork saved, which costs $47 per 1,000 feet B.M.

(34) Another fall on this canal is exhibited in Figs. 9 and 9a of an 11-feet drop in a secondary canal, which has a bed width of 18 feet.

This consists, like the last, of a narrow rapid followed by a laterally restricted fall, but exaggerated. It has the peculiarity of a kind of pen-stock or pit at the head of the rapid, the object of which is to hold up the water level and so limit the velocity of approach.

It is considered that this design would be improved if it consisted of two distinct falls of 5½ feet on the same principles as Fig. 8.

This has been worked out in Fig. 10.

The quantities of woodwork are as follows:—

	Piles, foot lineal.	Woodwork.	Fall.
Fig. 9 . .	1,620	54,160	11 ft.
Fig. 10 (doubled)	1,200	15,400	two of 5½ ft.

The puddle mentioned need not be brick clay, but any soil that would be deemed suitable for the embankment of a reservoir, and should be thrown in thoroughly wetted when, like hydraulic fill, it will settle down into a solid and impervious mass. If this is carefully done no better stanching material could be found; all it requires is protection from the erosive action of water, which is amply secured by the plank sheathing.

(35) Nature having provided earth and stones close to hand, it is desirable that their qualities should be utilised to the utmost, with a saving of the perishable wood. When permanent works of masonry are substituted for these frail and temporary structures of soft wood and tar paper, the stone used in the filling and revetted slopes will be the only material remaining to be again made use of. Engineers accustomed to work in wood do not take kindly to masonry structures. This was particularly noticeable in Burma, where the old "wood and mud" Engineers of the lower Province, before the expansion took place, had a holy horror of bricks and mortar; the reason is not far to seek. Timber structures are quickly and easily put up and require little supervision, whereas in masonry and concrete work, the arrangements for and collection of, mortar, stone, brick and cement, give a great deal of trouble and require careful and thorough supervision; the responsibility therefore is much greater.

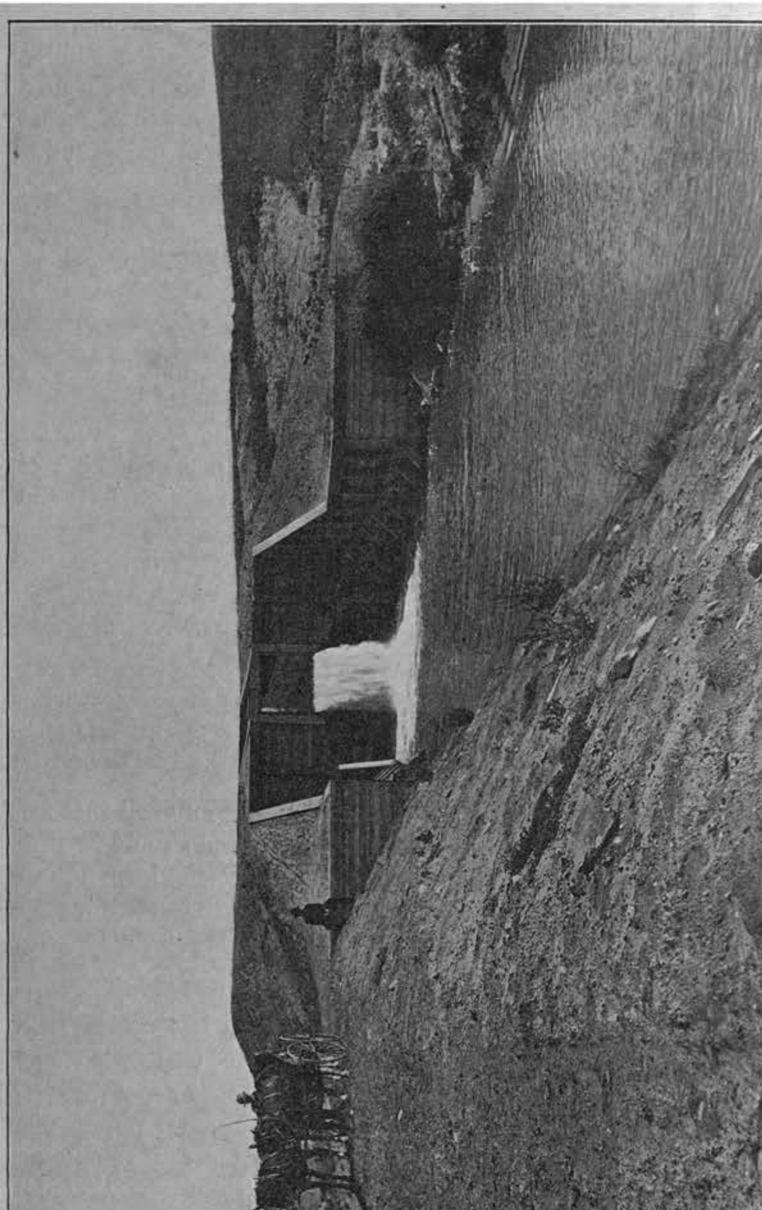


FIG. 9a.—Combined Fall and Rapid, Calgary Canal.

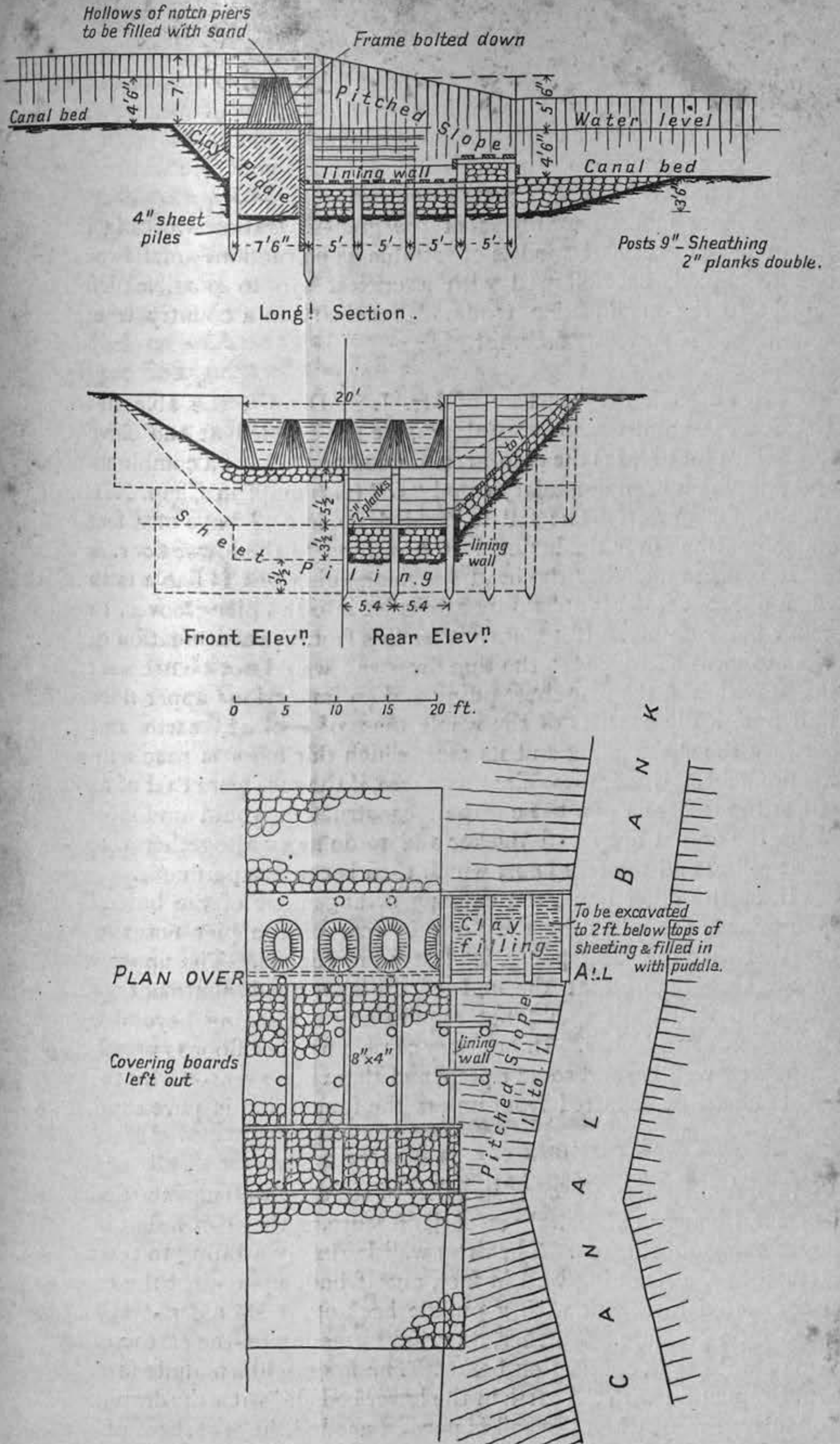


Fig. 10.—Design of $5\frac{1}{2}$-foot Notch Fall (Wingless Type).

In the States the idea has gained ground that if a structure be of reinforced concrete it must be suitable in point of cost as well as efficiency. This is by no means the case, as has been proved in several instances taken up in this work. To copy wooden structures in reinforced concrete will generally be found on analysis to be very poor Engineering.

The more modern examples of the great irrigation works in India and Egypt afford excellent models of suitable construction, not, however, to be blindly copied, but followed with a critical eye to possible defects, and subject to the modification rendered necessary in a country where labour conditions are so vastly different.

(36) Through the courtesy of Mr. J. S. Dennis, the able head of the Irrigation Administration at Calgary, we are enabled at the eleventh hour to produce (in Fig. 11) the plan in reinforced concrete of a combined regulator and fall which is on secondary canal "*C*" (*vide* map) in Chap. XIII., Fig. 2. The drop is $5\frac{1}{2}$ feet. The fall is divided up into $7\frac{1}{4}$ bays of 4 feet by piers, which continue over the inclined drop wall on to the lower floor.

It is presumed that the head to which this work is liable is that acting when 6 feet of water is upheld by baulks let into the pier grooves, the channel below being empty. If it were not for this fact, the continuation of the piers so as to form buttresses to the thin drop wall would not be necessary. With the provision of the rear sheet piling and an impervious upper floor, the drop wall practically consists of the whole mass of enclosed earth and concrete between the sheet piling and its face, which thickness is much in excess of requirement. Under these circumstances, if the case were that of an ordinary fall, without regulation, the cheapest construction would undoubtedly be to make the actual drop wall thicker and to do away altogether with the rear sheet piling and the rear floor, which then become superfluous.

If, on the other hand, the existing arrangement of the buttress piers is maintained, the whole of the floor in rear of the pier noses marked *A*, together with the sheet piling, is clearly superfluous. The upper wing walls would then take off at the end of the shortened abutments, *i.e.*, at *A*. The drop wall itself would not require any thickening beyond its present dimension of 18 inches. In ordinary cases, no upper floor is provided in rear of the weir wall beyond some riprap, and there is no reason why this arrangement should be departed from unless the foundation is pure sand, which is not the case here.

(37) A fall practically consists solely of the drop wall and the floor, the side wings not being an absolutely essential feature, as they can be left out, pitched slopes being substituted. The drop wall is simply a lining to the step in the earthen bed, a retaining wall in fact, and if built up against the original clay or connected therewith with a puddle backing, it should not be subject to hydrostatic pressure. Further, its weight pressing on the earthen foundation forms a watertight connection below. The floor in like manner forms a watertight connection with the earth in the lower bed and with the drop wall. Consequently, except when the soil is porous, as sand, lines of sheet piling in rear

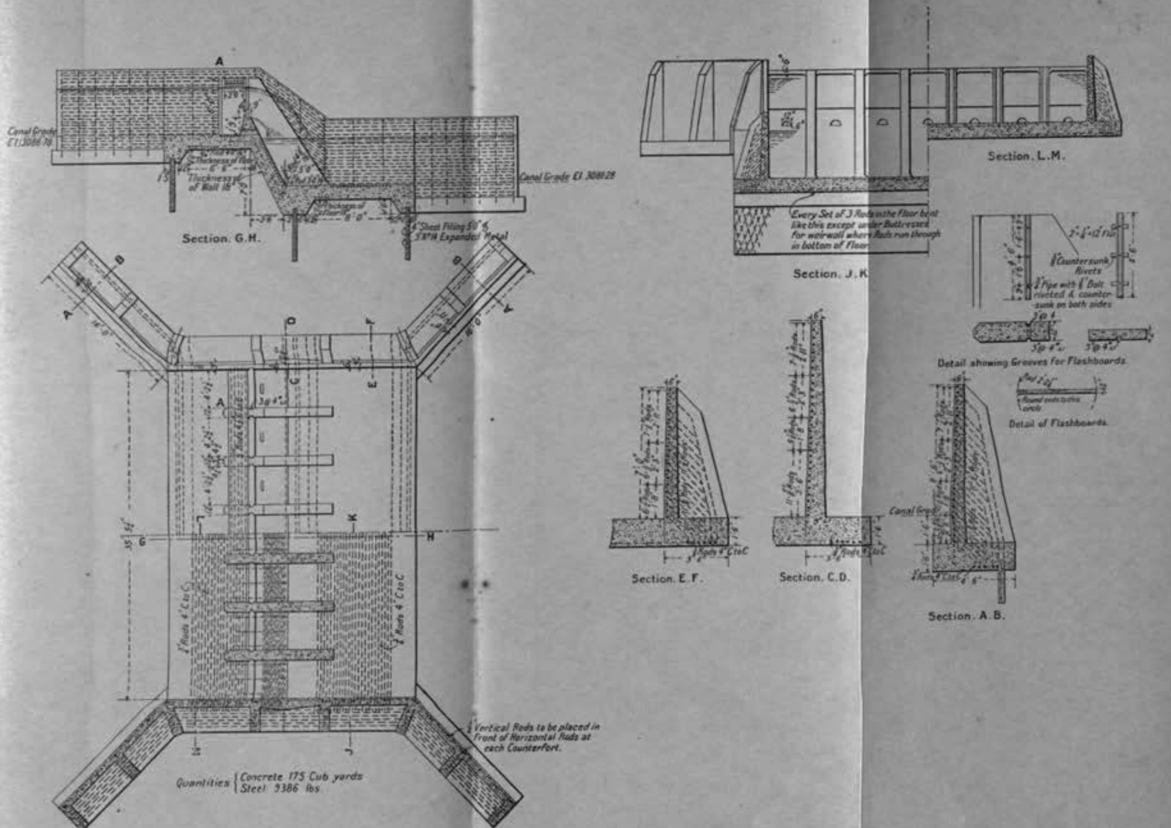
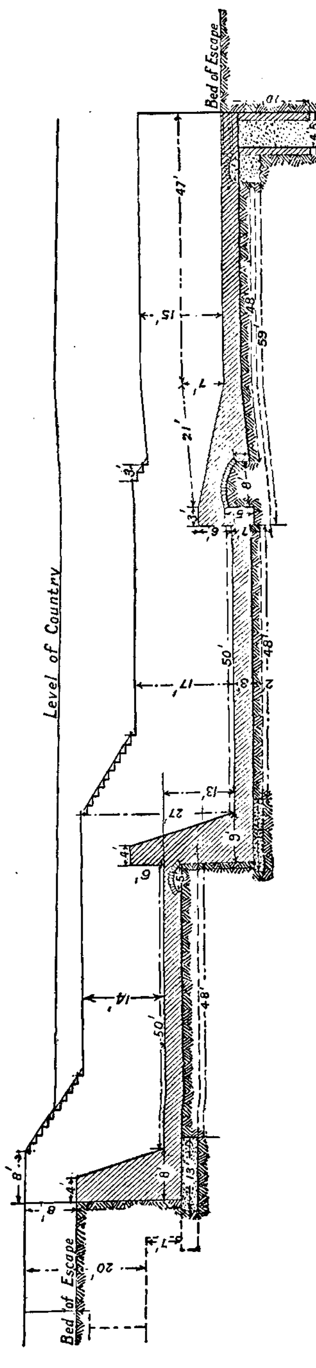


FIG. 11.—Reinforced Concrete Fall of 5½ feet, Calgary Canal.

[To face p. 290.

of the drop wall or below the foundation of the lower floor are not at all necessary; and in fact never are employed in any Indian works of this character.

A good design in a light material as timber may be quite unsuitable when a heavier material is employed. The plan under consideration adheres, it is contended, too closely to precedents of timber construction. It should be borne in mind that in hydraulic works sheer weight is always a desideratum.



In reinforced concrete work, weight is reduced as much as possible to save expense, and it will often be found that it is cheaper to employ the plain concrete of larger dimensions than reinforced concrete.

If a floor is reinforced at all, it is considered it should be with barbed wire, or wire netting, not with rods. It should not be exposed to tension, and if it rests on solid ground it will not be so.

It is considered that this floor should be made at least 3 feet thick, the lower half being of closely set riprap covered with 18 inches of plain cement concrete, the length also to be increased by half as much again by heavy riprap, the wings remaining the same.

The remarks regarding the superfluity of the sheet piling do not apply to that underneath the up-stream wings. In most falls, the foundation of these wings is carried down to the lower foundation level, and this can be most economically effected by sheeting piles forming the lower 5 feet. This line of sheeting should be carried under the abutments, and connect up with the drop wall curtain. The drop wall might be made vertical without disadvantage, with a battered face.

The piling shown at the termination of the down-stream floor is a useful, though not an indispensable adjunct, provided this floor were lengthened in concrete to about the same extent, as the upper floor is recommended to be shortened. The suggested alterations amount in fact to a redistribution of much the same material.

The panel counterfort retaining walls are an excellent feature, and effect considerable economy in section. It is here that reinforced concrete can, it is considered, be most suitably employed, not in the drop wall or floor.

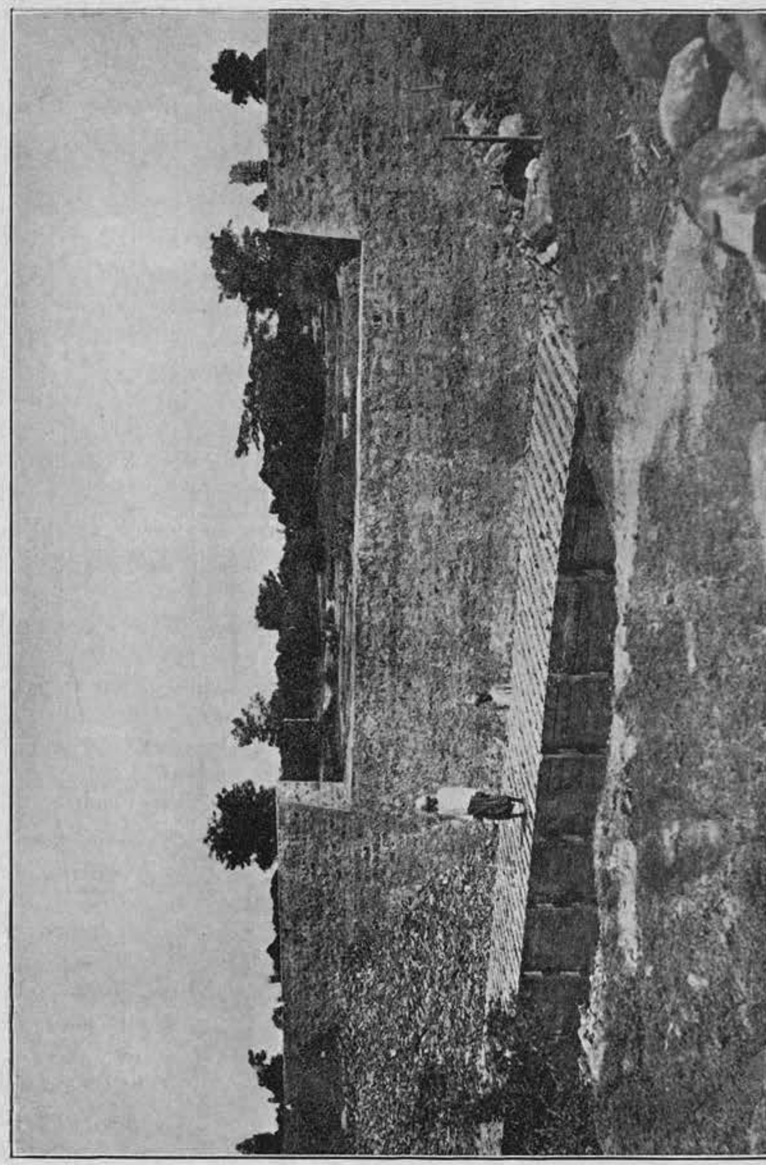


FIG. 13.—View of a "Wingless" Fall of 14 feet (Burma).

(38) A section of a canal escape fall ladder without notches which is worthy of attention is that of the Kushak Falls, Agra Canal, given in Fig. 12. The discharge of the escape, its bed width and other essential particulars are, as is so often the case, entirely wanting. Fig. 13 is a view of the "Wingless" Fall—drawn in Fig. 24, Chap. I.

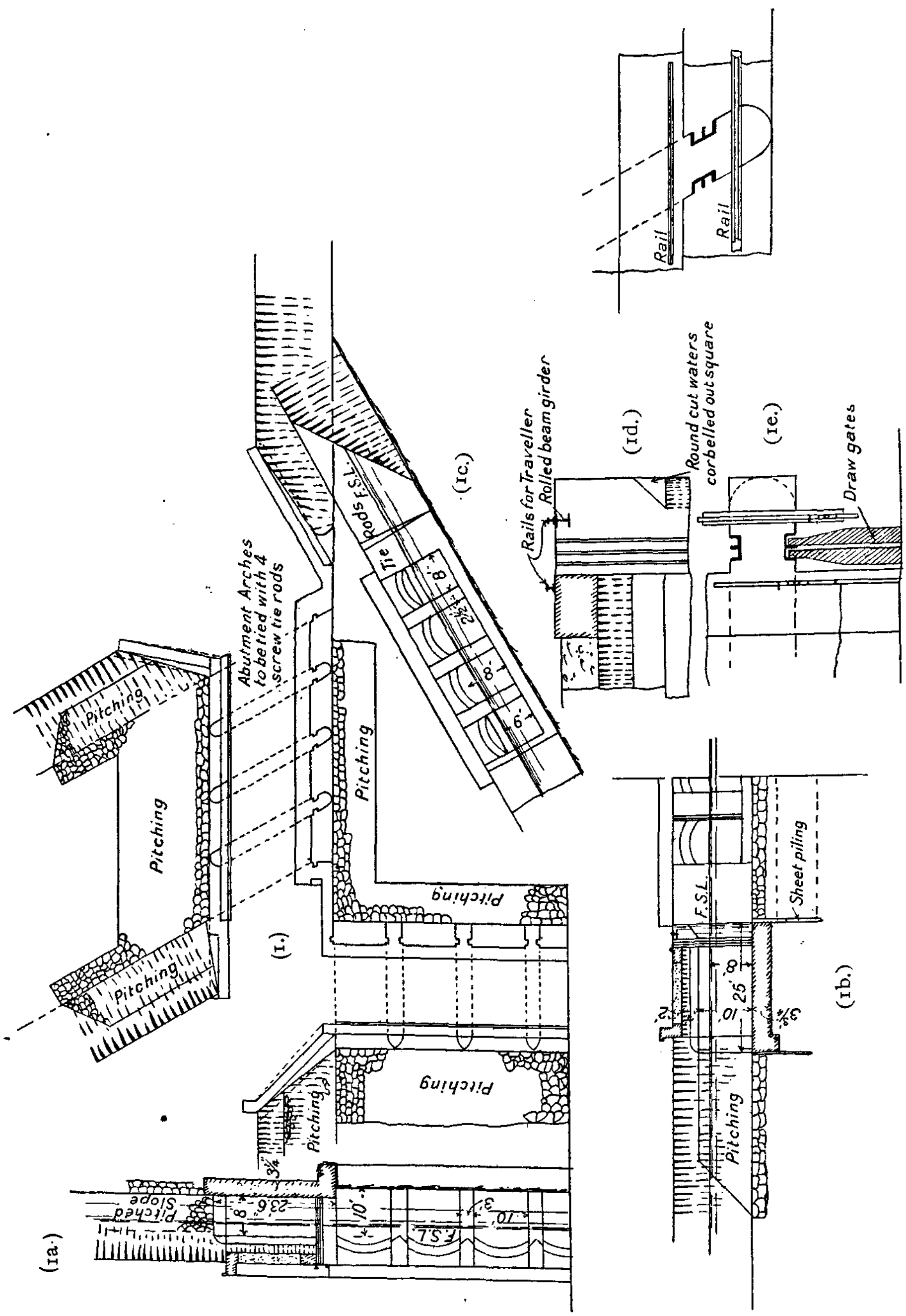
CHAPTER X

CANAL REGULATION BRIDGES AND ESCAPE HEADS

(1) In places on a canal where an escape or a branch takes off, a regulating bridge across both works is generally necessary. Distributaries very frequently take off above falls, and the two works are often combined. No object, however, is gained by placing the headwork of the branch directly at the fall and so amalgamating them into one, a better arrangement is to make the branch take off separately, higher up the canal. The regulation of falls by means of a superimposed bridge with shutters or baulks which drop on the weir, is hardly practicable where notch falls are concerned, and the system is generally to be deprecated as interfering with the regimen of the canal. Skew heads for branches have a decided advantage, or, as the question of silt deposit is not so pressing as in the case of head regulators on a river bank, the branch could leave the main canal at an angle, the head being recessed so as to admit of the spans being square to the axis of the branch. This arrangement, though less expensive, is not so good as a skew head, with the piers run forward to the canal side slope.

(2) The principles governing the design of canal regulation bridges are not quite identical with those which affect that of canal heads. In the latter, the object of the design is to enable the head work, if so required, either to completely cut off all ingress of the water from the river into the canal, or else to admit only as much as may be desirable.

The same functions have to be performed by the canal regulation bridges with this difference, that the maximum head of water dealt with is moderate in depth, seldom exceeding 10 feet at the outside, whereas in a river canal head the flood water may rise to anything up to, say, 30 feet above floor level. This fact modifies the style of design. The depressed arch, or a double series of arches at two levels, the lower carrying a breast wall, designs which are suitable for the latter works, need not be employed in the former. The springing of the arch, of which only one high level series is required, can be placed some short distance above the maximum water level. Within certain limits it is generally immaterial as regards cost, whether the arch be raised above full supply level or be depressed to that level or below it, the additional height of the piers in the one instance being nearly balanced in area by the greater depth given to the breast or flank walls. The depth of water to be regulated being small, the gates themselves can be made high enough to exclude the water without there having to be a watertight connection between the top of the gates and the face of the arch, the water being held up by the gates alone, irrespective of the spandrel walls above.



FIGS. 1, 1a, 1b, 1c, 1d, 1e.—Design for Regulator and Branch Head.

(3) A design for a regulation bridge and branch head is given in Fig. 1. In this case the water in the canal is assumed to be 8 feet deep and that in

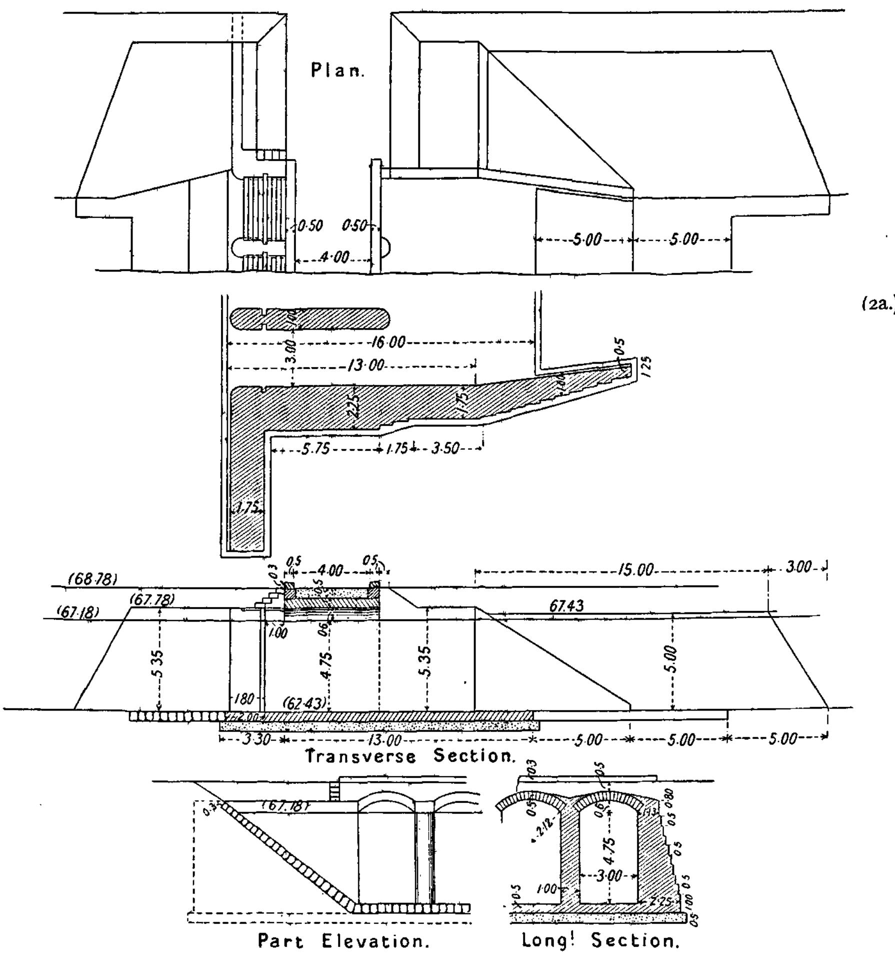
the branch 6 feet, the floor of the latter being placed 2 feet above that of the main channel. The banks, with cutting and spoil, are assumed to be 15 feet above canal bed level, and the bridge roadway or platform is placed at this level. The spring of the arches is fixed 2 feet above F.S.L., which is the same in the branch as in the canal. The space of 7 feet between F.S.L. and the road level is useful for hanging the gates when drawn up clear of the water line. Some space between the F.S.L. and the crown of the arches is also deemed advisable to allow of row boats passing underneath. In the canal regulation bridge the spans are made 10 feet. Large spans with high, heavy arches are quite out of place in irrigation canals, where headway need not be provided for laden barges. Most of the examples of existing canal regulators given in Mr. Buckley's "The Irrigation Works of India" are of this latter type, which it would be folly to imitate in a purely irrigation canal. The openings in the skew branch head, which takes off at an angle of 60° with the axis of the main canal, are made 8 feet wide, which will give an oblique span of about 9 feet. The skew arches have the same rise and thickness as in the canal regulator.

(4) The disposition of the wing walls in the combined work is arranged so as to allow of a suitable approach to the bridges from the canal bank road on either side, as well as from the country at the rear. The connection between the two works is formed by a short continuation of the abutment of the canal regulator, which joins on to the end of the left skew abutment of the branch. This is the only vertical retaining wall in the design. The width of the roadway between parapets is 12 feet, which is the least dimension suitable for cart traffic. This makes the length of the piers and the floor in both cases about 24 feet. The head of water being only 8 feet, the proportional length of the masonry floor comes to $3H = 24$ feet, or measuring from the grooves, over $2H$, so that the width of the masonry floor is, if anything, in excess of requirements, and need not be extended beyond the down stream pier noses. The thickness of the floor is $\sqrt{H} = \sqrt{8} = 2{\cdot}8$ feet $+ \dfrac{S}{10}$ $= 3{\cdot}8$ feet in the main regulator, and $\sqrt{6} + \dfrac{S}{10} = 2{\cdot}5 + {\cdot}8 = 3\frac{1}{4}$ feet in the branch, the subsoil being assumed to be firm clay. H is the depth of full supply above floor, and S the span. The addition of $\dfrac{S}{10}$ is for the piers, to allow increased depth for distribution of weight, which answers well in practice.

(5) The gates will be lifted by a travelling winch, which will run on rails clear of the roadway. One rail will rest on the dwarf parapet wall provided for that purpose, and the other will be bolted on to a rolled beam some 9 inches deep built into the top of the up stream cut-waters, which for this purpose are continued up to road level. The detail of this arrangement is shown on a larger scale in Figs. 1d and 1e. The gates, which only require

a watertight connection between each other, will be in plan as shown in Fig. 1e. They will, as a matter of course, be fitted with roller attachments.

(6) The arrangement of the pitching below the two works is shown in Figs. 1, 1a and 1b. That on the canal slopes is taken 2 feet above the water line, and terminates in a slope of 45°. Where the sloped pitching projects



Figs. 2, 2a, 2b, 2c.—Raswaniya Regulator (Egypt).

beyond that in the canal bed, a narrow slip of horizontal pitching extends on either side to form a foundation for the toe of the slope.

(7) Fig. 2 is a presentment of the Raswaniya Regulator on the canal of that name in Lower Egypt. With the exception of the down-stream wings, the alignment of which presents some novelty, the design is but a copy of the old Ganges Canal types.

The following points are noted :—

(1) Considering the depth of water, viz., over 15 feet, the spans are very narrow.

The width of openings, except in the case of weir sluice heads, should not as a rule be much less than the depth of water, and as the regulation of this work is believed to be only partial, spans of 4 or $4\frac{1}{2}$ metres, *i.e.*, 13 to 15 feet, could well have been adopted. The regulation is effected by the old primitive method of wooden sleepers or baulks dropped into grooves cut in the brick piers, which require a large and experienced gang of men to properly manipulate.

(2) In conformity with the practice in Upper India, the wing walls are vertical in section, which is the least economical form. The direct return up-stream wings are likewise an unsuitable disposition unless for a head regulator taking off from a river bank, which is apparently not the case here. Reference to the elevation in Fig. 2 will demonstrate how injuriously the sudden narrowing of the water way, due to the abrupt entire suppression of the side slopes, must affect the current. A good arrangement of the water wings will, on the other hand, ensure as gradual a change as possible in the sectional area of the water way, and this can only be effected by either adopting splayed walls which gradually absorb the side slopes, or else a direct continuation of the abutments by means of a sloping crested wall, similar to what is adopted down stream in this design, but parallel to the axis of the canal, which type of water wing effects a similar gradual widening, but in a different manner.

(3) The down-stream wings in this work converge on plan, the object being to force the current into the centre of the water way. It is believed that this device will defeat the very object which it is intended to secure. The contracting of the water way will tend to cause the current, on being released from the restraint of the wings, to spread out on both sides, as water flowing through an adjutage undoubtedly does.

A good point in the design is the arrangement of the pitching on the slopes, which is as it should be.

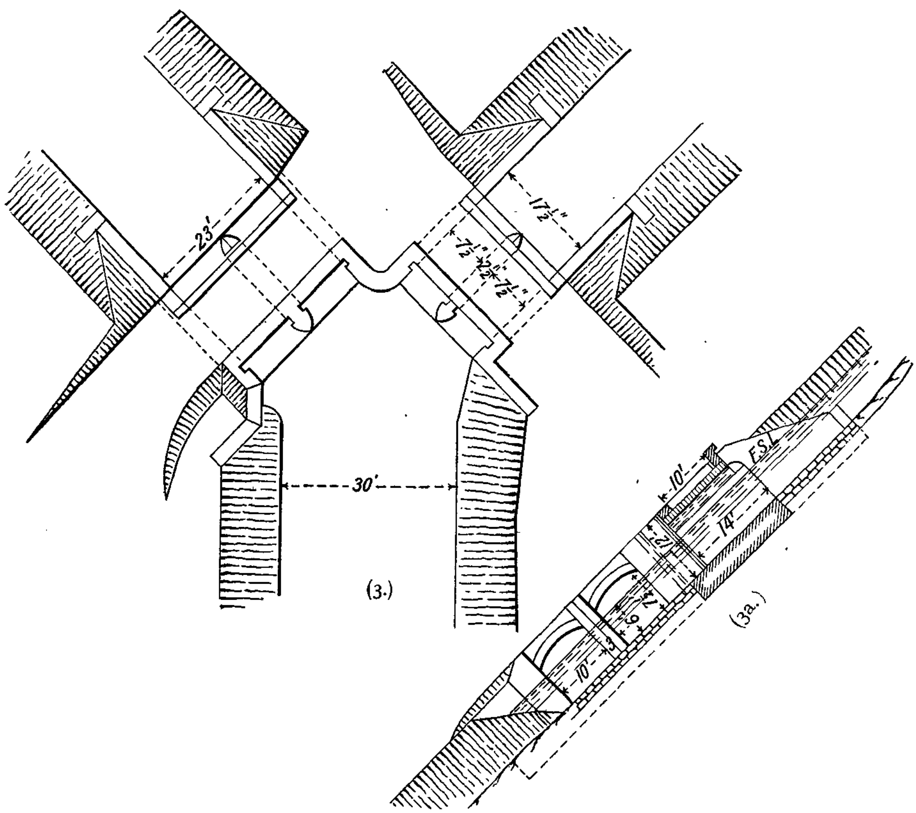
(4) The floor of the work is only 1 metre or $3\frac{1}{4}$ feet thick. This is rather too light.

The floor should have a thickness of $\sqrt{H}$ or 3·8 feet; if we add to this $\frac{S}{10}$ or 1 foot, the total depth will be 4·8 feet. The length of the floor is, if anything, in excess of requirements.

(8) Fig. 3 is a design of a bifurcation. The depth of water is taken as 6 feet in the canal and also in the two branches. It does not call for much comment; the wing walls have been shown as vertical in order not to complicate the sketch unnecessarily; they should certainly be battered in face in an actual case. A design for a regulator without bridge to pass traffic is given in the next section on Canal Escape Heads (Fig. 5).

Canal Escape Heads.

(9) An escape leading out of a canal within one or two miles of the head is an indispensable adjunct wherever silt deposit threatens to block the water way. In addition to this cause, several similar works are required on a canal of any considerable length. Notwithstanding the fact that the admission of water at the head is under complete control, still the supply on some of the lower reaches may be in excess of requirements, owing to heavy local rain, which would stop cultivators from using the canal water. For this



FIGS. 3, 3a.—Design for Bifurcation.

reason escapes in suitable positions, whereby the surplus supply can be got rid of, are necessary.

(10) An escape head should, if practicable, be combined with a fall, as a less width of water way will then be requisite, diminishing thus the cost of the work and the number of bays to be worked. The design will then be a combination of a canal regulation bridge and a fall.

The position of an escape head should be as close as possible to a drainage line, so as to diminish the length of the excavation in the channel. As a rule, several falls will have to be constructed along its course, in order to overcome the difference of level between the bed of the canal and that of the river or stream into which the escape channel will tail.

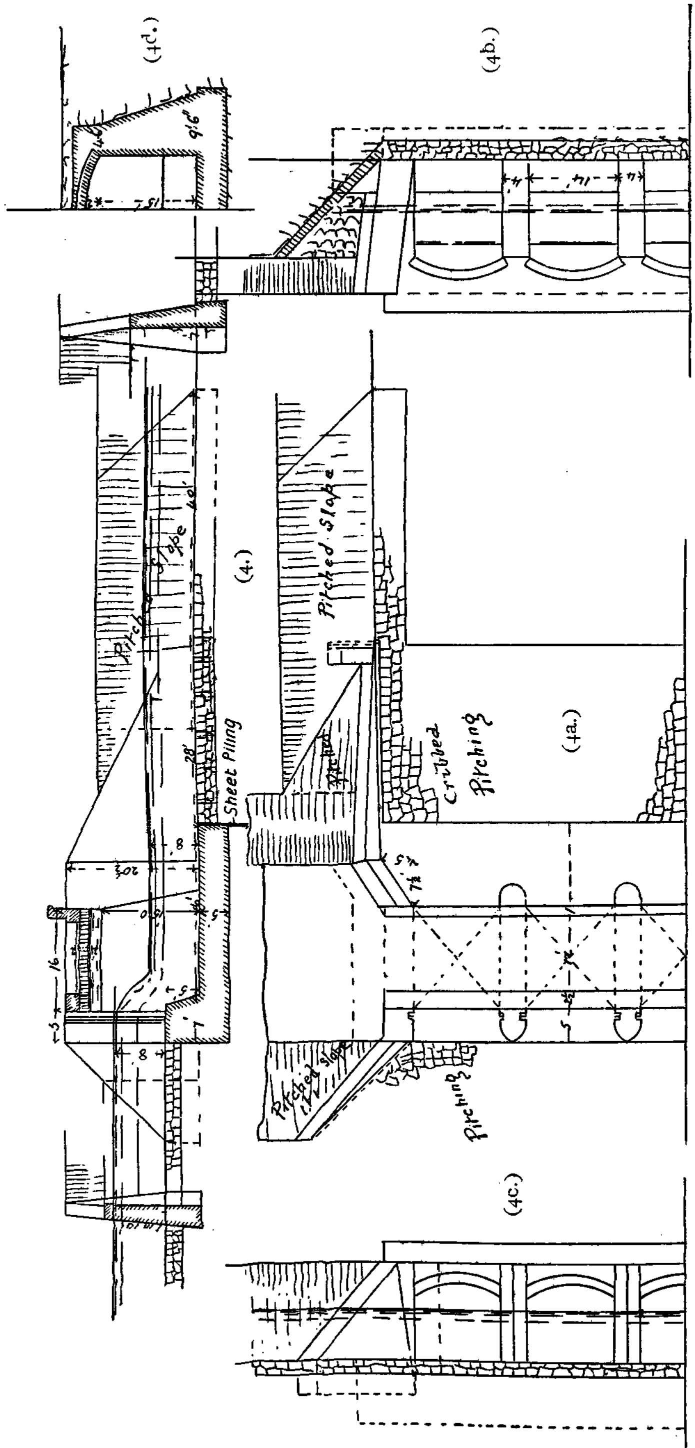
Canal aqueducts crossing drainage lines can sometimes be utilised with advantage to pass off surplus water. An example of such a work is given in Fig. 7, to which reference will subsequently be made.

(11) Fig. 4 is a design for a canal escape head under the following assumed conditions:—

The bed width of the canal is 100 feet, side slopes 1 to 1, the depth of water, full supply, is 8 feet, and mean velocity is $2\frac{1}{2}$ cubic feet per second, having a discharge of 2,160 cubic feet. Supposing the silt deposit to be of so heavy a nature that it will take a mean velocity of above 5 feet per second to scour it, then the escape water way will have to be designed of such a width as to induce this velocity in the approach channel, when running full supply.

The site of the escape head is assumed at 5,000 feet distant from the canal head, with a free overfall, the crest of which is flush with the canal bed. The slope of the canal bed is estimated to be close upon ·1 per 1,000 feet (with $n = \cdot 0225$), hence the drop in level, from the head sluice floor to the escape, will be ·5 feet. Increasing the velocity from $2\frac{1}{2}$ to 5 feet per second will nearly double the discharge. It is, however, presumed that the maximum supply through the head regulator is limited to 2,500 cubic feet, *i.e.*, about 25 per cent. in excess of full canal supply. The head being only designed to pass 2,160 cubic feet through its openings with a depth of 8 feet, in order to accommodate the additional quantity there will have to be a less depth of water below the work than above it—in fact, a fall. It is presumed that this diminished depth will not exceed 7 feet, giving a fall through the regulator of about 1 foot.

(12) The effective length of the escape weir will be tentatively assumed at 100 feet, equal to the canal bed width. The discharge passing over being, as before stated, limited to 2,500 cubic feet, that per foot run will be 25 cubic feet per second. We now require to find out what will be the depth of film over the crest to discharge 25 cubic feet with a velocity of approach of between 5 and 6 feet per second. According to Tables of discharges of overfalls (Table II., Chap. V.), the required depth is at once found to be $3\frac{1}{2}$ feet. Thus the surface level at the escape will be 3 feet above the head regulator floor. We now have to ascertain what surface slope will produce a velocity of 5 feet per second at an average depth of, say, 5 feet in the channel. With this depth R is estimated to be 4·6. Taking n, the coefficient of rugosity, as ·250 (it might well be assumed higher for silt-laden water), the coefficient c, suitable for a value of $R = 4{\cdot}6$ (and among high values of S as ·7 to 1·0 per 1,000) is ·78. The mean velocity (V) of the channel $= 100c\sqrt{RS}$. In this case $V = 5$ and $c = \cdot 78$, whence $100\sqrt{RS}$ (with $R = 4{\cdot}6$) should $= \frac{5}{\cdot 78} = 6{\cdot}4$. By reference to Table VII., Part II., Jackson's



FIGS. 4, 4a, 4b, 4c, 4d.—Design for Canal Escape Head.

"Hyd. Manual," we can find out what value of S per 1,000 will correspond to one of $100\sqrt{RS}$ of 6·4. This is found to be ·85 per 1,000. Thus in a length of 5,000 feet the surface fall will be $4\frac{1}{4}$ feet; this, added to the 3 feet surface level at the escape head, will make the depth of water at the head regulator $7\frac{1}{4}$ feet to produce a velocity of about 5 feet per second midway.

This is near enough for our purposes. When the canal is clear of silt the mean velocity of the current will be least at the head of the canal and greatest at the escape head. The velocities will vary as $\frac{Q}{A} : \frac{Q}{a}$, *i.e.*, to $\frac{2{,}500}{750} = 3·33$ feet per second at the canal head and $\frac{2{,}500}{310} = 8$ feet per second at the escape head. This will somewhat modify the value of d, which was adopted on the hypothesis of a lower velocity of approach, about $5\frac{1}{2}$ feet, but not to any very great extent, probably bringing it ·2 feet lower, and will reduce the $7\frac{1}{4}$ feet at the head to 7 feet. The bed and sides of the canal will require pitching all the way, particularly near the escape head, where large blocks carefully set will have to be used to withstand the great velocity of the current.

These conditions of velocity and discharge will, however, only apply when the canal is clear of silt, that is, at the very close of the scouring operation. At the commencement, the depth, particularly near the head, will be greatly reduced owing to the silt deposit. Thus the bed slope will at first approximate to the surface slope and the current will have a fairly uniform depth of from 4 feet to 5 feet. Also the velocity throughout will be about 5 feet per second except close to the escape fall, while the amount of water passed through will be the same as at the close of the operation. The construction of a powerful scouring escape near a canal head, if judiciously worked and with an ample supply to draw from, must effect great economy in silt clearance; as not only does it keep the head reach clear, but the periodical removal of deposit at this place will altogether prevent or greatly ameliorate the deposit of deleterious sand in the canal bed generally, and consequently effect economy in annual maintenance. The precise estimation of the velocity is best effected by use of formula (18), par. 48, Chap. V., as the method above sketched assumes a uniform water slope which is not exactly correct.

(13) The design of the escape head in Fig. 4 consists of a bridged overfall weir, the length being divided into seven bays of 14 feet. As the bridge will be used for traffic, being on the canal bank road, the width will be 12 feet between parapets. The arrangement for the gates will be similar to that in the example given in Fig. 1, the outer rail for the traveller being supported by a rolled girder built into the top of the front cut-waters. The depth of film (d) is taken as 8 feet (that of full supply in the canal), for purposes of sectional calculation.

The disposition of the wings down stream (Fig. 4a) is of the dog-legged type; the up-stream right wing is of the splayed sloping crested type, to allow easy access of entry for the water in the channel; with regard to the left wing down stream (*vide* Fig. 5), the junction with the regulator is formed by a continuation of the abutment on a reduced section.

The dimensions of all the parts are in exact accordance with the formulas and rules adopted by the author as detailed below :—

Rise of arch . . . 2 feet $= \frac{S.}{7}$

Thickness of arch . . $1\frac{1}{2}$ feet $= \cdot 4\sqrt{r} = \cdot 4\sqrt{12} = 1\cdot 4.$

Thickness of abutment at springing $= \frac{r}{5} + \frac{V}{10} + 2 = \frac{12}{5} + \frac{2}{10} + 2$ $= 4\cdot 6$ feet.

Thickness of floor $= \sqrt{H} + \sqrt{d} = \sqrt{8} + \sqrt{5} = 5\cdot 0$ (par. 20, Chap. IV.).

Length of floor beyond weir wall $= 2\ (H + d) = 2 \times 13 = 26$ feet, (par. 20, Chap. IV.).

Thickness of piers $= \cdot 3S = \cdot 3 \times 14 = 4\cdot 2$ (taken as 4 feet).

The thickness of the base of the retaining walls is in accordance with the formula $\cdot 4H - 1$ for stone walls with a face batter of 1 in 10. The up-stream wing has this proportional thickness at the canal bed level; below ground the back is carried down vertical, the face continuing to batter to the foundation.

(**14**) Fig. 5 is the design for the canal regulation bridge just below the escape head. It is not supposed to be required to cross traffic, but only for regulation purposes. As will be seen in the transverse section Fig. 5a, the cut-waters are corbelled out square above F.S.L., to carry the outer tier of arches which is provided, similar to the arrangements common to partial regulators, as the Assiût works, and undersluices generally.

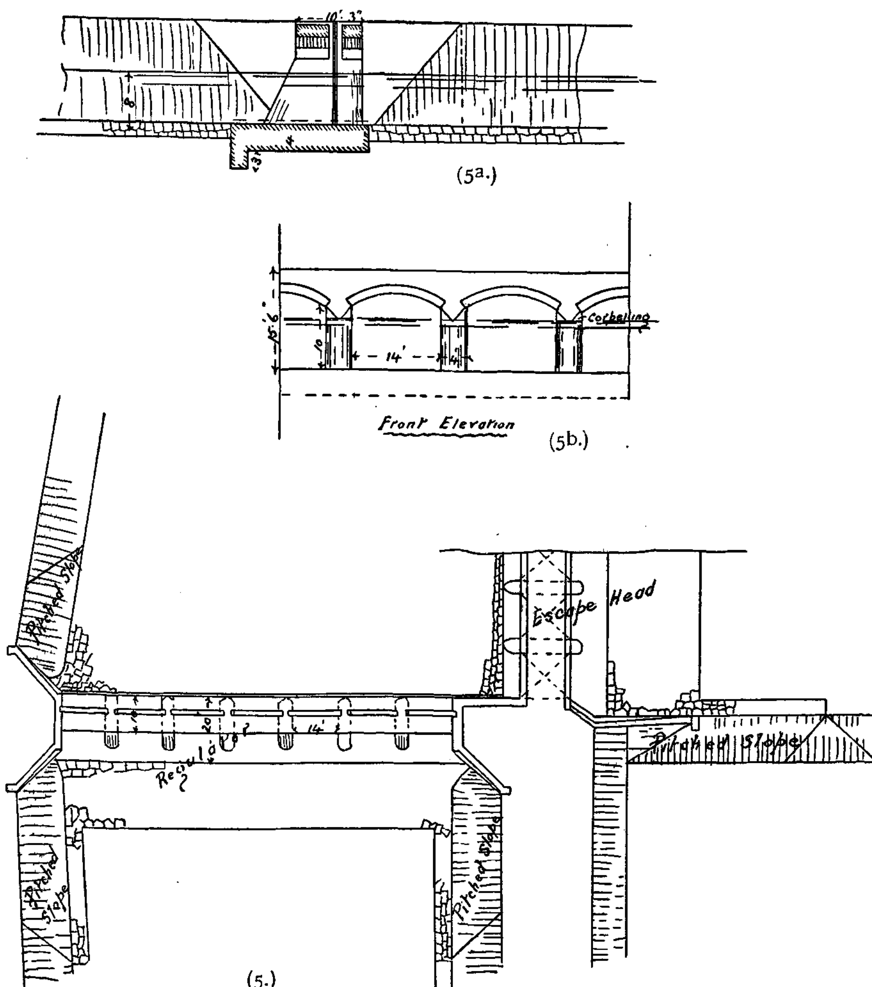
The spans adopted are the same as in the escape head, viz., 14 feet, of which there are seven, giving a combined width of water way of 98 feet, *i.e.*, closely approximating to the bed width of the canal, which is quite sufficient for bridges and cross canal works. The canal bed is shown widened out on one side only.

A skew regulator would really be more in place in combination with the escape head, but would cost more.

(**15**) We will now proceed to give an example of an existing escape head with fall which is on the Godaveri Eastern Canal.

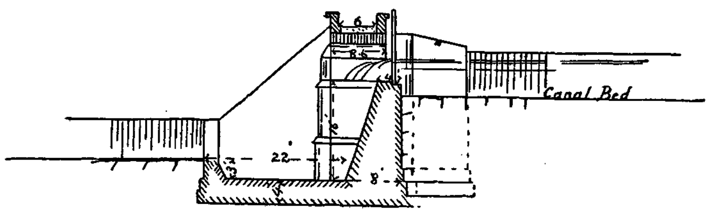
Figs. 6, 6a and 6b are drawings of this work. The depth of the fall is exceptionally great, being 16 feet above floor of fall and 13 feet above the level of the escape channel below. The latter being 3 feet higher than the floor, a water cushion 3 feet deep is formed by means of a raised curtain wall. The weir is divided by piers into five spans of 10 feet. The piers are partly built over the weir, and the roadway is only 6 feet wide between parapets. From this it is evident that the bridge is not required for traffic, but is merely a foot regulation bridge. This work seems on the whole to be well planned, although the curved wings are not commendable.

Straight battered wings would answer the purpose just as well and be easier to build, besides which, the railway under-bridge style of curved wings adopted down stream, which is so common in all Madras works, is objectionable; their curved shape tending to encourage a rotary movement in the water below the weir, termed "pooling," which is very destructive in its action.

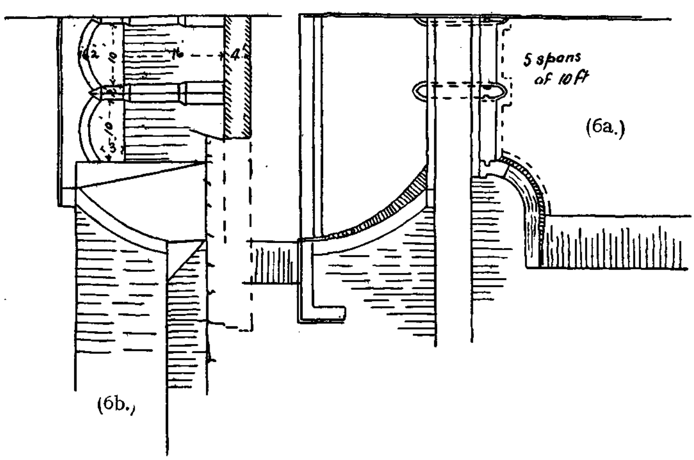


FIGS. 5, 5a, 5b.—Design for Regulator below Escape Head.

The piers are economically designed; the thickness is ·3S at the base and they taper by insets to 2 feet or ·2S at the springing of the arch of the bridge. Piers in a fall are subjected to very slight water pressure, end on, and receive great support from the weir wall into which they are built. They are not likely ever to be subjected to lateral pressure, as in the case of a regulating head or bridge, consequently the thickness can safely be diminished to that suitable for an ordinary bridge. The depth of water liable to pass over the weir is probably not more than 3 feet or 4 feet, judging from the height of the banks of the approach channel. The closure is effected by wooden gates raised by screw gear. A small movable winch on wheels, which could be taken away and housed when not required, would probably form a better lifting apparatus; or else a row of simple wooden windlasses, fixed on the cut-waters. Screw gear is only suitable for reservoir sluice gates in deep water, which are moved 2 feet or 3 feet up and down at the outside, but in the Madras Irrigation Department it is applied to every kind of water gate from undersluices to waste weirs.



(6.)



(6a.)

(6b.)

FIGS. 6, 6a, 6b.—Escape and Fall on the Godaveri Eastern Canal (Madras).

(16) The floor of this work is only 4 feet in thickness. This is considered insufficient; by the rule $t = \sqrt{H + d}$ the thickness should be $\sqrt{20} = 4{\cdot}5$ feet, par. 20, Chap. IV.

The length is also deficient: it should be $2H$ at least, or 32 feet measuring from the toe of the weir wall; it is actually only 22 feet. The water cushion of 3 feet, which necessitates deepening the foundation of the whole of the work, is considered to be quite unnecessary. A floor of rubble stone or brick on edge or end, set in Portland cement mortar with a good solid foundation of concrete below, will not require any water cushion beyond what is naturally provided by the filling up of the escape channel.

The weir wall is 4 feet thick at top and 8 feet at the bottom. Being

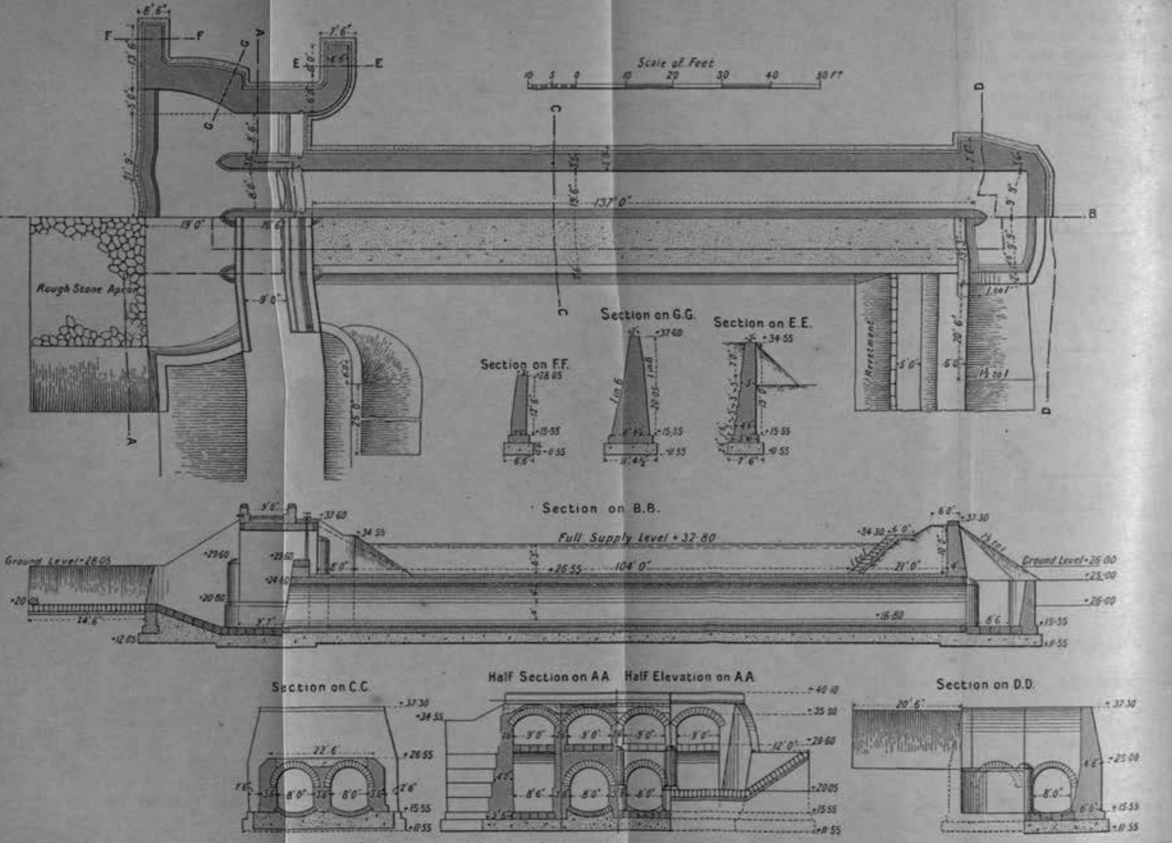


FIG. 7.—Combined Syphon and Escape, Connamur Canal (Madras).

[To face p. 305.

evidently of the "hybrid" type, its base thickness should be $\frac{H+d}{\rho}$, or taking ρ as $2{\cdot}1$, the required thickness comes to $\frac{20}{2{\cdot}1} = 9\frac{1}{2}$ feet. The 3 feet depth of water cushion in front of the weir wall is not of any material assistance, but it adds to the depth of the weir wall, and the latter's section should have been increased accordingly.

This weir evidently does not lie on the side of the canal, but is situated on an escape channel. There is probably no escape head proper, the escape channel leading out at bed level without a regulation bridge, the regulation being effected at the weir, which is some way off. The objection to this arrangement is that the approach channel to the weir will always be full of water, and if not very short will cause wastage, unless it could be utilised as a supply channel to one of two distributaries as well as an escape channel.

(17) It has already been noted that escapes for surplus water can easily be arranged for at aqueducts or drainage crossings, the escape or surplus weir being combined with the latter work. An excellent example of such combination is given (Fig. 7) of a work on the Connamur Canal, Madras.

The canal here crosses a drainage line, which is taken underneath it by a syphon or syphon aqueduct. This consists of two culverts 8 feet wide and about the same height. The waste weir and regulating and traffic bridges are built at the end of these two culverts, half the weir being simply a dwarf wall built over the end of the barrels, the other half lying beyond them. The three centre piers are spaced 9 feet apart, and are built in part directly over the central pier and the two abutments of the syphon. The weir consists, however, of five spans of 9 feet, two of which lie outside the syphon altogether, and here the weir wall is of the full depth of 14 feet. Both abutments are therefore quite clear of the syphon.

The question now arises, if this is the best arrangement possible, whether it would not be preferable to lengthen the culvert and place the whole length of the three bridge piers on top of the syphon, thus saving a depth of 11 feet off two-thirds of the three piers of the regulating bridge, the abutments remaining the same depth as before, but recessed behind the face of the syphon? A great deal of space, which means increase in the length of the syphon, seems to be occupied by the canal slopes on either side. That on the left could well be reduced from $1\frac{1}{2}$ to 1, as any soil, even pure sand, will stand at 45° if pitched. This 10 feet is just equal to the lengthening required to bring the bridge entirely over the syphon. Another 10 feet in length could very well be saved on the other side; this would be sheer gain. By thus recessing the bridge, the floor beyond the weir, which is unnecessarily wide, could be reduced, as the difference of level between canal full supply and the bed of the drain is 13 feet, and a length of $2H_1$, or 26 feet, would be sufficient. The actual length of the floor is 34 feet, hence 8 feet can well come off here. If the design were thus modified, the saving would be 10 feet length of syphon, 8 feet of flooring 5 feet thick, and the whole of the piers below weir crest level, a by no means inconsiderable amount, equivalent to 12 or 15 per cent. of the whole.

The general arrangements of this design are otherwise decidedly good, and it forms a very instructive example.

(18) An example of a very large escape, which belongs more properly to reservoirs than to canal works, is the Koshesha Escape in lower Egypt.

Figs. 8 are plans of this immense work, which consists of sixty spans of 3 metres, or 10 feet, and it upholds 6·2 metres, or, say, 22 feet, of water, although it is stated in "Egyptian Irrigation" to be subjected to a head of 4 metres only. This is too vague to enable a calculation to be made of the horizontal thrust of the water on the work, though it is sufficient for the vertical pressure on which the design of the floor may be made to depend. The soil is good hard clay.

The main peculiarity of the design is the division of the waterway into two parts by arches thrown between the piers, which thus form an intermediate platform. The upper and larger portion of the vent above this platform is closed by iron gates, hinged at the bottom, which, when released, fall automatically on to the platform, to which they are hinged. They cannot be raised when the level of the water is much above that of the platform, but are intended, when the reservoir has to be emptied, to remain down till the water has run off, no regulation by them being possible. The vents below the platform are closed by ordinary draw gates working in grooves; these only come into use when the water above the level of the platform has been drained off. A 12-feet roadway between parapets is provided, so that evidently provision has to be made for crossing traffic. This and the considerable projection of the piers in front of the gates, apparently to allow a wide space between an outer set of emergency grooves and the face of the barrier gates, causes the piers to be of exceptional length, which is not required for statical resistance. This work is evidently erected in the middle of a long embankment crossing flat country, similar to a sluice in a tank embankment; it is not situated in a depression or marked natural watercourse.

The arrangement of longitudinal rubble spurs above and below the escape to guide the current in a direction normal to the axis of the work, is admirable, and so is the disposition of the abutment wings.

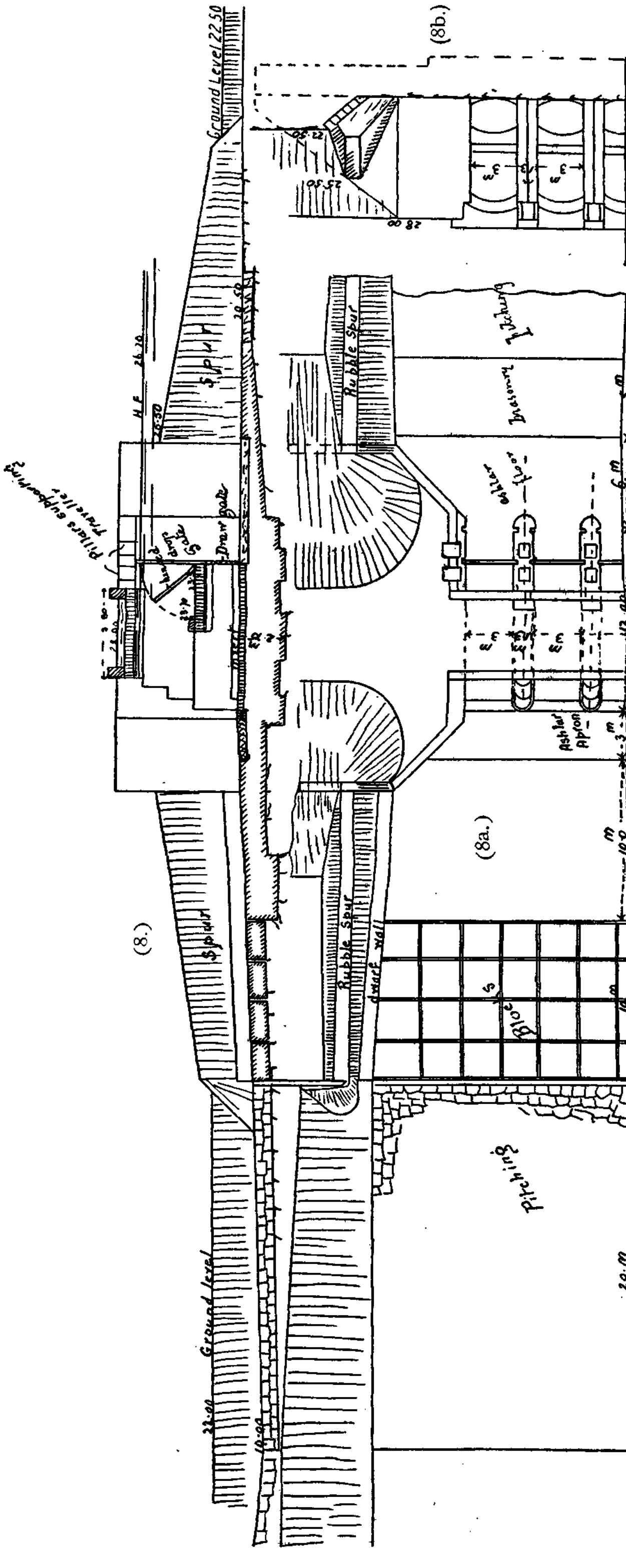
(19) In some other points the design of this work is, however, open to criticism.

Taking the floor first, the depth under the piers, which depth runs right through, is considerable, being 2·75 metres, or 9 feet. Considering that the soil is hard clay, and the head of water nothing very remarkable, being 7 metres, or 22 feet, when the bed below is dry, but stated not to exceed 4 metres, or 13 feet, this mass of rubble masonry appears excessive.

The Assiût works, built not on hard clay but Nile mud, have a depth of foundation of 3 metres, or 10 feet, with not far short of the same head, viz., 3·25 metres in the Ibramiya Head.

With regard to the superstructure. The provision of spare grooves and the consequent extension of the piers up stream, a construction always to be avoided if possible, appears an unnecessary precaution, particularly in view of the fact that the work will only be used once a year, and will doubtless remain clear of water for several months, when any repairs to gates, etc., could be carried out. The floor should, however, be so constructed that it will never require repairs; this can always be ensured by using good cement masonry of sufficient depth. The ashlar slabs used in this and many similar works are considered objectionable, being very liable to be ripped up, and being besides expensive. Good hard rubble on edge with thick joints of cement will stand erosive action much better than any more expensive material.

(20) The division of the spans horizontally, though doubtless an excellent



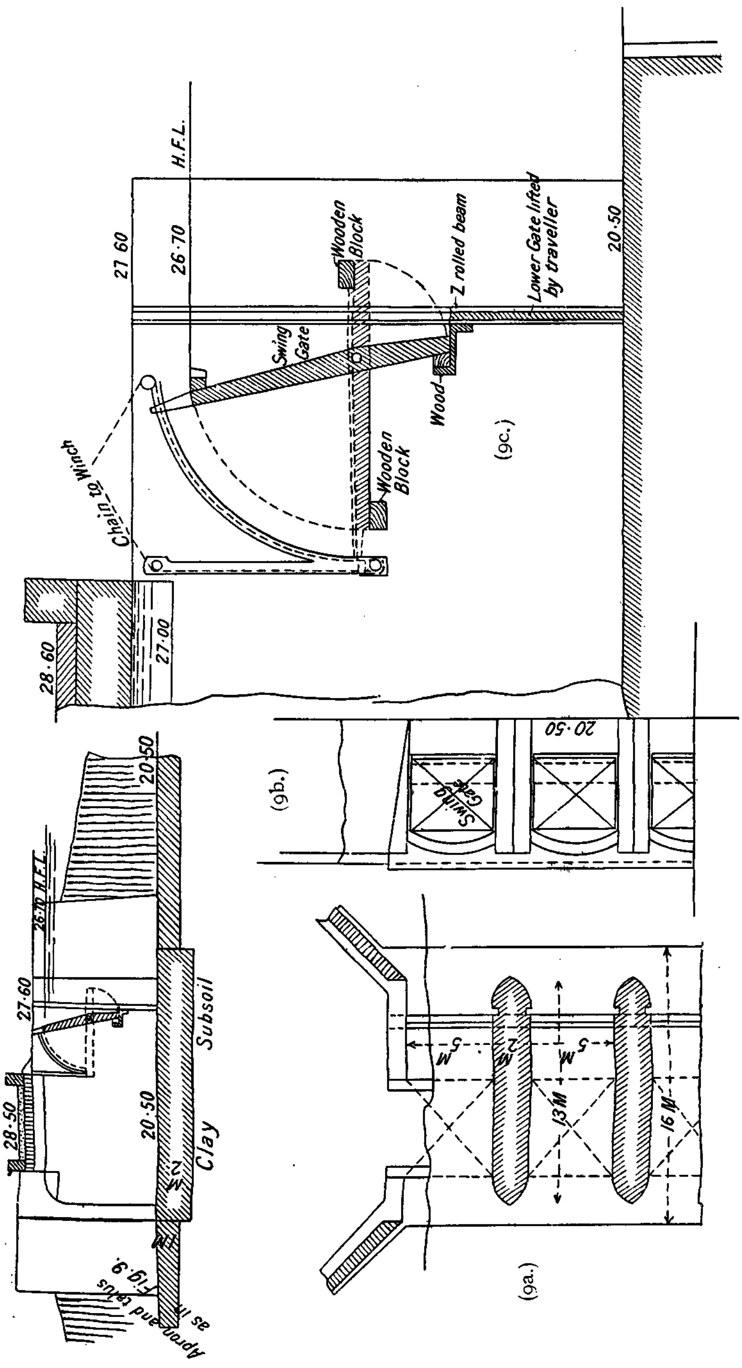
FIGS. 8, 8a, 8b.—Koshesha Escape (Egypt).

arrangement in itself, is carried out by the insertion of a great thick vaulted masonry platform, which takes up a great deal of the waterway, necessitating more spans to carry off the discharge than would otherwise be requisite. This obstruction is not short of 5 feet in depth. Here is a case in which iron could be used with great advantage in place of masonry. The falling gate, hinged at the bottom, necessitates a long platform to receive it when down. If a gate hinged at the centre of pressure, *i.e.*, at one-third of the height, or thereabouts, be substituted, it could be manipulated with great ease, even under full pressure, that on the upper and lower leaves being balanced. It could also be arranged to fall automatically when the impounded water rises above a fixed level. A balance gate of this description is hinged at both ends on a steel axle, which is either built into the piers or else is received into a movable plummer block, which, provided with anti-friction rollers, can slide up and down an iron groove. The gate, after being pulled into a horizontal position, can then be raised by chains attached to the blocks in the grooves when offering the least resistance to the water pressure. This, in certain cases, is a more suitable arrangement than that adopted in Fig. 9 (*post*). The total pressure on each axle will be $\frac{wlh^2}{4}$ or, in the case shown in Fig. 9c, $l = 16$ feet, $h = 13$ feet and $w = \frac{1}{36}$ ton, whence the expression becomes $\frac{16 \times 169}{4 \times 36} = 19$ tons, or taking 6 tons as safe shearing stress for steel, the sectional area of the axle will be $\frac{19}{6} = 3\frac{1}{6}$ square inches nearly, requiring to be 2 inches in diameter.

(21) Considering the depth of water (viz., 22 feet), the spans of 10 feet width are decidedly too narrow. They could be well increased to 5 metres, or $16\frac{1}{4}$ feet, the pier being 2 metres wide, or ·4*S*. A revised design comprising these alterations is given in Fig. 9.

The spans are increased to 5 metres, and in place of the heavy masonry platform, a *Z*-shaped rolled beam is substituted, on the upper flange of which the base of the swing gate abuts, while the lower draw gate presses on its lower flange. The upper swing gate is 4 metres, or 13 feet, the lower draw gates $2\frac{1}{2}$ metres, or $8\frac{1}{4}$ feet, deep. The latter can be raised $3\frac{1}{2}$ feet while the upper is down. The depth of the free waterway is greatly increased by this arrangement, viz., from 5 metres to 6 metres, or from $16\frac{1}{2}$ to 20 feet. The sixty spans of 3 metres give an effective waterway of $60 \times 3 \times 5 = 900$ square metres, or say 10,000 square feet. With the alterations shown in Fig. 2, each span of 5 metres has a free waterway of $5 \times 6 = 30$ square metres; the required number of spans of 5 metres will then be $\frac{900}{30} = 30$ spans, giving a length between abutments of $(30 \times 5) + (29 \times 2) = 208$ metres. The existing work has a length between abutments of $(60 \times 3) + (59 \times 1{\cdot}3) = 257$ metres; the revised design will thus cost 20 per cent. less, and with the further reductions in the superstructure and floor (if the latter were reduced to reasonable dimensions), the saving would be quite 25 per cent.

(22) A rough sketch of the swing gate, an invention of the author, is given in Figs. 10 and 10c. It is shown clear of the archway, but could be fixed underneath it if so desired.



FIGS. 9, 9a, 9b, 9c.—Alternative Design.

The arrangement for revolving the gate consists of two endless chains attached to projections on either side of the top of the gate, which chains work inside curved iron grooves. Each chain at the top of the piers is carried round a winch drum connected by a horizontal shaft, revolving which either way causes the gate to be lowered or raised to any extent. If the gate is required to fall automatically, or at once when released, in the former case the axle can be fixed at such a height that when the water reaches a certain point the gate will revolve of its own accord, overcoming the friction of the chain and winches. If, on the other hand, the gates are required to be suddenly released with full supply on, the winch drums can be prevented from turning by a catch, which being knocked away, the gate will be free to revolve.

In the design in Figs. 9b and 9c the gate is not intended to be lifted, but when horizontal to remain in that position, with water passing above and below it. If it is deemed desirable to lift the gate, grooves can be provided as shown dotted, with blocks for containing the axles of the gate. These blocks are attached to chains lying in the vertical grooves, by which the whole gate can be lifted up by a traveller. The arrangement adopted in the plan of pivoting the gate on a fixed axle on which it swings, and leaving it *in situ*, is simpler and better, the only objection to this arrangement being the possible accumulation of floating *débris* on the end of gate. This, however, can be easily got rid of by turning the gate round a little for a time, when it will be swept away by the current.

(23) When an escape is not required for scouring purposes, a regulation bridge across the canal is not essential. The escape head will then consist either of a simple regulating bridge with its sill level with the canal bed, or raised a foot or two above that level, forming a dwarf weir wall on which the regulating gates fall; or else of a bridged fall similar to Fig. 4, only with the crest of the weir raised 2 or 3 feet above canal bed level. The length of the weir should be such as to take off the whole or a part of the full discharge as may be required.

(24) Distributary heads require but little cement; their construction should be similar to that of canal branch heads or cross canal regulators, *i.e.*, an open waterway with or without a raised sill, closed by draw gates.

For such works double wooden gates can well be used. These can easily be fitted with small iron rollers, the lateral stanching being effected by vertical stanching strips fastened to the gate at their upper extremity only. The pressure of the water forces these against either the base or the side corners of the tables of the iron grooves. These gates can be easily manipulated by a wooden windlass fixed over each span, the barrel fitted with ratchet and pawl. The windlass can be turned by detachable wooden bars fitted into holes, or else a spoke wheel can be fixed in the centre of the drum. The system so much in vogue in Madras, of constructing canal and distributary heads on the principle of small sluice ways situated at floor level, which are closed by a gate operated by screw gear, is much to be deprecated. This system is only suitable for outlets from tanks and reservoirs under a considerable head of water.

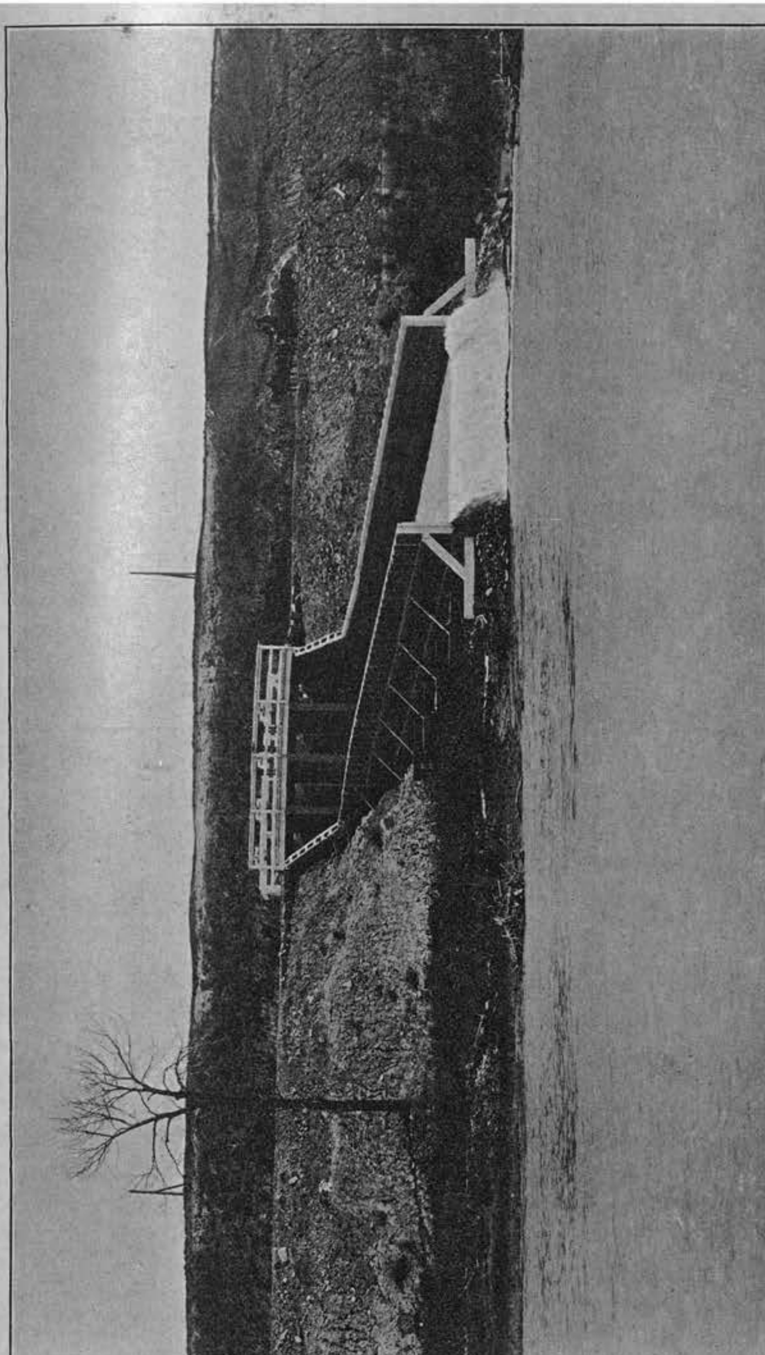


FIG. 10.—View of Escape Head on Calgary Canal, Alberta, Canada.

(**25**) A view of an escape head, situated a mile or so below the intake of the Calgary Canal, in Alberta, Canada, is given in Fig. 10.

The structure, as will be seen, is of wood and consists apparently of 5 vents, each 5 or 6 feet in width, closed by draw gates, which are operated by rack and pinion hand gear. It forms a very instructive example as to the use to which timber can successfully be adapted. This work, together with all other timber erections on this canal, will, however, eventually have to be replaced by permanent structures in masonry or concrete.

Possibly in some future day these arid canal banks will be planted with umbrageous trees, and thus come to bear some resemblance to the shady

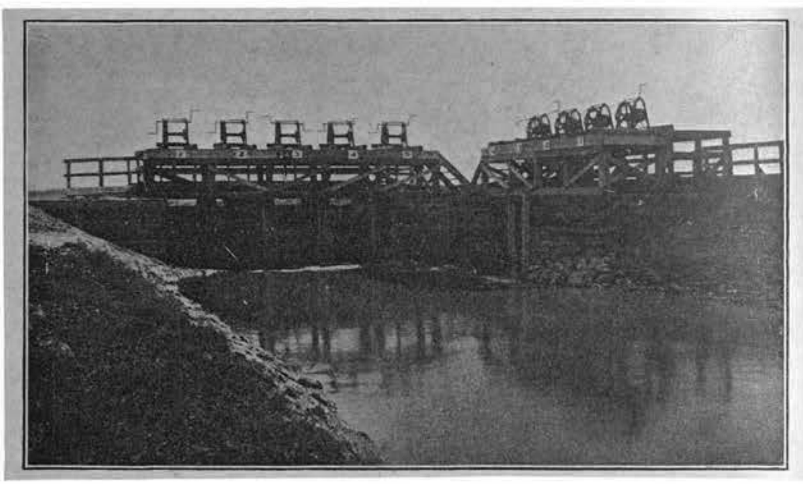


FIG. 11.—Bifurcation on Calgary Canal.

avenues on the older Indian canals which in a thirsty land add such a charm to the landscape.

(**26**) Fig. 11 is a view of a bifurcation on the same canal. The regulators are fitted with the rear pivoted balance gates illustrated and described in Chap. VIII. This arrangement, though doubtless effective and easy of manipulation, has, it must be confessed, an extremely crude appearance. These temporary structures are, it may be observed, by no means inexpensive. It is believed that a properly designed masonry work would cost very little if any more.

The policy of the Canadian Pacific Railway in this canal, as in their own particular case was, however, to get the work *through* somehow as soon as possible, and then subsequently improve gradually when leisure and opportunity offered. Who will say that in a pioneer project of this kind this was not the proper policy to adopt ?

CHAPTER XI

CANAL CROSS-DRAINAGE WORKS

(1) THE disposal of drainage that is intercepted by a canal can be classified as the following :—

I. By lateral diversion, *i.e.*, by excavating a channel parallel to the canal, the stream can be thrown into another drainage line, for the disposal of which provision has been made.

II. By passing it underneath the canal, the canal either crossing the stream on a raised aqueduct, or, if the headway is insufficient for a clear passage, the bed of the stream is depressed below normal level, and the water passes in a tunnel underneath, rising again on the further side. This latter is termed a syphon or syphon aqueduct. (Figs. 1, 3 and 8.)

III. The drainage water can be admitted into the canal itself. This is termed an inlet.

IV. The drainage can be taken into and across the canal at the level of the bed of the latter, the inlet on one side and the exit on the opposite side. This involves one regulator across the canal and one at the further bank across the exit of the drainage. This is termed a level crossing. (Figs. 18 and 19.)

V. The drainage can be taken over the canal by an aqueduct. This is termed a superpassage, to distinguish it from an aqueduct proper. (Fig. 10.)

VI. The canal can be taken under the drainage line by a depressed syphon or syphon-superpassage. (Fig. 9.)

(2) In Fig. 1 we have an example of Class I., being the Thora Nala Aqueduct on the Midnapur Canal. In this work the canal bed is 25 feet above that of the river, giving sufficient headway to pass the highest flood, which is 18 feet deep.

Like all masonry aqueducts, the construction mainly consists of an arched bridge with platform at canal bed level, and provided with two solid parapets, which retain the water flowing through.

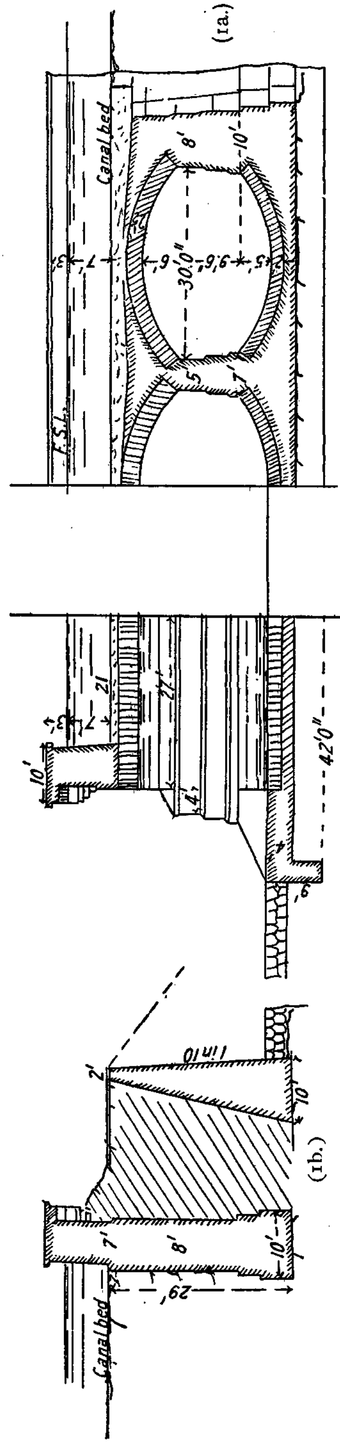
To reduce expense, the waterway of an aqueduct is made narrower than the average width of the canal in earthen banks. Owing to the smoothness of the sides, the coefficient of rugosity (n) is much less than that applicable to channels with earthen sides and bed, being ·013 in the former against ·025 or ·0225 in the latter. This alone greatly increases the velocity, so that a considerable reduction in section can be effected, even if the original mean velocity of the current were retained. As, however, that velocity can safely be increased to 5 feet per second, if sufficient bed slope be given, a still further reduction in the width of the water way in the aqueduct can be effected. In the example we are considering, the reduction in bed width is not shown on the general plan in Fig. 1d.

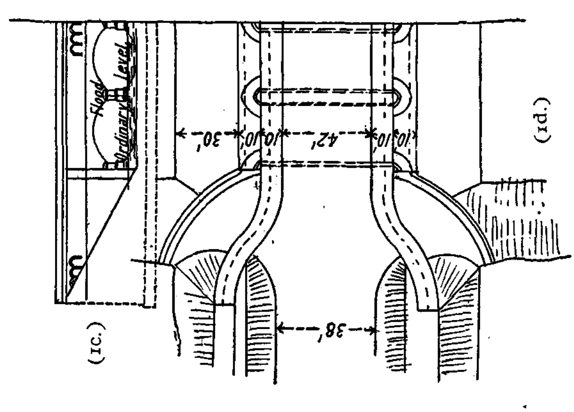
(3) The thickness of the arch throughout is $2\frac{1}{2}$ feet, the radius being 20 feet. The value given to n in the formula $n\sqrt{r}$, adopted for crown thickness (*vide* par. 17, Chap. IV.), is therefore ·55. The weight of water overlaying the arch is $7 \times \frac{1}{36}$ tons $=$ ·2 ton per square foot.

The live load in ordinary bridges is assumed at about 120 lbs. per square foot. This represents an equivalent weight of water of 2 feet in depth. Therefore the extra live load, which, however, is not suddenly applied and removed, as in the case of railway and road bridges, will be the weight of the depth of water carried, minus 2.

The value of the coefficient n in the formula $n\sqrt{r}$ can then be approximately increased in proportion to the depth of water carried; in ordinary cases $n =$ ·4.

The following rule for deducing the increase to the value of n will suit in most cases. Let d = depth of water, then $n =$ ·4 + ·02 $(d - 2)$. When the depth of water is 2 feet, n will remain ·4; with 5 feet carried $n =$ ·46, with 6 feet $n =$ ·48, with 7 feet $n =$ ·50, with 10 feet $n =$ ·56.





FIGS. 1, 1a, 1b, 1c, 1d.—Thora Nala Aqueduct, Midnapur Canal.

This is for large spans of over 25 feet. For smaller spans n should be taken as $\cdot 5$, no matter what the depth of water is.

In the example of the Thora Nala Aqueduct the correct crown thickness would then be, with 7 feet water carried, $\cdot 5 \times \sqrt{r}$ or $\cdot 5 \times 4\cdot 5 = 2\frac{1}{4}$ feet. This would increase to $2\frac{1}{2}$ feet at the springing.

The parapets, as is usual in large aqueducts, are widened out to carry a roadway, as communication for cart traffic must be kept up along canal banks. The parapets here are 7 feet wide at base and 6 feet at top, corbelled out to provide a 10 feet roadway, with an iron rail fence on either side. The thickness of the parapet in this case is excessive. It can be made half, but should not exceed two-thirds, the depth of water. In the next example we shall see it is made about $\frac{2}{3}D$ in width, D being the depth of water in channel.

(4) The piers are $\frac{S}{6}$ or $\cdot 167S$ in thickness. They widen out by offsets to 7 feet, or $\cdot 23S$, at the base, S being the span.

There is no definite rule regarding the ratio of the thickness of piers proportional to the span in the case of large span bridges. It may be taken to vary from $\frac{S}{10}$ to $\frac{S}{5}$.

For heavy works of this description the proportion $\frac{S}{6}$, as in this case would not be excessive. In the Kali Nadi Aqueduct of 60 feet spans (Fig. 3) the proportion is $\frac{1}{8\frac{1}{2}}$, and in the Gunneram Aqueduct (Fig. 4), with spans of 40 feet, the proportion is $\frac{1}{7\frac{1}{2}}$ nearly, or, to be exact, $\cdot 15S$. In the Budki Superpassage (Fig. 10) the proportion is $\frac{1}{8}$. All these carry about 7 feet of water, and are all built of brickwork. A safe rule to adopt would be to make the top thickness of piers of large span aqueducts for 25 feet span and over $\sqrt{S}$, below 25 feet $\cdot 2S$ (*vide* par. 12, Chap. IV.).

The piers of large span bridges should increase in thickness towards the base with the object of better distributing the pressure on the foundations, and further of inducing a uniform stress at all points in the height of the pier. A formula for effecting this is given in "Molesworth's Pocket Book," p. 89.

In the example we are now examining, the width of the pier increases by two offsets from 5 feet to 7 feet at the base. A straight or curved batter would have been a simpler and better construction.

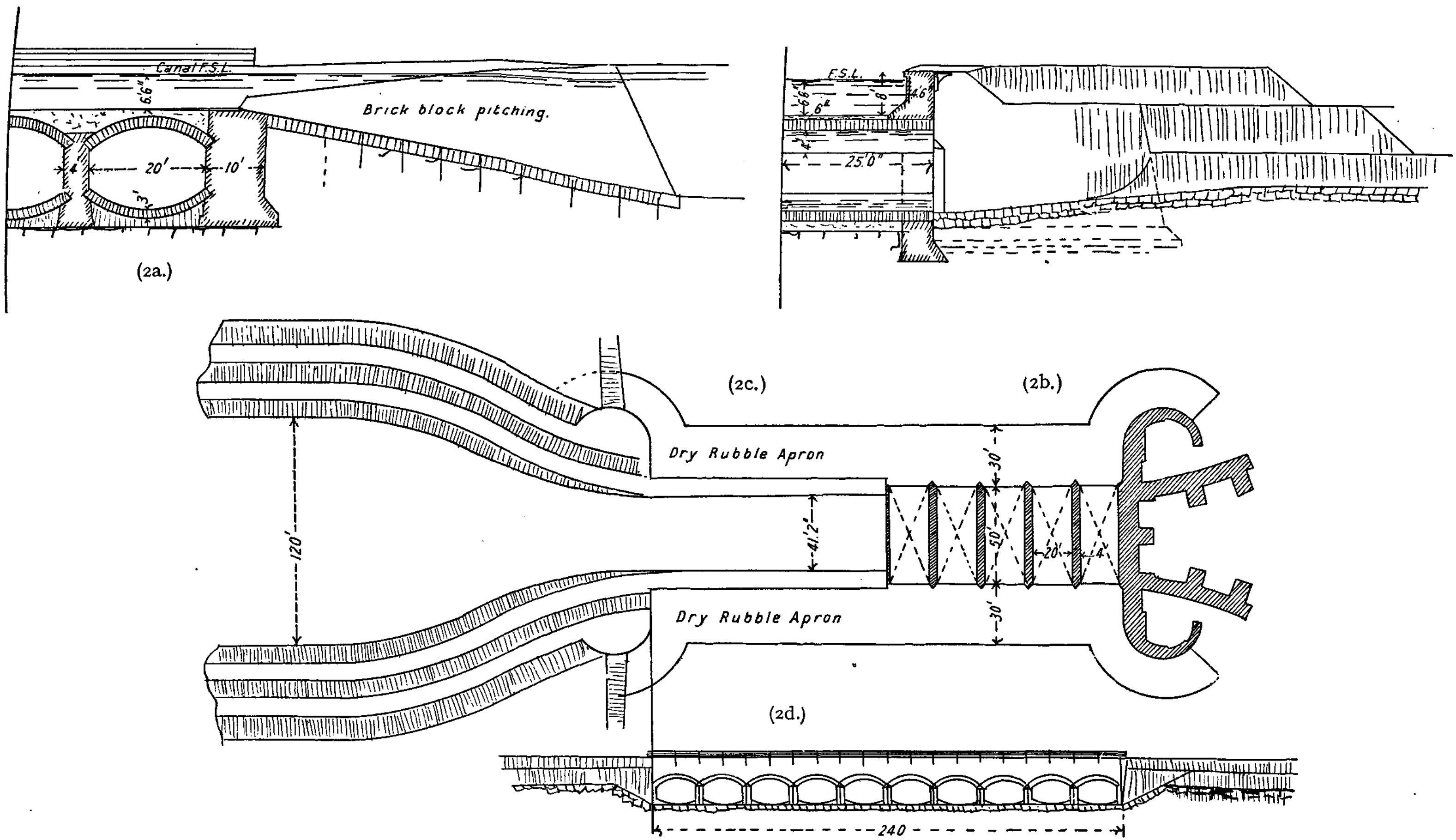
(5) The floor is composed of inverted arches with a versed sine of 5 feet, the thickness in the centre of the span being 4 feet, and that at the spring line of the inverts 9 feet. The object of this invert is evidently to distribute the weight on the piers evenly over the somewhat shallow foundation. It is very doubtful whether the inverted arch does really act is this way ; the use

of an invert is more to prevent a floor from blowing upwards from water pressure underneath, and it is used with advantage for this purpose in works subjected to a head of water. In the cases of bridges, however, there is no appreciable head of water against the work, consequently inverts are, apparently, not necessary.

The objection to their use is the great obstruction they offer to the free passage of water, by decreasing the effective depth to the extent of nearly 3 feet. It would have been far better to have made the foundations thicker all through, say 5 or 6 feet deep, with a level floor flush with the bed of the river. The projection of the invert is eased off by a sloping apron at both sides and in front (Figs. 1 and 1d).

(6) The sections of the abutment and of the wing walls (Fig. 1b) are good. Figs. 1c and 1d are part elevation and plan at a smaller scale. Reference to Fig. 1d will show the disposition of the wings, always a most important point. In almost all aqueducts and super-passages double sets of wings are required, viz., two long curved land wings to form the connection between the masonry aqueduct and the earthen banks of the approach channel, and two water wings connecting the face of the abutments with the river banks on either side. The land wings form really a continuation of the parapet walls, and are of the same section at the top. Being subjected to hardly any earth pressure, they can be built with vertical sides, of the same width throughout as the top. The section in Fig. 1b is wasteful on account of the parapet being designed of unnecessary solidity, which involves the land wings in the same fault. We shall see that in other examples, by a better arrangement, an equally wide roadway can be provided without unduly thickening the parapet and wings.

(7) Figs. 2, 2a, 2b, 2c and 2d are different views of the Kerai Aqueduct on the Sôn Canal. The construction is very similar to the last example, the main peculiarity of the design being this, that to obtain the necessary headway, the spring lines of the invert arches are made on a level with the bed of the drainage line, the crown and floor proper being depressed 3 feet below the normal bed. A flat pitched slope connects the two levels. If the flood line rises above the archway, the aqueduct will become a syphon aqueduct. The drawing in "The Irrigation Works of India," from which this figure is prepared, does not, however, show the flood line. This device of depressing the floor to avoid the obstruction offered by the invert is a good one, *i.e.*, on the assumption that the invert is necessary. The author is, however, strongly of the opinion, stated with reference to the same matter in the last examples that a better design would be to increase the thickness of the floor, which is only 3 feet, to, say, 5 feet, and do away with the invert altogether. The bases of the piers are carried through the concrete to the ground—a bad arrangement, which tends to concentrate, instead of distributing the weight of the superstructure. The thickness of this mass floor is determined by the limit pressure allowable on the soil. The bearing area of the pier base increases 1 foot in width for every foot in depth of the floor (par. 9, Chap. IV.).



FIGS. 2, 2a, 2b, 2c, 2d.—Kerai Aqueduct, Sôn Canal.

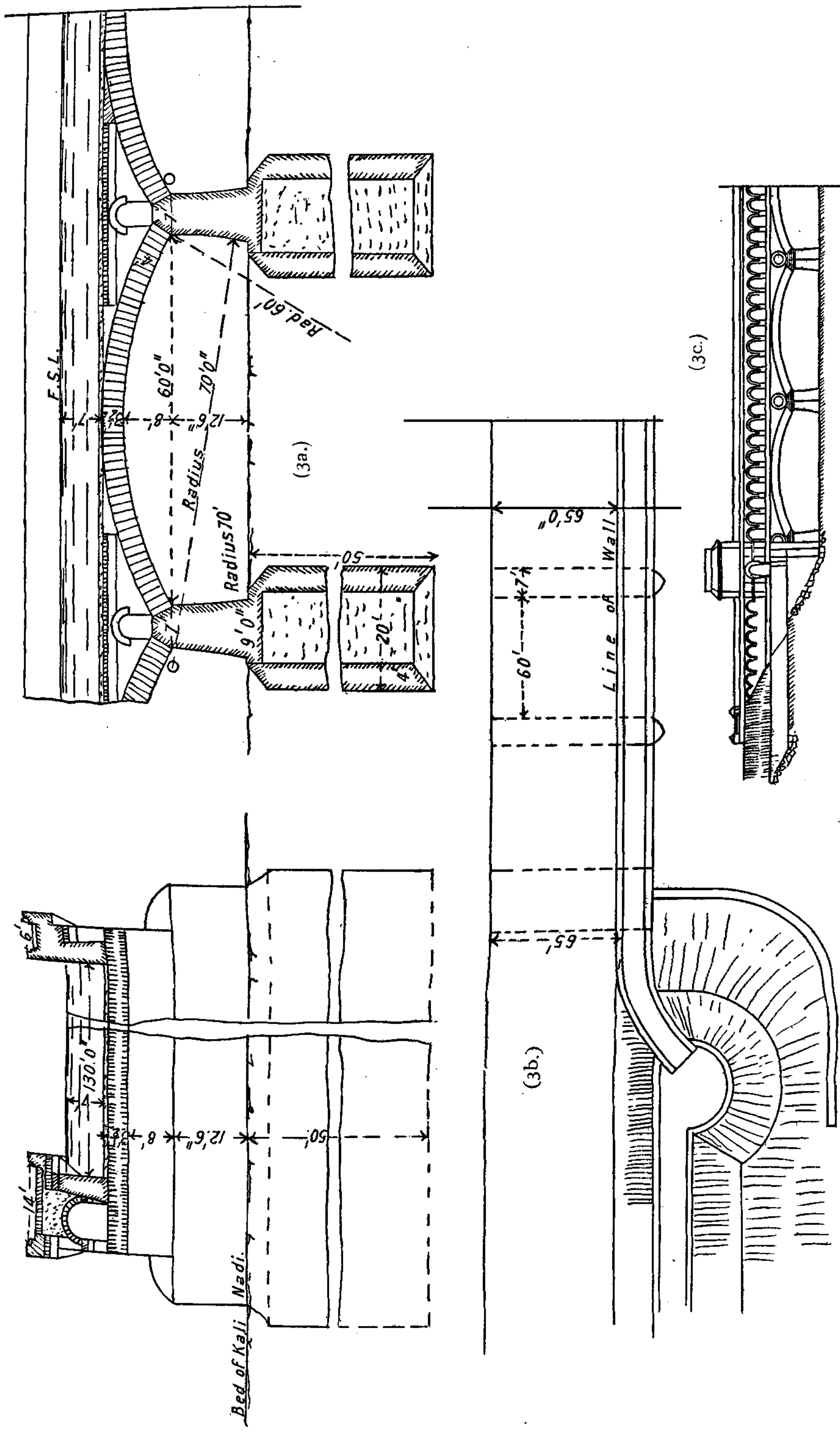
The invert stops dead short at the termination of the piers, the sloping continuation peculiar to the last example not being adopted.

(8) The parapet in this example is $4\frac{1}{2}$ feet thick, or just about $\frac{2}{3}D$; it is widened inwards at junction with the arch by the addition of a triangular strip, thickening the actual base to 8 feet and filling up the corner. This is an excellent arrangement. The widened roadway along the top of the parapet is formed by a projecting cornice on the inner side and by iron brackets, probably planked over, with a fence on the outside. This forms a roadway 8 feet wide, giving just room for a cart to pass. The arch is 2 feet thick, *i.e.*, very nearly $\cdot 54\sqrt{r}$, the radius being 14 feet.

The section of the abutment, which is 10 feet thick, or half the span, and with the buttress (*vide* Fig. 2b) is 16 feet thick in the centre, is a sheer absurdity. If made 5 feet wide at top and 10 feet at base, the dimensions would be ample. The bed of the canal is apparently not made up level with the top of the abutment, but slopes downwards towards the ground surface. This slope is pitched, and likewise both the inner slopes of the canal banks. This hollow is probably left to silt up naturally.

The disposition of the wings is shown in Figs. 2b and 2c. The water wings are semicircular in plan and are very high, so that the land wings are subjected to very slight unbalanced earth pressure. The widening of the canal from 40 feet to 120 feet is shown in Fig. 2c. These dispositions must be pronounced good. The semicircular water wings forms a very efficient protection against the scour of the river in flood.

(9) Fig. 3 is a representation of the Nadrai Aqueduct over the Kali Nadi, on the Lower Ganges Canal. This is one of the largest single works erected in Upper India, consisting of fifteen spans of 60 feet and a width between parapets of 130 feet. This is the second work erected at this site within a short period. The first aqueduct was designed to pass a maximum discharge of 5,000 cubic feet per second in the stream, and was completed and in working order when, after a period of exceptionally heavy rainfall, the Kali Nadi, which is ordinarily a simple depression, the banks and bed being cultivated, came down in an unprecedented flood. With the exception of one railway bridge, which was founded on deep sunk iron caissons, every single bridge on this long watercourse was destroyed; the aqueduct also was washed clean away and disappeared completely from sight, being buried in an immense hole scoured out owing to the obstruction. The loss of this aqueduct was a most serious disaster, as, being situated not far from the head work, the irrigation of the greater part of the canal was completely closed, involving great loss of revenue and interest on the capital charges. A new aqueduct near the old site was immediately put in hand, the design being entirely recast to provide for passing 140,000 cubic feet per second. The new design consists of 15 spans of 60 feet, the piers, abutments and wings being founded on circular wells 20 feet in diameter, sunk 50 feet below the bed of the stream. This structure is not provided with a floor, such indeed



FIGS. 3, 3a, 3b, 3c.—Kali Nadi Aqueduct, Lower Ganges Canal. Fifteen Spans of 60 feet.

being unnecessary owing to the great depth of the pier foundations; which are below the influence of any scour.

(10) The area of clear waterway in each span is 1,140 feet, and there are fifteen spans—total, 17,100 square feet. This area has to pass 140,000 cubic feet, hence the velocity of current passing the bridge will be $\frac{140,000}{17,100} = 8\frac{1}{4}$ feet per second. This immense velocity would speedily scour out the bed till equilibrium was produced. In the original project it was proposed to place a floor 10 feet below the normal bed level of the stream to allow for scour down to this level. This would enlarge the area of the waterway to 24,000 square feet and decrease the velocity to 6 feet per second. This velocity again is excessive, and if the floor, as proposed, were actually constructed, which is doubtful, further scour must necessarily occur in the unprotected river bed immediately below. This hypothesis is borne out by the recorded action of subsequent floods, one of which is stated to have scoured out a hole 30 feet deep below the work. From the above it is evident that the depth of pier foundations designed, viz., 50 feet, is in no way in excess of requirements. The afflux is estimated to be 6 inches.

(11) With regard to the superstructure, as the flood rises as high as the bed level of the canal, the arch and spandrels of the bridge openings cause considerable obstruction to the waterway. This obstruction, exclusive of the piers, is no less than 400 square feet in a water area of 1,600 feet, *i.e.*, one-quarter of the area of the waterway above the normal bed level of the river. The obstruction due to the pier cut-waters is not included in this estimate. This constriction of the passage is unavoidable in the arched type of superstructure in all cases where the flood level is higher than the crown of the arch, but the adoption of a steel girder superstructure for the aqueduct would very largely reduce the obstruction. In that case the only obstruction below the water line would be the joists and longitudinals below the plate floor; these would not take up more than 2 feet in depth, and the proportionate obstruction would be only one-twelfth of the normal waterway. However, under the peculiar circumstances of this particular case the relief thus afforded would be comparatively small. The congestion occurs mainly in the base of the water section, which is relieved by scour of the bed at each flood. Consequently the necessity of deep isolated foundations to the piers would still remain, as also the inadvisability of attempting to curb the bed scour by the imposition of a floor, even if sunk below the normal bed.

(12) In rivers, whose bed is composed of micaceous sand of low specific gravity, as the Ganges, Jumna, and the Kali Nadi in question, the bed level is being constantly altered by flood action, the water section automatically adjusting itself to varying discharge and velocity. During the prevalence of high floods the sandy bed is scoured out and carried along by the current, and this process continues until equilibrium is produced by the fall of the flood. Then with the consequent decrease in the velocity, the silt in

suspension begins to deposit and fill up the bed, till eventually the normal low water bed level is again reached. This action takes place in all rivers, but is most marked in those which run in soil of a very light character. In bridging a river of this description the only safe plan is the adoption of very deep isolated pier foundations, generally associated, in consequence of the expense of these foundations, with wide spans.

(**13**) In the design under review the deep foundations are rightly provided but owing to the arched type of superstructure adopted, the spans have to be limited in width, resulting in the foundations of the piers being too close together for economy. If laid flat, the well foundations would form a continuous solid floor nearly 20 feet in depth. Consequently, in this and in all similar cases wider spans are a distinct economy, to effect which a steel girder superstructure is a necessity. Such an aqueduct would not cost more than a railway bridge of similar span and width, because the load carried, although greater in amount than in the former case, is practically entirely dead load, and the aqueduct would probably be even a lighter structure than a railway bridge subject to a heavy vibrating live load. By adopting moderate spans of, say, 120 feet, the expense of pier foundations could be reduced by one-half, or nearly so; while, on the other hand, the safety of the work would be considerably increased by the lessened obstruction of the waterway and consequently diminished scour, which would be decidedly a desideratum, as a 30 feet deep scour, previously alluded to, can by no means be regarded with equanimity even with the 50 feet deep foundations adopted. The length of the aqueduct piers with a girder superstructure would have to be increased, as the canal water section would be split up into several channels, each separated by the girders, the vertical box plates of the lower flanges of the latter being carried up to above full supply level to form the sides of the channels, the rest being open truss work. With eight spans of 120 feet, the available waterway above normal bed would be $8 \times 120 \times 22 = 21{,}120$ square feet. The velocity of passage before the bed is scoured out would then be $\frac{140{,}000}{21{,}120} = 6{\cdot}6$ feet per second, and at 10 feet lower level $\frac{140{,}000}{30{,}720} = 4\frac{1}{2}$ feet per second nearly. This allows of some increase in the width of the waterway as executed, which is evidently desirable.

(**14**) This increase in width must, however, be limited so as not greatly to exceed the normal width of the river channel. Scouring action in the bed of a river cannot be prevented by greatly widening the natural channel and lengthening the bridge. If this is done in the mistaken idea that the natural scour of the bed would cease, owing to the provision of ample waterway laterally, the certain action of the first falling flood would be to deposit silt on both sides, thus reducing the channel to its normal proportions. In future floods this lateral deposit would either remain undisturbed—the channel reverting to its normal regimen—or else the current would bore it out on one side, heaping deposit on the other, thus causing great danger to the work from cross-currents. This property of rivers is well known, and in the Punjab, for this reason, the bridges over several large rivers have had to be curtailed in length and the channel restricted by protective works into a straight, comparatively narrow reach, more in accordance with the normal channel.

(**15**) We will now proceed to review the details of this work. Excepting the sections of the wings, which, as is invariably the case in Upper India, are designed with vertical faces, the details of this work are excellent. The arch is $3\frac{1}{2}$ feet thick at the crown, or $\cdot 42\sqrt{r}$, increasing to 4 feet at the springing.

The thickness of the piers at springing is 7 feet, or $\cdot 116S$, or $\frac{S}{8\frac{1}{2}}$. This thickness must be considered as below the average. As $\sqrt{S}$ the thickness would be 7·7 feet. The piers widen out in a curve to 9 feet at the base. The spandrels of the arches are lightened by a series of jack arches supported by the longitudinal piers shown in Figs. 3 and 3a. The parapets are corbelled out to form the roadways on either side. One provides a 12 feet cart track, and the other a 6 feet wide footway. The corbelling and arching is ingeniously arranged, but it takes up a great deal of room, necessitating a lengthening of the arch, piers and abutment, which could be reduced, if no regard were paid to architectural features.

The wings consist of a pair of curved landings ending in the canal earthen banks, which are here of semicircular shape to receive them. On the river side are two large water wings, curving a full quadrant and continued to well beyond the termination of the land wings, forming a very efficient protection to the flanks of the work.

The discharge of the canal is 4,100 cubic feet per second, which has a velocity of 4 feet per second in passing the aqueduct; the curves narrowing the canal are not shown on the plan; the ordinary bed width of the canal is 230 feet. The banks on each side of the narrowest part are revetted, and enclose a puddle core of large dimensions.

(**16**) Fig. 4 contains the plans of the Gunneram Aqueduct over a branch of the River Godaveri, taken from the "Madras Irrigation Manual." It has forty-eight spans of 39 feet, and the width between parapets is $23\frac{1}{2}$ feet, with a depth of water of $6\frac{1}{4}$ feet. The length of this structure, which is 2,250 feet between abutments, or nearly half a mile, is such that some slope is necessarily given to the canal bed; this is made 18 inches, the spring line of the arches being built lower in each span. The surface level of canal, however, shows only a slope of 6 inches (*vide* sections on AA and CC), which would be ample.

The river flood has risen as high as R.L. 23·91, *i.e.*, nearly 24 feet above bed, and actually as high as the surface of the canal supply in the aqueduct (*vide* section on AA). This must have subjected the work to a very severe test, and clearly proved the sufficiency of the shallow foundations adopted. The piers and abutments are built on circular wells 6 feet in diameter and 7 feet deep, leaving an interval of 3 feet to floor level, which

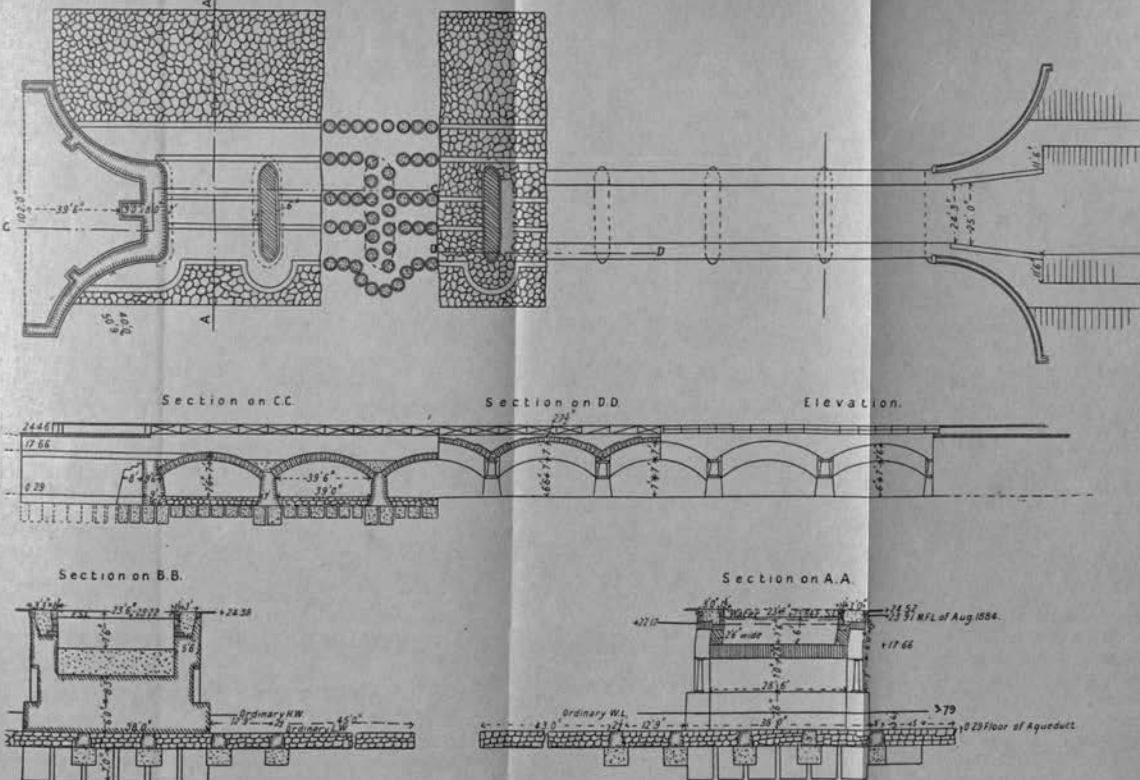


FIG. 4.—Gunneram Aqueduct, over Branch of Godaveri River. Forty-eight Spans of 40 feet.

[To face p. 322.

is occupied by the spreading bases of the piers. A continuous floor of pitched rubble 3 feet deep, protected by a series of horizontal curtain walls founded on shallow wells, the whole 6 feet deep, is provided. The existence of this floor is what undoubtedly prevented the pier foundations from being undermined during the exceptional flood alluded to. Deep reinforced concrete sheet piling fore and aft of the floor would, however, make a better protection with mass concrete filled in between.

Owing to the great difference in the material of which the respective river beds are composed, no useful comparison between a work which is the exponent of shallow foundations and the Nadrai Aqueduct can well be made. Under the conditions prevailing in the case of the Kali Nadi, shallow foundations, as have proved so successful in the Gunneram Aqueduct, would undoubtedly be a failure, owing to the great velocity engendered and the non-resisting character of the material of the river bed, unless they were continued for a very long distance below the work and reinforced by sheet piling. Such a construction, though it would interfere considerably with the regimen of the river, would doubtless stand, but its cost would probably be equal to that of the deep foundations adopted, in addition to which the point of dangerous scour would only be transferred lower down the river bed.

(**17**) As regards details, the parapets and spandrel walls are remarkably slight, being only $2\frac{1}{2}$ feet thick. The spandrels are, however, filled up solid with concrete to the level of the intrados of the arches. The widening of the parapet to form a roadway is effected by external arches springing at a higher level, which level is horizontal throughout, not on a falling incline as is the case with the main arches. These arches are carried by stone columns built on the cut-waters. This projection is 3 feet wide, allowing a top width of $5\frac{1}{2}$ feet for the roadway on either side. A better arrangement and much lighter, would have been to have made the parapet walls thicker say 4 feet, and used iron brackets for the widening at top. The thickness of the arches at the crown is 2 feet 9 inches, or $\cdot 45 \sqrt{r}$.

The piers are 6 feet wide at the spring line, or $\cdot 15S$, and widen out to 7 feet at bed level, and to 10 feet at top of the well foundation. These appear to be in good proportion, $\sqrt{S}$ being 6·32 feet. The section of the abutments is very light, due credit being given to the large buttress in the centre.

These abutments, with 40 feet spans, are of four-fifths less sectional area than those of the Kerai Aqueduct, with only 20 feet spans, the absurdity of which has been already noticed when reviewing the details of that work.

The disposition of the wings is peculiar. There are two large concave level crested wings, which start from the outside of the abutments, forming water wings, inside which is the earth of the embankments, while the water is apparently contained by two inner walls, which are not shown to have any foundation, and may be just vertical walls of dry masonry.

This arrangement is probably due to there being no natural banks to the river, the canal embankment itself confining the river within the bridge

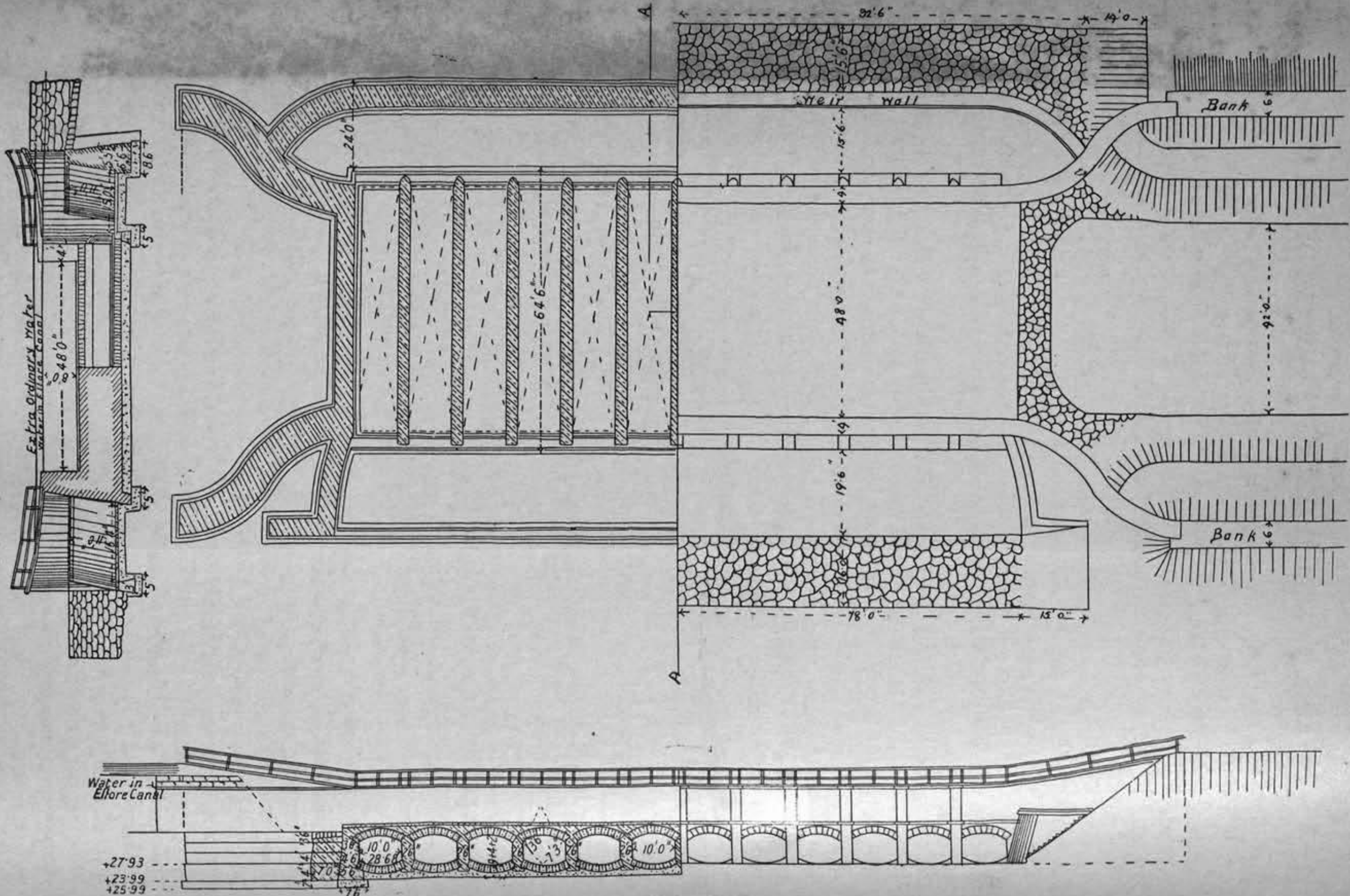


Fig. 5.—Kesarapali Aqueduct, Ellore Canal.

abutments, and these run at right angles to the axis of the stream. Without a cross-section at this point the exact arrangement of the junction of the earthen bank with the masonry of the bridges cannot be ascertained, the plan over-all being certainly defective. This work is contemporary, or nearly so, with the Solani Aqueduct on the old Ganges Canal, having been finished in 1852.

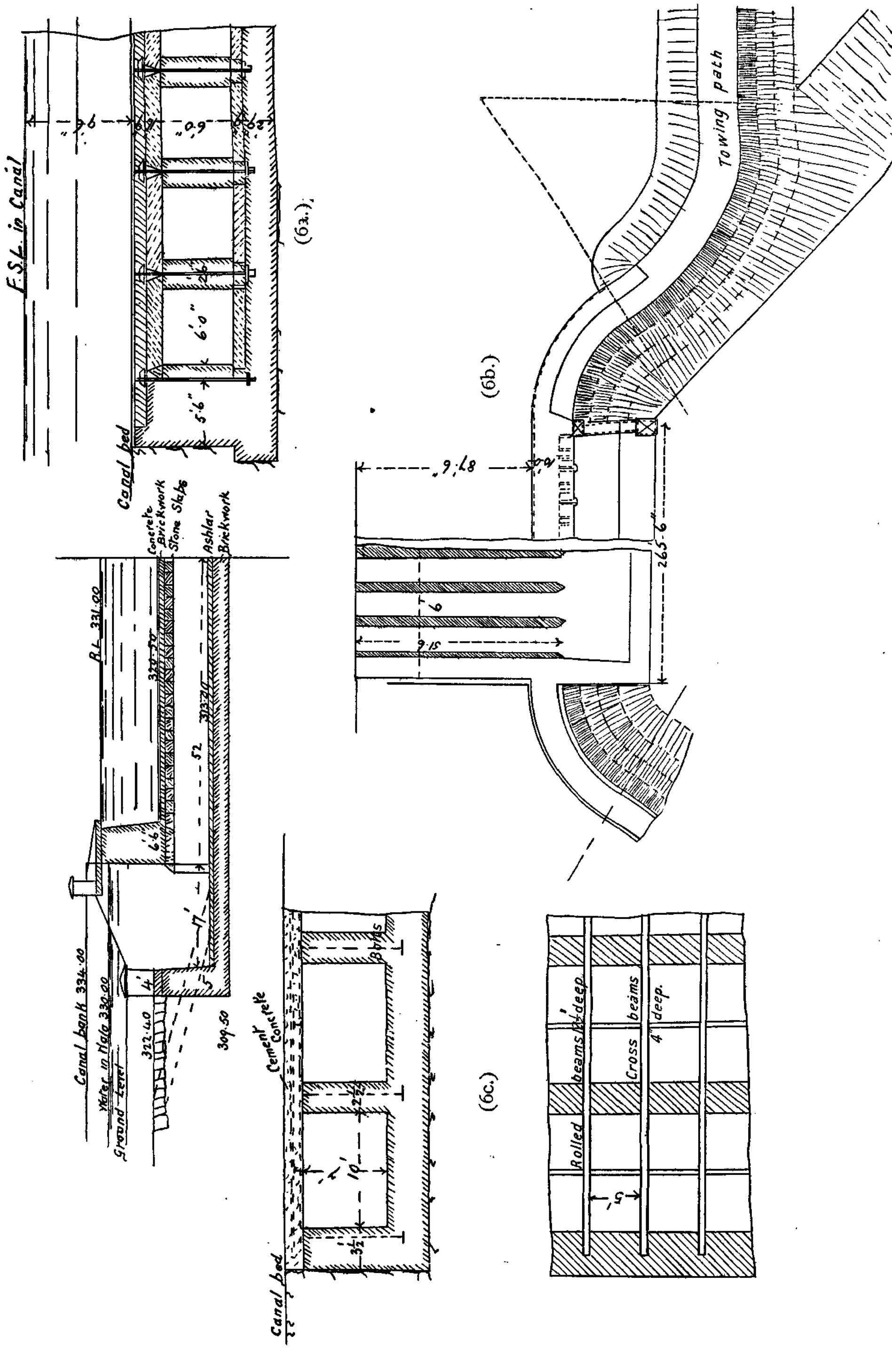
(18) Figs. 5 contain the plans of the Kesarapali Aqueduct on the Ellore Canal, Madras. This work is rather remarkable in having a clear overfall of 12 feet in the bed of the drainage line ; the design thus resolves itself into a bridged fall, with the bed of the canal below that of the upper reach of the drainage line.

A depressed bed involving a so-called syphon has been avoided, which is always to be effected if practicable, as syphons are very liable to fill up with detritus and cause a breach in the canal. Another marked peculiarity in this design is that the crests of the parapet of the aqueduct are purposely kept only just above F.S.L. ; this is to enable surplus canal water to spill over either side of the aqueduct into the drain. The work then fills the *rôle* of an escape as well as an aqueduct.

The openings are 10 feet wide by $5\frac{1}{2}$ feet high, and are provided with inverts which take up a great deal of waterway and could well be dispensed with, the floor being made thicker, if necessary. There is little or no upward pressure on the roof, so that arched vaulting is suitable. The design as a whole and in detail must be pronounced excellent.

(19) Figs. 6, 6a and 6b are a representation of the Kao Nadi Syphon Aqueduct, carrying drainage underneath the Sôn Canal. Both halves of the work are exactly similar. In order to save headway the tunnels are vaulted with flat ashlar slabs, which are held down by bolts passing through the piers, and are covered with a layer of brickwork and concrete. A 10 feet roadway is provided on each side, consisting of slabs of stone resting on stone corbels.

(20) The flood level of the outside water is R.L. 330; that of the underside of the slab roof is 319. There is consequently a head of 11 feet of water acting upwards, tending to lift or break the roof when the canal is empty. When the canal is full and the drain empty, the weight of water pressing on the slabs is that due to $9\frac{1}{2}$ feet depth of water. The former pressure is the greater, but in calculating the thickness required for the slabs the weight of the slabs and the superimposed masonry and concrete is in their favour, whereas in the latter case it is against their resistance to rupture, so that the slabs are subjected to a uniform load of, say, 10 feet of water plus their own weight. Taking this at a specific gravity of $2\frac{1}{4}$ and the thickness being $2\frac{1}{2}$ feet, the load is equivalent to a weight of $10 + (2\frac{1}{2} \times 2\frac{1}{4}) = 16$ feet of water, weighing $\frac{16}{36} \times 20$ cwt. $= 9$ cwt. per square foot, the span, or L, being 6 feet, or 72 inches, the width B being 12 inches. To find the required depth, using a factor of safety of 4, the breaking weight, or W, is 36 cwt. per



FIGS. 6, 6a, 6b, 6c.—Kao Nadi Syphon Aqueduct, Sôn Canal.

square foot. Then the following formula for strength of rectangular beams uniformly loaded, taken from "Molesworth's Pocket Book," p. 136, will state the case, viz., $W = \frac{8KBD^2}{L}$. This involves two unknown quantities, D and K, of which K, *i.e.*, the coefficient of rupture, must be obtained by experiment. This is easily done by having some beams, say 1 or 2 inches square, cut off the stone to be used, laid out on supports at a convenient distance apart, say 4 feet, and loaded in the centre by a hook passed over carrying a weighing platform on which weights are placed by degrees till the beam breaks. The weight of the platform, etc., should naturally be included in the count. Then K, or the modulus of rupture, will equal $\frac{LW}{4BD^2}$, either in inch lbs. or inch cwts. as W is taken in lbs. or cwts. In this case L is the length between supports of the beam operated on, W the average breaking weight of several experiments, and B and D the breadth and depth of the beam placed under loading. Having thus obtained a reliable value for K, it can be substituted in the formula $W = \frac{8KBD^2}{L}$, whence W being four times the distributed load on the slab 1 foot wide in cwts. or lbs., B being the breadth 1 foot or 12 inches, and L the length between the piers, likewise in inches. D, the required depth, will be $\sqrt{\frac{LW}{8B}}$.

We have seen that the upward pressure below the slabs is that due to a head of 11 feet of water; from this should be deducted the weight of the slabs and superimposed material equal to a corresponding weight of water per square foot of $2\frac{1}{2} + 2\frac{1}{4} = 6$ feet. The balance of head acting will then be $11 - 6 = 5$ feet of water or $\frac{5}{36}$ tons per square foot. This will have to be taken by the tie rods. Supposing the ties to be 5 feet apart, the tension on each will be $5 \times 6 + \frac{5}{36} = 4$ tons nearly (the length of the slabs being 6 feet), requiring a sectional area of $\frac{4}{5}$ square inches, or $1\frac{1}{8}$ inches diameter.

(21) In this example the levels of the inlet and outlet of the syphon are the same, hence the water will be higher one side than the other to allow for the increased velocity which is generally allowed for in designing these works.

The discharge or the velocity through a syphon or culvert is determined by the use of the Chezy formula $V = 100c\sqrt{RS}$, or $100c\sqrt{R\frac{h}{l}}$, using Kutter's coefficients. In the latter expression h is the head, l the length of the barrel. From this $h = \frac{lv^2}{(100c)^2 \times R}$.

The value of c is obtained from tables of Kutter's coefficients, that of h being assumed and subsequently corrected. The coefficient is but slightly affected by the value of S, the hydraulic slope, but is mainly a function of R, the hydraulic mean depth, or of $\frac{A}{WP}$, *i.e.*, the area ÷ wetted perimeter.

To the value of h thus obtained (v being given) must be added the loss of head at entry which is $h = \left\{\left(\frac{1}{c_1}\right)^2 - 1\right\} \frac{v^2}{2g}$. (1)

In this the coefficient c_1 is that appertaining to the velocity formula of a jet given in par. 1, Chap. V., and may be said to vary from ·62 upwards ; v is the velocity in the syphon barrel.

A selection of the proper coefficient will have to be made from the list in par. 8, Chap. V., modified by the shape of the barrel according to the table given in par. 7. Thus for an oblong-shaped vent a higher coefficient should be adopted than for a square one.

As a general rule ·8 is an average value for c if the orifice is square, $\left\{\left(\frac{1}{c_1}\right)^2 - 1\right\}$ in this case would be ·5625. If the width is twice the depth, c will be increased, according to par. 7, Chap. V., by 1·03, or become ·824, and the expression $\left\{\left(\frac{1}{c^1}\right)^2 - 1\right\}$ will be ·456. A value of ·5 is often adopted.

With velocity of approach taken into consideration, the above formula should read approximately $h_1 = \left\{\left(\frac{1}{c_1}\right)^2 - 1\right\} \frac{v^2}{2g} - \frac{{v_1}^2}{2g}$, v_1 being the velocity of approach.

The value of h to produce the velocity v in the syphon should also be reduced by that of the velocity of approach which, not strictly accurate, is sufficiently approximate for practical use.

(22) A practical example will now be worked out, one barrel of the Kao Nadi Syphon, Fig. 6, being taken. The area of this is 36 feet, the wetted perimeter 24 ; consequently $R = 36 \div 24 = 1·5$. The velocity adopted will be 8 feet per second; in this case $\frac{v_2}{2g} =$ unity. The discharge will then be $36 \times 8 = 288$ second-feet. The length of barrel (l) is 104 feet. It is required to find the head to produce this velocity. As before we have $h = \frac{lv^2}{(100c)^2 + R}$.

Here the value of c for $R = 1·5$ and $S = 1$ in 1,000 with n taken at the usual value of ·013 is, according to Table XII., Part II., page 142, "Hyd. Manual," is 1·249, say 1·25. The true value of S, or of $\frac{h}{l}$, in this case will be much in excess of 1 in 100, being more like 5 in 1,000, but in Jackson's tables slopes steeper than 1 in 1,000 are all accounted as of that slope, as the coefficient varies but slightly : h will then be $\frac{104 \times 64}{(1·25)^2 \times 1·5} = \frac{6,626}{234,375} =$ ·283 feet.

To this we must add loss of head due to entry, or

$$\text{with } c_1 = 8$$
$$h_1 = ·5625 \times 1 = ·562$$

Total ·84 feet.

The head due to velocity of approach, assumed at 3 feet per second, will be $3 \times \cdot 01555 = \cdot 04$ feet. If this be twice deducted, the head required would be reduced from ·84 to ·76 feet. A somewhat easier method with same result is by use of formulus 23—26, par. 67, Chap. V.

(23) In designing a syphon in which a natural stream or river is carried underneath a canal, care must be taken that the channel below the work is of sufficient slope and sectional area to carry off the accumulation of water that is backed up above the work, as for a time the channel will be called upon to carry a discharge in excess of the maximum flood discharge; this involves the straightening or widening of the tail channel.

(24) Referring again to Fig. 6, the disposition of the wings and the narrowing of the waterway of the canal is shown in the general half plan (Fig. 6b). The land wings, as usual, are curved on plan, which in the case of aqueducts is undoubtedly the only suitable form, while two straight flank walls with sloping crests enclose the cistern at the head and tail of the syphon. The vertical drop at the exit end is a bad form, as it prevents detritus being washed through the syphon, necessitating its removal by excavation. The exit end should not be a dead wall but an easy incline, as shown by the dotted lines in Fig. 6. In most modern designs this sloping approach is provided at the inlet end as well as the outlet of the syphon, as exemplified in Fig. 9, *post*, though it is not absolutely necessary at the upper end.

(25) The drawback to a slabbed culvert of this description is the limit it imposes on the span of the openings and the great expense of the ashlar slabs.

A far better construction would be to form a flat concrete roof supported by rolled iron beams, or one of reinforced concrete ; the span could then easily be increased to 10 feet, and the cost would be very much less.

Fig. 6c shows the construction.

The beams are 10 feet span and 5 feet apart. The weight on them will be equivalent to 15 feet of water, or the total distributed load $= 10 \times 5 \times 15 \times \frac{20}{36} = 417$ cwt., the weight of a cubic foot of water being $\frac{1}{36}$ of a ton. From the Table given in " Molesworth's Pocket Book," p. 146, a beam $12\frac{1}{2}$ inches deep with 6-inch flanges and $\frac{11}{16}$-inch web will safely carry this load.

Cross-beams of small section will be laid along the centre of the span between each main girder, thus dividing the platform into 5 feet squares. The total depth of the roof can be reduced from $2\frac{1}{2}$ feet to $1\frac{1}{2}$ feet, giving an additional 1 foot in the headway of the sluices.

(26) Another instructive example of a syphon passing drainage underneath a canal is given in Fig. 7. In the transverse section it will be seen that there is a fall in the syphon of $3\frac{1}{2}$ feet. This will enable a large discharge to be passed at increased velocity through the work, the waterway being considerably curtailed to what would otherwise be required. The spans are

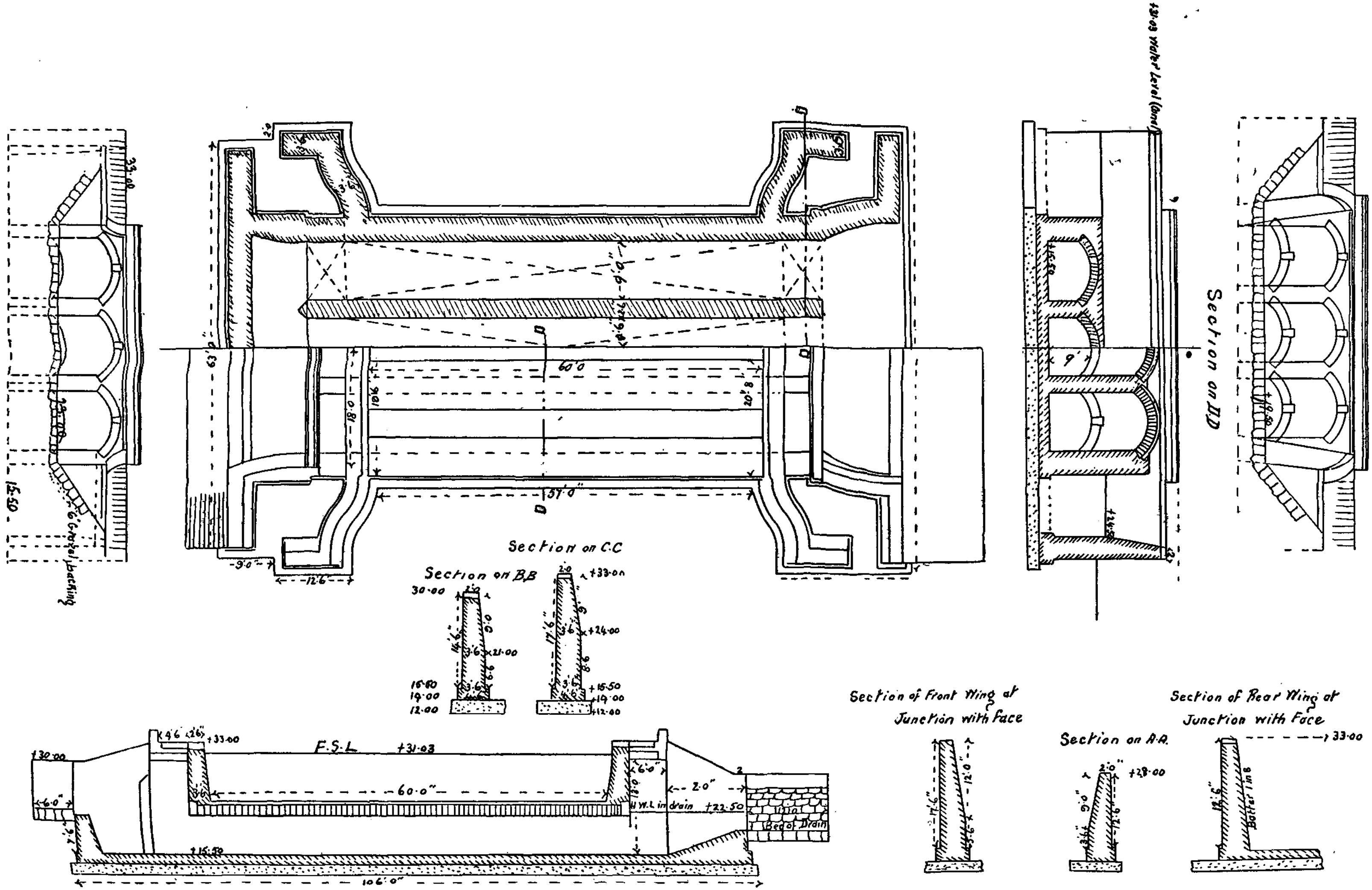


Fig. 7.—Syphon Aqueduct on Ellore Canal, East Godaveri (Madras).

9 feet, the piers $2\frac{1}{2}$ feet thick, the headway 6 feet, and the floor $3\frac{1}{2}$ feet thick. The abutment is of reasonable dimensions. The disposition of the wings is apparently good. The earth lines, so necessary to form a proper idea of the suitability of the wings, are not given.

The canal is 7 feet deep and the parapets are 3 feet 6 inches wide at base and 2 feet 6 inches at top. The roadway is formed by an outer archway of 6 feet in width, the piers being lengthened to carry it. This provides a roadway of 7 feet. The exit side of the work has a sloping rise to the bed level of the stream, which is as it should be. The head producing upward pressure is 4 feet. The arch, with concrete covering at the crown, is 2 feet thick, and with a specific gravity of two will just balance this pressure. No exception can be taken to any of the details, except possibly that a thicker parapet corbelled out on both sides would provide an equally wide roadway, and thus save the lengthening of the piers and the external high level arch.

(27) A very recent instance of a design to pass a canal under a river is illustrated in Fig. 8 of the Burra Bubsa Syphon, Trebeni Canal.

This is a better design than any hitherto given, the main point in betterment being the double approach slope provided. This is obviously the best arrangement as it encourages the scour of material, which in the case of the Kao Nadi syphon must be left in the barrels or in the approach cisterns, and require clearance possibly several times in the year. The greatest possible head acting on the roof of the barrel is about 6 feet. The counter-weight is only $2\frac{3}{4}$ feet depth of masonry at specific gravity 2, equivalent to $5\frac{1}{2}$ feet of water. This leaves a little over $\frac{1}{2}$ foot of water pressure unbalanced, producing tension in the platform. In this calculation the drop in the water surface or hydraulic gradient is allowed for.

A peculiarity in this design is that the F.S.L. of the canal and high flood in the river are at the same level. On the canal side up stream, the parapet of the superpassage is provided with vents 4 feet by 4 feet by which water can be allowed to fall into the canal syphon, or else the canal can be allowed to overflow through these vents into the river above. In the approach slope of 1 in 10, a horizontal step is provided for the water from above to fall on. These vents can be closed by baulks, and another set of grooves at the head of the syphon are for flashboards to prevent the river from flowing up the canal if the latter is empty when the overflow takes place.

(28) The Ravi Syphon, the section of one barrel of which is given in Fig. 9, forms a part of one of the largest irrigation projects ever undertaken. It is now in progress, and is estimated to cost £5,300,000, or 26 million dollars. The irrigation effected will amount to 1,876,000 acres from 500 miles of main and branch canals and 2,714 miles of distributaries. The revenue is expected to reach a net amount of £520,000 and a gross amount of £640,000, the former yielding 10 per cent. interest on the capital expended.

The accompanying map (Fig. 9a), for which, with all this information, we are indebted to Mr. Buckley's "The Irrigation Works of India," is explanatory of the scheme. The Jhelum River has a large surplus "low" supply of water which cannot be utilised by the existing canal, which fully irrigates the Jhelum-Chenab, or Jech Doab, excepting 1,100 square miles of country in the upper part of the canal, which is too high to be reached by

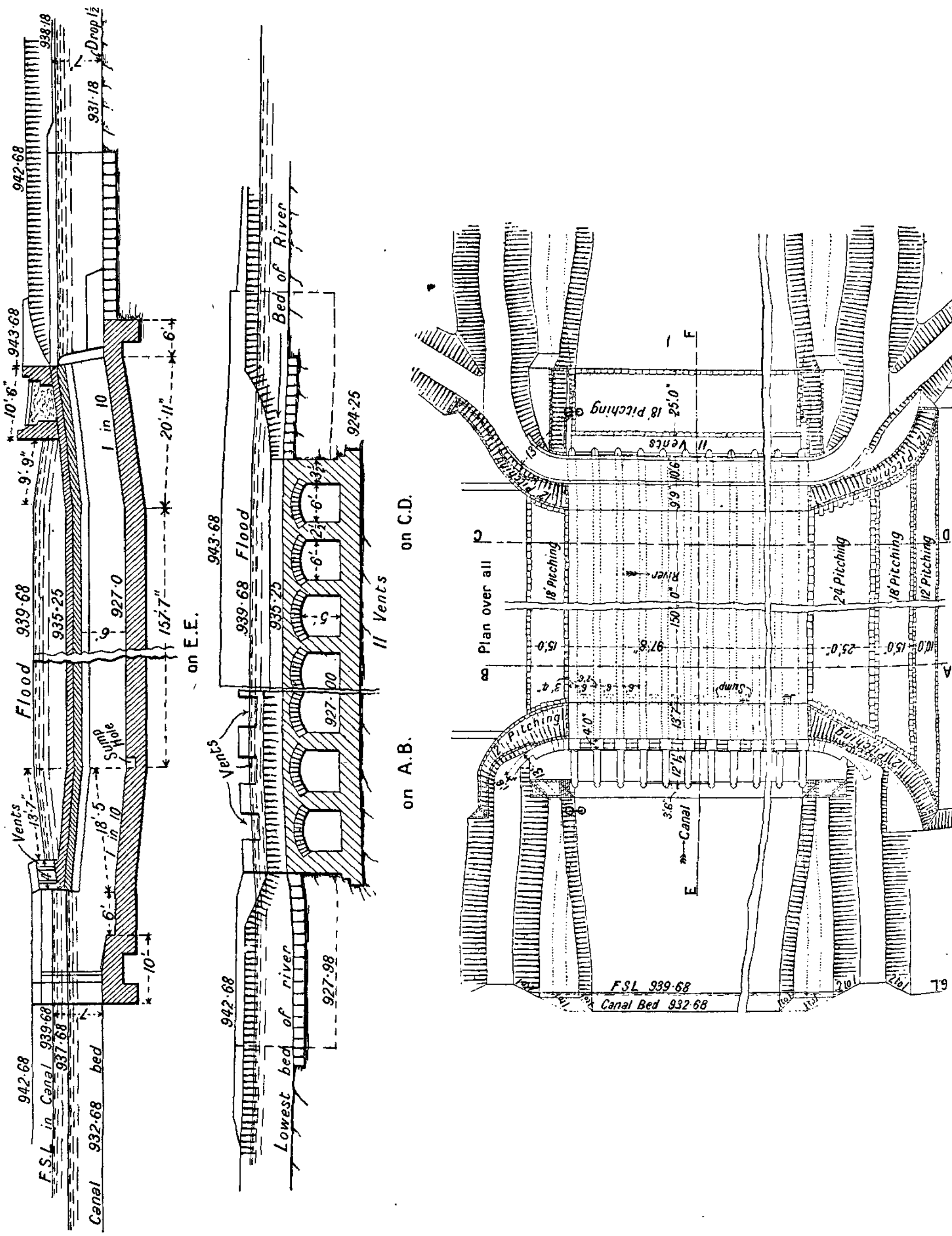


FIG. 8.—Burra Bubsa Syphon, Trebeni Canal (Bengal).

the water. As much as 6,000 to 8,000 second-feet can be taken out of this river in the winter months after giving the existing canal all it wants. The project thus consists of a new weir over the Jhelum at a much higher level than the one at Rasul. The water drawn from this will be thrown into the

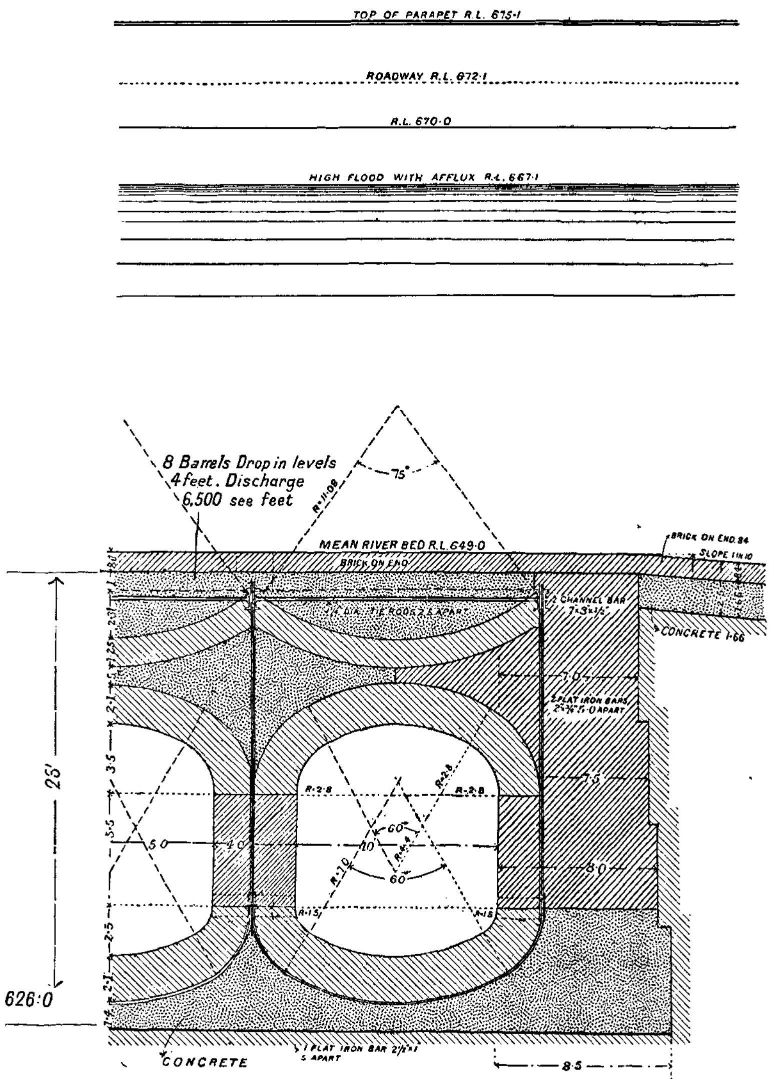


FIG. 9.—Section of part of the Ravi River Syphon, Upper Chenab Canal.

Chenab River above the present Khanki Weir, and on the way will irrigate the 1,159 square miles of the so-called Jech Doab. It may be mentioned that "Doab" means literally two waters, and designates the tract lying between two rivers.

(29) The 6,000 cubic feet thrown into the Chenab Canal will all be used

in the existing canal, while its former supply will be partly cut off by another weir to be constructed higher up the river. Above this weir a canal, of carrying capacity of some 12,000 second-feet, called the Upper Chenab Canal, will irrigate another Doab, the Ravi-Chenab, or Upper Rechna, and the surplus water, after effecting this irrigation, 6,500 second-feet, will be

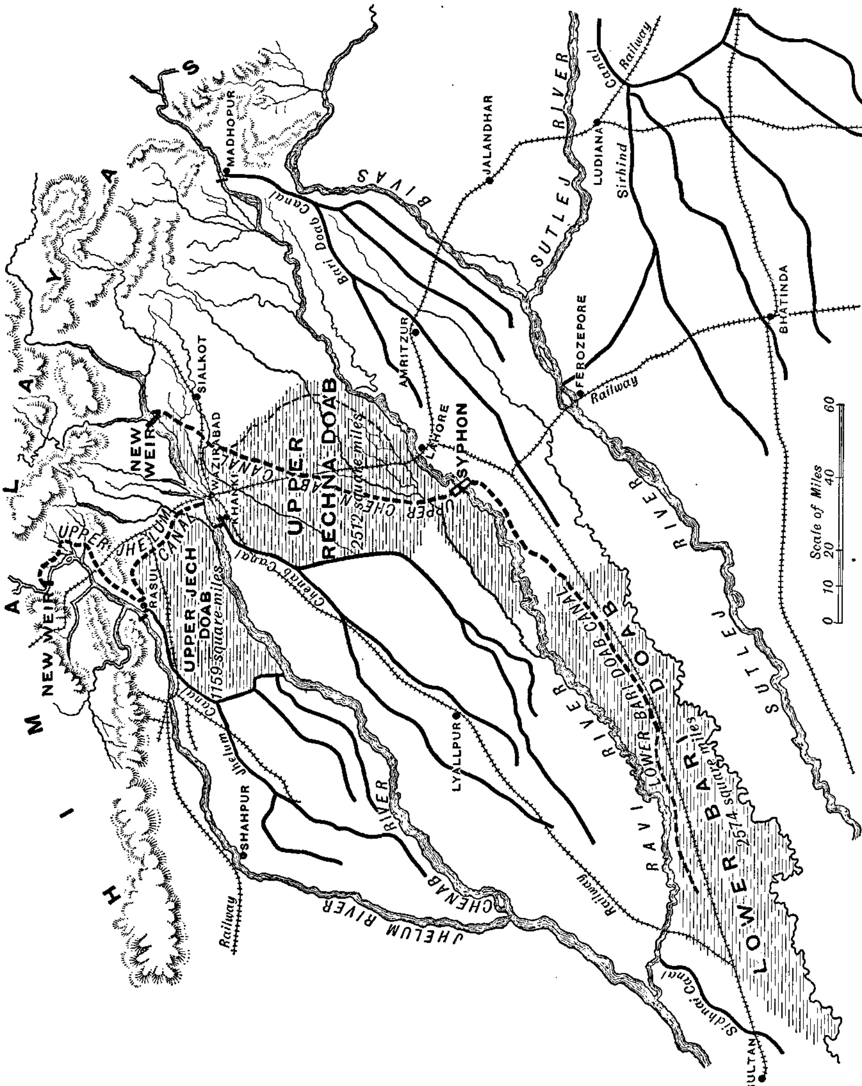


FIG. 9a.—Map of New Canals projected in the Punjab, now under construction.

carried under the Ravi River and will then irrigate the Lower Bari or Beas-Ravi Doab.

The Full Supply Level in the canal above the syphon is 668·75, while the soffit of the arch is 641·00; the head acting upwards is therefore 27·75 feet of water. This could not possibly be met by dead weight, as the cost would be prohibitive, consequently each barrel roof is tied down with steel rods and straps.

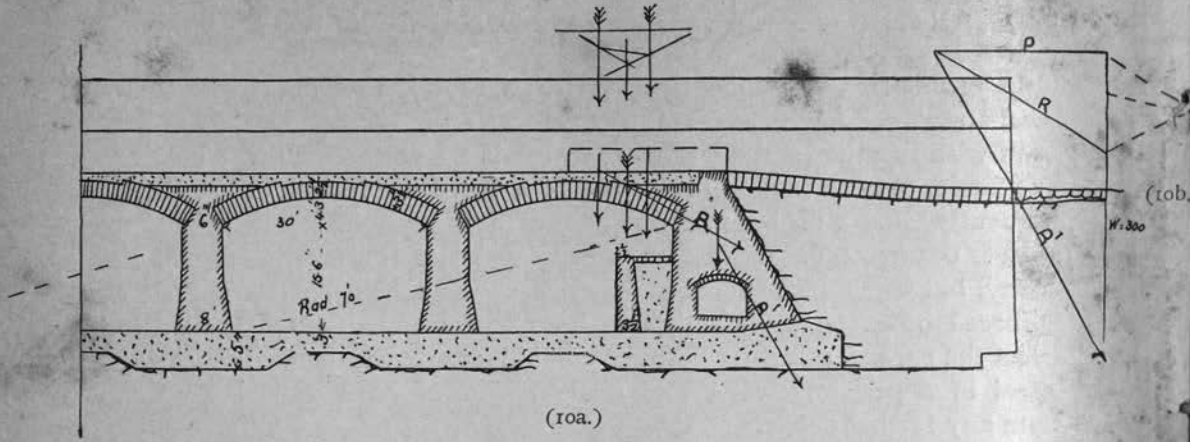
Even with this the total depth of the work is 23 feet, which will involve great difficulty in construction in the bed of a hill torrent such as the Ravi. Consequently this will become one of the most notable single irrigation works ever constructed. Some fear is expressed lest the steel work should rust away, but experience gained in America seems conclusive that the steel in reinforced concrete is perfectly free from corrosion, although soaked in water, as the surface rust first formed combines with the cement, encasing the steel in a perfectly water and airtight skin of hard material, which prevents any further decay.

The Ravi River has a flood discharge of 200,000 second-feet. The syphon will be 1,400 feet long, and will have 8 barrels or vents, each having a superficial area of about 100 square feet; the velocity will therefore be $\frac{6500}{800} =$ 8 feet per second nearly.

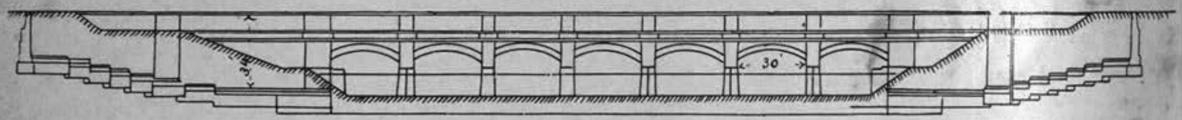
(30) Superpassages, where the levels are such that rivers have to be taken over a canal, are comparatively rare, as they involve a very heavy work if the stream or torrent is large, and so are avoided if possible. They generally occur in the upper section of a canal, the head work of which is situated among the lower hills high up in the course of a river. There are some very large works of this description in the old Ganges Canal. An excellent modern example is given in Fig. 10 of the Budki Superpassage on the Sirhind Canal in the Punjab.

The length of the work is seven spans of 30 feet, the width between parapets 400 feet. The arches have to be raised well above the canal bed on account of the exigencies of navigation, and also of the level of the bed of the torrent. The 30 feet wide spans are therefore suitable. The thickness of the arches at crown is $\cdot 45 \sqrt{r}.$ The thickness of the piers at springing is 6 feet or $\cdot 2S$. We have already seen that a proportion of $\sqrt{S}$, giving a thickness of $5\frac{1}{2}$ feet, is sufficient. The piers are therefore too thick. The bottom width could be retained at 8 feet, the piers having either a straight or a curved batter as has been designed. The foundations, which are probably on good soil, are admirable (*vide* par. 11, Chap. IV.).

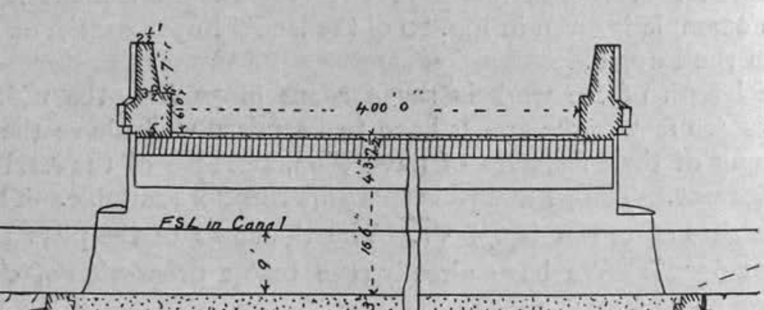
The abutment, by the look of the section, is clearly much too heavy, a very common fault among designers. To prove this the actual incidence of the resultant line of pressures on the base has been graphically found; the method of working, which has already been fully explained in Chap. IV., consists in first finding the centre of gravity of the half arch and its load of water, the latter reduced in depth, as shown by the horizontal dotted line, to an equivalent mass of masonry. This process is shown in Fig. 10b and the reciprocal funicular polygon above Fig. 10a. The forces 1 and 2 are the areas of the two halves into which the half arch has been divided. Having found the centre of gravity of the half arch, a horizontal line is drawn through the centre of the arch crown to intersect the vertical through this centre of

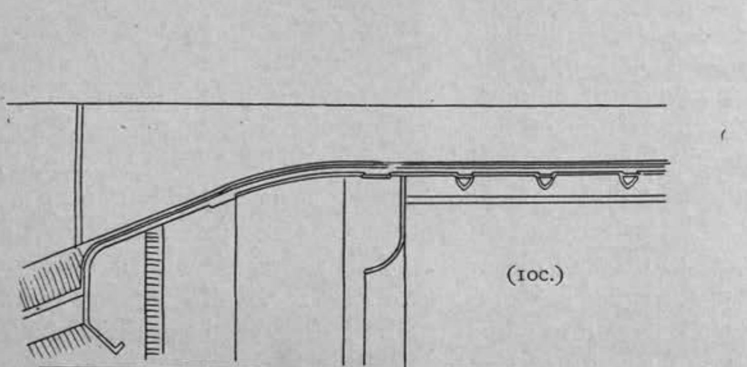


(10a.)



(10d.)





(10c.)

FIGS. 10, 10a, 10b, 10c, 10d.—Budki Superpassage, Sirhind Canal.

gravity, and from the point thus found the line R is drawn in Fig. 10a through the centre of the arch at its springing, till it intersects a vertical line through the centre of gravity of the abutment and its water load. In the force polygon (Fig. 10b), the load line, composed of the areas 1 and 2, is continued down to measure 300 square feet, the area of the abutment with its load of water, and the line R is then drawn from the termination of 1 and 2, parallel to its reciprocal in Fig. 10a, cutting the horizontal P at a point. From this point another line, R_1, joining the termination of the vertical load line 1, 2 and 3 just obtained, gives the final resultant R_1. This projected on the profile of the abutment in Fig. 10a from its proper starting point, viz., the

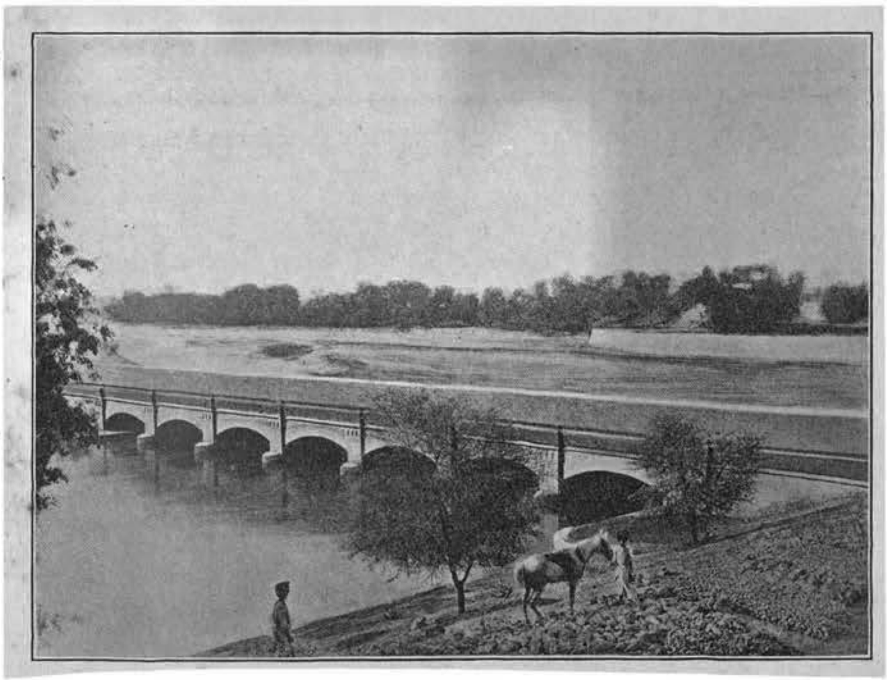
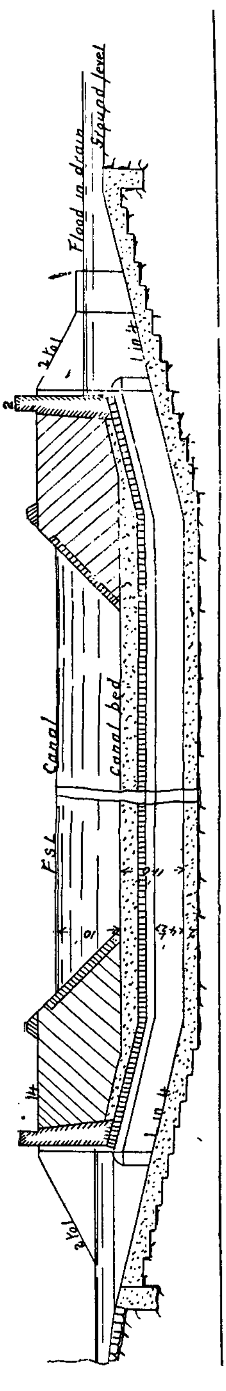


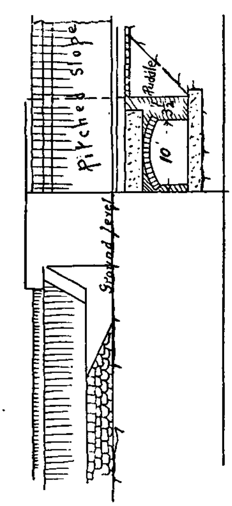
FIG. 10e.—Budki Superpassage, Sirhind Canal.

last intersection found, cuts the base of the abutment at a point some 5 feet within its heel. As no credit has been given to the weight of the earth backing with water above it, the resultant line R_1 need only just fall within the base. This proves that the abutment is unnecessarily thick. The line P represents the horizontal thrust of the arch, and if measured will be found to closely correspond with rt or $8 \times 30 = 240$, *i.e.*, the calculated horizontal thrust of the arch (*vide* par. 13, Chap. IV.). If the abutment were made 8 feet wide at the springing and 13 feet at the base it would probably be of sufficient section. As built, it is half as wide again as these dimensions, and besides is provided with a large buttress of which no account has been taken.

It might here be mentioned that the calculation of the effect of buttresses in an abutment or retaining wall is effected as follows :—The wall



(11.)



(11a.) (11b.)

FIGS. 11, 11a, 11b.—Syphon on Chenab Canal.

should be considered as having a base equal to its normal thickness plus the length of the buttresses, but formed of two materials of different specific gravities, the solid portion being of the proper specific gravity of the material and the part behind of a lighter specific gravity, equivalent to that of a material spread over the space of the same weight as the solid buttress only. Thus, supposing a wall is 6 feet thick and is provided with buttresses projecting 4 feet and 4 feet thick and 6 feet apart, *i.e.*, at 10 feet intervals, and let the specific gravity of the wall be 2, then the specific gravity of the 4 feet wide space behind will be $2 \times \frac{4}{10} = \frac{4}{5} = \cdot 8$, and the effective base width of the wall be 10 feet, not 6 feet.

The disposition of the wings is generally similar to that usually adopted in aqueducts, consisting of water wings as curved continuations of the faces of the abutments, and splayed land wings which carry parapets in continuation of those in the aqueduct proper. These wings are shown in the plan over all (Fig. 10c). As a further precaution, the ends of the land wings

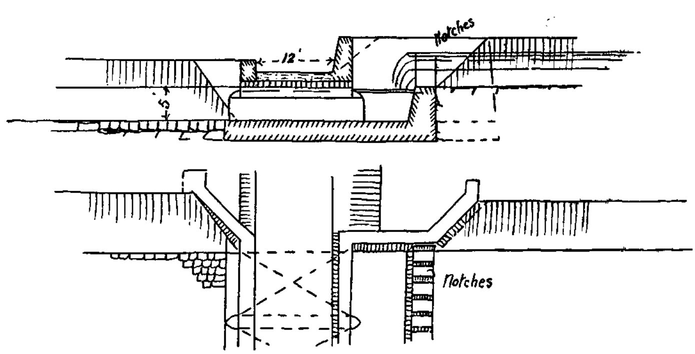


FIG. 12.

are connected by a cross wall at bed level, which apparently goes down to the full depth of the foundations. The land wings being in solid ground are stepped up in foundation, which is shown in the elevation Fig. 10d. A photograph of this work is given in Fig. 10e.

(31) Fig. 11 shows longitudinal and cross section of the type design used for syphons crossing drainage, adopted in the Chenab Canal, which, being a comparatively recent work, will probably contain better designs than those on the older canals. It will be noticed that the canal banks are carried right across the syphon with a pitched slope on the water side and a revetment wall outside. This is in order to admit cart communication across the aqueduct, which is always a necessity. This arrangement for a low culvert, two or three spans only in width, is probably the most economical, as it does away with the necessity of any land wings and of additional arches to carry the roadway, as adopted in Fig. 7. The upward head of water below the crown is 9 feet. To balance this a thickness of roof of $\frac{9}{\rho}$ is required; taking

the specific gravity or ρ as 1·8, the thickness should be $\frac{9}{1 \cdot 8} = 5$ feet. The actual average thickness of the arch and concrete above is about $4\frac{1}{2}$ feet. There is therefore tension in the masonry equivalent to the weight of a layer of brickwork, etc., of $\frac{1}{2}$ foot in thickness, which in lbs. is $\frac{1}{2} \times 1 \cdot 8 \times 62 \cdot 4 = 56$ lbs. per square foot. A few tie rods are thus required to carry this unbalanced stress, which otherwise would exert a transverse upward stress on the arch, which, from its construction, it is least adapted to bear. No doubt the cohesion of the mortar in the concrete above is more than sufficient to withstand this pressure, but in all masonry works, particularly those in connection with water, any tension should be avoided.

The general arrangements in this design are simple and excellent. The floor is only 2 feet thick, and the whole is made entirely of concrete. The proportionally wide spans are also a good point, as the wider they are the

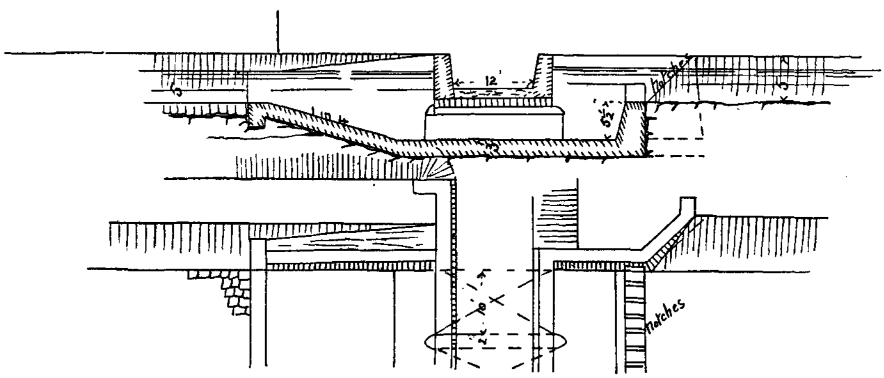


FIG. 13.

higher the coefficient of discharge will become. The double slope is a good arrangement for drainage lines, as it gives an impetus to the water and tends to carry detritus through the culvert. A direct overfall on the up-stream side, on the other hand, absorbs a great deal of the velocity of approach. The form of vertical drop above and slope below is suitable where a canal is taken under a drainage line, as being more economical.

(**32**) Distributaries and branch canals are often taken by a syphon underneath a low roadway or drainage line. In the former case an over-bridge is avoided as also the interception of the drainage that generally passes down a road. An example of such a work is given in Fig. 12, which represents a notch overfall weir with a low level bridge or culvert beyond, through which the water passes. Fig. 13 is a design for a similar work, but without any drop in the bed; the culverts in this case act as a syphon. In such cases it is usual to give a small drop to prevent any heading up. In this instance the notches are not carried up the full depth of the canal water, but only half-way, to allow a free passage when the syphon is full.

(33) Two Egyptian works are illustrated in Figs. 14 and 15. The first is a plan of a syphon under the Sohagiah Canal, taken from "Egyptian Irrigation." It has a vertical fall up stream and a slope down stream.

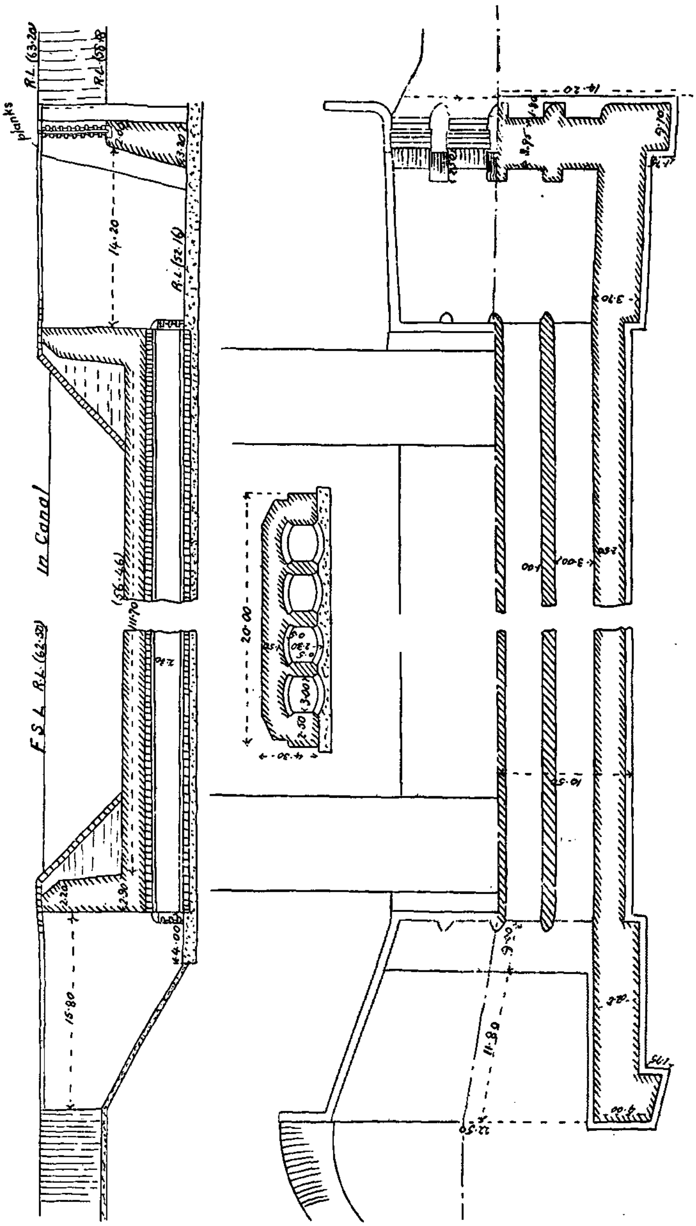


FIG. 14.—Sohagiah Canal Syphon (Egypt).

The fall is provided with grooved piers and a plank platform, possibly with a view of holding up the water to cause a scour when the stream is in low discharge. The well between the weir wall and the face of the syphon is of very great width, viz., 14 metres, or nearly 50 feet. This appears quite unnecessary. A width equal to the depth of water, *i.e.*, of about 10 metres, or 30 feet, would be ample. There does not appear to be any fall in the bed of the drain. The openings are about 10 feet wide and $7\frac{1}{2}$ feet high, the piers $3\frac{1}{4}$ feet thick, which is somewhat excessive. The thickness of the roof is 2 metres, or $6\frac{1}{2}$ feet, which is sufficient for a head of 13 feet. The flood level of the drain is not shown, but from the height of the banks it is probably about R.L. 61 at least. That of the intrados of the culvert arch is 54·50; the head then will be $6\frac{1}{2}$ metres, or $21\frac{1}{2}$ feet. The syphon is evidently not designed to run full with the canal empty, as it undoubtedly should be. This state of things is mentioned by Sir Wm. Willcocks, in

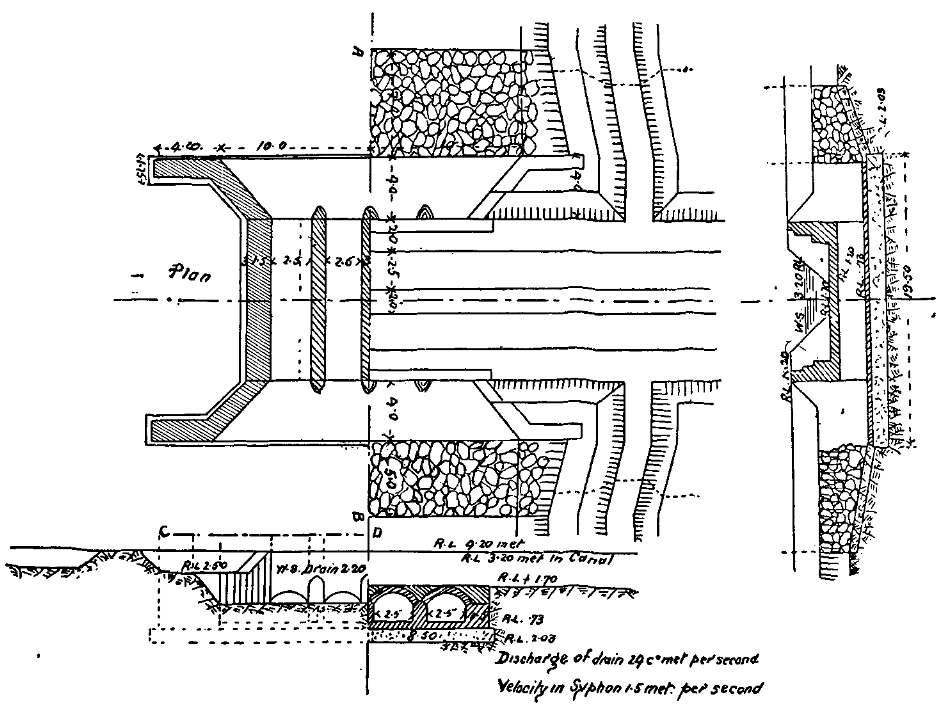


FIG. 15.—Nizam Drainage Syphon (Egypt).

"Egyptian Irrigation," many of the older works in Lower Egypt having been designed on the hypothesis that the canals would never run dry, *i.e.*, the supply would never be cut off from the head. Now, however, that head regulation is possible, these works are not equal to the pressure brought upon them, and this doubtless is one of them.

The section of the vents could easily be enlarged to great advantage, by adopting the system of a thin concrete roof, reinforced by iron beams or rods and bolted down as shown in Fig. 6c.

A clear headway of about 11 feet could then be secured, or else the floor could be raised a metre higher. The abutment, as is so commonly the case, is unnecessarily thick. The slope of the earthen banks is carried through the work. This saves land wings, but it is doubtful if any economy accrues considering the lengthening of the heavy syphon barrel which this system involves. On a small work it is doubtless suitable, but this is a very large work, though it does not appear so from the drawing, the scale being small. The distance apart of the parapets of the aqueduct is no less than 350 feet, and it carries 20 feet of water.

(34) Fig. 15 is another Egyptian syphon, but of quite small dimensions. This is provided with a double slope, which is of pitching. The disposition of the wings and the banks are given in the plan over-all, which is most

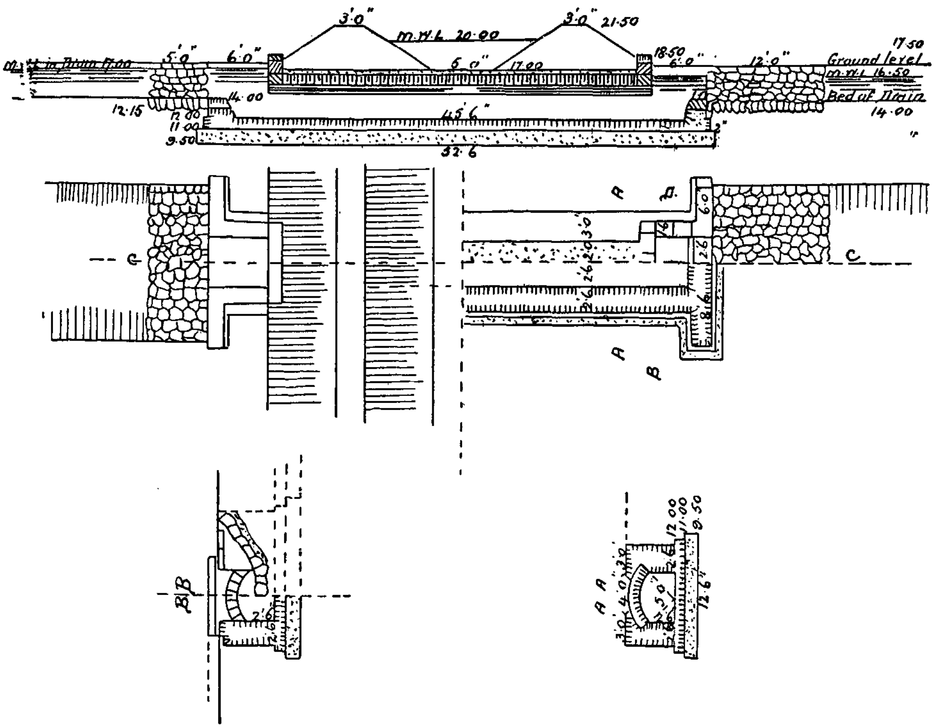


FIG. 16.—Canal Syphon (Madras).

useful as a guide in similar cases. In this design the banks of the canal are carried right across. The outer parapet walls are of too heavy a section, being nearly as wide at the base as the depth of water in the canal. The arches of the barrels might well have been made flatter.

(35) Fig. 16 is a type section of a syphon under a distributary from the "Madras Irrigation Manual." As will be seen, both earthen banks are carried entire across the syphon; the general arrangement is simple and effective.

(36) Fig. 17 is a type diagram of a superpassage on a small scale taken from the same source as Fig. 16. This is provided with land and water wings

of the usual Madras pattern, the battered section of which is a vast improvement on the unscientific vertical faced revetment walls, so absolutely universal in Upper India and Egypt. The design appears a good and economical one.

(37) In Egypt of late years wrought-iron pipes have been used for both

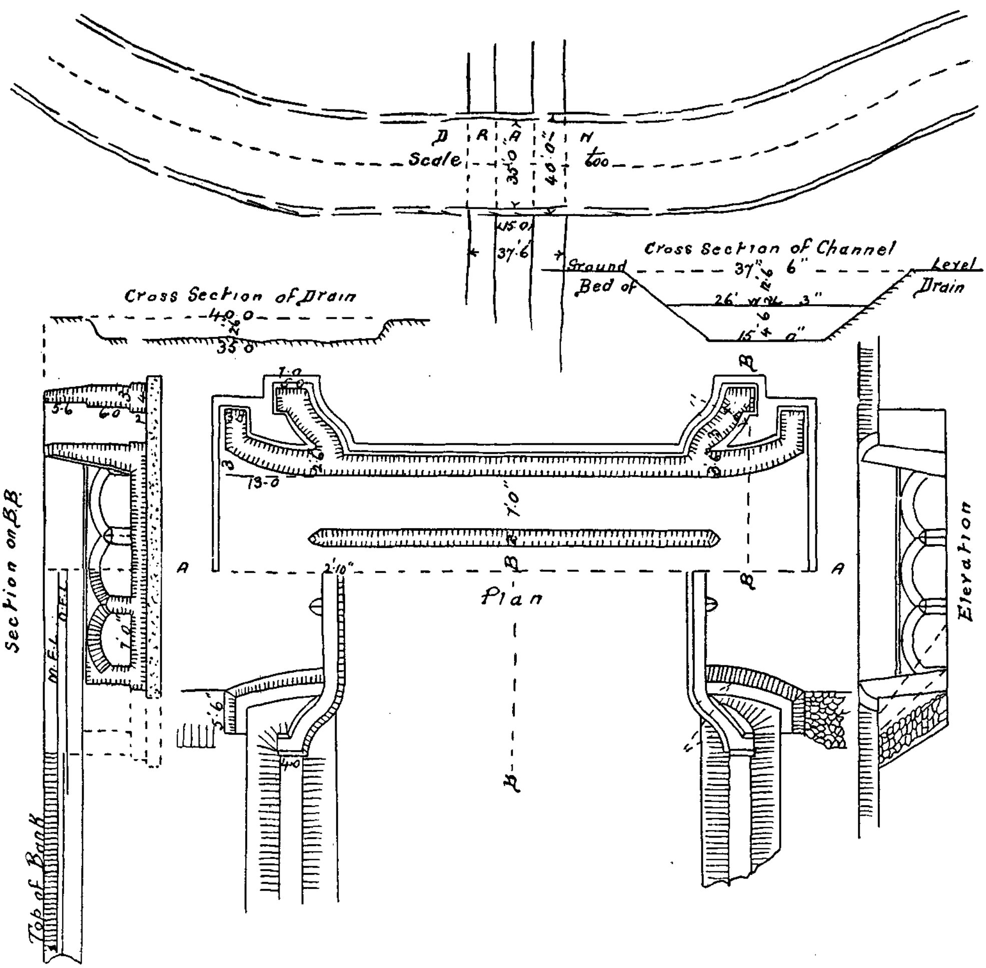


FIG. 17.—Type Diagram of Superpassage (Madras Canals).

aqueducts and syphons. The following is a description of them from "Egyptian Irrigation":—

"They are generally constructed of ¼-inch sheet iron, butt jointed, stiffened with angle irons at every alternate joint if over 12 feet in circumference, and lap jointed if under 12 feet. Since the sheets in the market are 8 feet by 4 feet, or 6 feet by 3 feet, the pipes are always constructed with their circumference some multiple of the length or width, so that there may be no cutting of plates. The pipes are sometimes laid on a bed of concrete, varying from 1 metre to 25 centimetres in thickness, according to the quality of the soil, or they are laid on the hard clay soil and well pitched round with clay balls. Where the pipes are used as aqueducts they are generally supported on wooden trestles. The great advantage of using wrought-iron pipes is that they can easily be transported; they do not need expensive supervision during construction, and can be put together so rapidly that the cost and trouble of a diversion for the canal during the time of construction is avoided. These pipes can be closed at the ends and floated to their destination. By dredging the foundations where they have to be laid they can be floated over the site and then sunk without shutting the canal head."

A pipe of 16 feet circumference will discharge 300 cubic feet per second, with a head of 2 metres or 6½ feet, and 156 cubic feet with a head of half a metre or 1·6 feet. One of 12 feet circumference, 180 cubic feet per second, with a head of 2 metres, and 87 cubic feet with a head of half a metre.

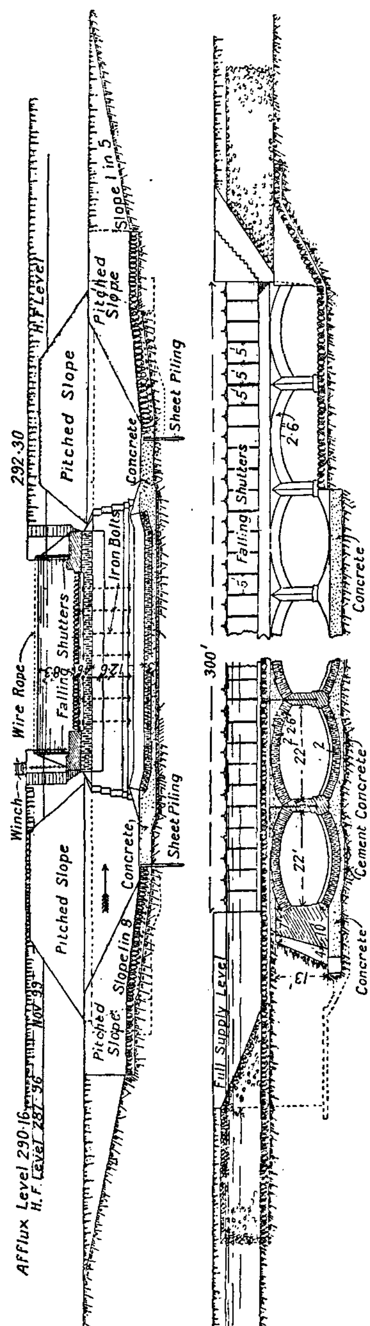


FIG. 18.—Thapangaing Aqueduct and Level Crossing (Burma).

(38) Where the drainage is slight, inlets can be provided; they simply consist of a small fall or pitched rapid, protected by flank walls, which conducts the drainage water into the canal, the floor and banks of the latter being pitched all round to prevent damage. If the inlet occurs at the roadway bank, it has to be bridged. There are no plans of any level crossings available, and it is believed that there is only one example extant. The Dhanauri on the Ganges Canal, which design is too antiquated to be of any use as a guide. A level crossing consists of an open inlet on the side of the torrent, a regulating bridge across the canal, and another at right angles across the exit of the torrent, which is generally provided with a fall to expedite the speedy removal

of the unwelcome guest. It is very essential that quick-acting falling gates be supplied to the outlet bridge; draw gates would be much too slow in manipulation. The balanced pivoted gates recommended for the Koshesha Escape would, it is believed, answer admirably for such a purpose.

(**39**) Cases in which the passage of drainage across a canal is beset with the following difficulties are enumerated below.

(1) The drainage is too extensive to be wholly admitted into the canal.

(2) It cannot be taken underneath in a syphon drain, owing to the presence of the backwater of the parent river when in flood, which comes up to the very canal bank, hindering the free discharge.

(3) It thus must be disposed of by a superpassage; but this involves the formation of a large deep reservoir on one side of the canal, which has to fill

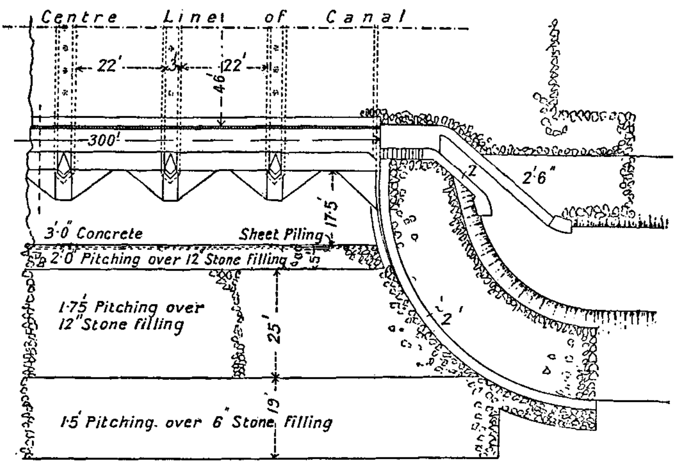


FIG. 18a.—Thapangaing Aqueduct.

up before sufficient elevation is attained to admit of surplus passing at a high level over the canal.

This problem was satisfactorily solved in the case of the Ali Superpassage and concomitant works in the third or fourth mile of the Agra Canal. The works constructed consisted of, firstly, a solid embankment on the upper side of the canal which crossed a large depression; secondly, a water tower built some way off the canal in the depression, provided with external sluice openings at different levels; thirdly, a culvert which connected the base of the tower with an inlet into the canal itself; lastly, an iron girder superpassage with a masonry fall in the further (river side) bank. This arrangement has answered well. When the depression fills up to a moderate extent all its water can be gradually drawn off through the tower into the canal. When, however, as happens not annually but occasionally, the tributary streams bring down so much water as to quite fill the reservoir, the surplus is then disposed of automatically by passing over the superpassage into the river. When the crisis of the flood is over, the deep reservoir can be gradually tapped into the canal by means of the water tower and inlet culvert. Unfortunately no plans are available.

(40) The subject of this chapter cannot be considered as exhausted without the insertion of the plans of the Thapangaing so-called aqueduct. Figs. 18 and 18a contain the plan of the remarkable work which is really a combined syphon and level crossing. The work was originally designed as a syphon to carry a hill-torrent underneath the Mandalay Canal. During its construction an immense flood came down of an estimated discharge of 57,000 second-feet. The previously estimated flood discharge on which the design of the syphon was based amounted to 24,000 second-feet; con-

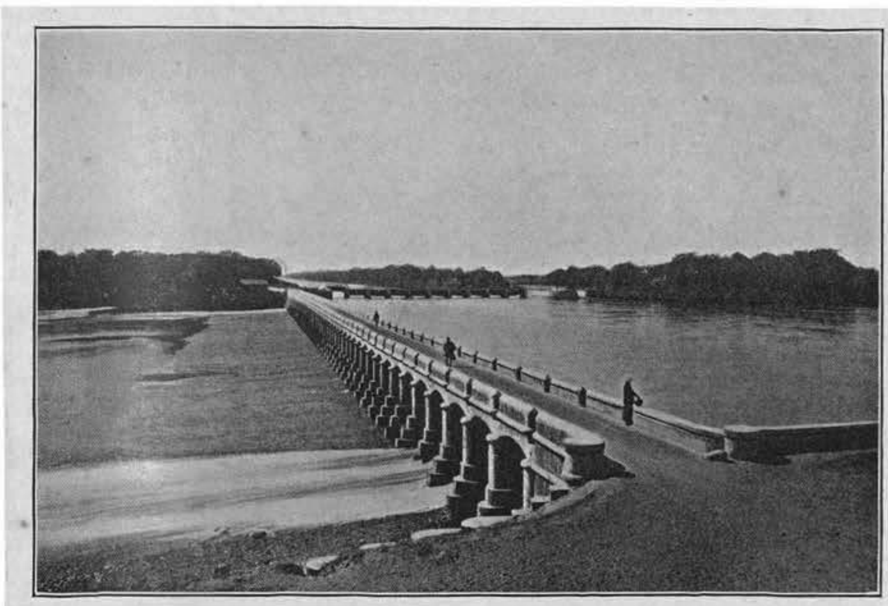


FIG. 19.—Dhanauri Level Crossing, Ganges Canal.

sequently, the design had to be remodelled. In effecting this it was wisely decided to leave the syphon as originally projected, but to supplement it by a level crossing. The parapets of the canal superpassage were removed for 300 feet, and a series of wooden falling shutters substituted. There are sixty pairs of these, each being 5 feet wide by 7 feet high. The width of the canal here is 46 feet. Each pair is connected across the canal by a wire rope, which passes over brackets raised above the shutters, and side tension rods prevent the shutters from falling inwards. The tension rods on the up-stream side are held in position by a let-go gear, and when released both shutters fall, one inwards and one outwards, allowing the flood to pass over them. The canal head must previously be closed and the water emptied all but 3 feet to act as a cushion.

This arrangement has been twice tested and found to act well. A successful precedent of novel construction is always most useful, as it enables similar works to be designed with confidence and with possible improvements on the original.

The general alignment of the wings (Fig. 18a) appears to be excellent. There are the quadrant outer wings, affording a good approach to the syphon, while the splayed inner wings connect the canal banks with the walls of the superpassage. A second wall is introduced here, with the object of allowing the level berm to be continued between it and the canal wing, up to the abutment. The head of water acting below the soffit of the arch is 13 feet, to meet which is 4·6 depth of arch and pitching equivalent to 9 feet of water; consequently, holding-down bolts have been inserted to assist in meeting this unbalanced hydrostatic pressure of 4 feet at the crown. The sloping approaches to the syphon are good and worthy of imitation.

(41) A view is given of the Dhanauri Level Crossing on the old Ganges Canal in Fig. 19, for which we are indebted to Mr. Buckley. This is one of the few instances of this class of work and is on a very large scale. It was built in 1850.

CHAPTER XII

RESERVOIRS AND TANKS

(1) WHEN water is impounded for purposes of irrigation in a reservoir, this can either be effected by embanking a depression, which receives the run off of the rainfall from a catchment area, or else a defined river or stream is held up by a masonry dam or weir, or by an earthen bank. In the former case there are innumerable examples in the small tanks that have been constructed in eastern countries where the slope of the ground is favourable. Of the latter there are also many examples on a very large scale: for instance, the Bhatgarh Reservoir in Bombay, the Periyar project in the Madras Presidency, and the more recent example of the Assuan Dam in Upper Egypt. These three works have already been referred to in Chap. II., which treats of the sections of dams. The large storage reservoirs in the United States are too numerous to be catalogued. The Salt River Reservoir will impound $1\frac{1}{4}$ million acre-feet, a record quantity.

(2) In designing a reservoir, the first point requiring definite statistics is the maximum and minimum discharge to be expected from the catchment area, the former for the purpose of designing the works for the disposal of the surplus water which passes through the reservoir after filling the latter to its full capacity; that is, the required length of waste weir or by-wash or the discharging capacity of waste sluices, if such be adopted. The second determines the capacity of the reservoir, which naturally must be such that it will fill in ordinary years, and on this depends the area of land which it is capable of irrigating and the revenue which may be expected to accrue.

The subject of maximum discharge from catchment areas has already been investigated in Chap. V.; the minimum may be assumed as half the maximum, *i.e.*, with a maximum run off of $\frac{1}{4}$ of the rainfall, the minimum may be taken as $\frac{1}{8}$ or $\frac{1}{10}$ of the rainfall. There are, of course, many instances of small rain-fed tanks which naturally will not fill in a year of drought, though they may do excellent irrigation in ordinary years, so that in most cases the average rainfall has to be considered, not the absolute minimum. A tank however, situated on a stream which never fails to run intermittently during the rainy season, even in a year of drought, is naturally more valuable than one dependent entirely on local rain, which may be very scanty in some years.

The ideal site for a reservoir dam is one across a narrow gorge in a stream or drainage line which above this point widens out in a long level and broad depression. The discharge of the catchment should be such that the tank will fill more than once during the season. According to the "Madras Manual of Irrigation," the irrigating capacity of a tank thus supplied can be reckoned as $1\frac{3}{4}$ its actual capacity.

(3) The actual capacity of a tank is the cubic contents of the water impounded between full tank level (F.T.L.) or F.S.L. (Full Supply Level), and that of the sill of the lowest irrigating sluice. One of the first points requiring attention, after selecting the site for a proposed reservoir, is the determination of its storage capacity up to different proposed levels of escape, *i.e.*, to F.T.L. For this purpose marks should be fixed at differences of level of about 5 feet or 10 feet apart, on any convenient short lines of section; the contours of these levels should then be marked out and surveyed all round the basin in order to obtain the perimeters and areas at each contour. From these the contents of each lamina, can be calculated and the contents up to any contour. The same result can be obtained by a series of longitudinal and transverse sections taken up to the heights of the various contour levels. The former should be directed in conformity with the axis or axes of the figure of the basin, and tranverse sections at right angles to them. Should a winding river channel or depression form part of the basin, it is often more convenient and correct to estimate its contents independently and add it in afterwards.

(4) The formulas in use for obtaining the contents or capacity from the horizontal contour areas are as follows:—

With two contour areas only, viz., A_1, A_2, at a common vertical distance apart (d). The contents $= \frac{1}{3}d\ (A_1 + A_2 + \sqrt{A_1\ A_2})$.

If there be three equidistant horizontal sections the contents $= \frac{1}{3}d\ (A_1 + 4A_2 + A_3)$.

If there be any even number (n) of equidistant horizontal sections $A_1\ A_2$, etc., up to An at a common distance d, the contents $= d\ (\frac{1}{2}\ A_1 + A_2 +$ etc. $+ An - 1 + An)$.

The capacity of the reservoir being thus estimated, the amount of supply that can be expected annually from the catchment area can be obtained from the Tables given in Parts I. and II. of Table II., and Part I. Table III., Jackson's "Hydraulic Manual," where also some excellent practical examples of their application are given, or else the run off can be obtained from Tables given in Chap. V., of this work.

The irrigating capacity of tanks and duty of water impounded varies naturally with the amount of absorption and evaporation. In the Madras Presidency, where the system of irrigation from tanks and reservoirs is very extensive, the following is the quantity per irrigated acre required to be stored to bring the crop to perfection:—

	Cubic feet.	Acre-feet.
(1) Five months monsoon rice crop	216,000	5 nearly.
(2) Cold weather rice crop	175,000	4 "
(3) No. 1, if stored for next year's use	540,000	12·4 "

This includes all loss from evaporation and absorption in the reservoir and the irrigating channels. This estimate, though rough, is useful as a guide. For cold weather crops other than rice, such as wheat, etc., the storage in acre-feet will be about 2. The loss from evaporation varies with the climate of the country in which the reservoir is situated. In Upper Egypt the maximum is taken at ·39 inches per day, and that in India in the hottest and driest months is not believed to exceed ·4 inches, with ·2 inches per diem in the coldest month. The annual loss by absorption varies considerably according to the nature of the bed of the tank, and may be taken, according to "The Irrigation Works of India," at one-half the yearly loss by evaporation.

From the experiments made in tanks in Rajputana, which has a dry, hot climate, the average daily loss from both causes was—October to March, ·20; April to June, ·56; and July to September, ·41 inches per diem.

This is equivalent to a total annual loss in depth of 6·15 feet from evaporation and 3·62 from absorption, giving a total of 9·77 feet.

In Madras a continuous run of 1 cubic foot of water will irrigate $66\frac{2}{3}$ acres of rice and double that amount of dry crops. In new tank projects the duty per cubic foot per second is sometimes taken as—rice 50, dry crops 100 acres, inclusive of losses from evaporation and absorption.

(5) Whether a masonry dam or earthen bank is adopted to impound the water depends upon the local circumstances and comparative cost. The usual limit in height of earthen banks used to be considered as 60 feet, but this has been greatly exceeded in some reservoirs in Bombay, where the Waghad Reservoir embankment is 95 feet high, and another is projected of a maximum height of 110 feet. In England an embankment 125 feet high has been successfully constructed. Now that the new system termed "Hydraulic Fill" has come into general use there appears to be absolutely no limit to height adopted. The projected Oigawa Dam in Japan is to be 330 feet in height, exceeding even that of the Shoshone arched masonry dam which now holds the record at 310 feet.

The works connected with a tank or reservoir consist of (1) the embankment or dam, (2) the waste weir, bye-wash or escape sluices for the disposal of surplus water, and (3) the outlet sluices for irrigation.

(1) *Embankments.*

(6) The common practice in Europe is to construct a puddle core in the centre of an embankment, the remainder of the material being composed of earth, gravel and stones; in this case the imperviousness of the dam to leakage is entirely dependent on the puddle core.

Where suitable earth is not obtainable to form the mass of the embankment, the adoption of a puddle core is obligatory, but where good soil is available the puddle core for embankments of, say, 50 feet in height is not necessary. In India the use of a puddle core is generally limited to very large reservoirs, but even in the case of high banks it is often entirely dispensed with. A thoroughly consolidated homogeneous embankment of earth is undoubtedly preferable to one of infirm material rendered water-tight by a puddle wall. The large Waghad tank embankment, a section of which is shown in Fig. 1, was constructed with rammed layers of moist earth without a puddle core.

(7) Unless clay soil is not easily obtainable near the site of the embankment, this adoption of a puddle core for small or large depths of water is not absolutely essential. In forming an embankment without a puddle core, it is obligatory that the earth be at least damp and consolidated in layers by heavy rollers. The whole mass can be made of wet earth, *i.e.*,

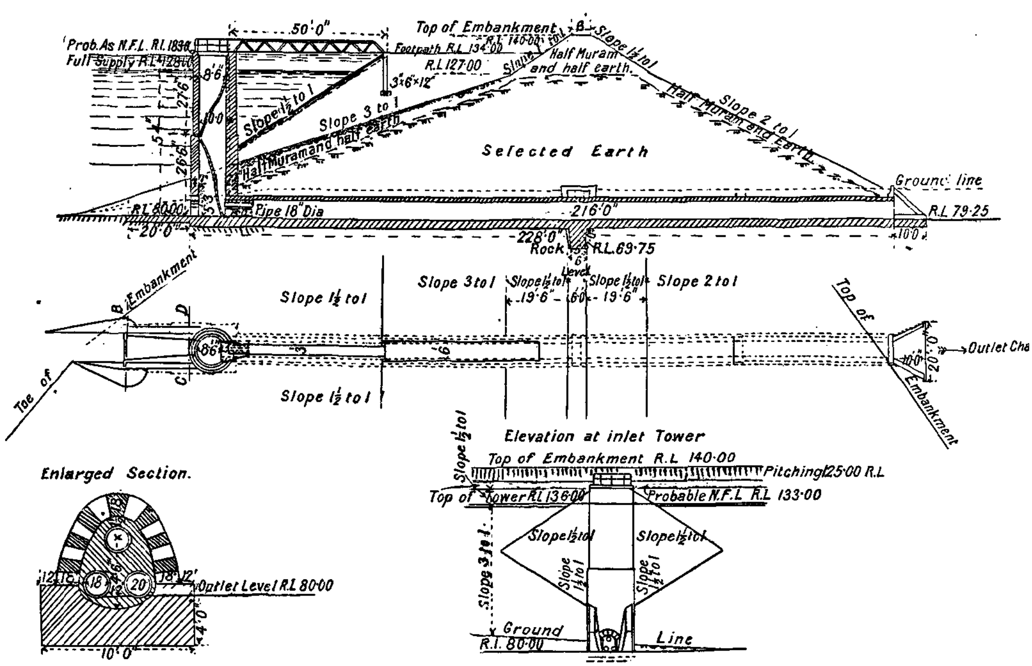


FIG. 1.—Waghad Tank Embankment.

practically puddled, if water is available. The method of procedure is as follows:—The bottom 10 feet of the base of the bank should first be thrown up, and a temporary cut made in the solid ground or rock at a lower level to pass off waste water, or else the masonry outlet culverts can be first built and adapted for this purpose. The work should then lie in abeyance until water has collected in the basin behind the bank. The surface of the banks should then be divided into shallow basins about 12 inches deep by narrow partition walls of earth. Into these enclosures the water should be pumped or baled up from the reservoir. As soon as a series of these shallow basins are full of water, the earth is thrown in to fill them up level with the top of the partition walls, after which another series of chequers are formed on top and again watered. While one part is being filled up, another is being watered or chequered, so that there is no intermission in the earth carrying. When the embankment is thus raised, the level of the bed of the escape cut can likewise be raised either by partially filling it up or cutting a new channel at a higher level, so as to allow the water to rise to a further height behind the bank. By the means thus described, each layer of earth is thoroughly soaked and clods dissolved, so that no ramming or clod breaking is requisite, and the new layer is further consolidated by having 6 or 9 inches of water laid over it, the result being that the whole bank is composed of wet earth devoid of air spaces, which are inseparable if dry earth is used, no matter how much it may be consolidated by rolling or ramming. Consequently when the tank fills, there can be no settlement whatever of the embankment. If during this process the bank shows signs of supersaturation by bulging, the work should be suspended at this part for a day or two until matters adjust themselves, and the layer of water subsequently reduced in depth.

The author has constructed several embankments on this system, and not the least sign of settlement ever appeared in the bank thus formed when fully tested. This system is probably best applicable where coolie labour is employed and earth carried on the head in baskets, when it can be deposited wherever required.

In many cases where stiff clay is the material, it is customary to mix a certain proportion of fine gravel with the clay, as gravelly soil is naturally far more watertight than pure clay, unless the latter is kept continually moist. This can be done by arranging a certain proportion of the carriers to convey gravel, so that the mixing takes place automatically. In the United States the universal practice was to construct the core walls of masonry, but they are now quite discredited. The embankments there are thrown up in layers of damped earth and rolled by heavy rollers. Now, however, the hydraulic method is superseding the ordinary system entirely.

(8) Undoubtedly by far the best method of forming reservoir embankments is that of "hydraulic fill." By this means immense embankments, not only for reservoir, but for high railway banks, have been constructed in America.

The hydraulic fill dam is now recognised as the most economical method of handling earth, as well as by far the only absolutely satisfactory means of compacting the earth of a dam into a perfectly solid mass, free from air particles. A bank is thus made entirely of wet puddle, and when settled is perfectly impervious to water, and, further, cannot settle any more. Consequently, a core wall, except as a temporary measure to prevent the liquid mud spreading out or percolating through a rock fill side which sometimes encloses it, is quite superfluous.

The conditions best suited for the economical construction of hydraulic dams are :—

(1) The existence of abundance of water at a sufficient elevation to form a sluicing head, that is, to afford a steep enough slope for the wooden frames which convey the sluice material on to the embankment.

(2) An abundant deposit of suitable material at a high level.

The volume of water required is from 5 to 20 cubic feet per second. One cubic foot per second will remove 80 to 200 cubic yards in 24 hours. The most suitable material is an admixture of soil, clay, sand and gravel of all sizes. Stones of 2,000 lbs. weight can be carried through the sluicing channels along with a current of thick clay and sand.

In the process of hydraulicking the slopes are first formed and are always kept higher than the interior. By means of check boards and side flumes the heavier material can always be diverted to the slopes, while the fine mud is kept in the centre. The open loose stone and gravel deposited outside allows of the drainage and gradual dessication of the mud, which forms the centre portion of the dam.

(9) The material is either loosened by a hydraulic jet from a "Giant" playing against

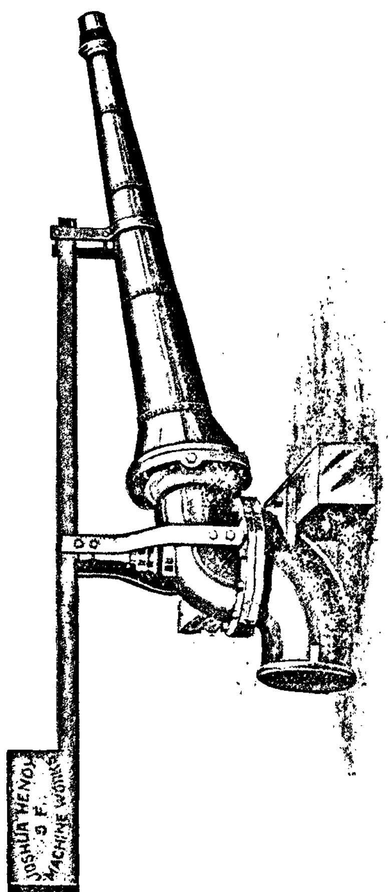


FIG. 2.—Hydraulic Giant Monitor (Double-jointed).

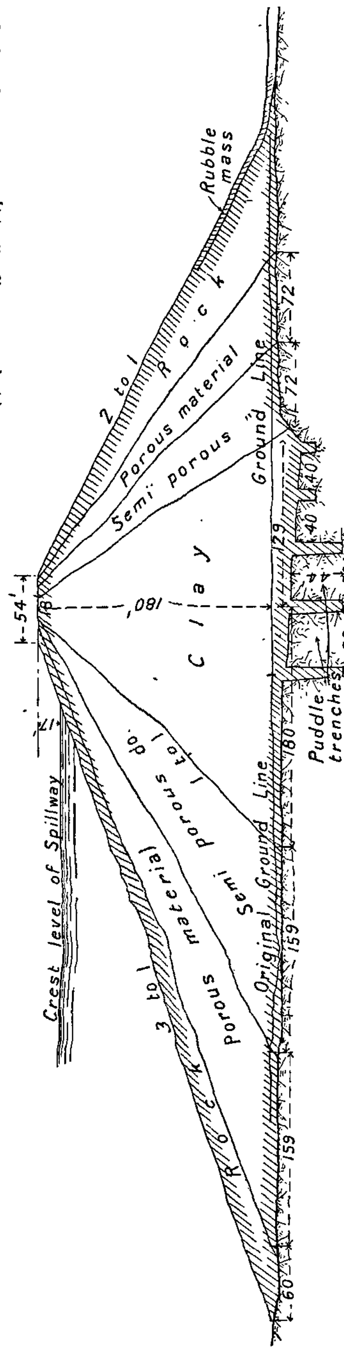


FIG. 3.—Necaxa Hydraulic Fill Dam, Mexico.

the bank, which is the best method, or if pressure is not available the ground can be loosened by plough, or by hand, and then sluiced.

Fig. 2 is a photograph of a hydraulic monitor. The giant is moved

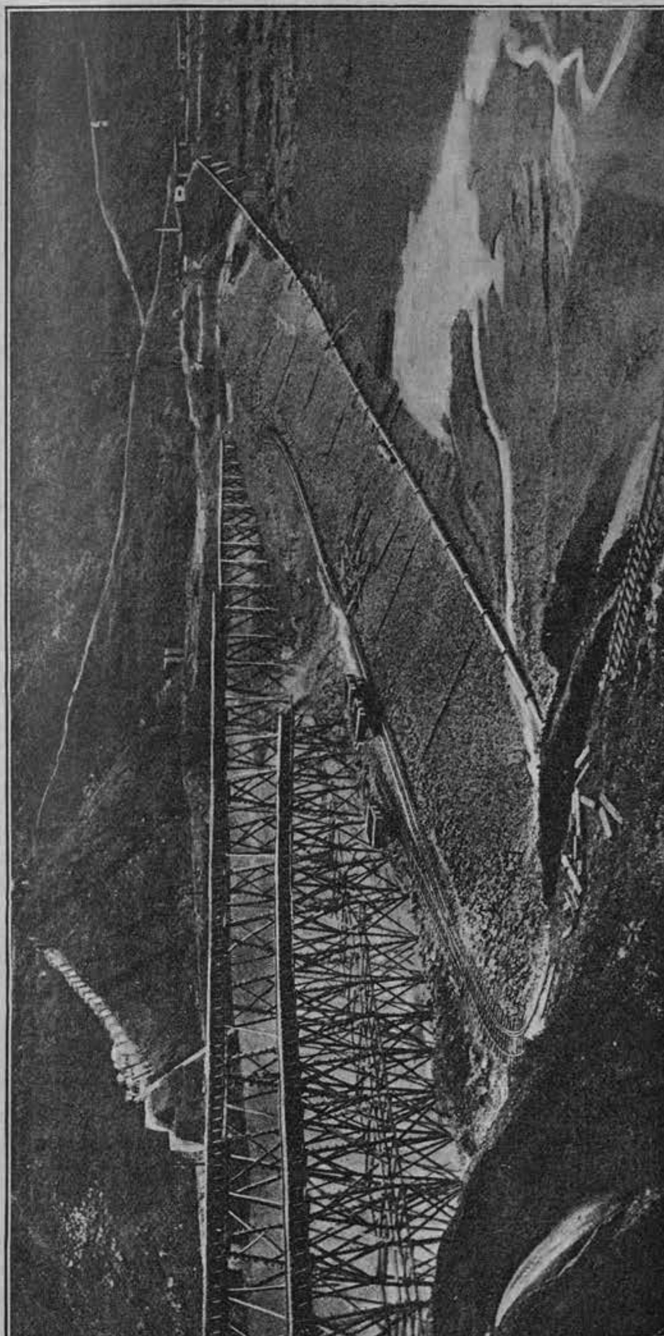


FIG. 3a.—View of construction of Necaxa Dam, Mexico.

round in any desired position by a deflecting nozzle, by which device the force of the issuing jet turns the machine on its pivot so that a child could operate it.

A long and detailed description of hydraulicking, with many photographic illustrations, is given in "Reservoirs for Irrigation, etc.," by J. D. Schuyler, an invaluable up-to-date book which should be consulted on this subject.

Wherever earth at a higher level than the dam crest is not available, low level borrow pits can be excavated by the jet and pumped up to a commanding level whence they are conveyed on to the dam by flumes.

In such cases, the outer casing will have to be made of rock fill, not by hydraulicking. Special arrangements require to be made to drain the internal mass, pipes being inserted in the stone casing with the object of removing the water, and at the same time retaining the mud in suspension.

The cost of hydraulic filling is from $3\frac{1}{2}$ to 6 cents per cubic yard, equivalent to $1\frac{3}{4}d.$ or $3d.$ in English money, or from Rs.4 As.6. to Rs.7 As.8 per 1,000 cubic feet in Indian currency.

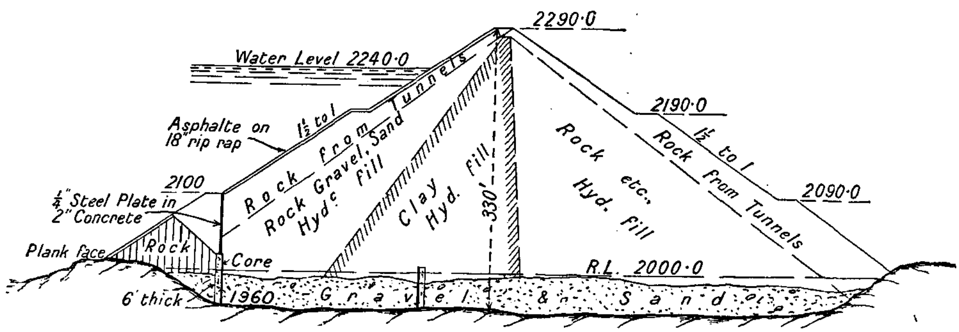


FIG. 4.—Projected Oigawa River Dam, Japan.

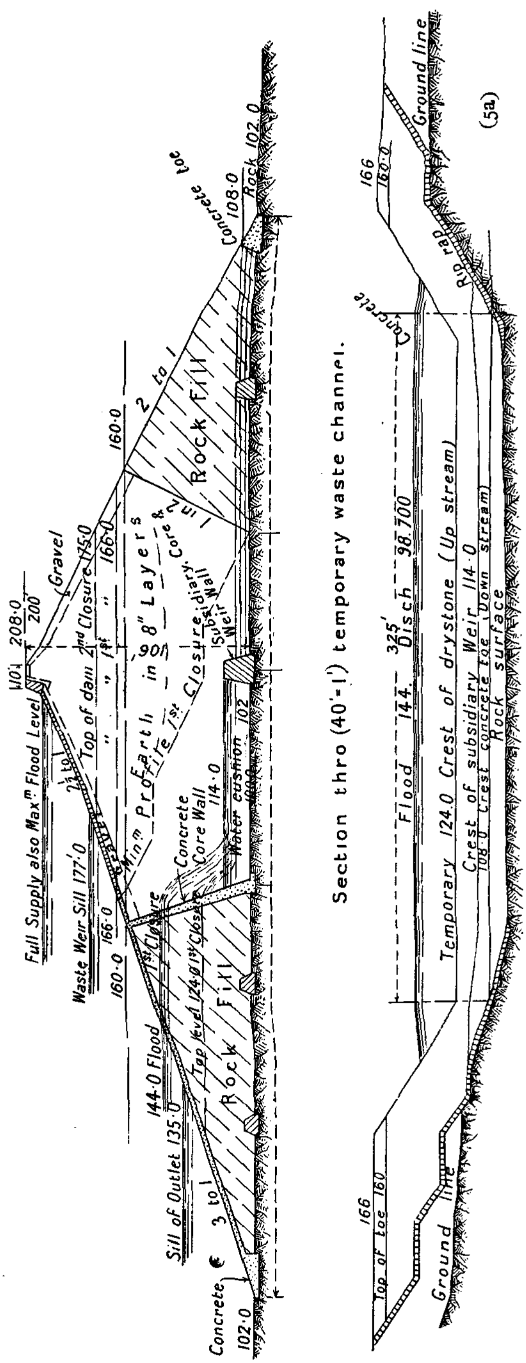
(**10**) A few modern examples of earthen embankments, whether made up by ordinary process or by hydraulicking alone, or by rock fill or stone walling backed by hydraulic fill, will now be given. Fig. 3 is a section of the Necaxa Dam in Mexico. It is one of a series of three now being constructed for the Mexican Power and Light Company in the State of Pueblo. It will store 34,850 acre-feet, and be used as a penstock for pressure pipes. It is thrown across the Necaxa River, which has a discharge varying from 70 to 7,000 second-feet.

It was originally intended to build a masonry dam here, but the softness of the rock substratum was considered unequal to the concentrated unit pressure of a dam of that description, so the hydraulic fill was substituted. The material was sluiced down from the adjacent heights where good clay gravel and masses of rock were met with. A photograph in "Reservoirs" of the deposition of rock on the slopes shows pieces of 18 cubic feet which had been sluiced down the flumes. This dam is 180 feet high, an altitude not long ago deemed impracticable for an earthen dam.

Fig. 3a is a photograph of the work during construction, derived the above work.

(**11**) Fig. 4 is the section of a dam projected to be built over the Oigawa River in Japan. It is to be of hydraulic fill throughout, except the outermost layer on the slopes which will be rock (presumably hand packed) from the tunnels. On the up-stream toe is a core wall of concrete with a steel diaphragm built, and the rock filling of the toe is covered with planking; above the point where the steel core intersects the slope, the latter will be covered with asphalt laid over 18 inches of pitching or riprap. This dam, when completed, will be the highest in the world. This is a notable example of the possibilities of, and the confidence placed in, the hydraulic fill dam.

(**12**) In Fig. 5 is a section of the Maladevi Dam, the most recent construction of that kind in India. Both toes are formed of rock fill, the up-stream one forming a temporary overfall for floods after the first year's work. The overfall slope is faced with a thick layer of concrete. The whole of the up-stream slope is covered with a layer of concrete overlying one of gravel. The plans are derived from Strange's "Indian Reservoirs."



FIGS. 5, 5a.—Maladevi Dam, Mysore, India.

The full and maximum levels are the same. This means that the weir sill can be closed or raised by gates, automatic or otherwise. The reservoir formed will impound 117,400 acre-feet.

The waste weir (Figs. 6 and 6a) is of the so-termed stepped type,

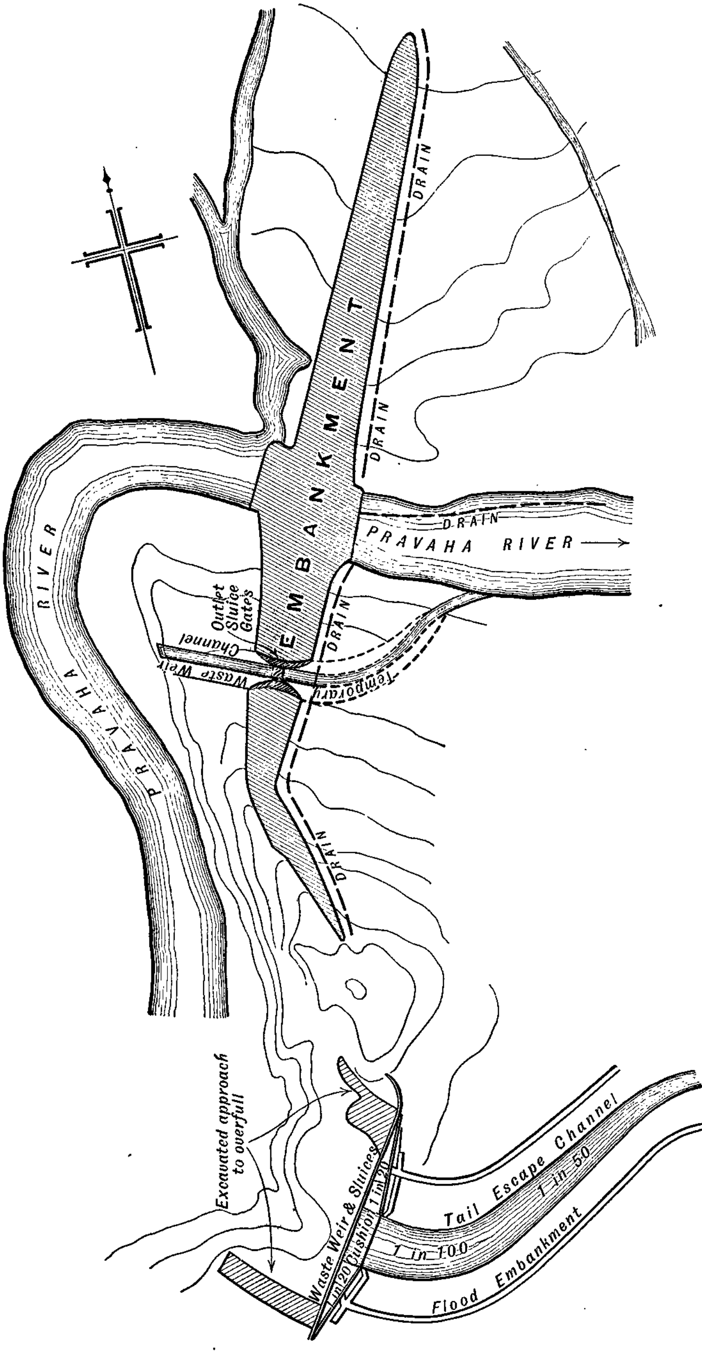


FIG. 5b.—Maladevi Dam, Site Plan.

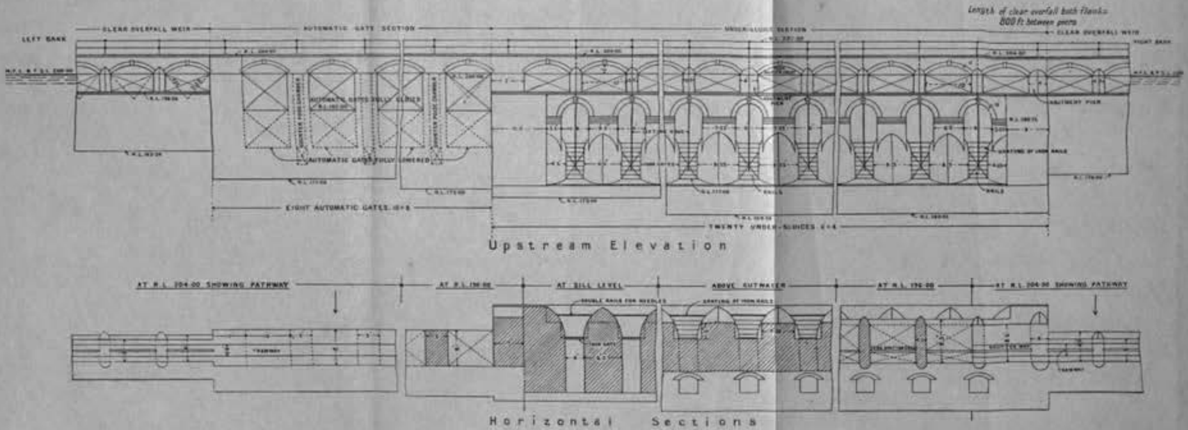


FIG. 6.—"Stepped" Weir in Maladevi Tank Project.

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having three sets of gates at the centre twenty sluices, 6 feet by 4 feet, worked by screws and capstans. Next is a battery of eight automatic gates,

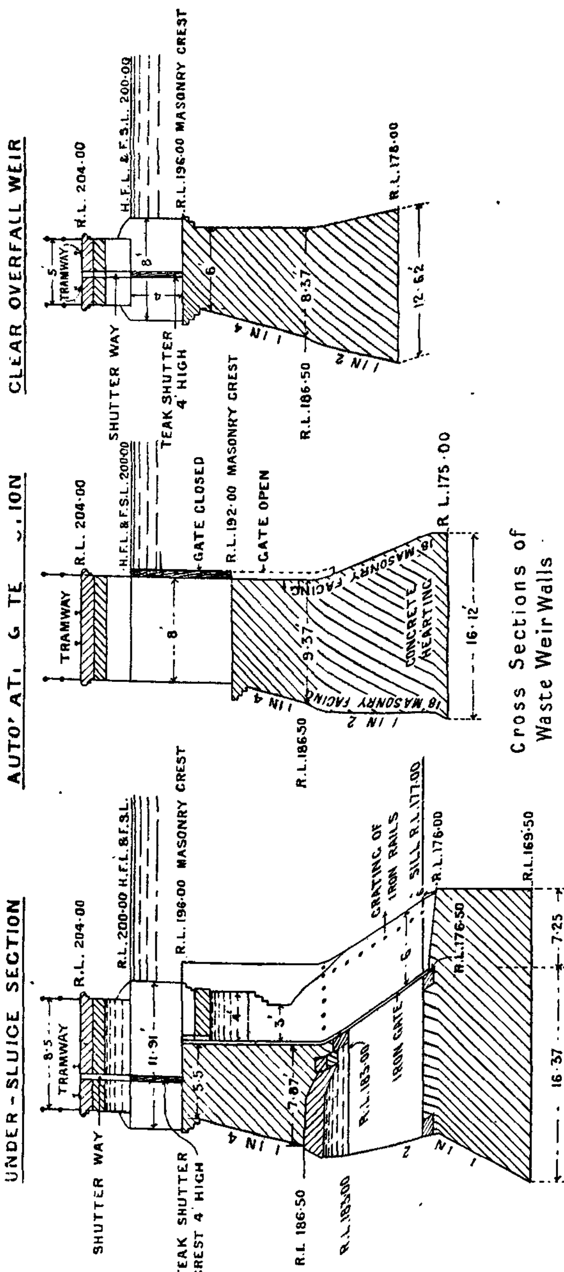


FIG. 6a.—Sections of Maladevi "Stepped" Weir.

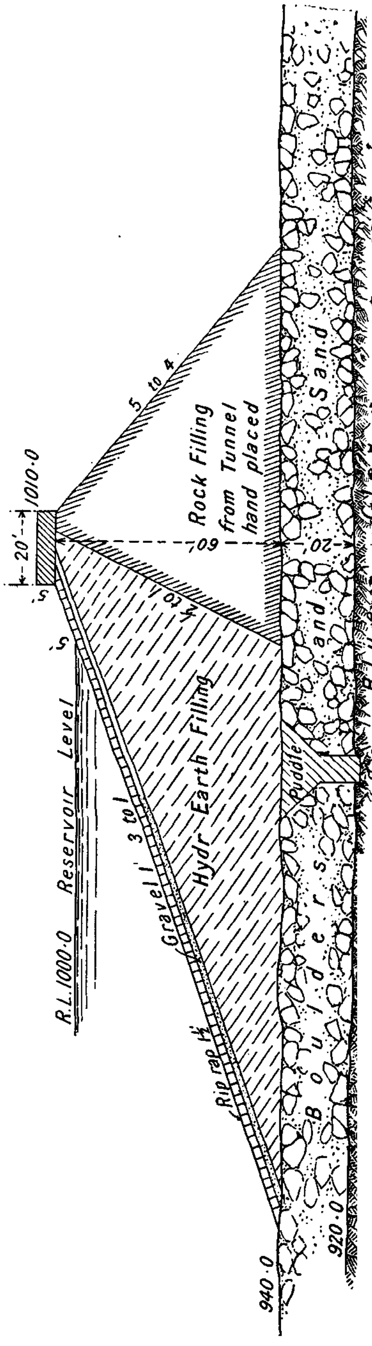


FIG. 7.—Zuni Dam, New Mexico.

8 feet by 10 feet. Above all, and extending for its whole length, 1,135 feet of the weir crest is an arcade carrying a footpath and tramway for travelling

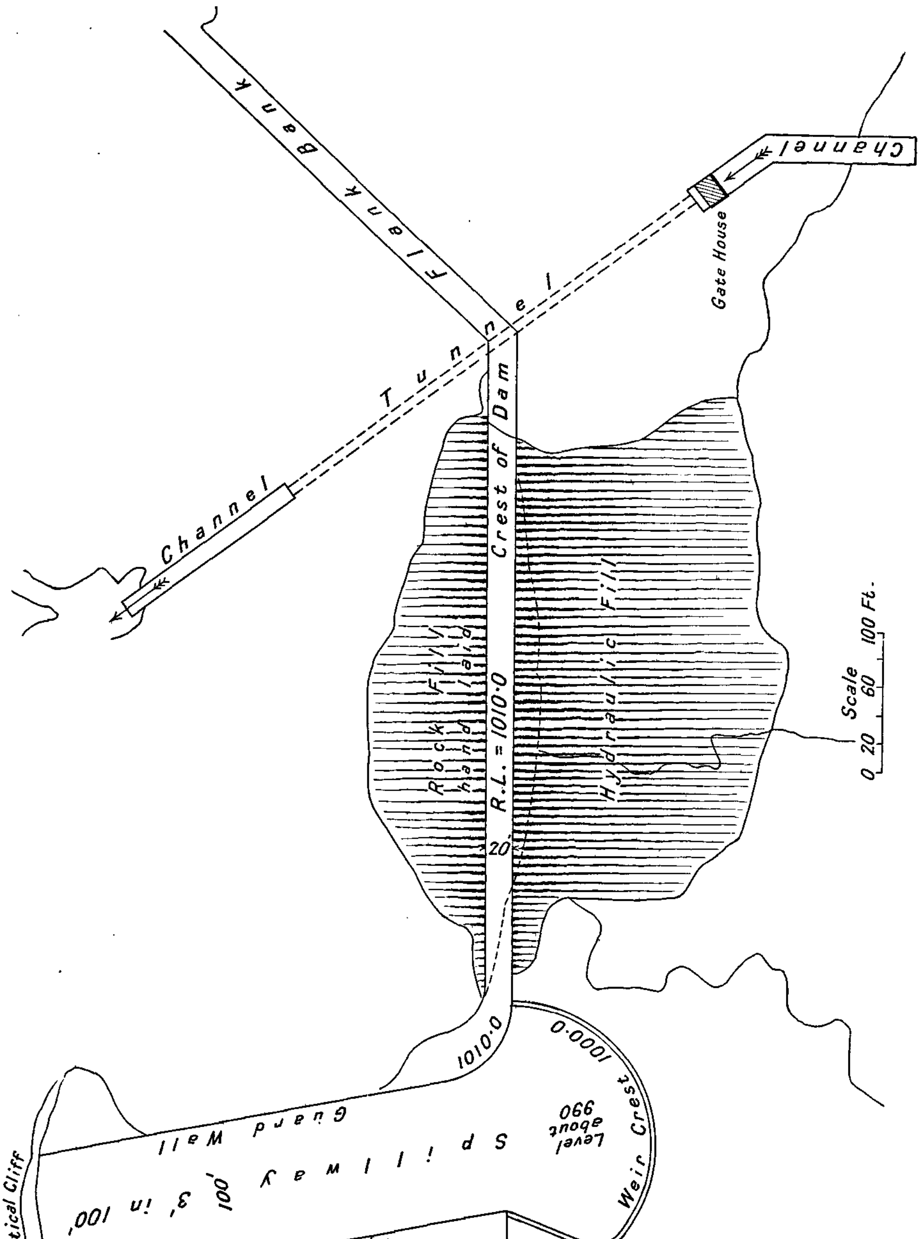


FIG. 7a.—Zuni Dam, Site Plan.

winches. These are provided with 10 × 4 feet teak wood shutters, which are used to temporarily raise the weir crest level by 4 feet. The automatic gates it is proposed to use are of Rheinhold's patent, shown in Fig. 14, *post*. The calculated discharges are as below:—

		Second-feet.
Free overfall, 800 feet × 4	. . .	22,827
Twenty undersluices, 6 × 4	. . .	12,000
Eight automatic gates 8 × 10	. . .	6,457
	Total .	41,284 second-feet.

This discharge is equivalent to a run off of ·418 inch an hour from the catchment of 153 square miles. It is doubtful whether this plan has been finally adopted, or whether several batteries of the automatic gates used on Lake Fife in the Bombay Presidency have not been substituted. This later form of automatic gate has been pronounced a great success. The outlets

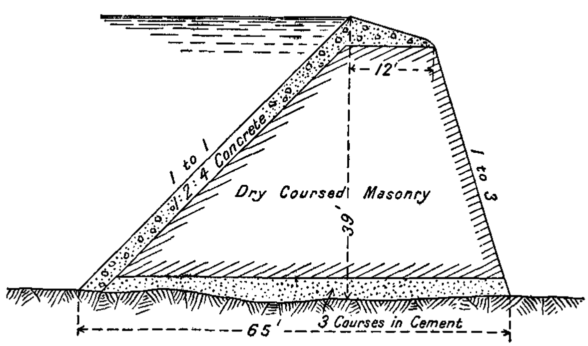


FIG. 8.—Alfred Dam, Maine.

from this latter reservoir consist of 6 vents, 8 feet deep by 4 feet wide, a plan of which is given in Fig. 16, *post*.

(13) The Zuni Dam (Fig. 7) is of a different type to the ordinary hydraulic fill earthen dam. It consists of a trapezoidal block of rubble masonry laid dry, backed by hydraulic earth fill. The connection with the solid clay substratum through the river bed of boulders and sand is made by a puddle trench.

In the site plan (Fig. 7a) the long circular weir crest of the spillway is an excellent feature found in several American works.

(14) An example of pure rock fill dam, or rather weir, built of dry coursed masonry, with a thick skin of concrete on the up-stream side, and the lower $2\frac{1}{2}$ to 3 feet laid in cement mortar, is given in Fig. 8 of the Alfred Dam in Maine.

Where good hydraulic lime is locally obtained a construction of this kind is not economical. A weir of the arch buttress type would probably cost less.

(**15**) Fig. 9 is of the Milner Dam, Twin Falls canal in Idaho. It consists of dumped rock thrown round a wooden plank core, which latter is embedded in dry rubble masonry. The up-stream side is made up of hydraulic earth fill.

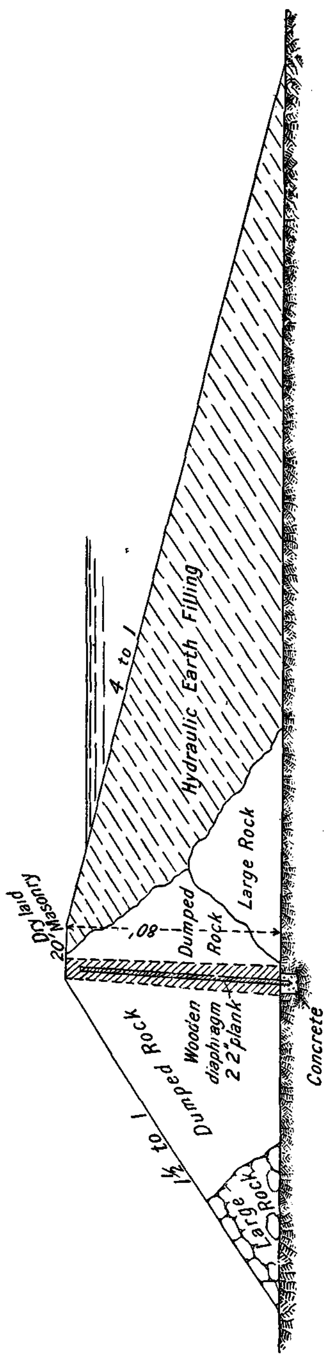


Fig. 9.—Milner Dam, Twin Falls Canal, Idaho.

The wooden diaphragm is for the temporary purpose of preventing the mud of the hydraulic fill washing away through the interstices of the loose rock. Once the hydraulic earth fill is drained and consolidated it will become perfectly watertight, and the diaphragm can be permitted to decay without any damage accruing.

(**16**) Fig. 10 is a section of the Lower Otay Dam in California. This dam spans a narrow canyon, and it was originally intended to build a masonry weir on the site. This accounts for the apparently unnecessary mass of concrete at the base of the dam. It is rendered watertight by a diaphragm of steel plate enclosed in a concrete wall 2 feet thick. Without acquaintance with all local circumstances it is impossible to judge whether such a dam is the most economical form possible. The only method of threshing the matter out is by making out comparative designs and estimates.

(**17**) Fig. 11 is a diagram illustrative of the method of rendering a dam, formed of sand, watertight by means of a puddle core and a puddle trench carried down to solid clay. The slopes also are protected by a layer of clay.

(**18**) A very instructive example of the capabilities of embankments made of pure sand is given in Fig. 12, of the embankment of the Jeypore Waterworks, taken from the "Min. Pro. Inst. C.E.," Vol. CXV. In this case the embankment is not only made entirely of sand, but also lies on a foundation of sand of great depth. The H.W.L. is 44 feet above the floor of the sluice, which is at the bed level of the original stream.

The free board or margin between H.W.L. and crest of bank is greater than usual, being 15 feet, *i.e.*, one-third depth of water, and the crest width is 32 feet.

The usual dimensions of a clay embankment, where a wide roadway is not required, would be, top width 15 to 20 feet and free board one-fourth to

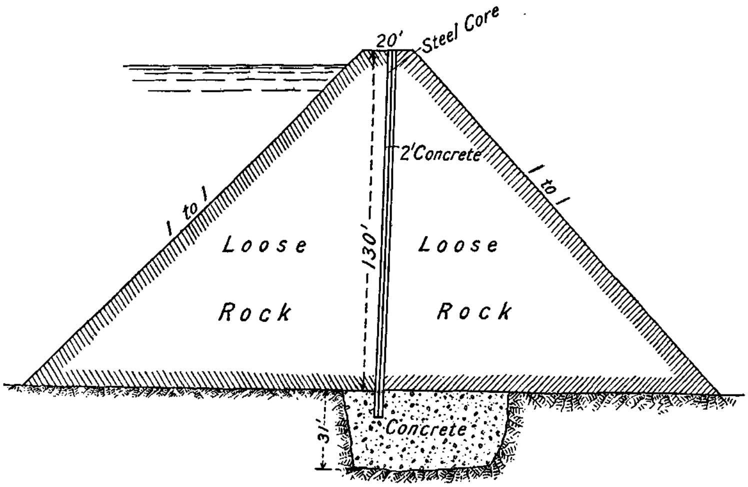


FIG. 10.—Lower Otay Dam, California.

one-fifth depth of water, with a minimum of 3 feet and a maximum of 6 feet, and face slope 3 to 1, back slope 2 to 1, whereas in this case the face slope is 4 to 1. The pipe culvert is founded on blocks of brickwork sunk 8 feet below the floor and filled with concrete; the inlet tower is founded in like

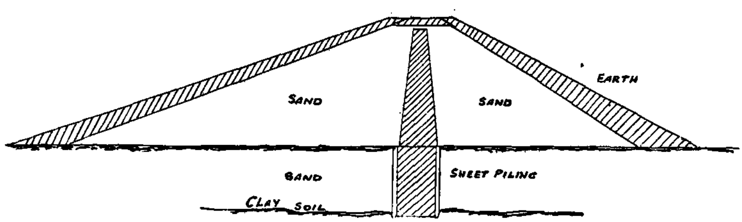


FIG. 11.

manner. The leakage is said to be insignificant, the silt deposit on the bed of the tank having stanched the sand. The value of (l) in this case is 370 feet, and the F.S.L. is 30 feet above sill of outlet. Consequently $c = 12{\cdot}3$.

(**19**) Fig. 13 shows the founding of a puddle wall on concrete which continues the impermeable core down to solid rock. In some embankments the puddle trench has to be carried through fissured or inferior rock in this way to even 100 feet in depth. Rock, unless of the nature of solid bed rock, is not so secure a foundation as sand, as pressure might disintegrate fissured

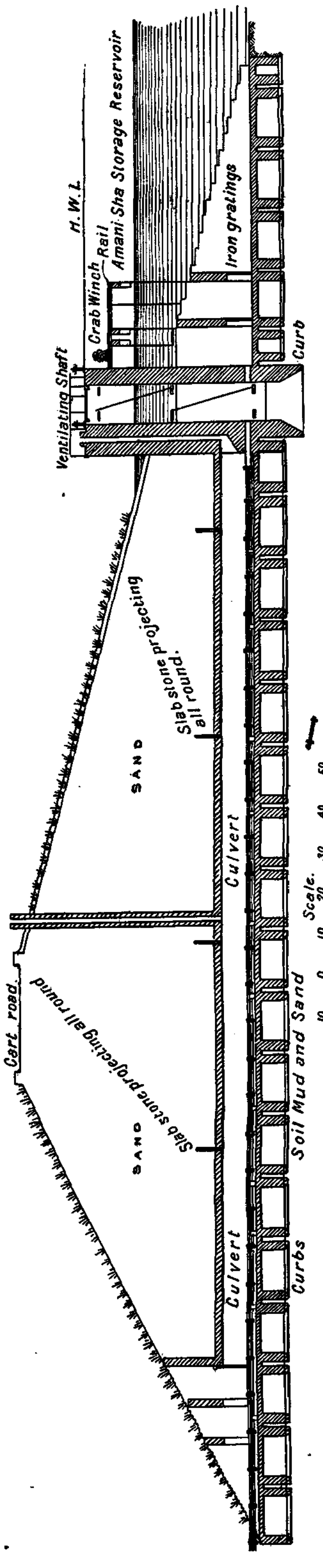


FIG. 12.—Sand Dam, Jeypore Waterworks, India.

rock and blow it out, causing the destruction of the embankment, whereas with sand the superimposed weight presses the particles together and ensures enforced percolation through the sand. Some leakage is sure to occur in the sand substratum, but it will generally be harmless.

With regard to the thickness of puddle walls, the following extract from "Waterworks Engineering," a recent standard work on the subject, will give a safe general rule:—"For general guidance it may be regarded as a safe rule to make the thickness of the puddle wall at the base of the embankment equal to one-third the depth of the impounded water in the reservoir, battering both sides at such an angle as will give a common thickness of 6 feet of puddle at the top of the wall." This dimension would only apply to a very large embankment. "It is unnecessary to carry the puddle wall above the highest wave level of the reservoir, and it is undesirable for it to have anywhere a less covering than 3 to 4 feet of ordinary earth to protect it from the co-operative influences of sun and wind. The puddle wall must be carried down through the subsoil to form a watertight joint with the impervious base of the reservoir. It is laid in a trench excavated in the ground, the sides of which, frequently

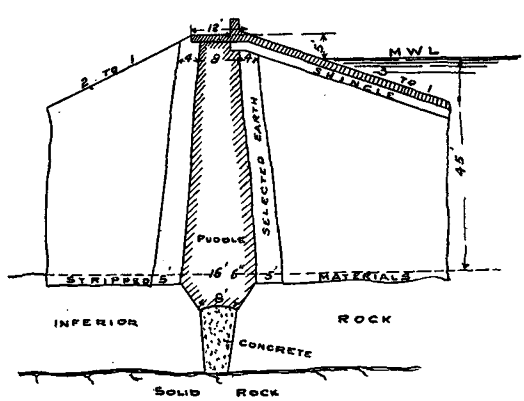


FIG. 13.

battered equally but always reversely to the sides of the puddle wall in the embankment, form the requisite lateral support to the material. In practice, the thickness of this wall at the base is seldom found to be less than half the thickness possessed by it at the ground level." The puddle core is sometimes formed of an intimate mixture of sand, gravel and clay. In the United States peat has been used for the same purpose, but in that country the core is now almost invariably made throughout of rubble masonry, or concrete, or steel plates enclosed in concrete, or a wooden diaphragm is made use of.

The Madras rule for the thickness of puddle walls is to make them 2 feet thick at top with a side batter of 1 in 8 or 1 in 6; the base width would then become $\frac{d}{4} + 2$, or $\frac{d}{4} + 2$; not very far from the rule quoted in last paragraph of $\frac{d}{3}$; the side batter is, however, much greater and the top thinner. It should be borne in mind, however, that the earth when carried by coolie labour is far more evenly distributed and much better consolidated than when tipped by trucks in the usual European method, hence a thinner section of puddle wall can be adopted.

(20) The usual safe side slopes to embankments are 3 to 1 on the water side, with 2 to 1 at the rear. As regards top width, this should not be less than 8 feet in a small tank impounding 10 or 15 feet of water, increasing to 15 or 20 feet in a high embankment. It is evident that the top width should, as in the case of a masonry dam, be some function of the depth of M.W.L., and the general rule adopted in Madras that the width of an embankment at M.W.L. should not be less than the maximum depth appears worthy of general use. Thus with 50 feet depth, and 6 feet free board, and 3 to 1 fore, and 2 to 1 back slopes, the thickness of crest will become $50 - (5 \times 6) = 20$ feet. But this rule would not be operative for depths much below 40 feet. A direct rule for the top width, irrespective of the height of free board, of $2\sqrt{d} + 2$ would seem to be in agreement with ordinary practice. Thus with M.W.L. at 16 feet depth the crest width will be $2 \times 4 + 2 = 10$ feet; at 25 feet depth $(2 \times 5) + 2 = 12$ feet; at 40 feet depth $(2 \times 6) + 2 = 14$ feet, and at 50 feet depth $(2 \times 7) + 2 = 16$ feet.

The Madras rule for top width is $\frac{d}{4}$, with a minimum of 8 feet in long tank embankments with varying depths.

The height of the free board above M.W.L. is seldom less than 10 feet in large embankments, or than 3 feet in small ones; the free board is subject to the conditions of wave action.

The section of the Waghad Embankment in Fig. 1, with a top width of only 6 feet, is not considered a good one by Col. Mullins, by whom the group of Bombay tanks are critically reviewed in the Appendix to the "Madras Irrigation Manual."

The formula for the height of the waves in feet above M.W.L. is 1·5

$\sqrt{F} + (2{\cdot}5 - \sqrt[4]{F})$ where F is the fetch or longest line of exposure of water surface to wind, expressed in statute miles.

Thus, if $F = 4$ miles the height will be

$$1{\cdot}5 \times 2 + (2{\cdot}5 - 1{\cdot}4) = 3 + 1{\cdot}1 = 4{\cdot}1 \text{ feet.}$$

With F = 10 miles the height will be $1{\cdot}5 \times 3{\cdot}1 + (2{\cdot}5 - 1{\cdot}7) = 5\frac{1}{2}$ feet.

Embankments, where the soil is unfavourable, are often provided with drains longitudinal behind the puddle wall with transverse ones at intervals. These are trenches filled with loose stone in order to carry off any leakage and prevent the rear portion being supersaturated, thus preventing slips.

(21) A masonry dam has this advantage over an earthen embankment in that it can act in whole or in part as a waste weir, as for instance the Vrynwy Weir, 60 feet high, which has an overflow: the Coolgardie Weir, 120 feet high, and the Bhatgarh overflow weir; or else waste or supply sluices can be built in the body of the wall, as has been done in the case of the Bhatgarh and Assuan Dams. This form of waste sluice, now that so successful an example on a very large scale has been constructed at Assuan, is bound in many cases to supersede the commonly adopted overflow waste weir distinct from the dam.

The design of the section of all kinds of masonry dams and the rules governing the shape of the profile have been fully treated in Chaps. II. and III.

The material of which a dam or weir is constructed is usually either rubble masonry or concrete. The following remarks on this subject, from "Waterworks Engineering" (Tudsbery and Brightmore), are well worthy of reproduction:—

"The construction of the vast masses of masonry or concrete of which dams are formed is an art that requires the exercise of judgment in the selection of the materials used, close attention to their preparation, and watchfulness during the process of building. The question as to whether concrete or rubble masonry is in any given case the preferable building material must largely depend upon the character of the rock available. The great tensile strength of Portland cement causes it to be in high favour as a matrix, but it is not entirely unexceptionable. Where a dam abuts, as is frequently the case, upon steep hill sides, the rapid variation in height tends to produce an unequal settlement in the masonry or concrete as the structure is gradually raised. This settlement is resisted by the shear of the material transmitted from the comparatively unyielding rock abutments. It is not improbable that action of this kind has produced some of the fractures that have too often occurred in such dams. If this be so, it points to the prudence of building in a slow-setting hydraulic lime, thereby permitting free settlement of the mass before it sets rigidly. The rapid settlement during construction of large masses of concrete must give rise to internal shearing stress of indeterminate amount. In concrete work, such settlement is much reduced by the introduction of large blocks of stone and boulders into the work, a practice that possesses the advantages of adding to the coherence of the entire work and generally of effecting some economy in its cost."

To which remarks might be added that the use of long header stones in rubble masonry laid crossways, which is often specified, tends to facilitate rupture of the wall. With a battering face, each face course overlaps the preceding one, thus automatically forming a bond between the face work and the interior filling, so that long bond stones are not only unnecessary to preserve transverse bond, but they have the disadvantage of breaking the longitudinal bond. A wall under pressure from the rear has no tendency to split longitudinally as a badly bonded wall of a building might have, but owing to unequal settlement the tendency is for transverse cracks to be formed. Thus, if used at all, which is to be deprecated, bond stones should lie longitudinally, not transversely. The author considers that the use of special bond stones in rubble masonry is a mistake as tending to destroy the homogeneity of the mass.

In the Bhatgarh Dam, a section of which is given in Fig. 15, Chap. II., the interior was formed of concrete with a plentiful inter-sprinkling of large stones which were embedded in the concrete. This raised the specific gravity of the mass considerably above what it would have been without the admixture of heavy stone blocks, the actual weight rising to 150 or 160 lbs. per cubic foot.

The Periyar Dam (Fig. 13, Chap. II.) is also formed of concrete.

It has already been stated that the construction of sluices in the body of a dam, as has been effected in the Assuan and Bhatgarh Dams, would often be a more economical procedure than that of a separate waste weir, unless natural conditions specially favoured the cutting of an inexpensive by-wash.

(2) *Disposal of Surplus Water.*

(22) Where the main dam of a reservoir does not itself act as a waste weir by allowing surplus water to pass over it, or through it (if waste sluices are adopted), a separate waste weir or by-wash or spillway has to be constructed. Such is naturally also the case where the dam is of earth. The safe disposal of surplus water passing through an irrigation tank, which is often the receptacle of a large stream, is a most important matter, and the ruin of most old native Indian irrigation tanks is generally due to proper means of escape of surplus water not having been provided. For designing such a work the maximum possible inflow into the tank under the assumption that the latter is already full has to be ascertained, and the waste weir built of such a length as to be able to discharge this quantity or rate of inflow at a defined depth of film of water passing over the crest. This depth (d) is very often 3 or 4 feet, but may be made anything in reason provided that the expense of raising the embankment all through to this extent does not exceed the cost of providing a longer waste weir with a less value of d.

It is evident that the crest level of the waste weir is that of full supply level or F.S.L. of the reservoir, while the maximum tank level or M.T.L. (to use the Madras phraseology) will be F.S.L. + *d*. Now the level of the crest of the embankment or dam is naturally dependent on the latter, not the former, so that the adoption of a waste weir as a means of disposal of surplus water involves the raising of the whole embankment to an extent equal to *d*, the greatest allowable depth of film passing the weir. This forms the great drawback to the adoption of waste weirs, which otherwise are an excellent provision for escapes, being self-acting.

In order to reduce the difference between F.S.L. and M.F.L. to a minimum, if not to abolish it altogether, several courses are open. Firstly: The weir could be provided with collapsible shutters, as are commonly employed on river weirs, and described in Chap. VI. These are balanced at

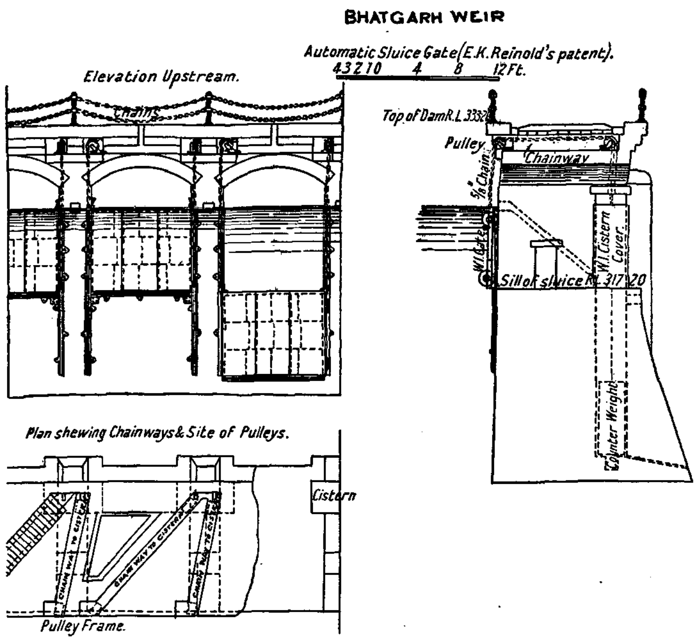


FIG. 14.—Bhatgarh Weir Automatic Shutters.

the centre of pressure, and fall automatically when overtopped. They will have to be raised by hand, which could be effected on the lowering of the water level in the reservoir by men hooking them up into position from a staging erected over the weir. This method is doubtless economical and safe, but requires very judicious regulation to prevent loss of storage water, and is practically limited to depth of 3 or 4 feet. Secondly: Automatic drop shutters could be adopted, as exemplified in the Bhatgarh waste weir (Fig. 14), or the Lake Fife waste gates (Fig. 16). This system is purely automatic in action, the gates being lowered on any increase of level in the water of the reservoir, while on a fall of the same below F.S.L. they rise and close the openings. Thirdly: Sluice bridge openings could be adopted, similar to what was suggested for the Koshesha Escape in Chap. X., with collapsible upper gates and lower draw gates, or only with the former.

Fourthly: An overfall of any kind could be abandoned and the regulation effected by low level sluices built in the body of a closed dam, as exemplified in the Bhatgarh Dam and the Assuan Dam (Chap. II.). Fifthly: The cylinder rolling on crest, as shown in Fig. 29. Sixthly: In the "stepped" weir (Fig. 6), where three systems are employed together.

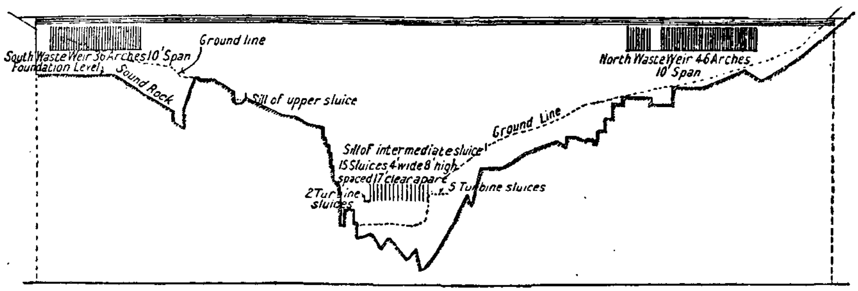


FIG. 14a.—Longitudinal Section, Bhatgarh Dam.

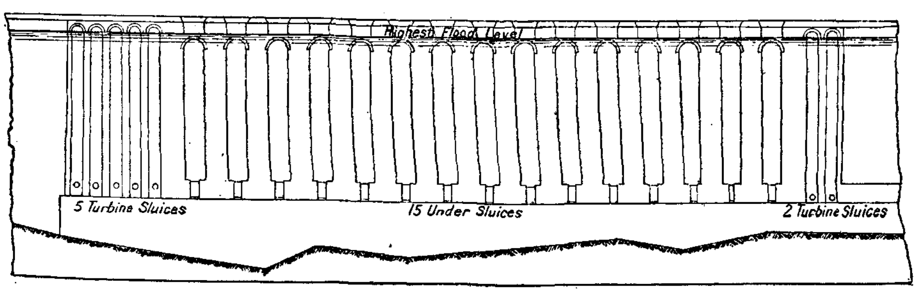


FIG. 14b.—Elevation of part of Bhatgarh Dam.

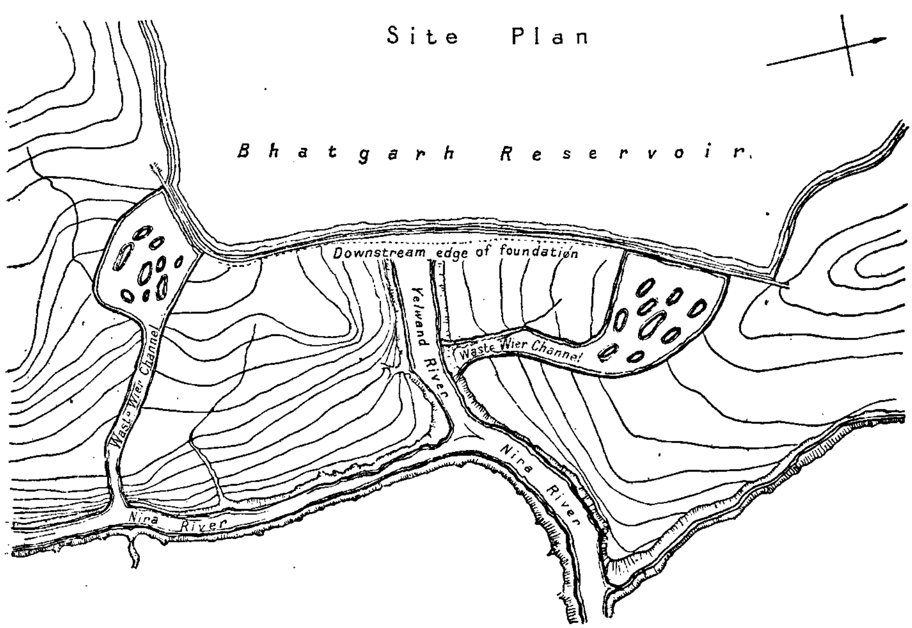


FIG. 14C.

(23) The design of the Bhatgarh self-acting shutters is illustrated in Fig. 14. The principle is that of a counterweight, placed in a deep cistern constructed under each pier, which counterweight is connected with the gate by a system of chains and pulleys. This gate is hung vertically, and runs on anti-friction rollers. An inclined channel through the pier connects an opening which is placed at F.S.L. with the base of the cistern; when water rises above F.S.L. it obtains access to the cistern and raises the counterweight and lowers the gate; when the level in the reservoir falls below F.S.L. the supply of water to the cistern is cut off and the counterweight falls, bringing the gate up into position again. There is a small outlet at the base of the cistern. These gates are said to have given complete satisfaction.

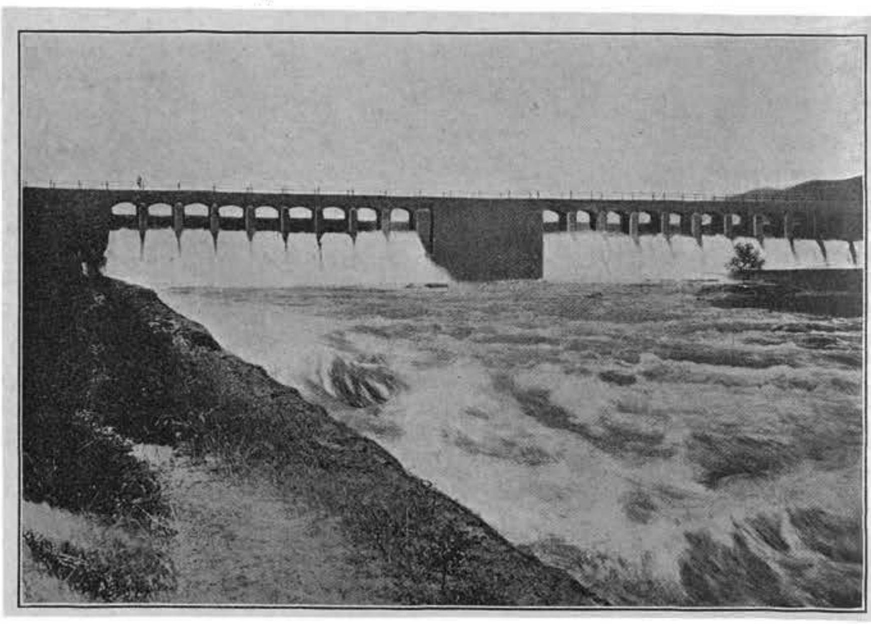


FIG. 14d.—Single Automatic Waste-weir Gates, Lake Whiting, or Bhatgarh.

(24) The section of the Bhatgarh Dam is given in Fig. 15, Chap. II. This dam is 3,020 feet long and 127 feet high above the lowest point in the foundations. It impounds 90,000 acre-feet available for irrigation. The dam has 15 low level vents 8 feet deep by 6 feet wide, and the waste weirs on either flank consist of an arcade with 103 openings of 10 feet. Automatic gates, illustrated in Fig. 14, are fitted to 88 of these, the rest being opened and closed by hand. The water from the undersluices and overfall weir falls into the bed of the Nira River, and further down fills up a low depression termed the Vir Basin, which also contributes in a certain degree to the storage capacity. At the termination of the Vir Basin is a long waste weir at the head of which the Nira Canal takes out. It irrigates at present 75,000 acres, with a length of main canal of 100 miles and 139 miles of distributaries.

This work is not a financial success. Fig. 14a is a longitudinal section of the Bhatgarh Dam, Fig. 14b a part elevation of the deep undersluices, and Fig. 14c a site plan. These sluices have proved effective in keeping down silt deposit, and the reservoir in twelve years' time has not yet silted up to sill level. When it does they are expected to prevent any further rise in the deposit. A photograph of the Bhatgarh gates is given in Fig. 14d.

(25) The lakes formed by the two dams (see map, Fig. 15)—the Bhatgarh Dam on Lake Whiting, formerly called Lake Bhatgarh, and the Lake Fife Dam, *alias* the Mutha Dam—are situated only twenty-five miles apart, in the Western Ghauts. They are built across the Nira and the Mutha rivers, the catchment area of the former of which is but 128 square miles, but the rainfall in the catchment varies from 250 inches in the ghauts themselves to 40 inches at the dam site. These rivers discharge immense volumes of water during the actual monsoon months, but from January to June practically run dry. The object of the reservoirs is to collect a proportion of the flood discharge and store it for the months when irrigation is mostly in demand. The catchment of the Lake Fife is somewhat larger

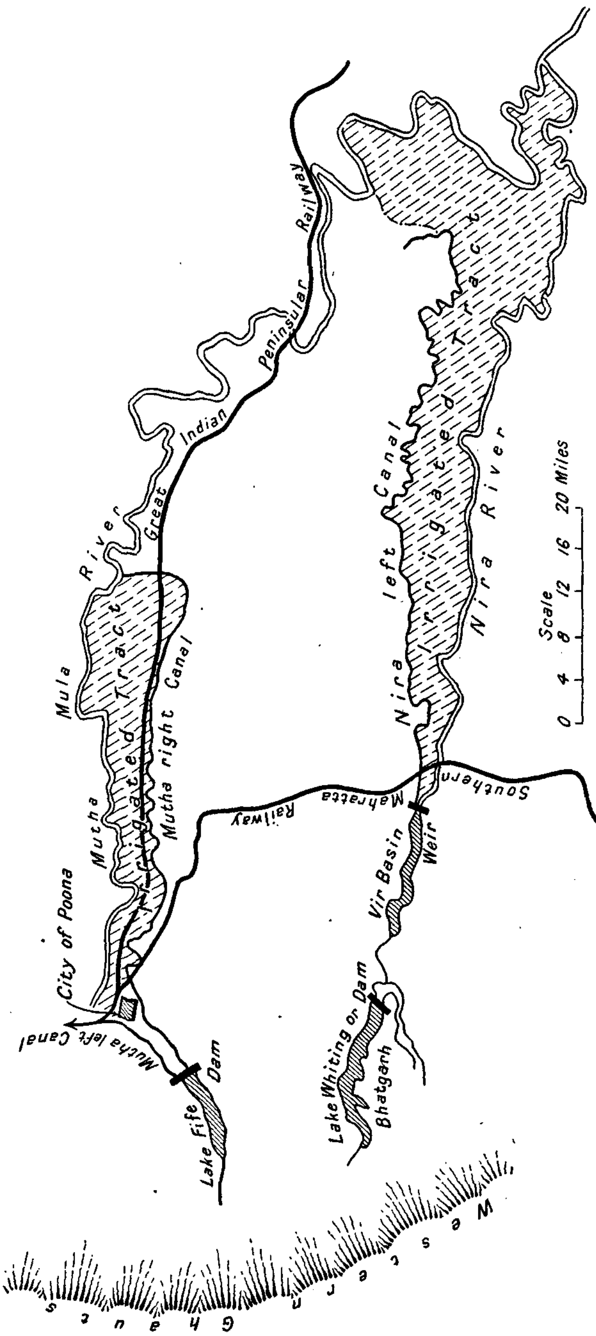


FIG. 15.—Location Map of Mutha and Nira Canals, Bombay.

than that of Lake Whiting. The Bhatgarh Dam is the first to be provided with undersluices, and is consequently the precursor of the Assuan Dam. These sluices, however, do not carry by any means the whole discharge of the river, as is the case with the Assuan, but are considered to be mere scouring sluices to keep the reservoir from silting up.

The work in the Vir Basin consists of a weir 2,273 feet long and 42 feet high. It has two sluice openings closed by needles and two deep undersluices.

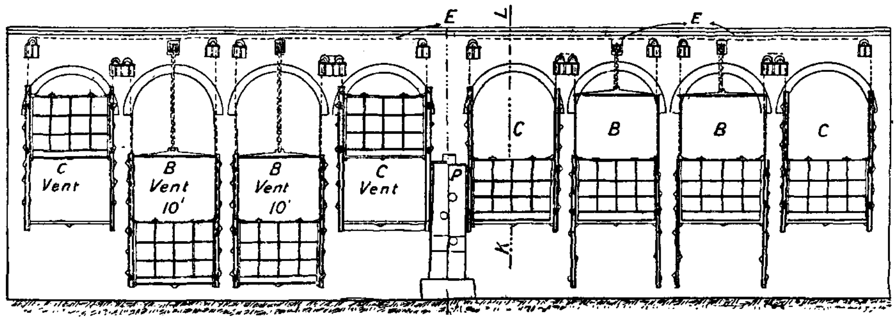


FIG. 16.—One Grouped Battery of Automatic Gates, Lake Fife (88 gates in all).

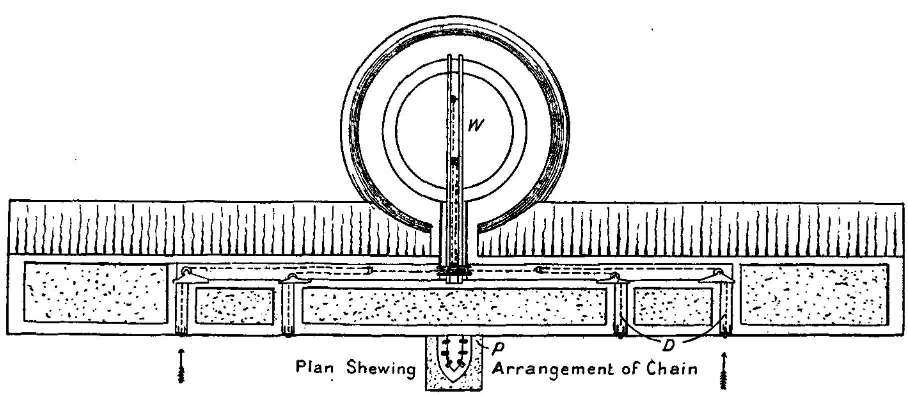


FIG. 16a.—Plan of Cistern, Lake Fife Weir.

The Mutha Dam is 3,700 feet long and 98 feet high. At its deepest part it impounds 76,000 acre-feet available for irrigation.

The map (Fig. 15) shows both canals and the areas irrigated by them, with their head works.

The whole of the irrigation is on sidelong ground. The canals cross many drainage lines, and consequently have proved very expensive to construct. These Bombay irrigation works resemble those in America more than the doab and delta canals in the flat country in Upper India and Madras.

(26) In the Lake Fife Surplus Escape Weir a new style of automatic

closure gates were introduced which have given more satisfaction than the Rheinhold pattern could do, as the latter requires a high weir in which to work properly. The following description is abridged from that given in "The Irrigation Works of India," from which source Fig. 16 is also derived:—The gates work in pairs, suspended from pulleys by chains. The lighter gate is drawn up by chains attached to its ends working inside the grooves, while the heavier gate is fitted with a square hollow frame, the area of which is the same as that of the gate itself. The chain is attached to the centre of this frame, and consequently does not get in the way of the current when the gate is lowered. The heavier gates weigh over 100 cwt., and the lighter ones 44 cwt.

The sluices open when the heavier gates are allowed to fall. When fully

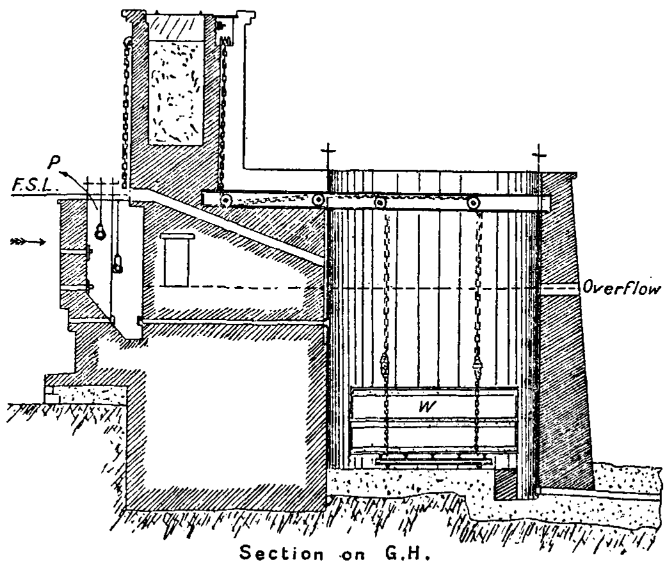


FIG. 16b.—Section of Cistern, Lake Fife Weir.

open the heavier gates lie below the sill of the weir, and the lighter ones rest above the flood water level. The gates are closed by the action of a counter weight, which is attached by chain or wire rope to the heavier gate. When the heavier gate is lifted the lighter one descends by its own weight. The counterweight works in a round cistern specially constructed for it below the weir. It has a cubic capacity of 760 to 800 cubic feet, but this varies with the depth of the overfall. The lower part below the extreme run of the counterweight is filled with sand.

The plan and section, Figs. 16a and 16b, illustrate the action of the counterweight. When full supply, which is also flood level, is reached, water runs down the channel *P* into the cistern, gradually immersing the counterweight, till at last it floats. The tension in the chains attached to the heavier gates is then relaxed, and the gates fall, opening their own vents, and by pulling up the adjoining lighter gates open their vents also. At the bottom of the cistern is an outlet pipe of smaller size than *P*. When the flood level falls the supply from *P* is reduced or else ceases altogether, and the water in the cistern then empties itself in a short time through the outlet, thus bringing the counterweight down and reversing the action previously described. The outlet as well as the inlet pipes are provided with throttle valves worked by hand when necessary from above. There are also some other inlets controlled by valves, by means of which the automatic action can be stopped and the gates held up at any required level.

The installation of these eighty-eight gates alone cost 5 lakhs of rupees, or £30,000 nearly (150,000 dollars). They can discharge 66,000 second-feet.

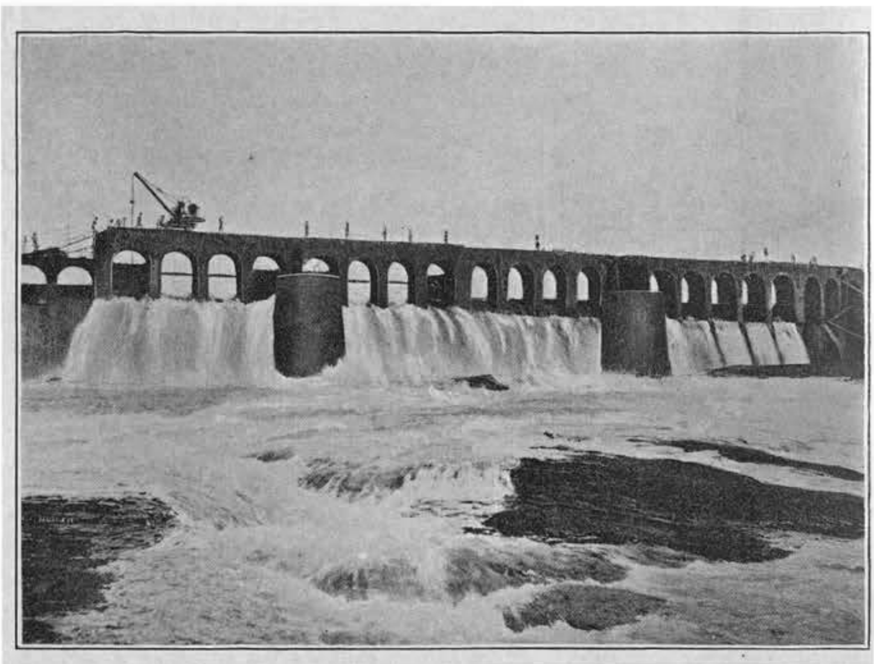


FIG. 16c.—Lake Fife, Grouped Batteries of Automatic Waste-weir Gates in action.

These gates have given perfect satisfaction in working. The instalment is undoubtedly very expensive, and would cost more in India than in America, but where the embankment or masonry dam is very long, it saves the cost of the whole work being raised 8 feet, which would be the case, if an open overfall weir were adopted, thus effecting a large saving, besides diminishing the risk. A view of the gates acting is given in Fig. 16c.

(27) A section of the Goulburn Weir at the head of the canal system of that name in Australia is given in Figs. 17, 17a, 17b. This quite recent work is a drop gate weir. Reinforced concrete piers 2 feet wide are erected on the crest 20 feet apart, and these spaces are closed by drop gates 20 feet long by 10 feet deep. The gates, which are manipulated by rack and pinion gearing, are lowered into recesses constructed below the weir crest. Owing to the extreme thinness of the piers and the weakening of the section by the gate recess below, the weir has to be strengthened by extensive steel reinforcement right down to the level of the base of the recess, and the framework of the piers is bolted down to this lower reinforcement. The view in Fig. 17b is derived from "Irrigation Engineering."

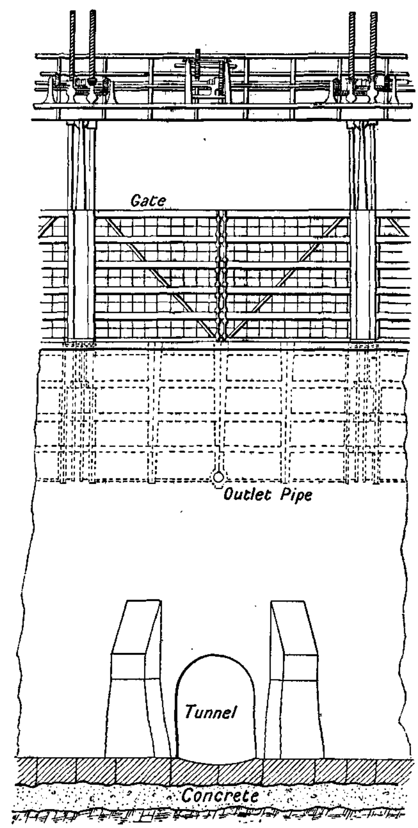


FIG. 17a.—Elevation of Goulburn Weir.

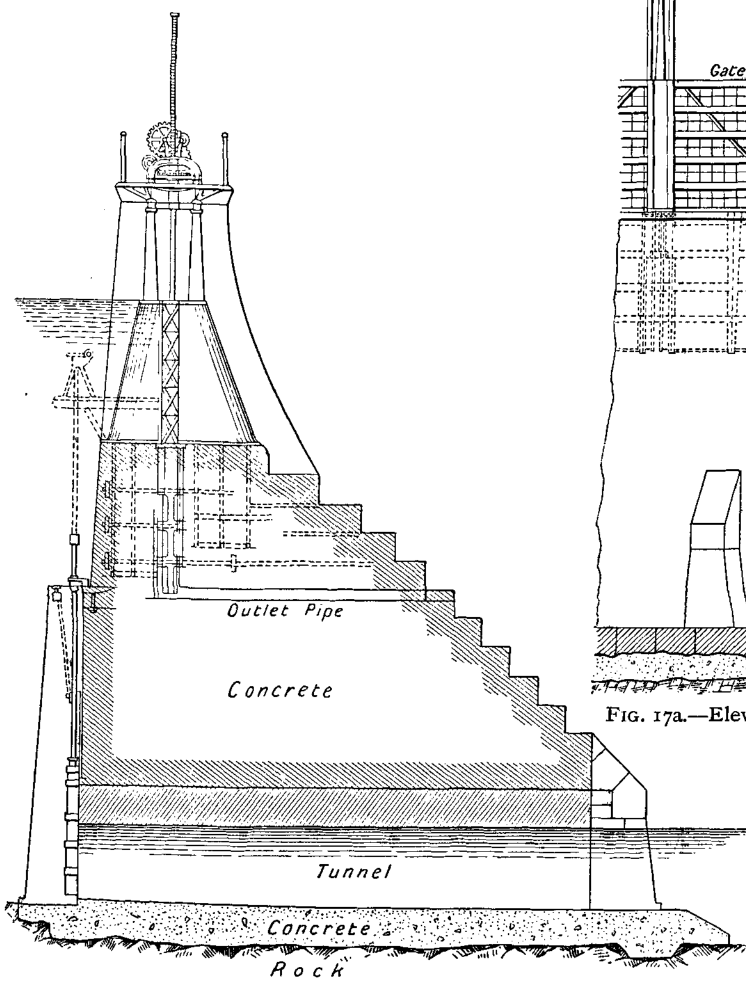


FIG. 17.—Section of Goulburn Canal Weir (Australia).

(28) This arrangement, which clearly involves considerable expense with no corresponding advantage, cannot be commended. A better arrangement undoubtedly would be to substitute solid wide masonry piers, arched over in the usual Indian style. The drop gates can be disposed of by being arranged to slide down the inner face of the weir, and when drawn up will run in roller paths up the face of the piers. The extra thickness given to the piers will necessitate lengthening the weir, but any increase of masonry

in this respect can be economised in the base thickness, which is unnecessarily great.

These suggested improvements are embodied in Figs. 18 and 18a. If the

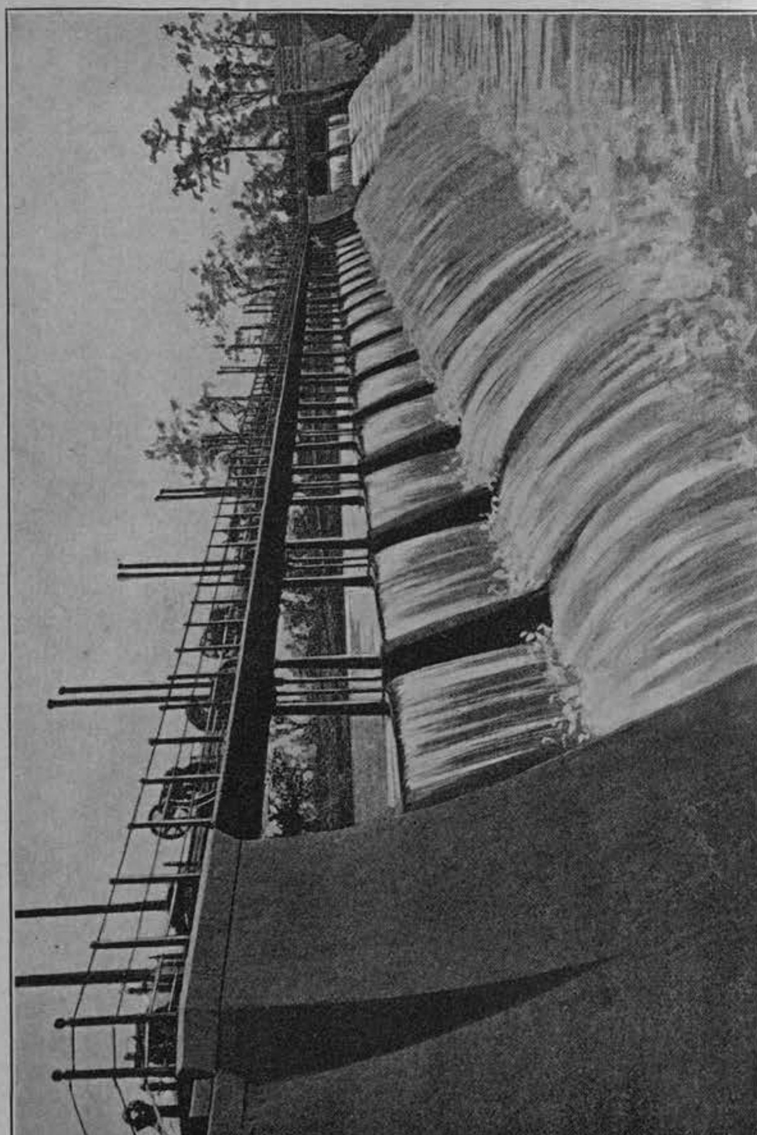


Fig. 17b.—Battery of Waste-weir Gates, Goulburn Canal, Victoria, Australia.

spans were reduced, automatic roller gates could be used, as in Figs. 14 or 16.

The base thickness of the weir is reduced from 44 feet to 30 feet, or $\frac{H + \cdot 6d}{\sqrt{\rho}}$, which is ample for purposes of stability, and the weir crest proper is increased to 15 feet width. A batter of 1 in 5 is given to the rear face. This will facilitate the manipulation of the roller gates, each of which will be lifted by chains passing up the pier grooves at either extremity, the chains being brought on to either a travelling or two fixed and connected winches, resting on the arched platform.

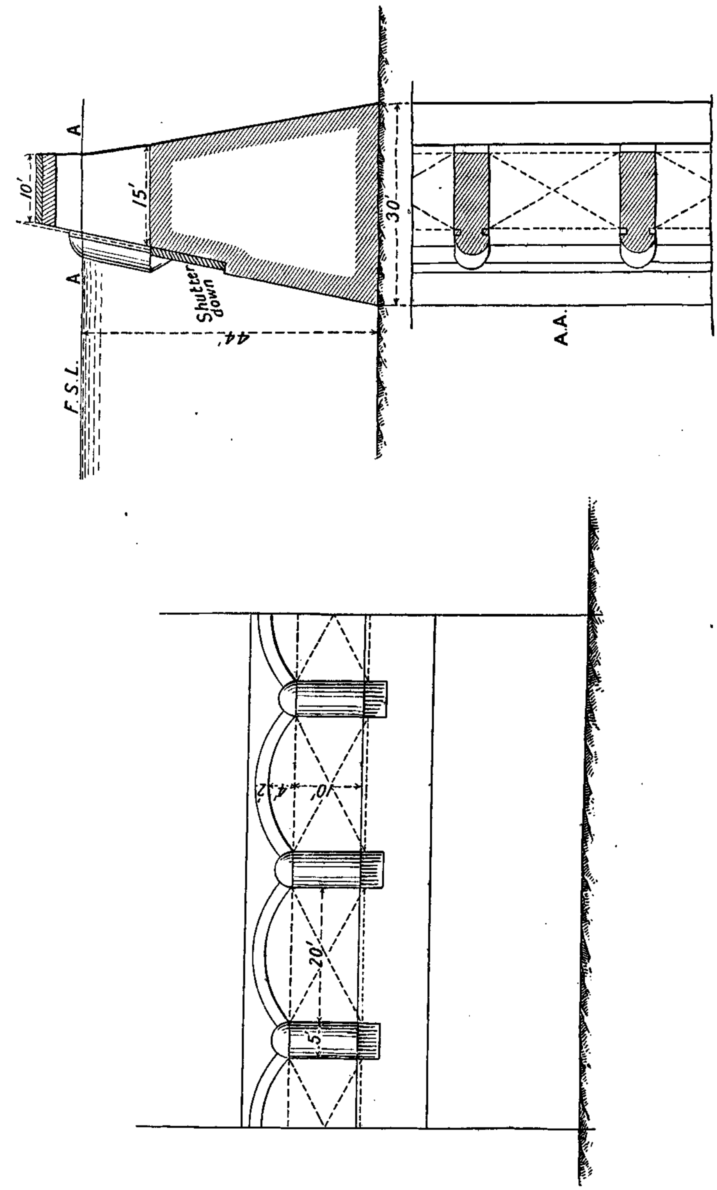


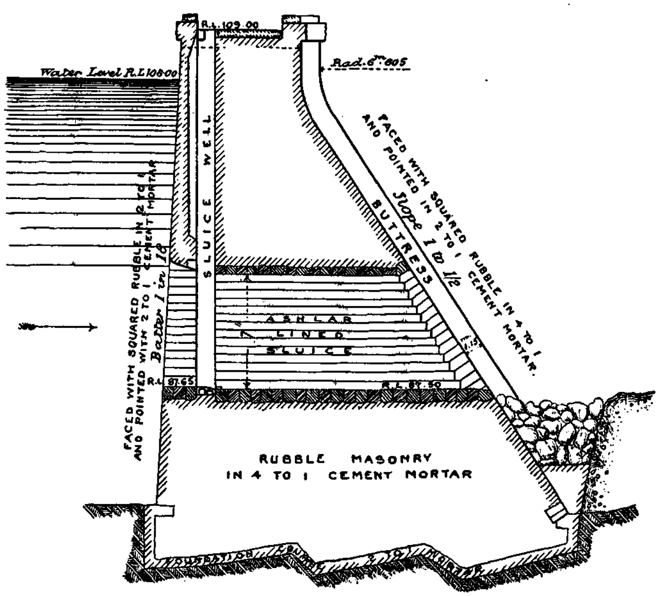
Fig. 18.—Goulburn Weir, Alternative Design.

This is undoubtedly a simpler and cheaper design than that shown in Fig. 17. The excessive base thickness given is due to the old exploded prejudice against allowing a free overfall, which still shows remarkable vitality in irrigation designs of the Western and Southern Hemispheres.

(29) Fig. 19 is a section of the Assuan Dam, as it stands at present, through the lowest tier of sluice openings. These sluices are 2 metres wide and 7 metres high, and the piers are 5 metres in width, with occasional abutment piers.

The lifting arrangement with Stoney's anti-friction rollers is illustrated in Chap. XIV.

An elevation of the dam showing the position of the various tiers of sluices is given in Fig. 19a, and a plan over all in Fig. 19b. This dam, of which a short account was given in par. 39, Chap. II., is now being raised 24 feet higher; it will then impound a volume double of what it now does and become a



SECTION OF PIERCED DAM.

FIG. 19.—Assuan Dam (Egypt).

really great work. Unfortunately plans of this project are not available, so it cannot be included in this edition.

(30) A short description will now be given of an example illustrated in Fig. 20 of an open sluice for supply or waste purposes fitted with the author's revolving gate. The gate is supposed to be of wood, and the drawing is little better than a sketch; it is, however, quite sufficient for the purposes of illustration. The chains at each end pass round drums which are connected by a horizontal shaft along the centre of the bridge. On the centre of shaft will be keyed a cog-wheel which is engaged by a pinion mounted on a frame and turned by a handle; this pinion can be easily disengaged; the shaft is then free to revolve with the gate. The chains passing over the end drums can be tightened or slacked as required by operating a screw which will raise or lower the standards carrying the drums.

Smooth cast iron plates projecting slightly are let into the wall in the

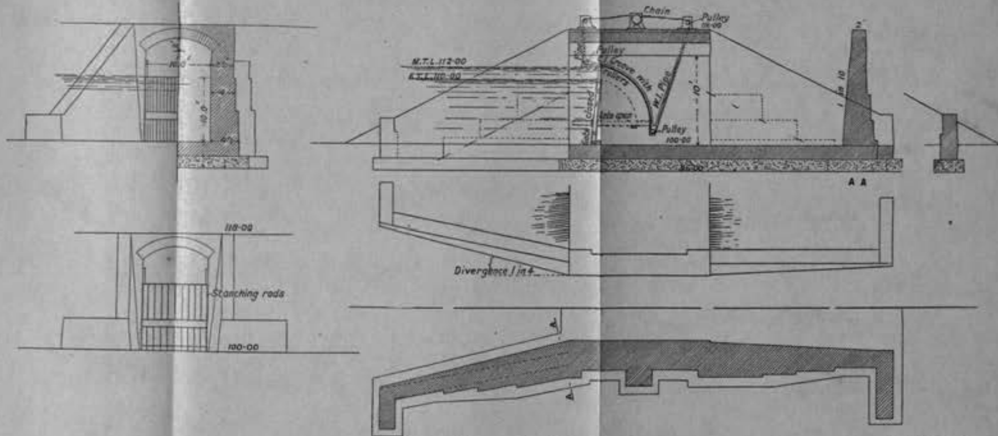


FIG. 20.—Waste-weir Revolving Gate.

[To face p. 378.

line of the gate when upright. The space left between the ends of the gate and these plates, about half an inch, will be closed and kept watertight by stanching rods of round iron attached to the gate. These are shown in the end elevations. A similar design is given in Fig. 9, Chap. X.

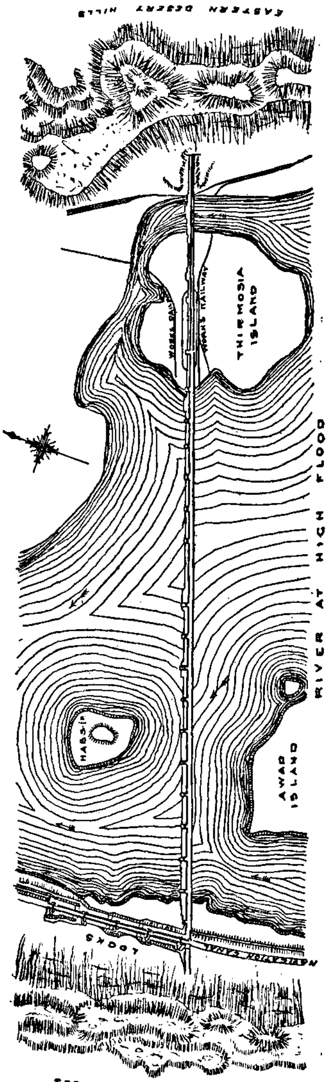


FIG. 19a.—Elevation, Assuan Dam.

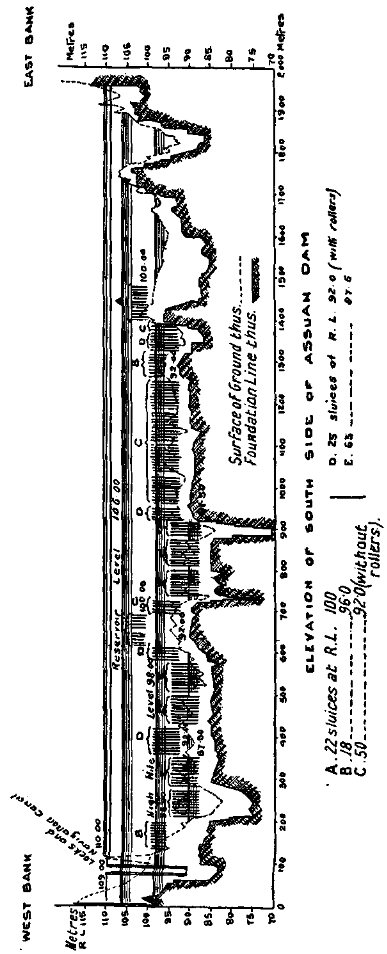


FIG. 19b.—Site Plan, Assuan Dam.

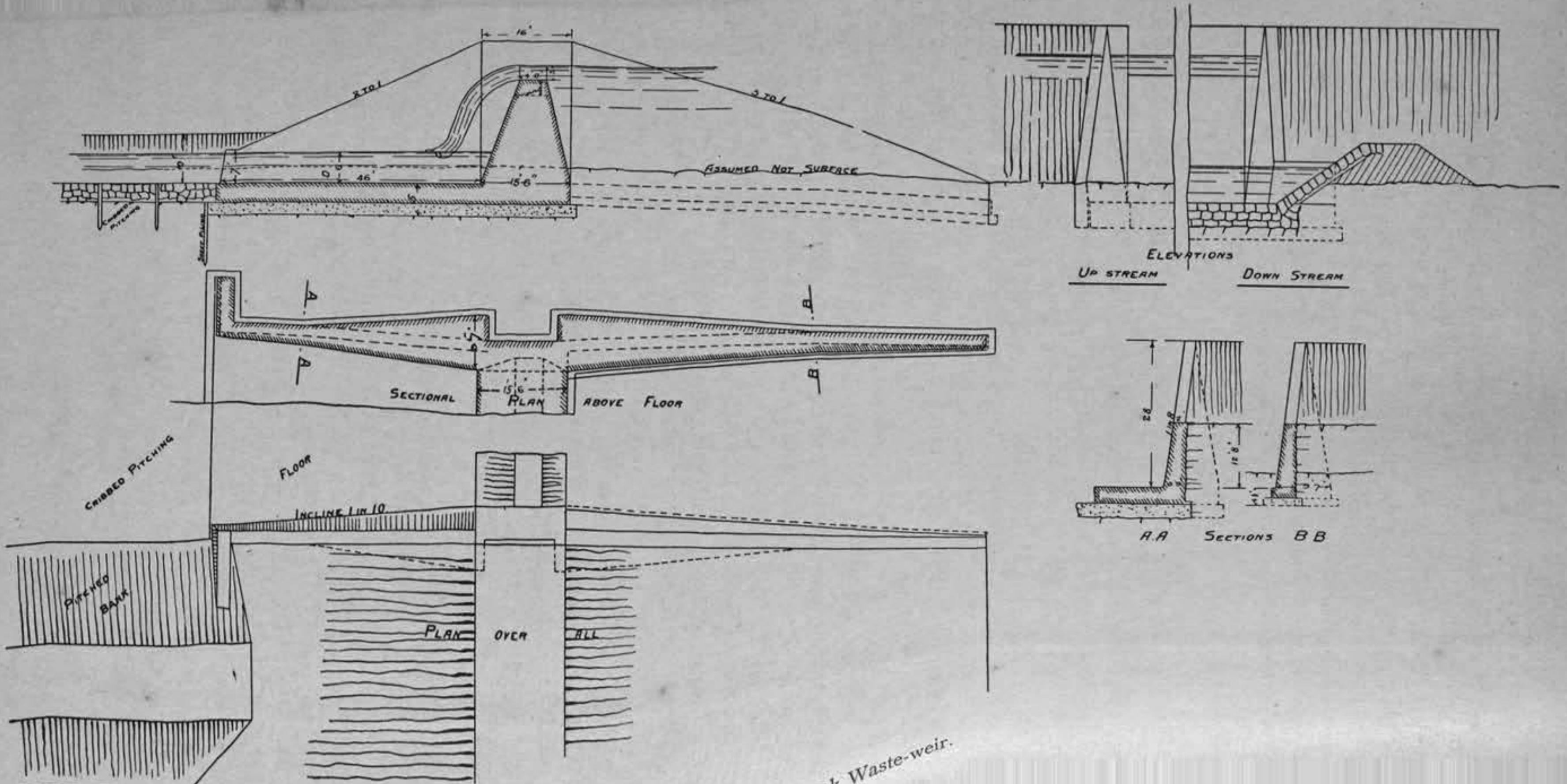
(**31**) An example of the design of a tank waste-weir is given in Fig. 21. The assumed data governing the design are—

d, or depth of film $= 3$ feet.

D the depth above floor in escape channel $= 6$ feet.

Then $r^\circ = \dfrac{d}{D} = \cdot 5$.

H or height of weir, 20 feet.



Design for Tank Waste-weir.

The surface level is assumed to be 3 feet above floor level, although this hardly affects the design. The free board is taken as 5 feet. The height of the abutments will then be $20 + 3 + 5 = 28$ feet. The section of the weir will be of the full, not the "hybrid" type, the use of which, except in cases where the weir wall itself is in cutting and the puddle at back can be securely filled in between the back of the wall and the face of the cutting under careful supervision, is to be deprecated as unsafe.

The crest width is made $\sqrt{H + d}$, or $\sqrt{23} = 4{\cdot}8$ feet.

The base width, according to par. 42, Chap. II., will be $\dfrac{H + d}{\sqrt{\rho}}$ $\left(D \text{ being less than } \dfrac{H}{2}\right)$ or $23 \times \frac{2}{3} = 15\frac{1}{3}$ feet (made 15 feet 6 inches). The profile of the weir wall is nearly equiangular.

The arrangement of wings adopted is a simple one. The embankment slopes are 3 to 1 rear and 2 to 1 fore. The approach wings will be inclined at 1 in 10 to the axis of the work and will be of the sloping crest variety, terminating at the intersection of the slope with the natural surface. The face wings will similarly slope down, their length being determined by that of the floor, and ending in a direct return dwarf wall. These will also be inclined on plan at 1 in 10. The wings will have a face batter of 1 in 8. Consequently their base thickness at starting point will be, according to Table I., Chap. I., $= {\cdot}4H - 1\frac{1}{2}$. This at the starting point at junction with the abutment will be $({\cdot}4 \times 28) - 1\frac{1}{2} = 9{\cdot}7$ feet. The back batter will disappear when the height of the wall is lowered to 12·8. This point will be 45 feet 6 inches from the abutment on the rear walls and 30 feet 6 inches on the face walls. The base width here will be 3·9 feet. As usual, the top width is taken at 2 feet.

The length of floor, which is measured from the toe of the weir wall, is $2(H + d) = 46$ feet. Its thickness will be $\sqrt{H + d} = 5{\cdot}0$ (nearly). The abutment, being mainly supported by the weir wall, is made 5 feet thick with vertical face and back. The battered backs of the wings overlap the sides of the abutments 1 foot.

The escape channel gradually widens at 1 in 10, and is shown embanked below the work, which would necessarily be the case if the natural ground were as assumed, at only 3 feet above flood level.

In tank falls the escape channel is generally made, if possible, wider than the weir, to reduce the depth D and consequent velocity of current. In such cases the depth in the escape channel is dependent not only on its bed slope, but its area. Examples of the type of fall most suitable for a waste weir in cutting are given in Fig. 24, Chap. I., and Fig. 4, Chap. IX.

In this design M.F.L. is assumed be 3 feet above F.S.L. Thus the whole embankment has to be made 3 feet higher than would be necessary did the full supply level coincide with the maximum. If collapsible crest shutters 3 feet deep were introduced as previously suggested, great saving would result in earth work. The masonry crest of the weir would then be 3 feet lower. The only difficulty in connection with the adoption of such gates is the necessity of an establishment to raise them at the proper time when the reservoir has been sufficiently relieved. Their provision, however, will not effect reduction in the length of the waste weir itself, as would be the case where the deep overfall sluice or low level sluice systems already described were adopted. For the purpose of lifting these gates, light short piers would have to be constructed at, say, 15 feet intervals, with one gate between, and a roadway formed by rolled beams and planks. The gates could be pulled up by attached chains passing over pulleys to a vertical drum or windlass revolved by gear or handspikes, which could be a fixture in each span; or a travelling winch running on rails could be adopted. A description of the crest shutters, 4 feet deep, of the Chenab Weir is given in "Min. Pro. Inst. C.E.," Vol. CXIII. These are hinged at base and raised from a platform above the weir.

The weir piers could project over the face or rear of the weir walls, so as to afford a sufficiently wide roadway without unduly widening the top width of the weir wall.

(**32**) Where local circumstances admit of it, the cutting of a wide spillway or by-wash will answer the purpose of disposal of surplus water from a reservoir. An example of such a work is illustrated in Fig. 22 of the spillway of Ashti Reservoir. The by-wash is a cutting 800 feet wide taken through the crest of hill which forms one boundary of the tank embankment; assuming no drop at the head, the slope is, roughly, about 1 in 700, or 1·4 per 1,000. With a 3 feet depth of water passing through, $A = 2{,}400$ square feet, $WP = 806$, $R = 3$ nearly, and $100\ RS = 6{\cdot}6$ nearly, whence $Q = 2{,}400 \times 6{\cdot}6 \times {\cdot}75 = 12{,}000$ cubic feet per second. ·75 is the coefficient c obtained from the Tables for a bed slope of 1 per 1,000, which will answer for all greater slopes (*vide* "Hydraulic Manual"). If a depth of 5 feet were adopted, the discharge through the by-wash channel would be $Q = 4{,}000 \times 8{\cdot}5 \times {\cdot}78 = 26{,}500$ cubic feet. What discharge the channel is intended to carry is not known. According to Table II., Chap. V., the discharge per foot run over a weir with free overfall is 17·3 cubic feet when $d = 3$, and 37·3 cubit feet when $d = 5$. The lengths of waste weir required to discharge the same amounts previously arrived at, viz., 12,000 cubic feet and 26,500 cubic feet, would be $\frac{12{,}000}{17{\cdot}3} = 694$ in the first case, and $\frac{26{,}500}{37{\cdot}3} = 710$ in the second case, the depths of water being the same in either case. Thus we see that with this steep slope the discharge is about seven-eighths of that with a free overfall.

If the ground is rocky, but will eventually wear away with the current, a good plan is to make but a shallow excavation, allowing the torrent to cut its own channel. If this is the case a weir should be built complete at the head of the work, but sunk in the ground, there to wait the retrogression of levels in the escape channel, which may not be fully developed for some years.

Distinctly the best type for by-wash falls in cutting is that illustrated in Fig. 24, Chap. I., and Fig. 4, Chap. IX.

In some cases, where the ground is favourable, rapids are a cheaper construction than a series of masonry drops, and are just as efficient. A rapid can often pass as much water over its crest as if the overfall were free.

Rapids are constructed either on a continuous slope, the inclination of which varies with the contour of the ground, or else are built in steps.

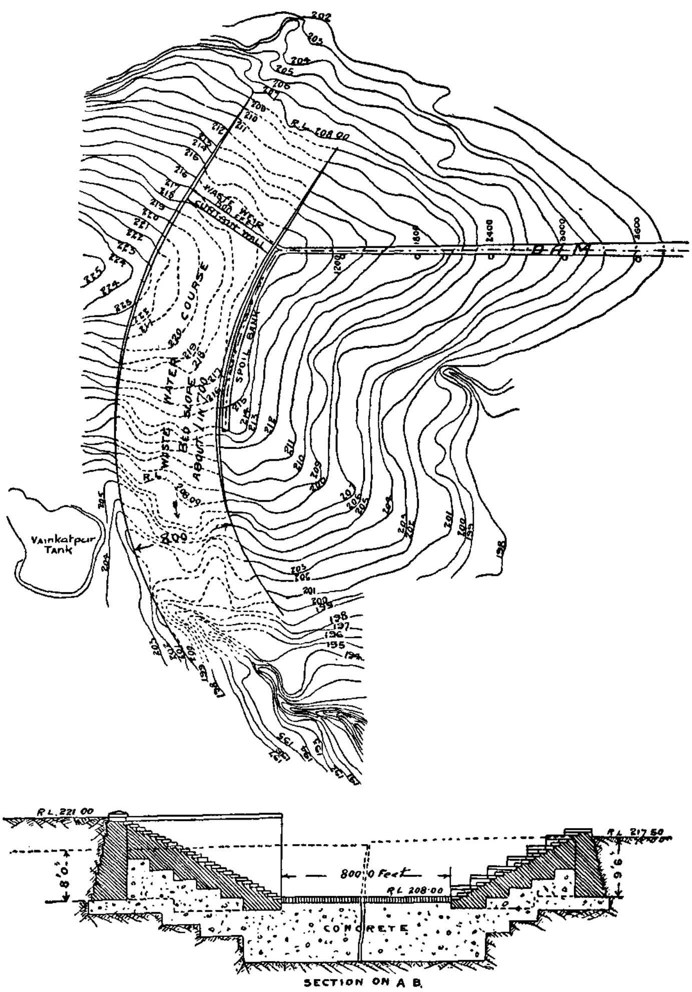


FIG. 22.—Spillway of Ashti Tank, Bombay.

Water cushions are often formed in each step by building a narrow wall above the tread at its edge; this holds up water over each step, and serves to dull the velocity of the current. A good example of an escape fall is that of the Kushuk Falls (Fig. 12, Chap. IX.). The water cushion here is necessary to check velocity.

(33) In the United States many excellent examples of storage reservoirs are to be met with, formed by damming up rivers, mostly in the upper

reaches, where rock exists in the flanks as well as in the bed of the river. Many such sites are not suitable for canals to take off. This is effected lower down in the course of the river, where another weir or dam of smaller dimensions, acting only as a diversion weir, is built with the concomitant canal head and possibly scouring sluiceway. In such cases the reservoir is tapped, as was effected in the Periyar project, by a tunnel, the entrance to which is controlled by sluice gates, or by a deep diversion cut in the solid rock at one flank of the weir, as in the case of the Minidoka Canal head works. Where canals can take off they often have to run in a winding course, cut as a bench in the hill side, starting parallel to the course of the river instead of

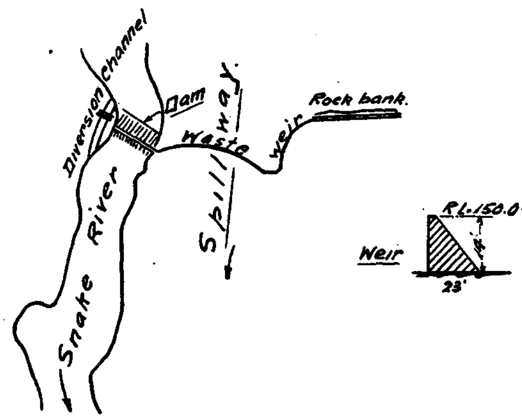


FIG. 23.—Minidoka Dam, Site Plan.

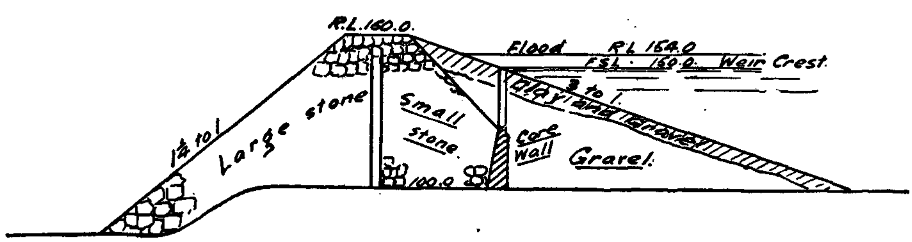


FIG. 23a.—Minidoka Dam, Section.

at right angles to it. This arrangement is rendered necessary by the configuration of the ground.

In Fig. 23 we have a section of the dam with a site plan of the head works of the Minidoka Canal, the head regulator of the canal of which has already been illustrated and commented on in Chap. VIII. The dam in this case is a combined loose rock and earth dam 60 feet high. The diversion channel, whereby the reservoir is tapped, is shown cut in the rock on the left flank of the weir. This is closed by five gates, 8 feet wide and 10 feet high, each operated by a pair of geared screws. The sill of these sluices is at R.L. 102·00, or 52 feet below crest of dam.

As is frequently the practice, the waste weir is situated on top of one or both the shoulders of the hill adjoining the canyon, a spillway being excavated if necessary to a depth sufficient to bring the crest of the waste weir 5 feet or more above the highest point of the spillway. If the waste weir is not raised well above the spillway, its free discharge will be greatly hindered by the shallowness of the approach channel. In the present instance no excavation seems to be requisite, as from the section given, the natural surface appears to be 14 feet below the weir crest and 18 feet below that of the dam. On this the weir is built to a curve on plan, following the highest ground line.

(**34**) A somewhat similar case is illustrated in Fig. 21, Chap. II., which is the site plan of the Roosevelt Dam across the Salt River in Arizona. This work is under construction at the present moment.

The dam is curved on plan, but is considered as a gravity dam, not depending on its curvature for stability. A roadway 13 feet wide inside parapets is provided along the crest, and continues across the spillways on, either flank over two large span bridges 200 feet in length.

These spillways are excavated in the solid rock, and the stone thus obtained will probably be used in the dam. They both start at about R.L. 190 and slope up to the weir, where the floor is at R.L. 205, and then immediately begin to fall down to R.L. 155, where the rapids merge in the walls of the canyon. The waste water is thus safely conveyed away, clear of the dam. These waste weirs are each 200 feet long and 5 feet high. The Salt River project is a very large one; the dam will impound 1,284,000 acre-feet, the area of its watershed is 6,260 square miles, and it will irrigate 200,000 acres.

The outlet of this reservoir is made by driving a tunnel 500 feet long through the solid rock at one side of the dam, in which six large gates weighing about 355 tons and operated by electric motors will control the outflow. It will also act as a silt scouring sluice. It would be a matter of interest to know if these are antifriction roller gates or whether the old contact system is still practised, as appears to be mostly the case in the States. The tunnel can discharge 10,000 second-feet, which falls into the river channel below. There are several other works on this large project, of which one, the Granite Reef Weir over the Salt River, is illustrated in Chap. II.

(**35**) A further example of a reservoir work is the Folsom Canal Weir, built across the American River in California (Fig. 24). The remarkable disposition of the head works is due to the necessities imposed by the site. The river is crossed at right angles by a masonry weir, which for a length of 180 feet has the crest lowered. This gap can be closed by one long hinged truss gate, operated by hydraulic presses, details of which are given in Wilson's "Irrigation Engineering." The higher portion ends on the west side with a head regulator, the canal of which takes off at right angles to the axis of the weir. On the east side the weir is turned in a curve of 90°, and thus continues parallel to the river up to another canal intake, that of the East Canal.

The crest of the higher part appears to be only 5 feet above that of the weir crest, *i.e.*, on the same level as the crest of the weir shutters, with the evident intention that in case of high flood the long arm of the weir could be overflowed and thus utilised as an emergency outlet for flood water. The maximum flood level is said to be 30 feet above weir crest, or at R.L. 225.

In both canals sluice gates are provided for the purpose of removing silt, while there are three culvert sluices in the body of the weir itself at a low level in order to scour deposit occurring in rear of the weir wall.

The Eastern Canal Head is a very massive structure, and has been already illustrated in Chap. VIII. This head is provided with what are termed sand traps, that is, silt is deposited in the sand traps and carried away by the sand gate shown at the side. This arrangement is all the more necessary as the scouring sluices are situated too far away to much influence deposit in front of the canal head work, and by the lifting system adopted the water can only

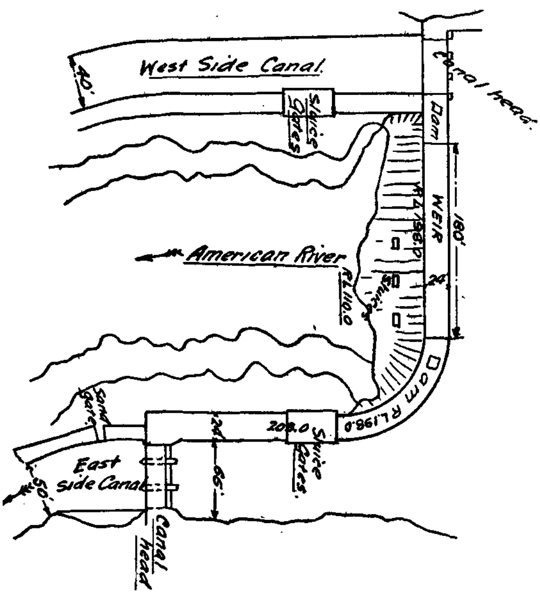


FIG. 24.—Site Plan of Folsom Canal Head Works.

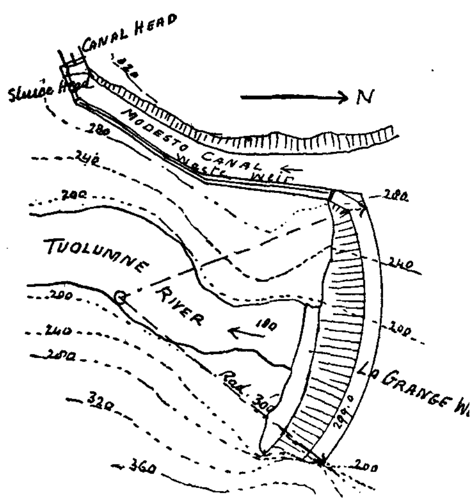


FIG. 25.—Site Plan, La Grange Weir.

be admitted at the bottom of the gates; thus any accumulation must be washed inside the canal. The section of the weir is given in Fig. 34, Chap. II.

(**36**) Fig. 25 is the site plan of the La Grange Weir in California and connected head works. A section of this weir has already been given in Fig. 29, Chap. II. The arrangement of the canal off-take is similar to that of the east side canal of the Folsom Dam. The canal runs parallel to the river on a bench cut in the shoulder of the hill, with a flank wall on the left side towards the river. This flank wall acts as a waste weir and is practically a continuation of the main weir, but at right angles to it. This subsidiary waste weir terminates with a scouring sluice head, close to which, at right angles, is the Modesto Canal Head Regulator. This disposition is clearly a better one than that at the Folsom Canal, where the sluice head is placed not close to the canal head, but higher up the approach channel near the weir. The clearance of all deposit in the approach channel, which also acts as a sluice way, is thus assured, which is not the case with the Folsom Canal Head. On the left of the weir a tunnel is bored through the rock for the supply of the Turlock Canal (not shown on site plan).

(37) Fig. 26 is a map showing the course of the Turlock Canal, which is not given in the larger scale site plan of Fig. 25.

The canal starts with a tunnel through the solid rock, it then falls into Dry Creek, filling up a large depression which is blocked at a low part of its

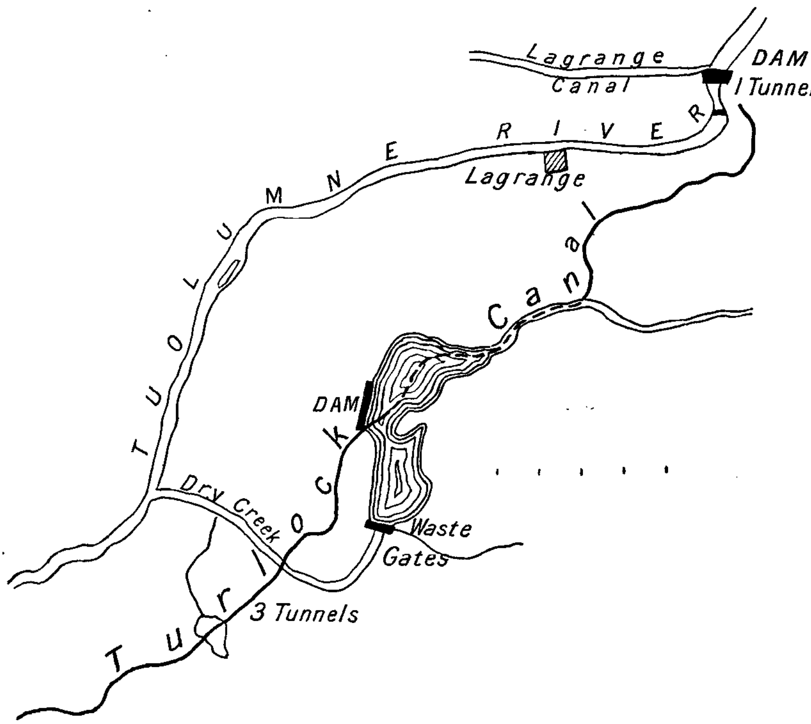


FIG. 26.—Site Plan of Turlock Canal.

banks by a dam. The canal, after leaving this depression which it fills up with water, crosses Dry Creek, being here at a higher level, and then passes through a watershed by means of three short tunnels, when it reaches comparatively open country. Any excess of water received through the tunnel, is got rid of by waste gates, or as they would be termed in Indian irrigation nomenclature, by an escape head, which lets it out back into the Tuolumne River by way of Dry Creek.

The whole scheme forms an instructive example of the difficulties connected with the alignment of canals in a hilly country.

(38) The Dhukwa Weir in Bundelkhund, United Provinces of India, the profile of which has already been given in Fig. 28, par. 56 of Chap. II., is a quite recent project, of which some account will be of interest.

The section of the weir, repeated in Fig. 27, is a great improvement on that of the old Betwa or Parichha Weir, the heavy profile of which is by no means to be commended. This latter has not been illustrated in this work, although the head regulator of the Betwa Canal, an adjunct to the old weir, has been reviewed in Chap. VIII.

Like many American examples, the weir will impound water to a level well raised above the shoulders of the flanks of the river channel proper. The work will thus consist of a central overfall weir crossing the defined river channel, its continuation on either flank being an insubmergible earthen dam provided with a masonry core wall. The two earth dam portions are about 1,000 feet long each and some 35 feet high, while the central masonry weir is about 60 feet high to crest of the shutters. This arrangement is exactly the reverse of that obtaining in the Minidoka Head Works (Fig. 23), or of the Roosevelt Dam (Fig. 21, Chap. II.), where the waste weirs are on the flanks, the central part being an insubmergible dam.

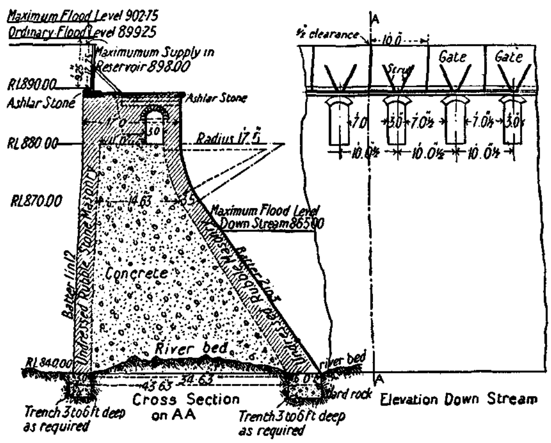


FIG. 27.—Section of Dhukwa Weir.

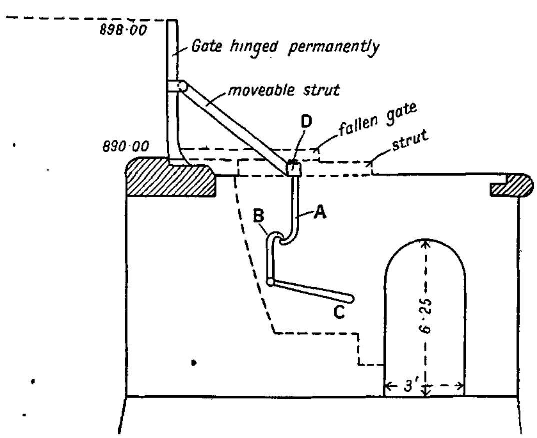


FIG. 27a.—Suggested Collapsible Shutters.

The flood discharge of the Betwa River is estimated at 800,000 second-feet, the available impounded supply will be 86,000 acre-feet. This reservoir will be tapped by three large body sluices, 8 × $6\frac{1}{2}$ and 8 × 10, worked by Stoney gates, and will act as a feeder to the lower old Betwa Reservoir and to the canal which takes off from it. How these sluices are to be worked, whether from the tunnel or from top of special piers built over each on the weir crest, is not stated.

(**39**) The arrangements adopted in this work of dropping and raising the collapsible weir shutters, present features of considerable interest, as they do not involve the necessity of an over-bridge. They are fully illustrated in Figs. 27 and 27a. In the latter figure it will be seen that a rod marked *A*, provided with an hooked end, is attached to the base of the movable struts of each shutter, this end is engaged by the similarly curved end of an arm of the lever *BC*.

The lever handles *C* are all situated in recesses connected with a tunnel which runs longitudinally throughout the weir. By pressing the handle *C* the rods *A* and *B* are disengaged and the shutter will fall. The gates can thus be let go either by an attendant or by the handles being connected with a cable worked from each end of the weir.

The raising of the shutters will be effected by means of a crane, running on rails on the weir crest itself. The crane as it moves along will be sheltered from the rush of water over the weir by being situated well in rear of the shutters it has already raised in its advance. The connection of each strut with the lever *BC* will simultaneously be effected by a man operating from the tunnel below.

The weir shutters are 8 feet high.

The obviously weak points in the design of the regulation apparatus consist in the multiplicity of the gates, which, not being automatically collapsible, have to be lowered as well as raised. The weir crest is no less than 4,000 feet in length, consequently 400 gates will have to be manipulated. Arrangements are contemplated whereby they can be lowered in batches, but the raising of the shutters will necessarily be a very slow operation. In addition to this, considerable leakage is bound to occur into the subway by means of the slots through which the vertical arm *A* attached to the longitudinal *D* must move back and forwards as the shutter is raised or lowered. These will doubtless be covered by stanching strips, but still, under a possible head of 13 feet of water, leakage is bound to occur. The disadvantages inherent in this system are considered as so formidable as to render even its trial a matter of doubt.

Two alternative systems presenting advantages from some points of view have already been illustrated in Fig. 34, Chap. II., and in Fig. 14. In the former, that of the Folsam Canal Weir, a hinged trussed shutter, 150 feet long and 5 feet deep, is raised and lowered by means of a series of hydraulic jacks, the plungers of which act as the supporting struts. This system could well be adopted for the Dhukwa Weir, with or even without the necessity of piers; the matter of expense for power installation being the only question to be considered. This would necessarily be very considerable, owing to the great length of the weir crest.

The second alternative system is that illustrated in Figs. 14 and 18, viz., of inclined automatic roller gates working up from below the crest on the inner face of the weir. In this system, however, piers will be necessary, and the spans could not well be very large, as the pressure instead of being distributed generally along the weir crest will be concentrated at the piers, so that unless buttresses were provided below the piers the spans could not well exceed 10 feet, which would not be sufficiently wide for the passage of a torrent in violent flood, as the Betwa River.

(40) The author believes that a modification of the principle exemplified in the case of the Schweinfurt Weir in Germany, which is illustrated in

Wilson's "Irrigation Engineering," page 184, will be found capable of satisfying the contradictory requirements of economy, simplicity, and rapidity of action in the regulation of this weir, and be found superior to either of the above systems. In the Schweinfurt Weir, the raising of the water level above the crest is effected by the adoption of a hollow steel cylinder internally braced, of a length of 150 feet, which lies on the weir crest between the two abutments. In flood-time this cylinder is raised clear of the weir and of the flood-water by being revolved, and it thus mounts a steep inclined shoulder, cut in the sides of the two abutments by means of encircling chains, one end of each of which is fixed and the other wound up on the drum of a motor winch. The cylinder is thus rolled bodily off the

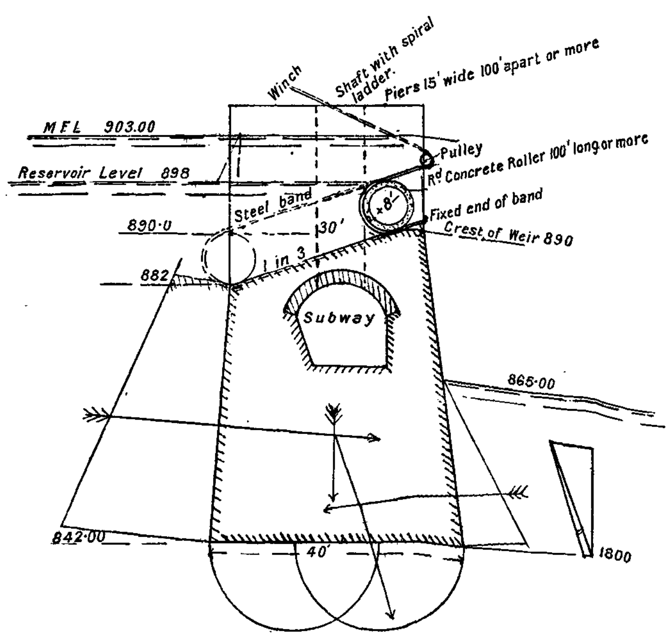


FIG. 28.—Alternative Design for Dhukwa Weir, with Rolling Cylinder.

support of the weir clear of the flood, and is similarly lowered when closure is required. The development of this principle, shown in Fig. 28, is to effect the required regulation, not by lifting the cylinders off the weir crest, as was previously done, but by retaining them on it, the crest being also inclined and built on an incline of 1 in 3. Regulation is then effected by rolling the cylinder up and down this inclined plane. The cylinders are never intended to be lifted clear of the support of the weir crest, and consequently are entirely free from the great bending stress induced were they supported only at each extremity, as in the example already noted. Further, the power required to move them is very slight, as the inclined plane is of moderate slope, and the friction is a rolling friction. They can then be made of considerable length and be formed of such economical a material as reinforced concrete. The cylinders will be revolved by steel ropes or chains encircling each end, which will run in a recess cut in the side of the piers. One end of the chains will be fixed to a hook at the summit of the run, while the other will be carried round the cylinder on to a pulley and thence to the drum of a hand winch fixed on the top of the pier. The cylinders will fall back down the inclined crest by their own weight, when the supporting bands or chains are unwound.

For the prevention of silt deposit on the inclined crest it is proposed to admit water through corrugations in the surface underneath the rollers, so that the only quite watertight connection will be at the summit of the run. No overbridge is needed, the subway system with the addition of vertical shafts at the piers being maintained.

In the diagram the summit of the trapezium of pressure should have been placed at R.L. 890, not 882. This correction would bring the centre of pressure just at the middle third of the base.

(3) *Reservoir and Tank Irrigation Outlets.*

(41) Next to pipes laid through an embankment, which as a rule are only suitable for small tanks upholding 6 feet to 8 feet of water, the simplest form of outlet is that illustrated in Fig. 29. This consists merely of a narrow bridge opening, which is regulated either by double wooden gates either superimposed or moving in double grooves, or by sleepers or baulks of wood, which are slipped into the grooves and removed or replaced one by one as required. The advantage of the latter system is its extreme simplicity and ease of manipulation. The water being drawn from the top and having a vertical overfall, the velocity of entry is mostly absorbed. On the other hand, when draw-gates are used, the water issues with considerable velocity, the opening acting as a sluice under a head of water. The use of double or triple gates obviates this to a considerable extent, as when the top gate is completely withdrawn the conditions become those of an overfall.

In Fig. 29 the opening is 5 feet wide; the floor for the length of the abutments is formed by an invert, a suitable arrangement in this case, as the abutments can be considered as a masonry beam supported at both ends and subjected to a load increasing from the top with the ordinates of the triangle of earth pressure (*vide* Chap. I.), and so can be designed of light section.

The arch is placed so as to be clear of the water spill at full supply. The wings are of the ordinary sloping crested type, and their bases are divergent, 1 in 20 (or the vertical batter multiplied by the slope of crest) from a parallel to the axis of the work. On both sides the wings have a crest slope of 2 to 1, that of the embankment on the rear side being reduced gradually from its normal inclination of 3 to 1 to the steeper slope. This intermediate portion of bank should be pitched. The rear wings slope right down to the ground level, which is the most economical arrangement, while the fore wings terminate in short returns so as to form a junction with the bank of the escape channel or canal.

(42) When the water is deeper than about 10 feet, this type of outlet will not answer. It will be more economical to adopt a barrel culvert for the

outlet, thus abolishing the abutments. The wings, however, will remain, with the addition of a breast wall on the fore side as connection.

Sluice outlets of this description are illustrated in Figs. 30 and 31, in both of which the arrangements are similar.

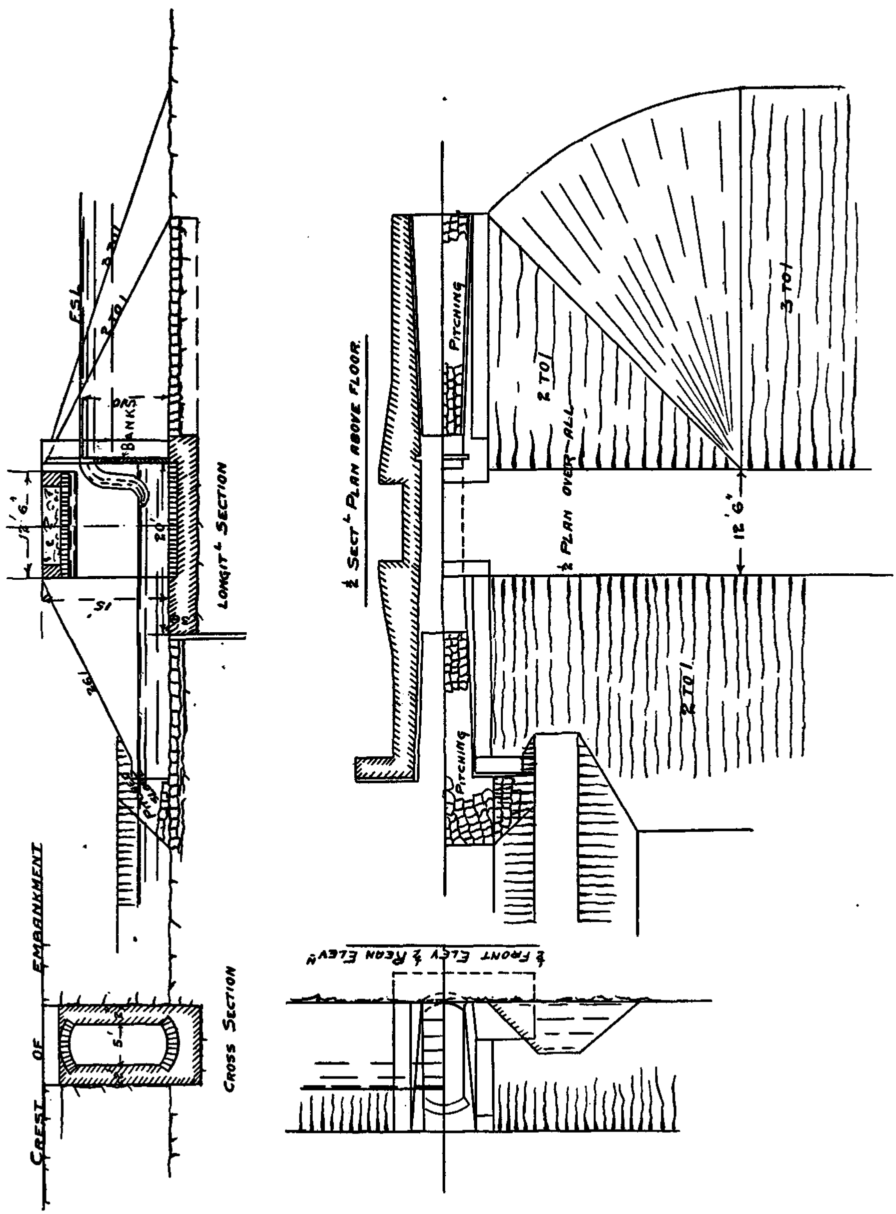


FIG. 29.—Tank Outlet.

The dimensions of the culvert are determined by the following considerations:—

Firstly: They should be sufficiently large to admit of a man entering them for examination or repairs.

Secondly: The maximum mean velocity of the current through the culvert should not exceed 5 feet per second. In Madras 15 feet per second is allowable.

In Fig. 30 the sluice gate is 2 feet square; the head of water, H, is 7 feet, *i.e.*, considering the opening to be just submerged, consequently the velocity of issue, according to Formula (1), Chap. V., will be $c\sqrt{2gH}$, or taking $c = \cdot 62$, the velocity of issue will be $\cdot 62 \times 8\cdot 025 \times 2\cdot 646 = 13$ feet per second nearly, and the discharge $Q = AV = 4 \times 13 = 52$ cubic feet per second. To pass this discharge at the limit velocity of 5 feet per second the sectional area of the culvert barrel will have to be $\frac{Q}{V} = \frac{52}{5} = 10\frac{1}{2}$ square

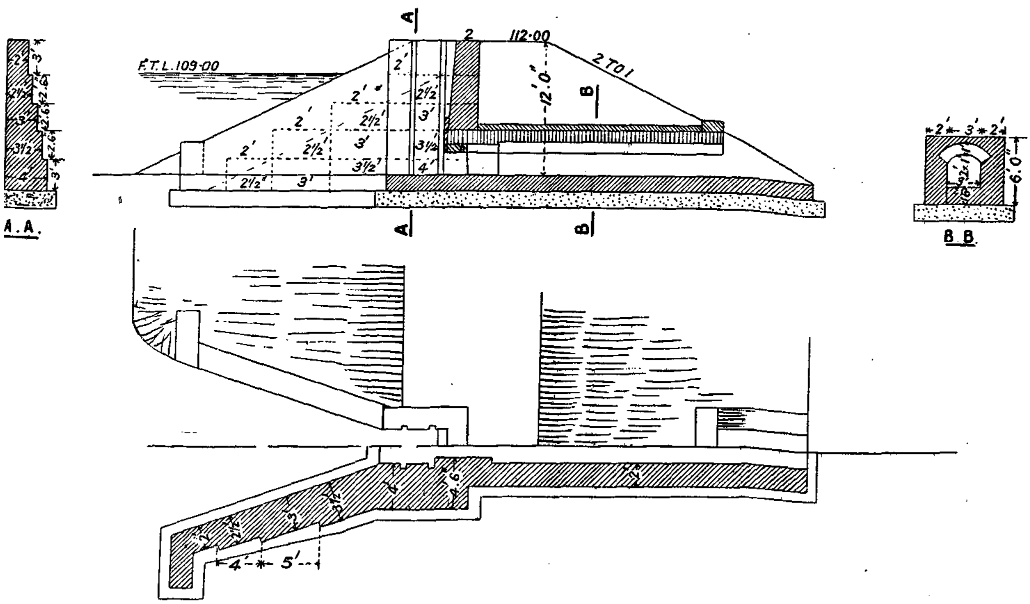


FIG. 30.—Tank Outlet Sluice.

feet nearly. The culvert then should be $3\frac{1}{2}$ feet wide $\times$ 3 feet high, to half-way up the arch.

In the design in question, however, the maximum discharge was limited to 30 cubic feet per second, so that the sluice gate will never be fully opened when the water level is at full supply or tank level, and the area of the outlet culvert was designed accordingly.

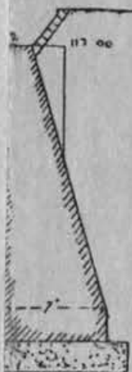
It is, however, too small for inspection purposes, being only 3 feet $\times$ 3 feet outside dimensions, and would have been better if made the larger size.

In this design one set of grooves is provided distinct from the gate for the purposes of blocking the outlet by baulks to admit of inspection and repairs to the gate. The shutter or gate should be of cast-iron strengthened with gussets on the outside, and the grooves should likewise be of cast-iron. Grooves made up of angle or channel iron, though sometimes used, are by no means satisfactory. The lifting apparatus should be screw gear, the best form of which is illustrated and described in Chap. XIV. The head of the sluice should consist of a square cast-iron flanged box built into the masonry, the hollow spandrel between it and the relieving arch being built up with brickwork in cement.'

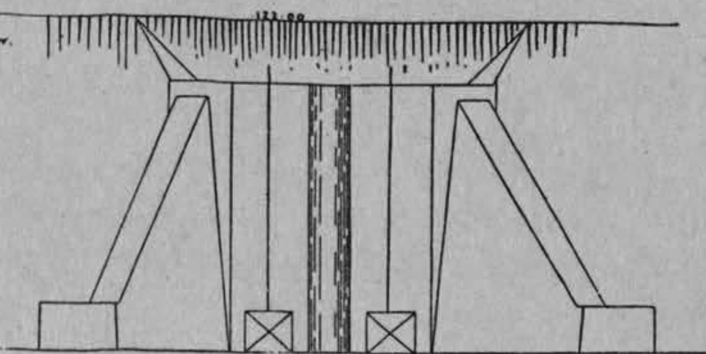
(**43**) Fig. 31 is a representation of a similar work, but on a larger scale. The sluice openings are double, 3 feet × $2\frac{1}{2}$ feet in size. In order to save masonry, the projecting piers and abutments terminate at *R*117, or 1 foot above maximum tank level, and space for the working platform, which is usually of wood, is obtained by sloping back the embankment. Access to this will be afforded by a step-ladder. Two sets of grooves are provided in order to enable either of the gates to be examined and repaired with a head of water in the tank. This can be effected by filling and consolidating earth in the space between the sets of baulks placed in the grooves. Figs. 31a and 31b are sketches of the gate and its frame. The sluice head openings can be lined with cast-iron plates or else built of ashlar. The roof can be made of rails or rolled beams set in cement concrete, or ashlar slabs could be used. Each of these sluice openings, with a maximum free head of 13 feet, will discharge $Ac\sqrt{2gH}$, or $3 \times 2\frac{1}{2} \times \cdot 65 \times 8\cdot 025 \times 3\cdot 6 = 140$ cubic feet nearly, or with both open, 280 cubic feet per second. The area of the culvert when full is about 28 square feet, so that the velocity of passage will be 10 feet per second with both sluices open. The channel to carry this discharge would have to be of 20 feet base width at least.

(**44**) In order to neutralise the velocity of issue a common practice is to build a square chamber in the rear of the culvert with bed sunk well below the level of the floor, and a low weir in front raised above the floor. The increased waterway thus given absorbs the velocity and the current issues from the chamber at a moderate pace. The outline of this is shown dotted on the plan and elevation in Fig. 31. The banks and bed of the issuing canal would have to be pitched for a considerable distance below the termination of the floor and wings of the sluice, unless some such remedy were adopted. The design was, however, made under the assumption that only one sluice would be opened when supply was at the maximum, the greatest allowable discharge being about 140 cubic feet per second. This would give a velocity of 5 feet through the culvert. Under such circumstances the well would be unnecessary.

(**45**) The disadvantages of draw-gates to reservoir sluices is that careful regulation is required to prevent waste of water, and the screw lifting apparatus is expensive. With deep water the gates require great pressure to force them down. The coefficient of friction in rusty gates certainly equals unity. This last objection can be obviated by introducing anti-friction rollers in the gates. The groove of the frame in this case will have to be made wider and deeper, or else could be abolished altogether. In any case, the use of vertical stanching rods will prevent leakage at the sides, and a horizontal one at the top between the upper edge of the gate and the sill.



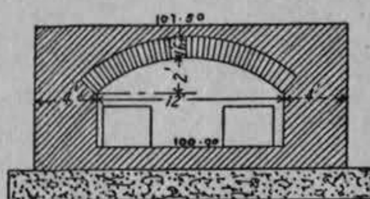
B B



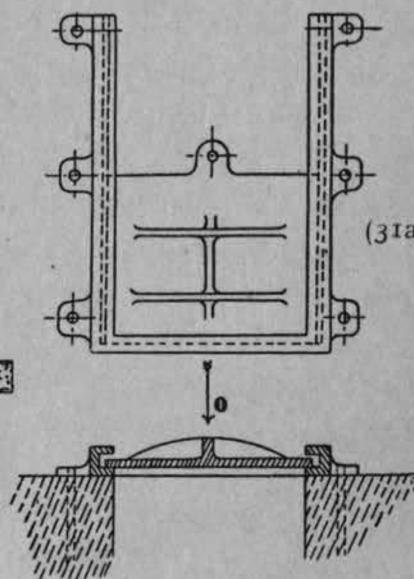
FRONT ELEVATION

Dotted lines shew position of sunk rear chamber.

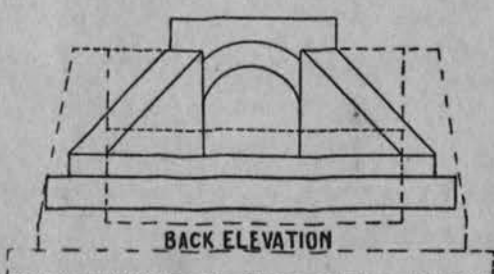
C.C



D.D



(31a)



BACK ELEVATION

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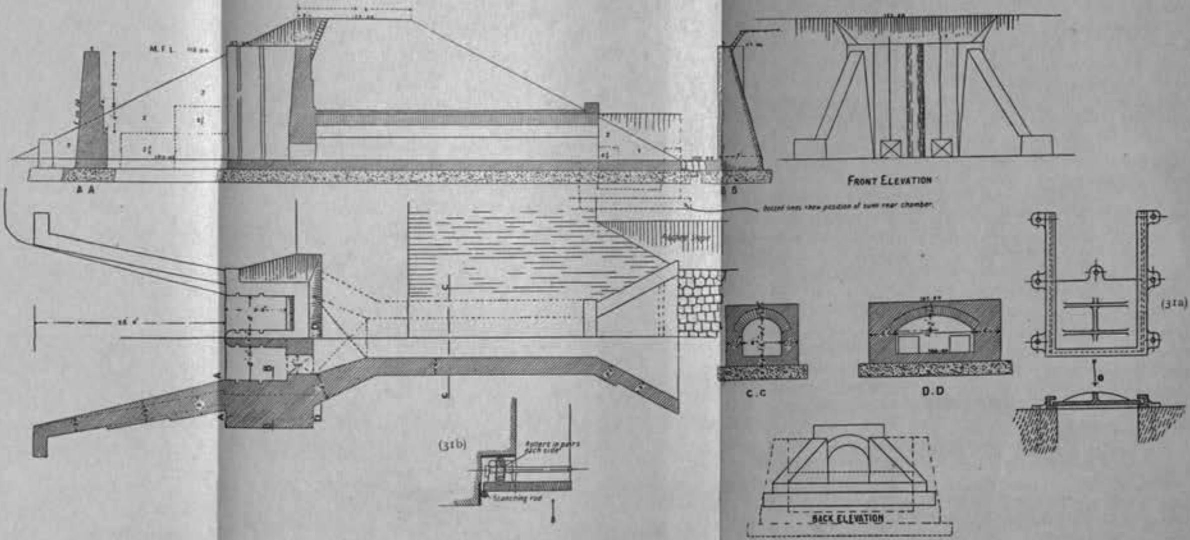

FIGS. 31, 31A, 31B.—Double Tank Sluice.

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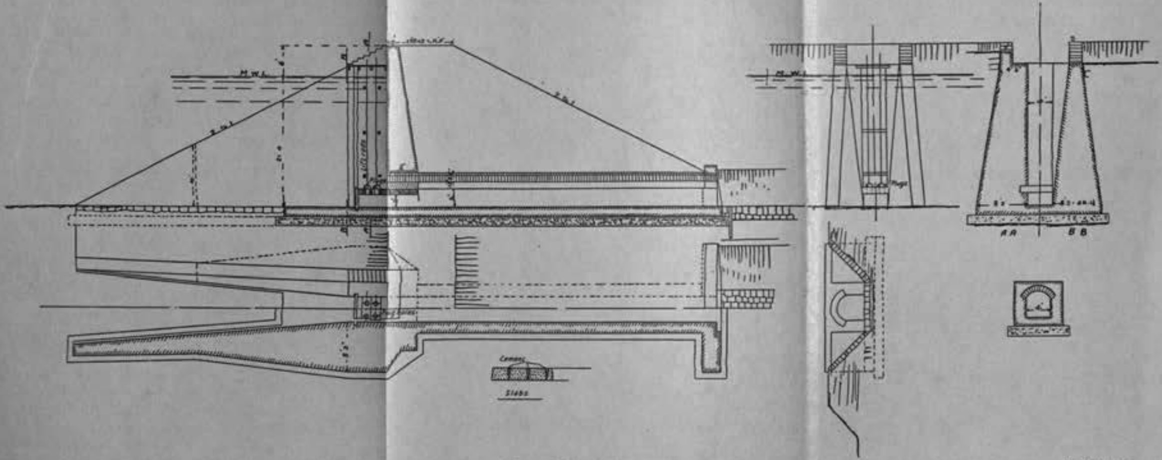

FIG. 32.—Sluice for Plugs.

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By this simple arrangement the frictional resistance would be reduced by 70 or 80 per cent., thus enormously relieving the pressure on the screw gearing, which could be made of a very light section in consequence.

(46) In the Madras Presidency a system has been successfully employed for many years in smaller tanks, which abolishes all frictional resistance, the pressure of water alone having to be met, and that on a very small area, and further enables very accurate regulation of the water to be effected. This system consists of admitting water into the culvert by means of circular holes cut in the flat roof of a chamber built in front of the sluice head, which holes are closed by conical plugs of wood. This is for small tanks of 20 feet depth.

Fig. 32 contains a design illustrating this construction. Before proceeding to discuss this design we will give the following extract from the "Madras Irrigation Manual" relative to this subject.

"Taking into account facility, simplicity, accuracy and safety, there are no better means of regulating the discharge of irrigation water from tanks of the ordinary kind than plug holes provided with suitable plugs. To secure convenient and fairly accurate regulation it is necessary that (1) the holes should be of a suitable size and sufficient in number; (2) proper plugs should be provided and a well-cut seat made for the shoulder of the plug; (3) the plugs should be regulated from a platform always accessible; (4) it should be known what the discharge is at any given time, and with the tank water at any given level; and that the way to secure any required discharge should be readily ascertainable. In order to facilitate calculation and comparisons, it is convenient to adopt one ratio of coning for all plugs, and this has been fixed at 1 in 4; also to select the following diameters of plug holes for general use, viz., 4, 6, 8, 10 and 12 inches.

"It will often be found that a very small expenditure on the improvement of an existing sluice will obviate the necessity for constructing additional sluices, which may be asked for by the cultivators. In addition to the plug holes, there is usually and properly a low level vent, which is closed and regulated by planks or a shutter sliding in grooves. The regulation of this lower vent cannot ordinarily be conveniently managed when the water is more than 2 feet deep on the plug stone, corresponding to about 3½ feet on the sill of the sluice. It is therefore desirable that the whole area of cultivation under a sluice should be capable of being efficiently supplied through the plug holes alone when the surface of the water is 2 feet above the plug stone.

"Again, although 2 cubic yards per acre per hour, or 1 cubic foot a second for 66 acres, is usually adequate as an average supply, this quantity, when much of the land is being prepared for cultivation, will be by no means enough. It is desirable, therefore, that the plug holes should be capable of discharging much more water at times, and this can be provided for by allowing one or two large, or several smaller, plug holes so as to admit of the discharge being varied according to circumstances.

"It is desirable to provide means for raising the plugs to a height just sufficient to secure a full discharge, and no more, and this will be the case when the bottom of the plug is raised to a distance, equal to the diameter of the hole, above the plug stone. An iron bar (round), passed through the plug and secured thereto by a key or cotter at top and bottom of the plug, is the best arrangement for the lower part of the attachment, whether a rigid bar be used throughout or a chain be the connection at the upper part with the platform. A cross-bar with a hole or a collar is the most suitable means of limiting the lift of the plug; and a second cross-bar should be placed just below the top of the rigid part of the lifting apparatus when a chain is used for the upper part. The plug will then be always vertically over the hole to

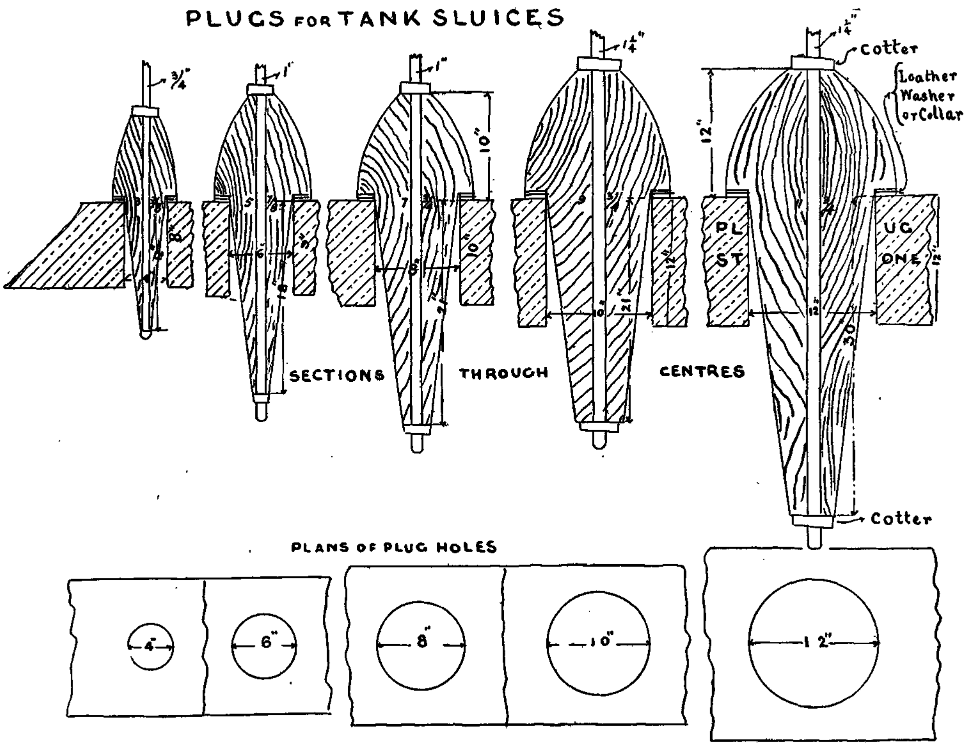


FIG. 33.—Plugs for Sluices.

whatever height it may be raised. When a rigid bar is used from the plug to the platform, the upper part should be flat and should be pierced with holes at short intervals (3 inches will usually be appropriate), as generally with some small and some larger holes in the plug stone the regulation can then be effected with sufficient accuracy. Similarly, when a chain is used for the upper part of the lifting gear, the links may be 3 inches long at the part within the range of the lift.

"When the plugs are of large diameter and the depth of the water great, they cannot be lifted by one or two men, and the provision of some sort of mechanical lifting gear will be necessary. This may be either a screw or a windlass or an overhead bar with small pulley tackle, either of which would enable the regulation to be readily and easily effected. A windlass should have a ratchet with pawl or detent, to keep the former in any required position.

"The plugs should be made of hard dense wood, and should be turned in lathe, so as to be true to gauge. Heart of Tamarind or of Vagay (Tamil), Darasippa (Teluga) makes good plugs. They should have iron flush straps at top and bottom of the coning to prevent splitting. The shoulders of plugs should be lined with leather, the thickness of the layer varying from $\frac{1}{4}$ inch to 1 inch, according to the size of the plug and the depth of the water (maximum). This layer of compressible material will diminish leakage and reduce the risk of fracture of the plug stone, should the plug be accidentally allowed to drop suddenly."

Drawings of different sized plugs are given in Fig. 33.

(**47**) The remarks above quoted concerning lifting apparatus require, it is considered, some modification. All plugs, small and great, should be lifted by screw gear. The rods for the larger sized openings under considerable pressure should be hollow, of gas or water piping. At the end of these rods, which should terminate above M.W.L., a brass female screw should be fitted. This is engaged by threads cut on a solid round bar; near the upper extremity of this latter a circular thrust plate is forged which works in a thrust box fitted with brass bearings, above which is the square rod head which takes the handle. Thus the power is applied to the male screw, which when revolved enters the hollow pipe, drawing it and the attached gate or plug upwards; in the same way by reverse action the pipe rod is forced downwards. This method of screw gear is far superior to that commonly employed, which is to adopt a solid rod threaded at the upper end. This screwed end is embraced at the top by a short female screw with a thrust flange and box. In this case power is applied to the female screw and the solid rod passes through it, rising above. This system necessitates a long, heavy, expensive solid rod, and, further, the screwed end rising above the platform is exposed to the weather; whereas in that first described, the screwed solid rod is only of a very short length, viz., that of the vertical play of the gate or plug, and can easily be replaced when worn, and the transmitting rod, being hollow, is well suited to resist torsional as well as compressive strain. The solid screwed rod is further protected from the weather and from dust, etc., as no portion appears above the platform. Both these systems are illustrated in Chap. XIV., Figs. 3 and 4. The second system is only suitable for the smaller sized plug holes and should never be adopted for sluice draw gates.

(**48**) Some other points require notice. The fore bay chamber is made only 2 feet in height and is shown such in the drawing in Fig. 32. This limit in depth is due to the want of proper grooves and lifting gear; the grooves arrangement shown in the original plan in the "Madras Irrigation Manual," which has been only followed as far as the section of the culvert and fore chamber, is exceedingly primitive. With proper iron grooves the wooden gate, even without rollers, could easily be lifted by means of an ordinary wooden windlass, under a very much greater mean head than 3 feet, which

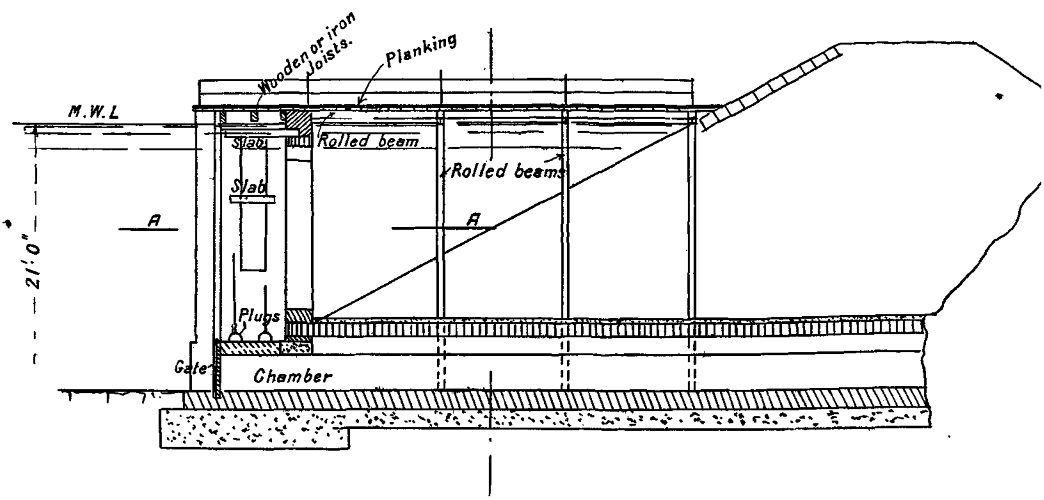
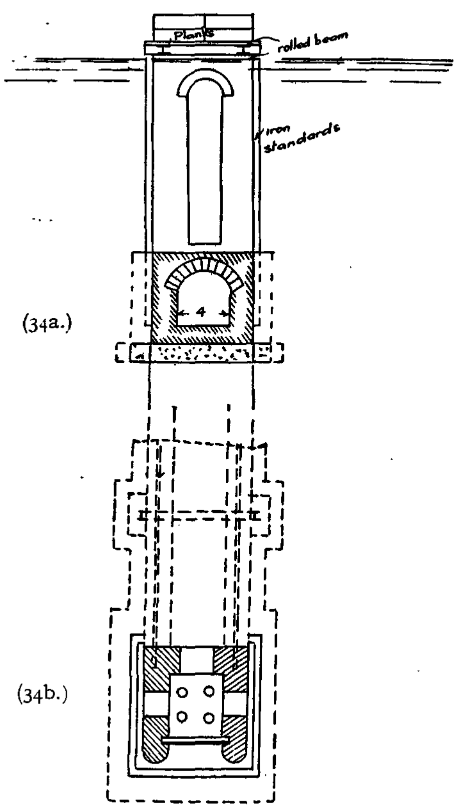


FIG. 34.



(34a.)

(34b.)

is apparently the limit. It is considered that the chamber should be at least 3 feet high to enable it to be inspected, except in the case of very small tanks.

The arrangements in design of the sluice in Fig. 32 show some differences from that in Fig. 31. The fore wing walls, as also the breast wall, are run right up to the crest of the embankment instead of stopping short at the platform level, which is undoubtedly a better arrangement. The amount of masonry saved in the former design is not much. The abutments shown in section *AA* have to be thicker at the top than the normal 2 feet, but are proportionately reduced at the base so as to keep the average width at a quarter the height, which is about the same ratio as in the trapezoidal profiles with vertical faces indicated in Table I., Chap. I., and exemplified in the section *BB*. The breast walls in both cases have a base width of about $\cdot 25H$, which is quite sufficient considering the extreme shortness of the unsupported length. The section of the wings in Fig. 32 have a fore and aft batter in accordance with the model profiles exhibited in Chap. I., whereas in the earlier design (Fig. 31) they have vertical stepped backs, an arrangement not so scientifically correct as when the base thickness at every point bears the same definite ratio to the height.

(49) To the critical observer it will be apparent that the amount of masonry involved in a high sluice head, as exemplified in Fig. 32, is very considerable. The only usual alternative is to run the culvert right on to near the fore toe of the embankment, and there connect it with a tower. This construction is, however, seldom adopted, except for much larger depths of water, as there would be no saving in cost. The author considers that for moderate depths, say up to 25 feet, a modification of the watertight tower fitted with sluice openings in the sides, as illustrated later, would be much more economical than the ordinarily adopted design of Figs. 31 and 32, and at the same time be equally efficient. A sketch of this is given in Figs. 34, 34a, and 34b.

The principle of this design is that the culvert is produced outwards to near the toe of the embankment. Outside is the closed chamber with draw gate and plugs, just as in Fig. 32. There being no earth to be upheld or water to be kept out, all that is required is an open staging erected over the chamber, from the top of which the draw gate and plugs can be worked, or, in the case of a sluice head, as in Figs. 30 and 31, for manipulating the draw gate. The depth of water is not such that a watertight tower is required at all, the object of which is to limit the pressure on the sluice gate or pipe valve, as the case may be, so that the structure, if of masonry, can be made very light, or simple open iron or wooden staging would answer equally well. The design for this masonry head, as shown in Fig. 34, consists of a U-shaped square well of 3 sides only, with openings in the walls, the back being towards the embankment. This well is covered in at top just above M.W.L. by a wooden planked platform, on which the screw gear heads and windlass will be placed to work the plugs and draw gate. The platform, which is isolated from the bank crest, is connected with it by a foot-bridge, supported on columns at intervals, which have separate foundations on either side abutting on the side walls of the culvert: the superstructure being of rolled joists and longitudinals covered by planking.

If necessary, the columns can be counterbraced transversely and longitudinally; the transverse bracing must, however, for obvious reasons, be above the fore slope of the embankment.

Excluding the concrete, the comparative quantity of masonry in the two designs of sluice heads of Figs. 32 and 34, including in the latter the lengthened portion of the culvert, are, roughly, as follows:—

Fig. 32 .	8,000 cubic feet.
,, 34 .	4,000 ,, ,,

Showing a saving of no less than 4,000 cubic feet effected by the alteration of the design. This comparative statement excludes all parts common to both.

From the value of the masonry saved must be deducted the cost of the planked foot-bridge, which, if built up of old rails, would be but trifling.

(**50**) The Maladevi Tank outlet (Figs. 35 and 36) is a good example of the design of an outlet through a very high embankment, the crest of which is 72 feet above sill. In such circumstances the usual plan followed would be

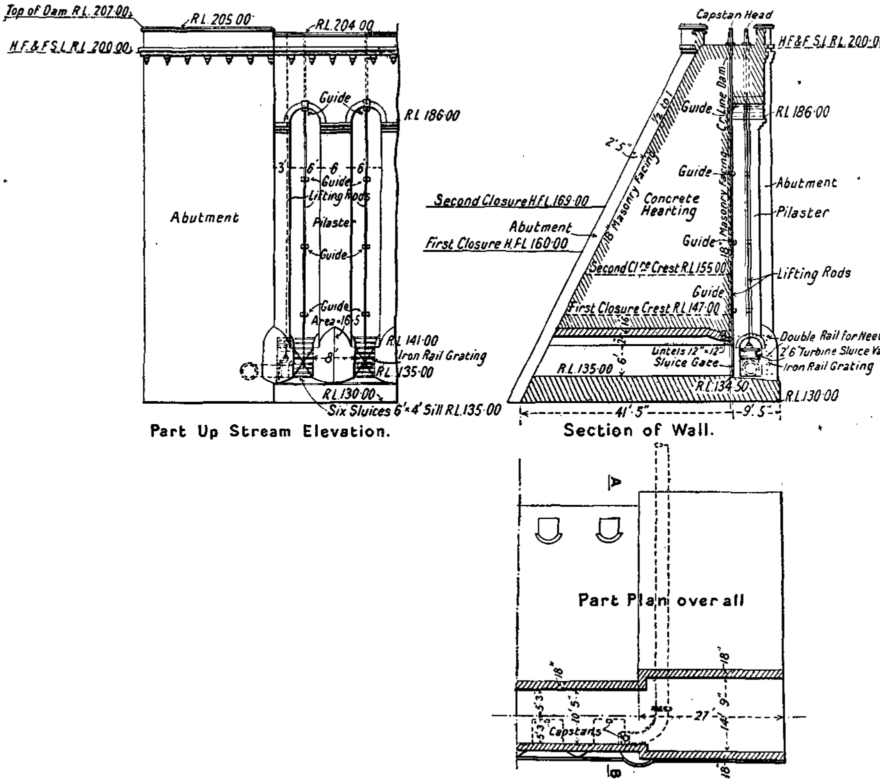


FIG. 36.—Maladevi Outlet Sluices (enlarged).

of pipes in a culvert or barrel drain with a valve tower—as shown in Fig. 38. In this case, however, a centre or breast wall has been adopted, but with wing cut short, the rest of the earth slopes being sloped down in a curve and pitched with stone. It is in fact a short length of masonry dam interposed in an earthen dam. The site was used as a temporary escape during the construction of the dam, and was consequently provided with a masonry floor. Further, the lower part of the breast wall was used as an overfall. These temporary works can be utilised in this style of outlet, which would not be the case if it consisted of a narrow culvert ending in a valve tower. If this were not so, this type of construction would prove more expensive than a pipe culvert with tower. The other works connected with this new project are given in Figs. 5, 5a, 5b, 6 and 6a of this chapter.

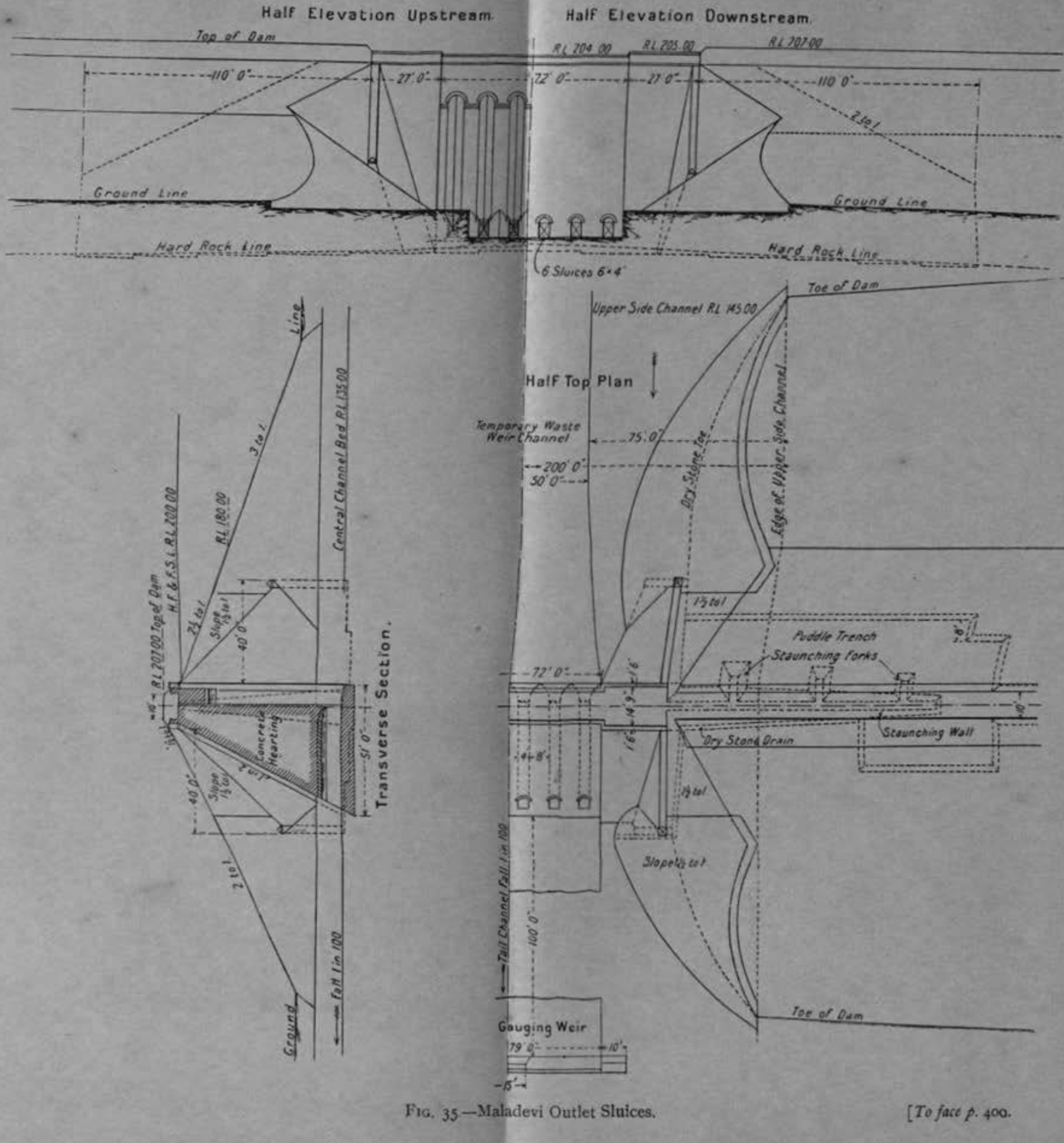


FIG. 35.—Maladevi Outlet Sluices.

[To face p. 400.

(**51**) The ordinary pattern of English waterworks towers is shown in Fig. 37, which is taken from "Waterworks Engineering." The position of the inlet valves to the tower is adapted to draw the supply from any one of three levels in the reservoir, as well as to scour or discharge the water from the bottom. The screen arrangement to ward off impurities would naturally not be required in an irrigation work. The dimensions of the wall of the tower, which is circular, are:—Top thickness, 2 feet, which increases $\frac{1}{2}$ foot for every 20 feet in depth below maximum surface level of the reservoir.

(**52**) In order to avoid all earth pressure on the tower, this latter is generally built well clear of the toe of the embankment. If, for economical reasons, it is deemed advisable to build it in the middle of the embankment, this can be done, if the section of the tower is increased to meet the stress thus liable to be brought upon it. The increase in thickness of the tower should naturally be on the far side from the embankment. This is well exemplified in Fig. 38, a section of the Hury Reservoir Embankment, for which we are likewise indebted to "Waterworks Engineering." As will be seen, the culvert is cut in the solid ground, a procedure which should be followed wherever possible.

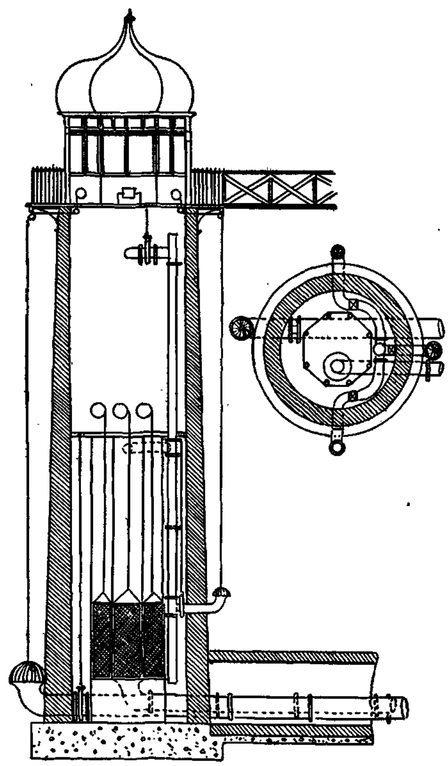


FIG. 37.

To allow for uneven settlement in the culvert due to the great pressure a slip joint is provided. This is shown in the section in question. The method of construction of the slip joint is fully explained in the text, to which the reader is referred.

The best section for a culvert to resist pressure is elliptical, with the shorter axis horizontal; the ratio is $\frac{hor.\ axis}{vert.\ axis} = \frac{2}{3}$, the lower third of the section truncated and closed with a flat invert.

The thickness of the arched wall of the culvert is obtained from the formula $t = b\sqrt{\frac{H}{2}}$, in which t = thickness required in inches; b = the smaller diameter of the culvert in feet; H = height of superincumbent earth in feet.

(**53**) The best method of carrying pipes in a culvert is not to lay them on

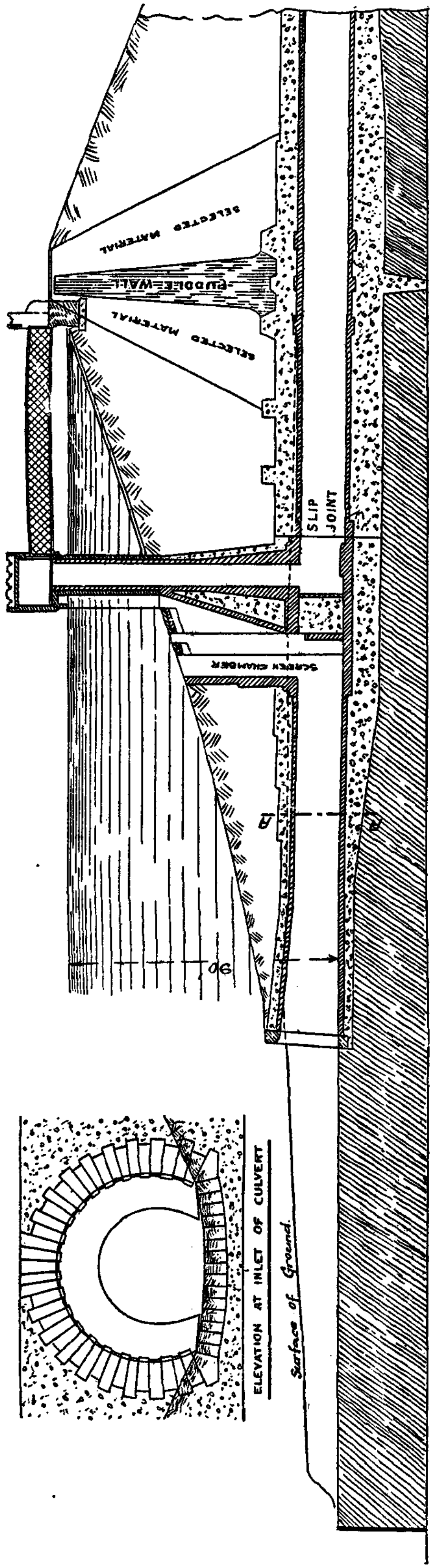


FIG. 38.—Hury Reservoir Valve Tower (England).

the floor, as shown in Fig. 37, but to suspend them from the roof; this enables inspection to be much more easily effected.

Fig. 39 is a section of the base of the Hubli Waterworks Tower, which shows the pipes hanging from cross **T** beams built into the sides of the tunnel.

(**54**) The subject of pipe outlets from small tanks will now be dealt with. These usually consist of earthenware pipes similar to the "Kolaba" pipes used in canal distributaries, with either spigot and faucet joints or else collar connections. In localities where earthenware pipes are difficult or expensive to obtain, excellent pipes can be made of fine cement concrete, which is stronger than earthenware, and can be moulded into any required shape. In the Ceylon Irrigation Department conical cement pipes are used. These are manufactured on the spot. While on a visit to Anurādhapura some fifteen years ago, the author inspected the manufacture of these pipes, details of which are given in Fig. 40.

As will be seen in the section of bank, the pipes are run up the fore slope of the embankment, and are removed and replaced by hand in accordance with the level of the water in the tank and the head required. This is a far superior arrangement to the ordinary pipe head at bed level, the closure of which is commonly effected by a man diving down under water and stuffing a handful of straw into the mouth of the outlet. Another arrangement for regulation is to erect a light wooden open staging over the outlet head. The conical pipes are then fixed vertically between three poles driven round them, and the cultivator standing on the cross-bars of the staging can remove or replace the pipes one by one as may be required, leaving the spare ones on the staging or on the adjacent bank.

When the height of the embankment is not great, corrugated or sheet iron piping could be used with advantage in place of a masonry culvert. The line of piping would have to be provided with large flanges at intervals to stop the creep of water, and should terminate at either end in a proper masonry fore-and-aft bay. It could be set in puddle or concrete. A chamber with plug holes or a flap valve over the pipe head should be added on the face; also, a wooden or iron staging carrying the lifting apparatus.

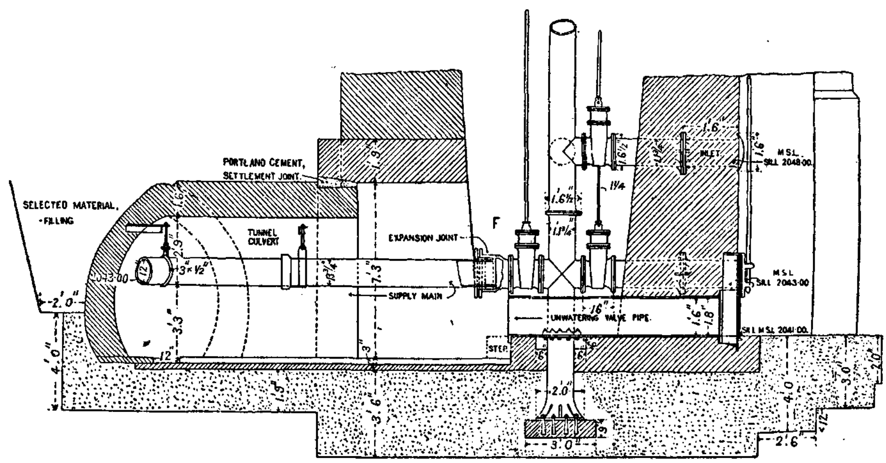


FIG. 39.—Part Section of Hubli Reservoir Tower (India).

(55) When a tank embankment dams the course of a large stream which in flood time has to be passed through the reservoir, the question of the possible prevention of silt deposit is a serious one. The river risen in flood pours water heavily charged with sediment into the reservoir, which sediment must nearly all deposit within it, thus gradually but surely diminishing its effective capacity. Scouring sluices, however powerful, will but palliate the evil, as their action is but local in its effect. In such cases the best remedy undoubtedly is to construct some works on the river before it reaches the area of the tank, whereby in heavy floods only the required proportion of the full flood discharge will be allowed to enter the tank, the rest being disposed of elsewhere into another drainage line. In cases when the whole discharge is required to fill the tank, the current can often, by means of a weir, be forced to spread over an area of flat country on either side, and made thus to deposit its silt on the land before it reaches the depression in which the reservoir is situated.

In a large reservoir a floating dredger will successfully keep down silt deposit. The possible prevention of silt deposit is a matter which should

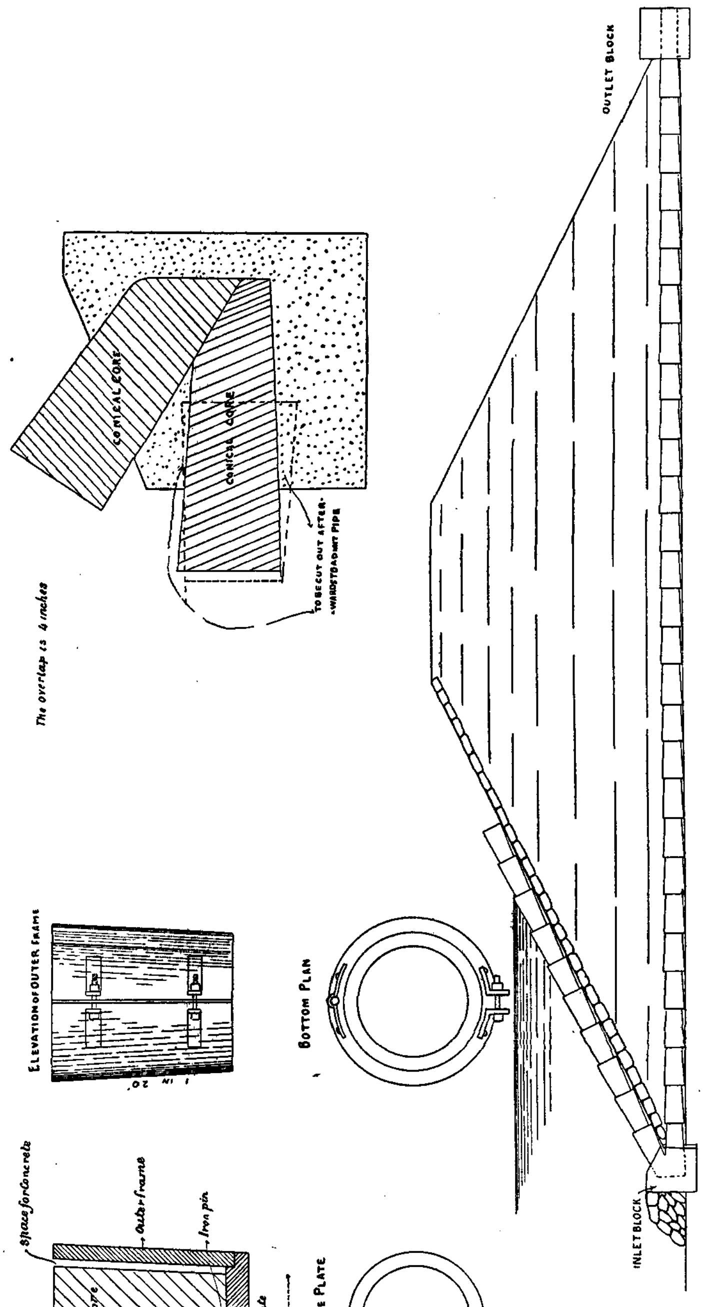
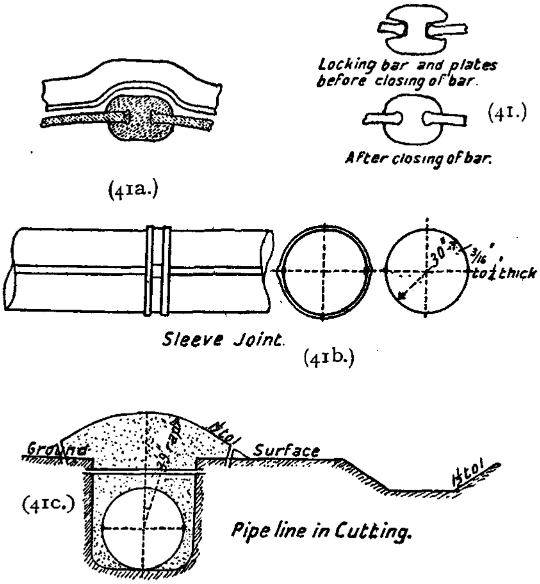


FIG. 40.—Ceylon—Concrete Outlet Pipes.

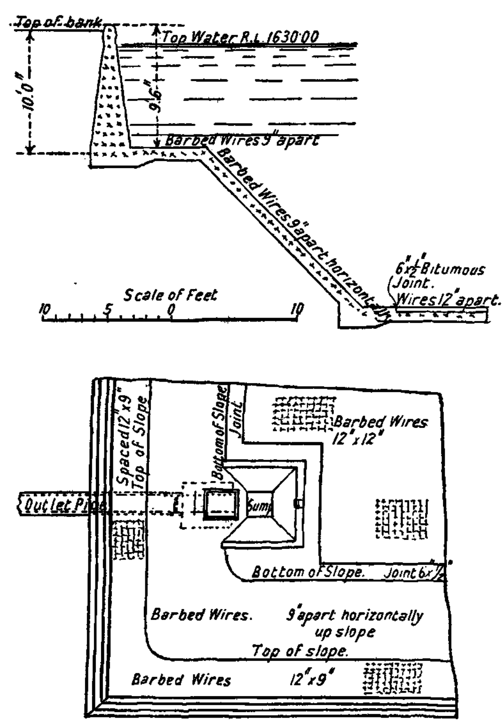
be carefully thought out on the initiation of any new reservoir project, as many old works have been rendered quite useless by silting up.

(**56**) An interesting paper on the Coolgardie Supply Reservoirs appeared in Vol. CLXII. of the "Minutes of the Proceedings of the Institute of Civil Engineers," from which the following information regarding a recent work, the largest ever constructed of its kind, is deemed of interest:—

To supply the mines at Coolgardie and Kalgoorlie, situated 350 miles from the West Coast, an immense reservoir was formed in the rainy belt near Fremantle, on the West Coast, having a maximum capacity of 4,600 million gallons. The surplus falls over a weir which is the highest ever constructed. Its section is given and discussed in Fig. 59, Chap. II. From this reservoir water has to be pumped, by eight pumping stations, the unprecedented distance of 360 miles.



FIGS. 41, 41a, 41b, 41c.—Coolgardie Pipe Line (Western Australia).



Cast-iron pipes being clearly out of the question, steel pipes 28 feet long and about 2·6 feet diameter were used. These are of the locking-bar pattern. To quote the paper: "The pipe consists of two plates, each the full length of the pipe and each bent to a semicircle. The edges are burred or beaded to the shape of a dovetail, and are inserted in the open jaws of heavy longitudinal bars, which are subsequently closed cold on the edges of the plates,

FIGS. 42, 42a.—Balla Bulling Reservoir.

thus forming longitudinal dovetail joints. The pipes when *in situ* were jointed by a simple sleeve joint, the ring being 8 inches wide and ½ inch larger diameter than the pipes bulging out to clear the lock bars."

Fig. 41 is a section of the locking bar, open and closed. In Fig. 41a is a section showing the sleeve joint enlarged, which is leaded, while Fig. 41b is an elevation and two sections of the joint and pipe. Fig. 41c is a section of the pipe way, showing the pipe as covered up by earth.

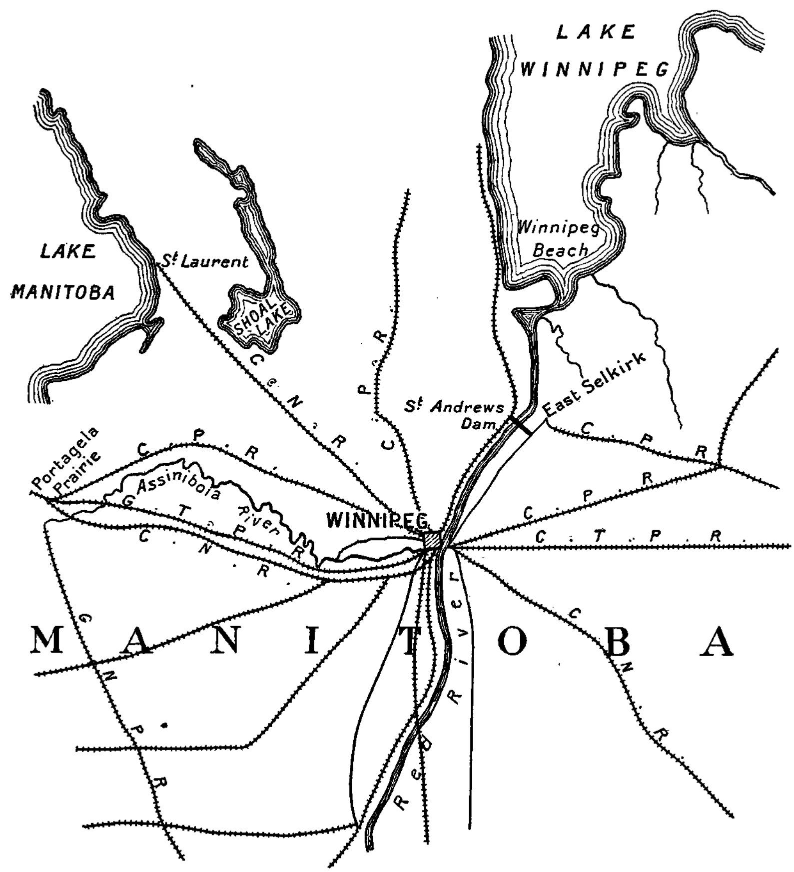


FIG. 43.—Location Map of St. Andrew's Rapid Dam.

(57) The largest of the service reservoirs, the one at Balla Bulling, is formed of concrete reinforced by barbed wire, and provided with bituminous joints right round near the toe of slope. This is to allow for expansion and contraction, as no concrete without such relief can stand the alternations of temperature in that climate without cracking.

In Figs. 42 and 42a we have a part section and plan of this reservoir.

The whole of this immense work cost £2,660,000.

(58) In Canada a notable dam has lately been erected over the Red River near Winnipeg, which, although strictly a navigation work, is still of interest

to Irrigation engineers as illustrative of a new type. The conditions of the Red River are peculiar; it rises south in the State of South Dakota, and consequently comes down in heavy flood in the spring at a time when Lake Winnipeg and the lower reaches of the river itself are still ice-bound. For this reason a movable dam, such as will offer no serious obstruction to the outflow, is a necessity. The object of the dam is to raise the water in the Red River to enable steamboats of large size to navigate in the summer months right up from Lake Winnipeg, which they are at present unable to do. To effect this the water level at St. Andrew's Rapids has to be raised about 20 feet, which will then increase the depth at Winnipeg City by 6 feet. The following figures are illustrative of the work:—Fig. 44 is a map showing the Red River and the location of the dam with regard to the City of Winnipeg and the lake of the same name. Fig. 45 is a section and elevation of the regulating bridge and movable dam, and Figs. 46 and 47 are photographs of the work, for all of which we are indebted to "Construction," an ably conducted Canadian professional magazine.

The dam is of the type known as the "Caméré" Curtain Dam, the closure being effected by a reticulated wooden curtain, which is rolled up and down a frame, thereby opening or closing the vent. It is a French invention, having been first constructed on the Seine. The principle of the movable dam consists in a large span girder bridge, from which vertical winged supports carrying the curtain frames are let drop on to a low weir. When not required for use these vertical girders are hauled up into a horizontal position below the girder bridge and fastened there. In fact the principle is very much that of a needle dam. The river is 800 feet wide, and the bridge is of six spans of 138 feet.

The bridge is composed of three trusses, two of which are free from internal cross-bracing, and carry tramlines with all the working apparatus of several sets of winches and hoists for manipulating the vertical girders and the curtain; the third truss is mainly to strengthen the bridge laterally, and to carry the hinged ends of the vertical girders.

It will be understood that the surface exposed to wind pressure is exceptionally great, so that the cross bracing is absolutely essential, as is also the lateral support afforded by a heavy projection of the pier itself above floor level.

In the cross section it will be seen that there is a foot-bridge opening in the pier. This foot-bridge will carry winches for winding and unwinding the curtains, and is formed by projections thrown out at the rear of each group of frames. It will afford through communication by a tramway. The curtains can be detached altogether from the frames and housed in a chamber in the pier clear of the flood-line.

The lower part of the works consists in a submerged weir of solid construction which runs right across the river; its crest is 7 feet 6 inches above L.W.L., at R.L. 689·50. The top of the curtains to which water is upheld is at R.L. 703·6 or 14 feet higher. The dam actually holds up 31 feet of water above bed of river.

(59) The Caméré curtain has given satisfaction in France, and may possibly do so here. Certain grave disadvantages are, however, inherent in this system. Firstly, the immense surface exposed to wind pressure, which must always be a continual menace to its safety; secondly, the expense of the

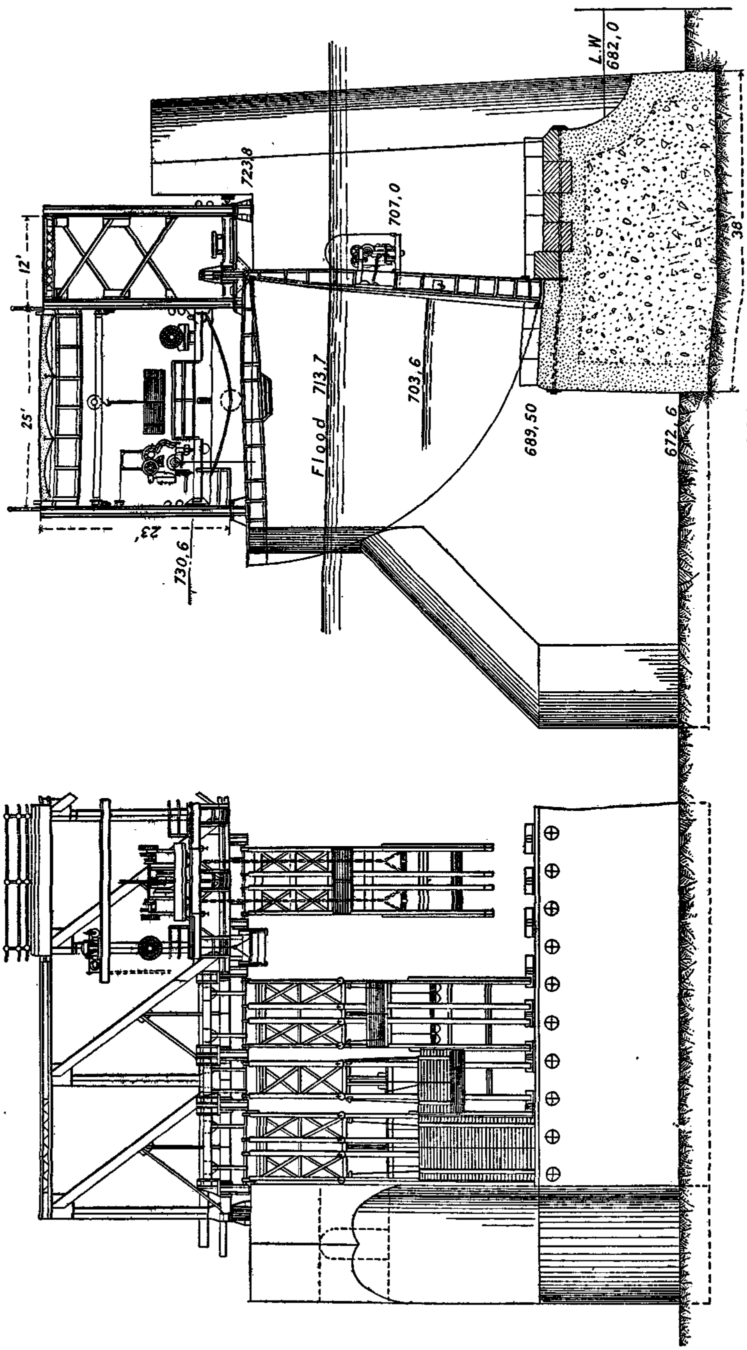


FIG. 44.—"Caméré" Curtain Dam at St. Andrew's Rapid, Red River, Man.

work, which must be enormous. It is believed that a very much cheaper and more effective design could be made by a combination of the principle of the Dhukwa Weir (Fig. 28) with that of the Folsam Weir (Fig. 34, Chap. II.). By this means the immensely expensive overbridge is not required at all,

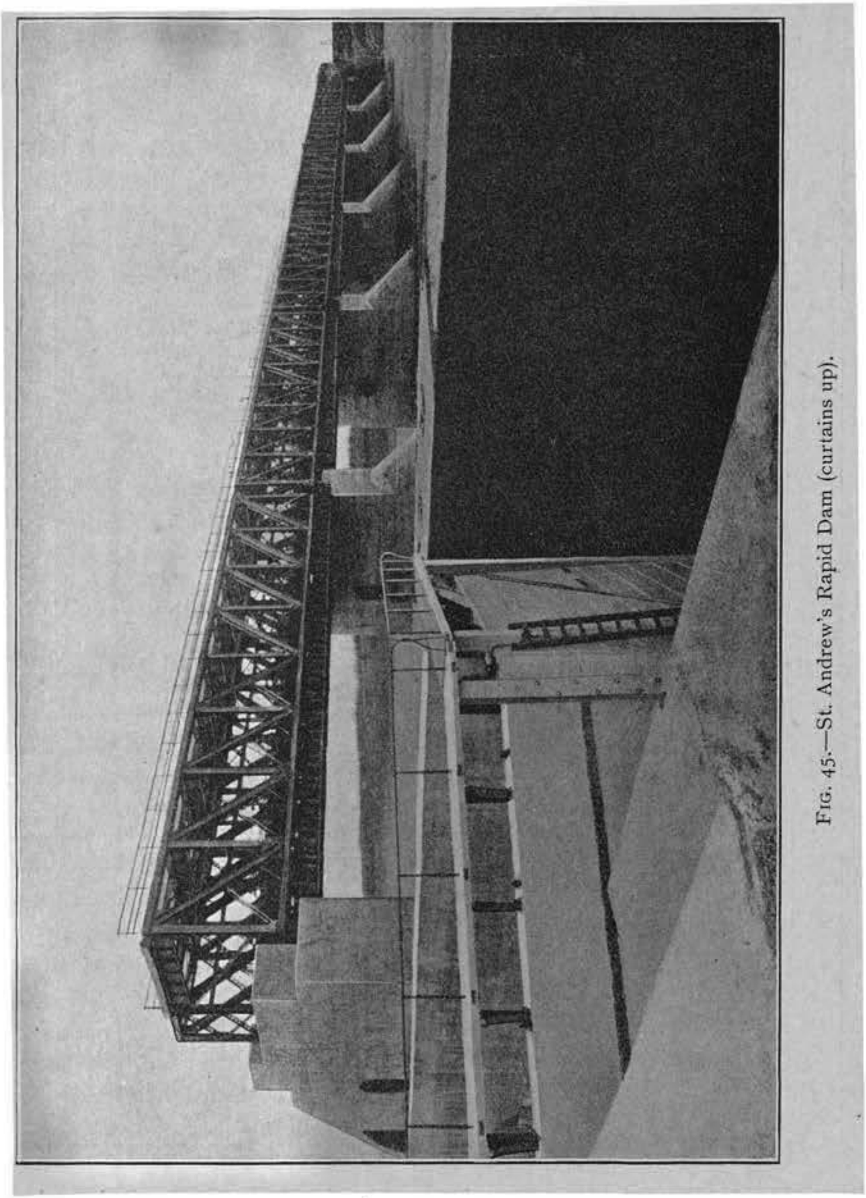


FIG. 45.—St. Andrew's Rapid Dam (curtains up).

the dam consisting merely of the submerged weir as already built, but pierced with a subway having at intervals lateral chambers containing hydraulic rams or jacks. Above the weir are collapsible gates, hinged at the base, which are raised by the piston-rods of the hydraulic cylinders pushing against the skin of the gate. These push rods will be strengthened with a cross-head and guide rods, and be mounted with a rolling wheel at their extremities, which wheel will revolve in a confined short channel provided against the gate, turning with it as it is lifted. This rigidity of the piston-

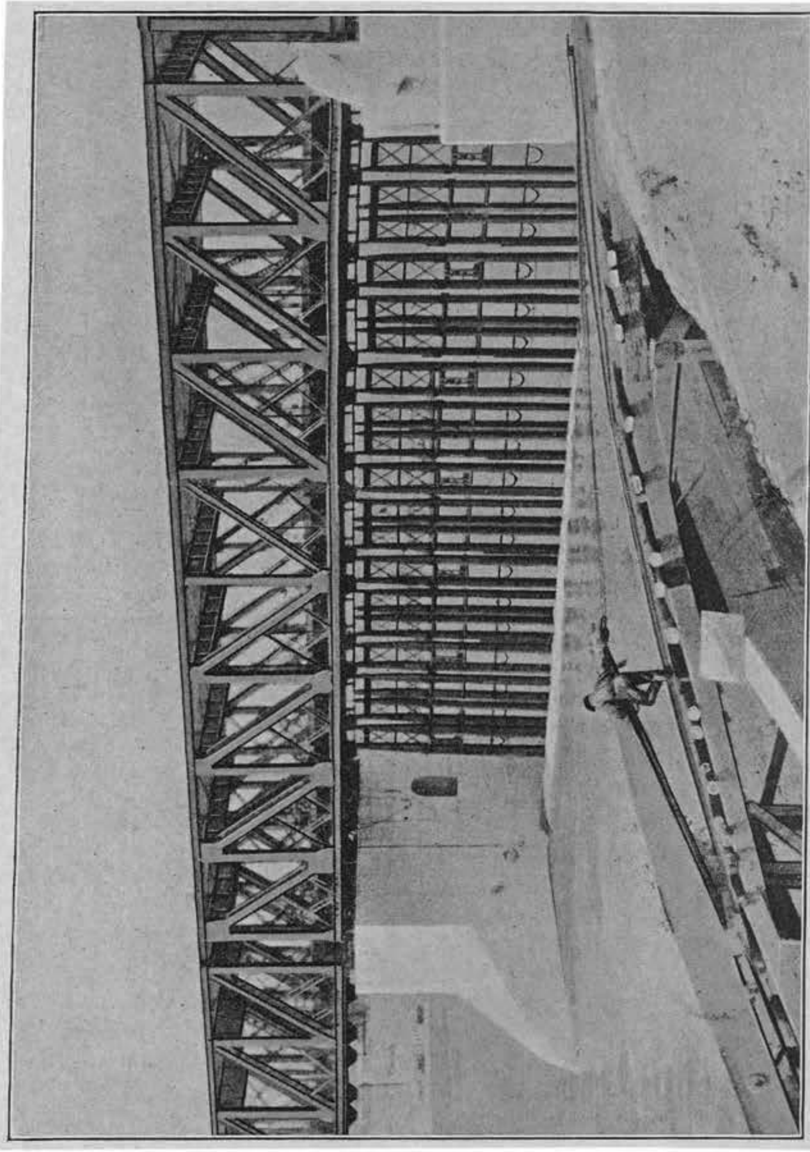


Fig. 46.—St. Andrew's Rapid Dam (curtains down).

rod enables a watertight stuffing box to be inserted at the point where it protrudes out of the chamber.

The subway and chambers will be always under water, and consequently must be rendered watertight. They will further require sump holes, side drains, and a pump in case of leakage. At about 250 feet intervals hollow piers will be erected, containing shafts for ventilation and communication with the subway. The hydraulic jacks can be worked and regulated from a power house above ground, as was done with the Folsom Dam. By turning levers in the power house it should be possible to raise or lower all the gates at once wholly or partially, or operate some only, thus affording complete control over the regulation by one man. The gates could be made of any length, say 100 feet, close jointed with stanching strips and with frequent pivot connections with the crest of the weir. When the gates are down they

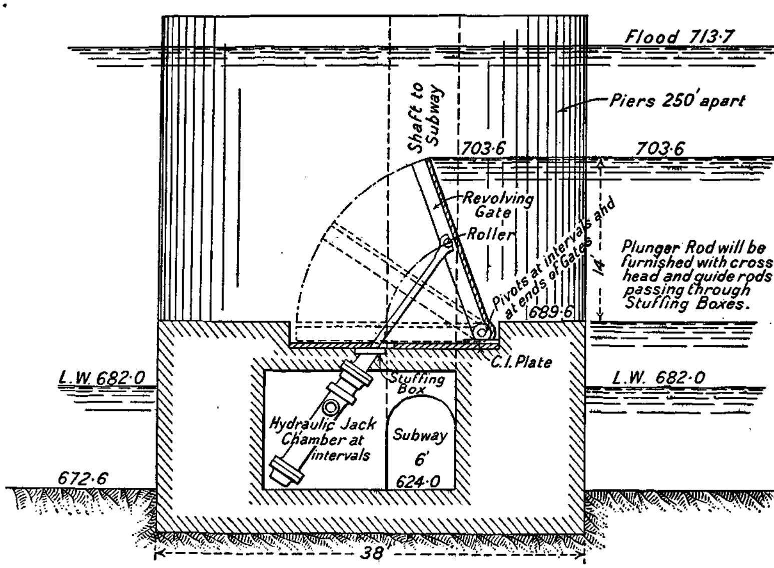


FIG. 47.—St. Andrew's Rapid Dam. Alternative Design, No. 1.

are housed in a recess below the crest of the weir out of harm's way. The rough sketch in Fig. 47 is illustrative of the above description.

(60) Another alternative method which is probably better than that just described would be to adopt the Dhukwa system of sliding hinged props. These could be pushed up or lowered by hydraulic jacks laid horizontally in a recess level with that of the gate itself. The hydraulic jacks would be situated outside not inside the subway or its chambers, and consequently no arrangements would be required to prevent leakage from above.

Both schemes are feasible and would cost very much less than the existing expensive installation.

CHAPTER XIII

DESIGN OF CHANNELS

(1) THE method pursued in designing a canal, irrespective of its masonry works, can only be treated in a general manner. In order to give actual examples of any value for instructive purposes, maps of an irrigation project would be required with a host of statistics, such as the levels of the country affected, the amount of water available at low and high supply in the river, the irrigable area, besides the cost of the headworks necessary to give the required depth of water in the canal, and many other matters. Such information belonging to an actual project not being available, it would be very difficult to produce imaginary conditions whereby an example could be worked out in detail.

(2) The first point to be decided on is the water available in the river of supply; with this information the designer will be able to form an opinion of the maximum and minimum discharge of the canal—regard being also had to the irrigable area commanded, and to financial considerations.

These being settled satisfactorily, the depth of water in the canal is one of the first points to be decided on, as also the sill level of the intake with regard to the weir crest and undersluice floor.

The limiting velocity with full supply will then determine the bed grade of the canal to carry its discharge with the fixed depth, and this slope can be obtained by use of tables.

The bed width of the canal and its side slopes, *i.e.*, the sectional area of the water-way, can be simultaneously worked out.

Main canals have to run for many miles from their source of supply without any irrigation branch taking off: how far, depends on the slope of the country. As soon as the bed slope of the canal begins sensibly to gain on that of the country, a fall will be required to keep the canal in cutting, and at such sites branches generally take off. As the canal proceeds, allowance must be made by reduction of the base width in each mile for absorption and evaporation, and the same applies to branches and distributaries.

(3) We have seen in Chap. VI. that the question of the quantity and quality of the silt and sand carried in suspension by the water entering the canal forms one of the principal points to be considered in designing an irrigation channel. In most rivers the silt is of a fertilising character, and consequently it is advantageous to adopt means whereby it can be conveyed right on to the fields irrigated, instead of being deposited in the channels near the head.

The inclination given to the bed should be such that a uniform velocity is induced right through the system, as any sudden changes or checks in the velocity of the current will immediately cause deposit. The matters in suspension are often composed in part of heavy sand and in part of light fertilising silt. The heavy sand should, if possible, be kept out of the canal. The only partially effective method to bring this about is to keep the lower gates of a head regulator closed during floods, and to admit water only at the top. This will exclude the heavy sand which is pushed along the bed of a river at every freshet. This precaution will, however, but mitigate the evil, which can only be satisfactorily dealt with by constructing a scouring escape head a short distance below the canal head. When more water is available than is required in the canal, the escape can be partially opened, or when the canal is closed during heavy rainfall, the cross regulator below the escape can be closed and water admitted at the head when it is fairly clear, which will pick up and scour out the deposit on the bed.

(4) The velocity in a main canal should be kept below that which would cause erosion in the bed and banks; the neglect of this important point has led to immense damage and loss, necessitating the eventual regrading of a canal in California, where the first canals were not designed on scientific principles. The following are the maximum velocities allowable :—

In light sandy soil . . .	$1\frac{1}{2}$ to 2 feet per second.
,, sandy loam . . .	$2\frac{1}{2}$,, ,, ,,
,, ordinary firm loam . .	3 ,, ,, ,,
,, stiff clay or gravelly soil .	4 ,, ,, ,,
,, pitched bed and banks, shingle and boulders .	5 to 6 ,, ,, ,,

In most Indian canals the maximum velocity does not exceed $2\frac{1}{2}$ or $2\frac{3}{4}$ feet per second.

The erosive power of water varies with the volume carried and with the amount of silt in suspension. For small volumes, as in distributaries, particularly if silt is carried, the safe velocity can be placed at a higher figure than in main channels of large capacity; thus, if $2\frac{1}{2}$ feet is the maximum allowable in the latter case, an increased velocity of 3 cubic feet per second would probably not be excessive in a narrow distributary, as the scouring capacity of a water section increases with its volume. The maximum velocity adopted should be as high as possible, in order to allow a fair velocity when a lower supply is running, otherwise the channels will become choked with water weeds.

(5) The discharge of a canal or distributary is based on the area irrigable and on the *duty of water*, *i.e.*, the acreage matured by 1 cubic foot per second flowing continuously for a defined time. The duty varies greatly, owing to the losses due to percolation and absorption and to variations in the rainfall in different years on the same canal.

In Upper India there are two distinct seasons for irrigation when different crops are raised. Firstly, the Kharif, that is, the summer season, when tropical crops, such as rice, indigo, cane, cotton, etc., and secondly, the Rabi or winter season, when crops common to Europe, such as wheat, barley, peas, etc., are raised.

The duty of the Kharif irrigation varies roughly from 50 to 120 acres, and of the Rabi season from 50 to 150 or even 250 acres. The loss of water between the head of a canal and the fields varies from 10 to 50 per cent.

To find the volume of water utilised, the number of days during which the crop is matured is required to be known. This is termed the "Base" of the duty. The volume utilised will then be found by the following formula :—

$V = \frac{B}{D} \times 86{,}400''$, V being in cubic feet (86,400 being the number of seconds in a day of twenty-four hours). Thus, supposing the duty D to be 60 acres, the base B 120 days, then $V = \frac{120}{60} \times 86{,}400 = 172{,}800$ cubic feet of water per acre. The depth in feet will be $\frac{V}{43{,}560}$, or $\frac{B}{D} \times 1{\cdot}51$, or in this case $\frac{120}{60} \times 1{\cdot}51 = 3{\cdot}2$ feet.

This depth can be conveniently termed acre-feet, or symbolically AF, and, being in smaller numbers, appeals to the eye better than the volume given in cubic feet per acre.

(6) The use of acres 1 foot deep to represent the volume of water used or stored is most convenient in the case of reservoir or tank storage, the cubic contents of which should be expressed in acre-feet instead of millions of cubic feet. The capacity will then bear a definite ratio to the area of irrigation to be effected. Thus if rice is the crop to be irrigated from a tank which requires 60 inches or 5 feet depth of water to bring to maturity, and the effective contents of the reservoir is 10,000 acre-feet, then the irrigable area will be $\frac{10{,}000}{5} = 2{,}000$ acres.

(7) As already noted, canals and distributaries should diminish gradually in bed width to allow for the loss due to percolation and absorption, in addition to the reduction in the required water-way due to the taking off of branches or distributaries in the first case, or to that of irrigation channels in the latter case. Thus, supposing a canal is estimated to lose 20 per cent. of its volume in a run of 100 miles; then, neglecting any loss from water drawn off into branches or distributaries, and assuming the maximum discharge at 3,000 cubic feet per second at the head, the water-way at 100 miles distant should be calculated for a discharge of $3{,}000\,(1 - \frac{1}{5}) = 2{,}400$ cubic feet; at 50 miles, $3{,}000\,(1 - \frac{1}{10}) = 2{,}700$ cubic feet; and at 25 miles, $3{,}000\,(1 - \frac{1}{20}) = 2{,}850$ cubic feet. With the same bed slope the reduction in water-way can be best effected by a corresponding gradual reduction of the bed width.

(8) There is no general rule applicable to every case for reduction in discharge due to absorption; each special case must be considered separately.

The author consulted a late Inspector-General of Irrigation (India) on the procedure adopted by him in designing the distributaries of the Punjab canals, and the following are his remarks :—

"There is, as far as I know, no general rule for the deduction of discharge in a channel due to absorption and evaporation, nor, indeed, can there be one. Quite apart from the wetted perimeter come the questions of slope of bed and, above all, the nature of the bed. There are lengths on the Bari Doab Canal, for instance, where little water is lost; again, there are lengths which are like a sieve. The designer must know his ground and apply such lessons of experience as he may have learnt. In any case, I never reduced my sections from above downwards, but always calculated them from the tail end, working upwards, *i.e.*, I *added* from the tail instead of subtracting from the head discharge. My method was as follows :—The area commanded by each distributary was first worked out, and an allowance of 2 cusecs at the distributary head to cover everything was then made for each square mile of commanded area. This gave the distributary discharge, and gives satisfaction in practice. On the branches of the Jhelum Canal I allowed ½ cusec per lineal mile for evaporation and absorption, and 2 cusecs per mile of the main canal. On the Jhelum this latter allowance meant about 1 cusec per 100 feet of wetted perimeter per mile of length, and the allowance has sufficed. In a channel, though, that carries 3,000 cusecs, as in the Jhelum, or, say, 10,000, as on the Chenab, these allowances are hardly worth consideration, and I do not suppose it would matter much whether they were given or not. On distributaries, the allowance of 2 cusecs per square mile amply covers everything in the Punjab. It would, of course, be different in a rice-growing country."

(9) According to Mr. Buckley, in the Punjab where the rainfall is low and the atmosphere dry, the loss by absorption, allowed for in the new canals, is 8 second-feet per million square feet of wetted perimeter. This has the merit of being a scientific method of estimating the loss sustained. Large canal systems have been designed on this principle.

The following table, from "The Irrigation Works of India," is most interesting, as showing the enormous losses from this item :—

Canal.	Area of Cold Weather Irrigation.	Mileage of Main Lands and Branches.	Wetted Area.	Loss.
	Acres.	Miles.	Millions of square feet.	Second-feet.
Upper Bari Doab . .	442,300	353	82	820
Lower Chenab . .	1,155,700	426	104	1,632
Lower Jhelum . .	383,100	180	78	624
Upper Jhelum . .	172,500	136	83	664
Upper Chenab . .	324,200	194	145	1,161
Lower Bari Doab . .	441,300	196	121	965

Of these canals, the first three are existing old canals, the last are under construction. (See map, Fig. 9a, Chap. XI., page 334.)

The losses thus estimated are only in the main and branch canals, which in India do no direct irrigation, but act solely as supply channels to the distributaries.

A canal system designed without due allowance being provided for losses by absorption and evaporation will be found very faulty, and will require entire remodelling as soon as the irrigation is fully developed. From 50 to 75 per cent. only of water admitted at the head of a canal actually reaches the fields.

(10) To ensure the economical distribution of water, the system of rotation, or, as termed in India, "tatils," has to be enforced. That is, that either the whole distributary is closed for a certain fixed number of days in each week, running full supply the rest of the time, or, if the distributary is kept continuously running at the head, certain lengths of it are closed in rotation. This system enables water to be carried down to the end of a distributary, and gives to all cultivators a fair share of the water.

In designing new canals this system of rotation should first be roughly worked out, as the design of the water-way is considerably affected thereby. Thus, if the full head supply has to be passed down a closured length, it is evident that the sole diminution in water-way will be that due to absorption, and the full, or nearly full water-way will have to be maintained right through this length. The same will apply to the main canal, if distributary heads are intended to be periodically closed. The length of closure usually adopted in Upper India is five days in fifteen.

(11) In designing canals from intermittent streams, the discharge will have to be considerably greater than for a continuous supply.

The following example will explain the method of calculating the discharging capacity of a channel under assumed conditions: Area to be supplied, 2,000 acres; duty of water, 60 acres; then the normal discharge of the channel should be $\frac{A}{D} = \frac{2,000}{60} = 33$ second-feet.

But, assuming that the supply falls short in one month, the river only supplying 33 second-feet during eighteen days, and during the remaining twelve days it can supply an average of 6 second-feet, and let x be the required discharge, then $18x = (30 \times 33) - (12 \times 6)$ $\therefore$ $x = 51$-second feet.

Again, supposing the river can supply the full discharge for ten days only in the thirty, the average supply of the remaining twenty days being one quarter of the whole, or 8 second-feet, then $10x = (30 \times 33) - (20 \times 8)$.

$\therefore$ $x = \frac{750}{10} = 75$ second-feet.

With regard to storage in fields, this must be limited to cases where the freshets occur at intervals of not more than a week, and in which the quantity equivalent to a ten days' supply can be distibuted over between six and seven days, while a discharge of $1\frac{1}{2}$ times the normal will suffice for the channel ("Madras Irrigation Manual"). For any greater interval between freshets tank storage will be needed.

(12) In India the whole subject of the periodical closure of distributaries, or sections of distributaries, and irrigated areas is worked out in advance in a complete manner with great exactitude, so that the system is ready for immediate application as soon as the canals are opened. Very great stress is necessarily laid upon the scientific distribution of water, and from a long course of many years practical experience this science has been brought to a very high degree of perfection. The financial prospects of a canal system may be said to depend more on the proper distribution of the water available by means of closure tables than on anything else. In India the water rate, which forms the direct revenue of a canal, is charged on the area irrigated, not on the amount of water used; this apparently unscientific system is made to work well and to produce a high duty solely by the application of rules of intermittent supply carefully framed to suit each particular case. The measurement of water itself by modules or weirs, as is practised in Italy and in the States, would be impossible in India, and, in any case, must involve great difficulty in practical application. In the case of the Calgary Canal, to be referred to later, water will eventually have to be measured out to some 6,000 customers, each legally entitled to a fixed amount; this is an appalling prospect from the point of view of a canal establishment.

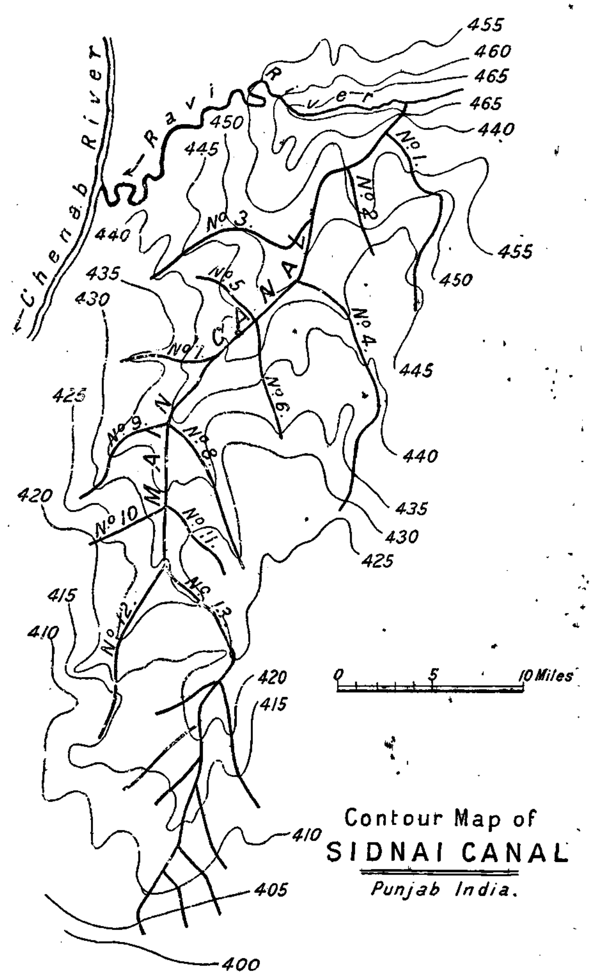


FIG. 1.

These matters of losses from absorption, and of the closure system adopted, are fully set out in Mr. Buckley's monumental work, "The Irrigation Works of India," and in Sir Hanbury Brown's "Irrigation," which latter valuable work deals with Egypt as well as India.

(13) Two examples will be given illustrative of the alignment of a canal and its branches.

Fig. 1 is a contour map of the Sidnai Canal in the Punjab. This is typical of a good disposition in a flat country. Distributaries, as a rule, should follow the lines of watershed, and thus be free from interference with cross drainage, and be enabled to irrigate on both sides. This ideal has been attained in several of the distributaries shown. The general fall of the country is a little over a foot per mile. The Sidnai is a small canal, discharging 2,400 second-feet and commanding 200,000 acres; it takes out of the Ravi River.

(14) The second example is that of the new canal now being developed in Western Canada by the Canadian Pacific Railway Company, for the irrigation of a large tract in the dry zone east of the Rockies. Reference has already been made to this very important and interesting work in Chap. IX., where three of the falls are illustrated and described, also in Chap. X.

The conditions here are wholly different to those prevailing in the "doabs" of upper India. The surface is undulating, and the slopes are steep. Nearly all the canals and distributaries are consequently in side cutting, and are only able to irrigate on one side. If a high ridge occurs, two distributaries have to compass it, one on each side, often not far apart. The heavy slopes add greatly to the expenses of the distribution of water, necessitating frequent small falls in the irrigating ditches, and as the winding contour of the ground has to be hugged, the length of the water-way is very much increased above what it would be if a straight alignment were possible; this not only adds to the expense of excavation but must considerably increase the losses of water due to absorption.

(15) The land grant belonging to the Canadian Pacific Railway, as shown in Fig. 2, consists of about 3 million acres. Of this area it is proposed to actually irrigate one-half, or $1\frac{1}{2}$ million acres. This huge block is divided into three main sections, the western, central, and eastern, of which the western, consisting of 350,000 acres of irrigable land, has already been brought partly under irrigation. The conditions of land tenure are peculiar. The company sells blocks of land to intending settlers at 25 dollars, or £5, an acre, a low enough rate, as it includes a water supply right. According to the irrigation laws promulgated in the province of Alberta, land sold as "irrigated" is legally entitled to a water supply of 1 second-foot for every 150 acres, and the company selling the land as "irrigated" enter into bond to supply this amount of water to the settler. The water supply thus stipulated is mis-termed the "Duty" of water. The company undertakes to deliver this *net* at a certain commanding point on each quarter section of 160 acres. The supply is therefore "net," not "gross," or even "utilised," *i.e.*, at distributary heads.

In the company's pamphlet it is assumed that a discharge of 2,000 second-feet, which is that of the present canal, will irrigate 300,000 acres of the western section, whereas under the conditions stated, allowing for inevitable losses in transit due to absorption, the discharge at the head will have to be

3,000 to 3,500 second-feet to effect the aforementioned area of irrigation, and to compass that of the whole project will necessitate a supply from the Bow River of 8,000 to 10,000 second-feet, if not more.

At present the land is not thickly settled, but when it becomes so, as is inevitable from the very favourable conditions of soil, climate, and price of land, then the difficulty of supplying fixed quantities of water will begin to be felt, and will present a problem that will tax to the utmost the organising powers of the present able irrigation establishment. In designing the channels no allowance was made for losses in transit, nor for intermittent supply or closure. All these matters, therefore, remain to be worked out, and will necessitate considerable eventual remodelling. It was probably in view of some such contingency that the works constructed are all of a temporary nature, the policy of the company being to get the water through and some start made. It may safely be stated that no Indian administration would ever have undertaken such legal responsibilities as have been imposed on this irrigation company. This obligation, unless considerably modified and relaxed, must eventually result in troublesome litigation, in which the settler has all the advantage.

(**16**) The Bow River has a minimum discharge of 6,000 second-feet, of which it is believed 4,000 have been granted to the company by the Government of Alberta. As we have seen, this will be inadequate even for two sections; consequently the whole low water supply of the river will eventually be required, and any further supply can only be obtained by storage through the formation of reservoirs in some large depressions, which can be filled by the canal during the winter months, and drawn upon in the irrigating season. Fortunately, the configuration of the ground lends itself to this arrangement.

The western canal head, or intake from the Bow River, is favourably situated clear of the high bluffs that fringe the river higher up, about a mile or more below the city of Calgary (see photo, page 265, Chap. VIII.). This location suffers, however, from one very serious defect which could only be remedied at considerable expense, and that is its position relative to the city. The water of the Bow river is here polluted by the sewage of a large town, which pollution, unless remedial measures are taken, will become wo se every year.

The result is that the canal water is not potable; and further, the numerous long reaches of natural water-course through which the canal water passes must likewise be similarly affected. This is a matter of public importance which the Government of the Province has apparently overlooked or disregarded.

(**17**) The full depth of the main canal is 10 feet, which it retains for its whole length of some 17 miles. The bed slope and width have three variations beginning with a 60-foot bed width, ending at 44 feet. This proportion of depth to bed width is unusually high; for a discharge of 2,000 second-feet a depth of 8 feet is the ordinarily accepted ratio. It will necessitate a weir some 6 or 8 feet in height across the Bow River to bring in full supply, which, however, has not yet been required. With a reduced depth of 8 feet in the canal a weir would still be a necessity, and the canal would besides have to be widened throughout, so that the matter of the most suitable depth is one of comparative cost, as some of the cutting in the main canal is of a heavy description.

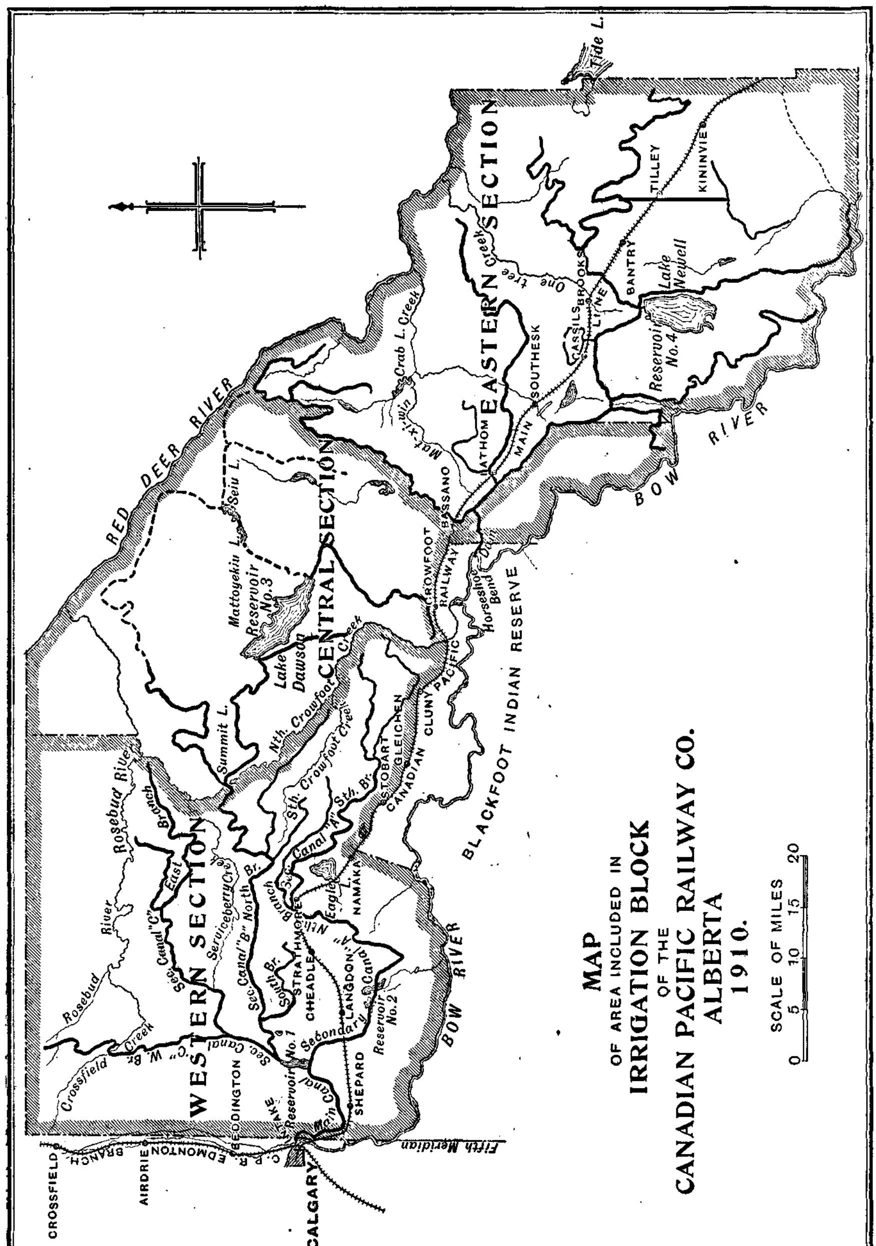


FIG. 2.

The main canal terminates at Reservoir No. 1, from which the so-called "Secondary" canals *A*, *B* and *C* take out. This reservoir is a long natural depression, which formation is a common feature in this undulating prairie country, and which has been fully taken advantage of in the alignment of the canal. The branches B and C are shown on part of their course on the map (Fig. 2) as dotted lines. This means that long natural depressions are made

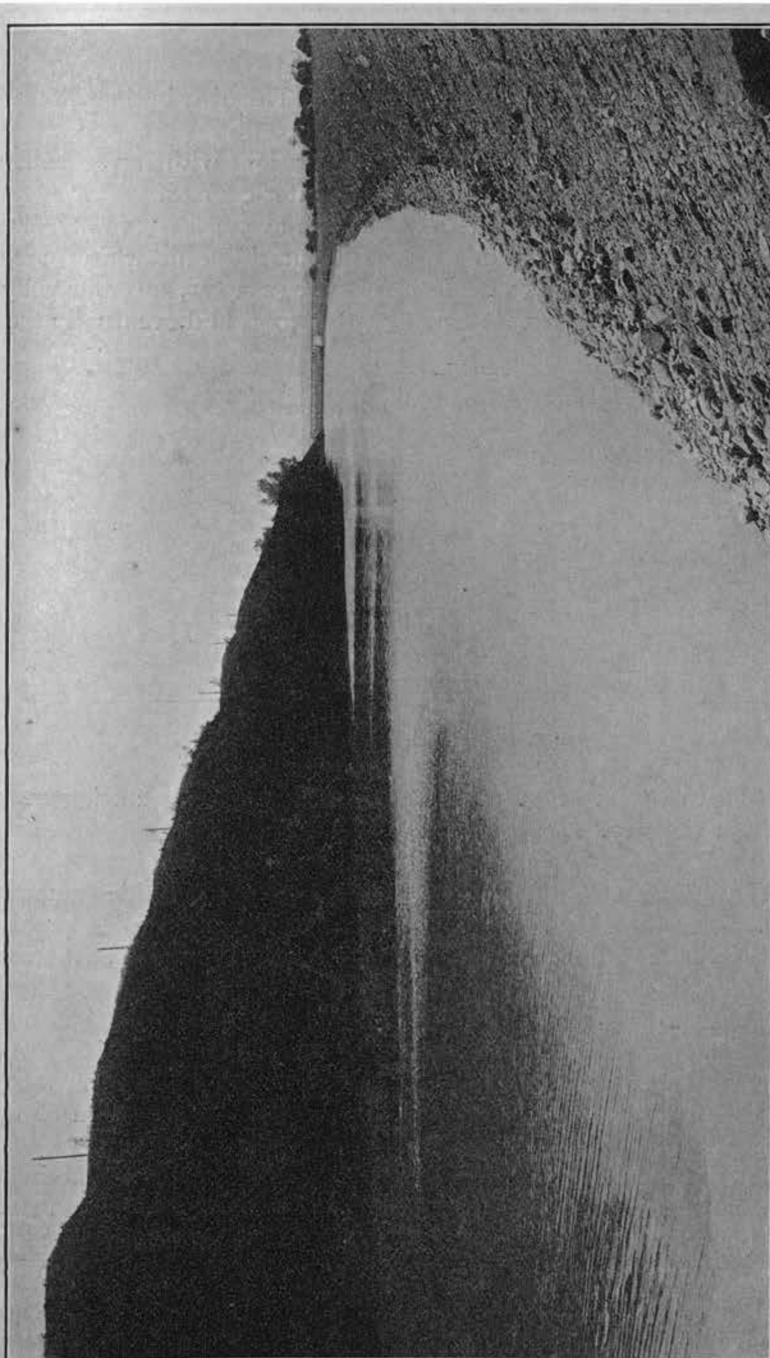


FIG. 3.—Side Cutting, first 2 miles Calgary Canal. Looking down stream.

use of. Some of these which are very deep can be embanked and used as reservoirs to be drawn upon as occasion demands, and they are of great advantage for watering stock. Reservoir No. 1 is 40 feet deep in places; only a few feet at the surface can, however, in this case be drawn upon as storage.

The secondary canals are of about 30 feet bed width and 8 feet deep at their heads, and are altogether 150 miles long. The distributing ditches will total 800 miles.

The secondary canals correspond more or less with large Indian so-called distributaries, the ditches with village water-courses.

The allowed mean velocity in the branches averages 2·3 to 2·7 feet per second, the soil being very friable this is about the limit which could be adopted without erosion. The side slopes are 2 to 1, an unusually flat inclination. Most Indian canals are dug out 1 to 1, and eventually the side

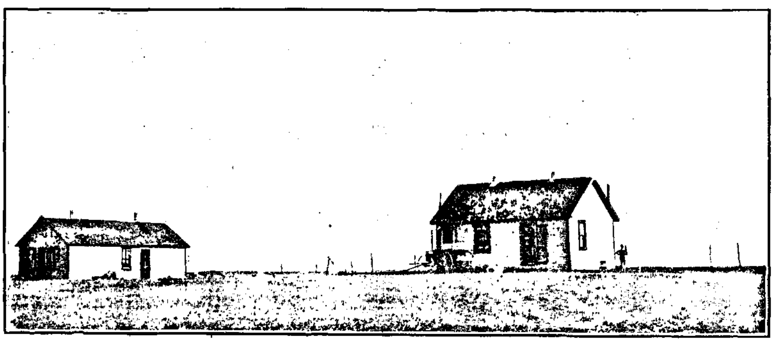


FIG. 4.—A Canal "Choki" on the prairies.

slopes silt up to ½ to 1. Fig. 3 is a view of the heavy side cutting near the canal head at Calgary.

(18) The western section already effects a considerable amount of irrigation. The soil differs from the black soil of the so-called prehistoric Lake Agassiz, which covers the greater parts of the provinces of Saskatchewan and Manitoba, in colour and texture. It is lighter, and has not the great depth peculiar to the more eastern provinces, the soil in which is nothing more or less than a variety of the familiar "black cotton soil" of India, sticky when wet, hard and split up when dry. The Alberta soil is, however, very fertile, and only needs water to raise the most astonishing outturn of wheat, sugar, beet and other valuable crops.

The canal extension in the eastern section has lately been put in hand. This portion will be supplied by a new intake from the Bow River, situated at Horse Shoe Bend, near the railway station of Bassano.

The river will be dammed by a masonry or concrete weir 40 feet in height, which is believed to be under construction. The settlement of this section is now being proceeded with. This reclamation closely resembles that so successfully brought about in the Lower Chenab Canal in India. The central section will be supplied from the existing Calgary intake, assisted by storage in natural reservoirs, the principal of which is the picturesquely termed "Dead Horse Lake," recently vulgarised into "Dawson Lake."

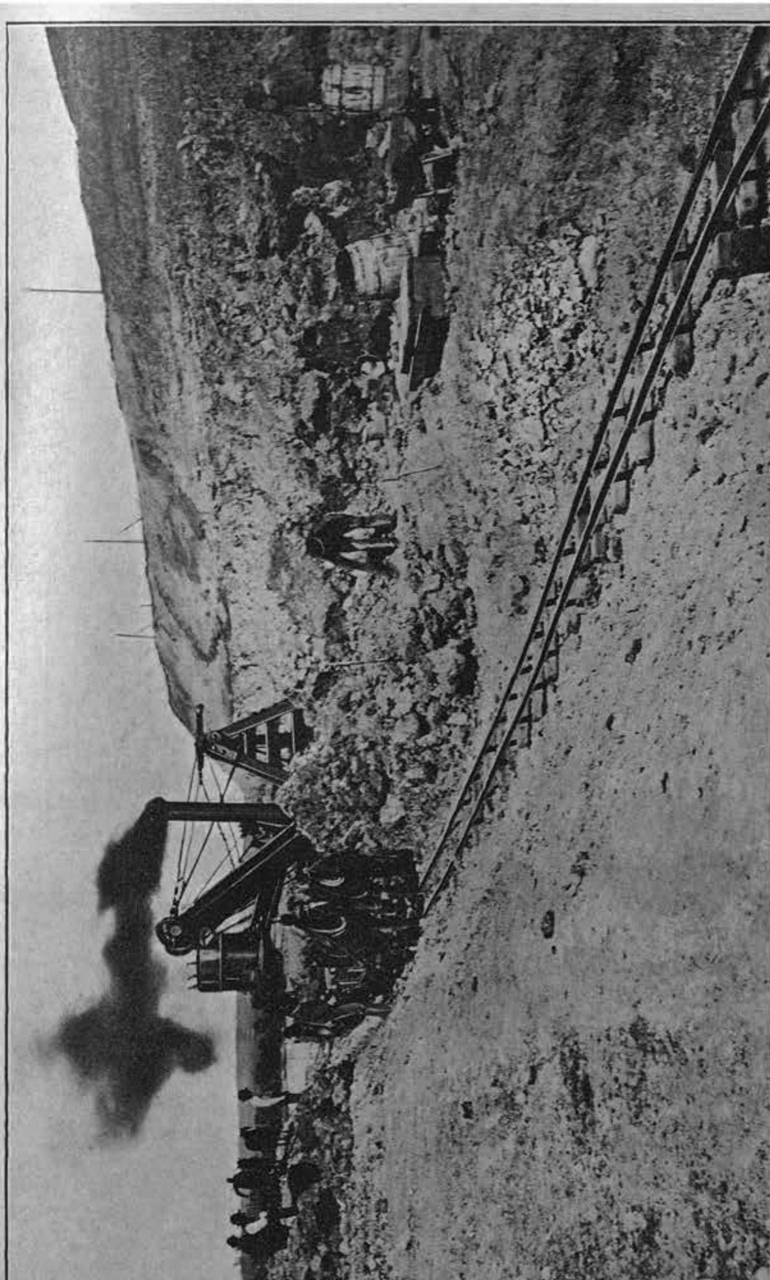


FIG. 5.—Excavation of side cutting near head of Calgary Canal.

These reservoirs can be filled in the non-irrigating season and drawn upon later. This system of intermediate storage is a novel and most

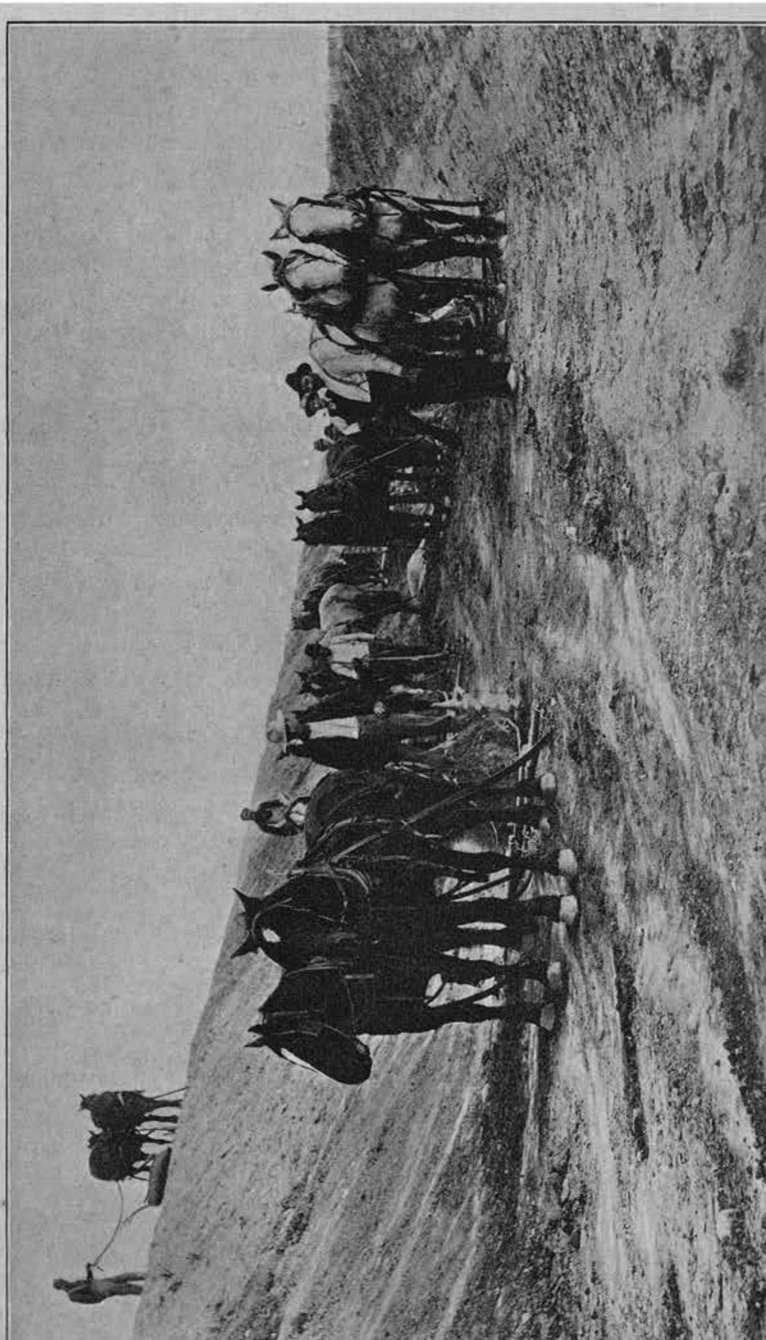


FIG. 6.—Horse Scrapers at work, Calgary Canal.

interesting feature, and its successful working out should be watched with interest by the profession.

The main canal and one of its branches will have to be widened to carry the additional supply of the central section, probably to the eventual total extent of 5,000 second-feet. The author had the privilege of being taken round a portion of the western section two years ago by Mr. H. B. Muckleston, "Chief Division Engineer," who, from having mainly to do with the design of these complicated works, was well qualified to afford information on technical points. This officer's first experience in velocity observations of a practical nature was in the Johnstown reservoir disaster, from the flood waters of which, like Moses of old, he was rescued when a child. The Johnstown (Pennsylvania) flood, brought about by the bursting of a reservoir, is notorious as the most disastrous on record. Fig. 4 is that of an Inspection House in the western section, and will be interesting to Indian irrigation officers who spend a large portion of their official lives in canal "chokis."

(**19**) Canals in the North-West have only one irrigating season, as ice and snow cover the country; the winters in Southern Alberta are, however, much milder than in Manitoba and Saskatchewan owing to the proximity of the Pacific Ocean and the warm Japan Gulf Stream which laves the western coast.

The canals can therefore be used for water carriage to reservoirs during the best part of the winter.

In British Columbia considerable orchard irrigation is effected, but the canals are on a very small scale. Mountain streams are intercepted and conveyed by ditches dug along a falling contour in the hill-sides to the orchards and fields. Irrigated land here sells for 200 or 300 dollars, *i.e.*, £40 to £60 an acre, so that irrigation is an exceedingly profitable undertaking. Water is, however, deficient, and storage works will soon become an urgent necessity.

The valleys with large lakes are occupied and cultivated, but the high plateaux on top of the adjacent hills are at present a waste except as a cattle range, being almost impenetrable owing to fallen timber. Here large lakes of water exist which could be made to form reservoirs for irrigation and mining purposes.

Some rivers, such as the Similikameen which flows into Washington State, presents tempting prospects of irrigation, and their wide deep beds could be transformed into a series of large reservoirs. Unfortunately the Northern Pacific, an American railroad, has been allowed to run their Vancouver approach line up this particular valley, thus effectively hindering any such future utilisation of its flood waters.

(**20**) The two illustrations, 5 and 6, depict Western methods of excavation which are of interest when contrasted with Asiatic practice in effecting the same purpose.

CHAPTER XIV

SCREW GEAR FOR TANK SLUICES, AND ROLLER GATES

(1) It is proposed in this chapter to give a short account of the best type of screw lifting gear suitable for tank sluices, as also of different types of rollers in use in vertical draw sluice gates.

Figs. 1 to 4 and Fig. 7 are rough sketches illustrative of the principle of different systems of screw lifting gear. In Fig. 1 the motive power is applied to a female screw cut in the base of the handle itself. On revolving the handle the solid screw, rising, lifts the gate, but there is no means of forcing

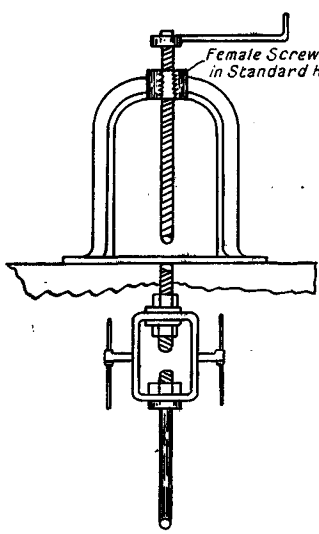


FIG. 1.

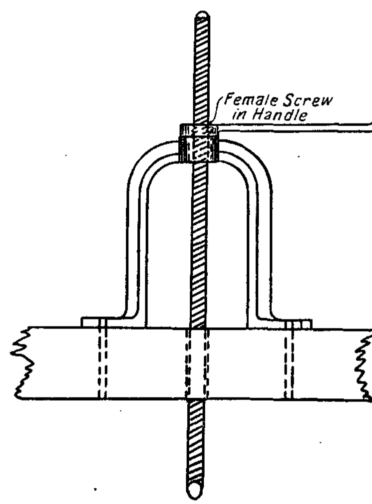


FIG. 2.

the gate down, unless its dead weight is sufficient for the purpose. The author has actually seen this extremely primitive gear applied to wooden sluices of Government tanks in Burma, and was privileged to witness the operation of lowering the gate against a head of water.

The procedure adopted was for one cultivator to stand and jump on the gate while his companion belaboured the top of the screw rod with a large stone, the handle having been previously run up to the top. The screw rod, it may be added, was fixed to the gate at its lower extremity and so could not itself revolve.

(2) In Fig. 2 the motive power is applied to the screw rod, the female screw being cut in the head of the standard. In this case the screw rod itself

revolves, and consequently has to be provided with a dog chain swivel joint either at the gate fastening, or preferably as near the screw head as possible, thus reducing the torsional stress on the rod. The handle rises and falls with the rod, to the extremity of which it is keyed.

This type is a great advance on the last, and is quite suitable for most plugs which are only lifted 2 feet at the outside. The swivel should be prevented from turning by two arms which slide up and down guide rods. This is shown roughly in the sketch.

(3) Figs. 3, 4 and 5 represent the ordinarily adopted lifting gear, in which the female screw is revolved. The solid screw rod connected with the gate

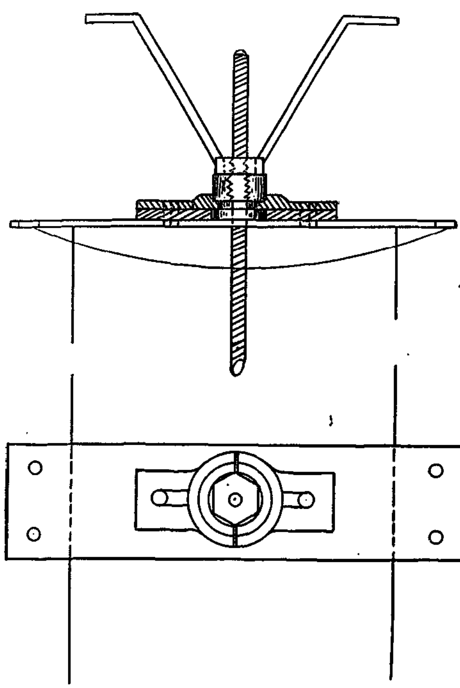


FIG. 3.

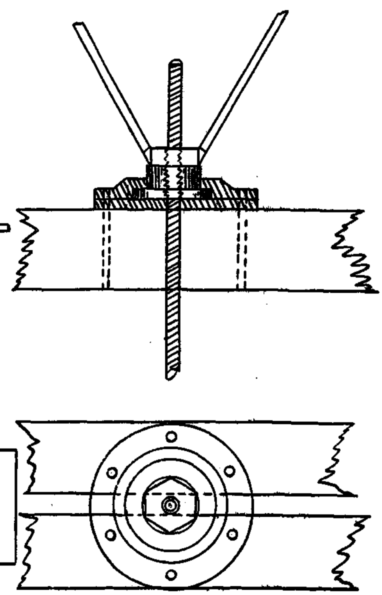


FIG. 4.

passes through the former and rises above the platform. This arrangement is on the same principle as Fig. 1, only the female screw itself is prevented from rising by the use of a thrust plate which either works in an annular groove cast in the brass screw head, as in Fig. 3, or itself is provided with a holding down, as well as a base plate, which embrace a circular collar cast or screwed on to the head piece as in Fig. 4.

In Fig. 3 the steel or iron plate is in two halves, and each half is provided with a slot as shown in plan. This enables the two halves of the thrust plate to be disengaged and removed by slackening the fastening nuts and pushing the plates backwards, the bolts traversing the slot. The author has seen several of these employed in lifting tank sluices, but they proved very hard to work. In Fig. 4 the holding down upper plate is in one piece, the brass female screw being provided in this case with a collar at its base. Fig. 5 is taken from the "Madras Manual," and exemplifies the usual Madras practice, where screws are very commonly employed. In this case the thrust plate also is in one piece and is slipped over the projecting shoulder of the

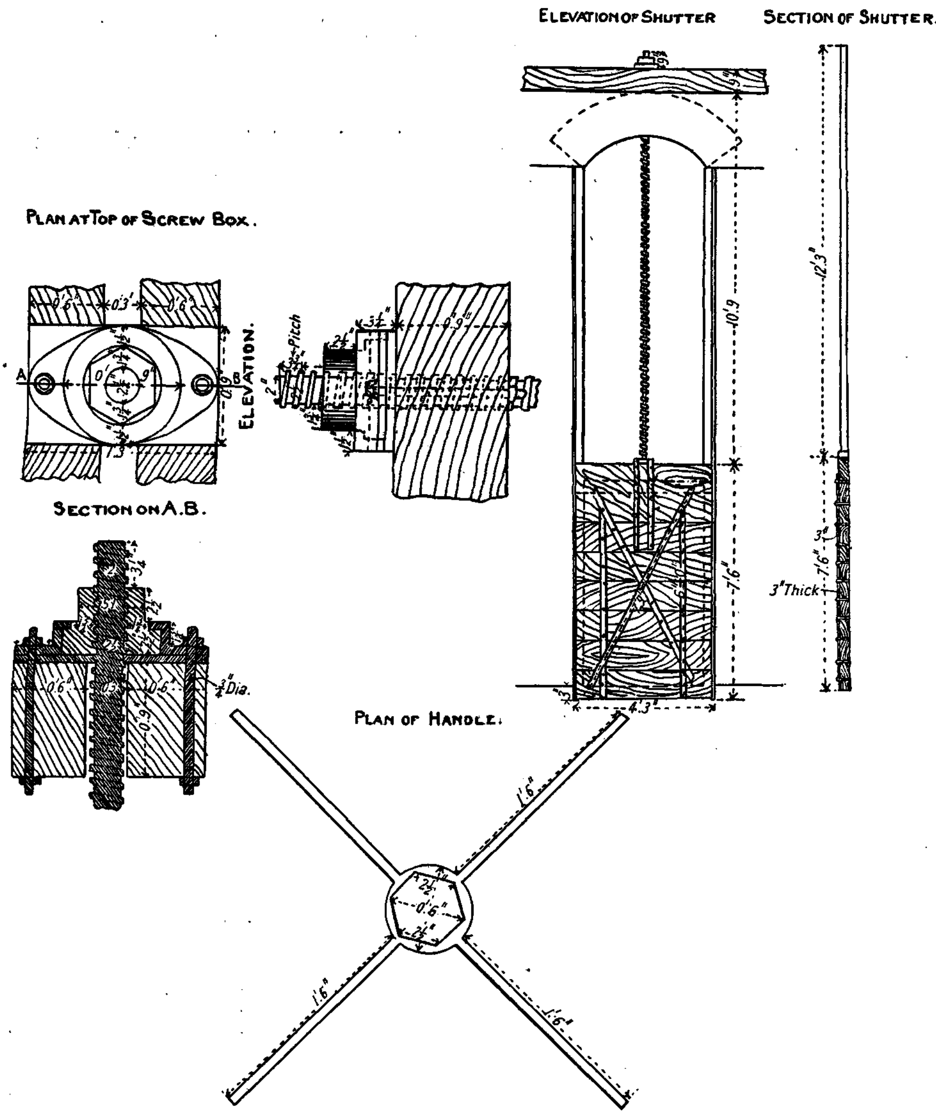


FIG. 5.

brass screw head, and bolted in position. If this thrust plate is truly cast and planed it should be as good as, if not preferable to, the form illustrated in Fig. 3.

(4) Fig. 6 represents another development of the same principle, and is used in Madras for lifting a series of gates one above another, which close

the larger openings of weir sluices or head regulators. As already mentioned in previous chapters, this style of lifting gear for such purposes is now completely obsolete. The introduction of anti-friction rollers enables gates when released to fall into position by their own weight without requiring the application of any power to force them down against a head of water. Hence they can be lifted by a travelling winch, an arrangement, much more economical than the lines of long expensive screw rods shown in Fig. 6. In this gear the female screw is formed at the end of a long pipe, at the top of which pipe is a solid head to which the power handle is applied and which is held fixed by a thrust plate. As the pipe is revolved to raise the gate, the solid rod attached to the latter is drawn up inside the pipe, being thus protected from dust or water.

As screws in future are never likely to be employed except for very short lifts, such as those of small deep reservoir sluices, of which the outside lift is 3 feet, it is quite clear that the principle of the motive power being applied to the short hollow screw, thus involving a long solid screw rod subjected to compressional and torsional stress, is not at all economical.

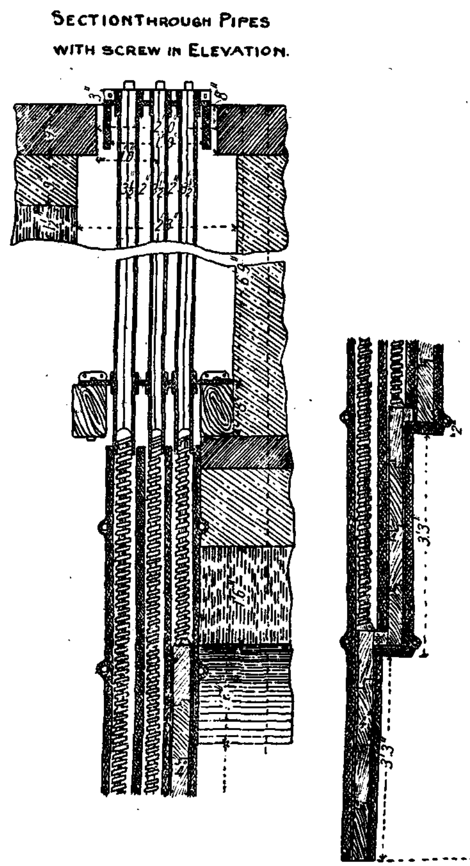


FIG. 6.

(5) The arrangement illustrated in Fig. 7 is undoubtedly far superior to the former. In this the power is applied to the *male* screw, which can be quite short, a little over the lift in length. This should be of solid cast steel. As shown in the illustration, the thrust collar and plates are situated at the rod head, and consist of three plates, the upper, the distance plate, and the base plate. The screw rod is threaded through the base plate, while the upper plates are superimposed, and the whole bolted through as shown on plan (Fig. 7). The upper and lower plates should be of brass, the middle distance plate, which is subjected to no strain of friction, being of planed iron, or else the middle and top plate could be combined in one piece of brass, as in Fig. 4. The solid screw rod passes into a pipe in the head of which is the female screw. This should be of brass, and in some cases the whole pipe is made of brass; but this seems a needless extravagance. The pipe head is held rigid by two arms running in guide slots cut in the frame of the standard.

Thus the torsional stress is at once absorbed, and the position of the ends of the arms, one of which can be made to project beyond the frame, indicate, by means of a pointer on a graduated scale, the exact height at which the gate stands above its sill. The pipe is rigidly connected with the gate, and from its ring section is clearly much better suited to withstand compression than a solid rod of greater weight of metal. It can also be made of ordinary gas or water piping. The introduction of ball bearings above and below the thrust discs would be a further improvement.

(6) This system for short lifts is by far the best of any other illustrated in these figures. It is remarkable that it is not given in the "Madras Manual," all the screw lifting gear in which are of the other types, in which the motive power is applied to the female screw. The only tank sluice fitted with this gear seen by the author was that in Kalawewa Tank, in Ceylon. In this a hand wheel turned a worm, or archimedean screw, which engaged a bevel wheel keyed on to the top of the shaft. The sluice gate in this case was 3 feet square under a head of 25 feet, and yet the gate was manipulated with the greatest ease by one man.

The screw heads, or standards, can be supported on wooden beams, or a cast-iron fish-bellied girder with double webs. Two rolled beams connected by a plate bolted to their upper flanges would suit equally well, the lifting rod passing between their webs.

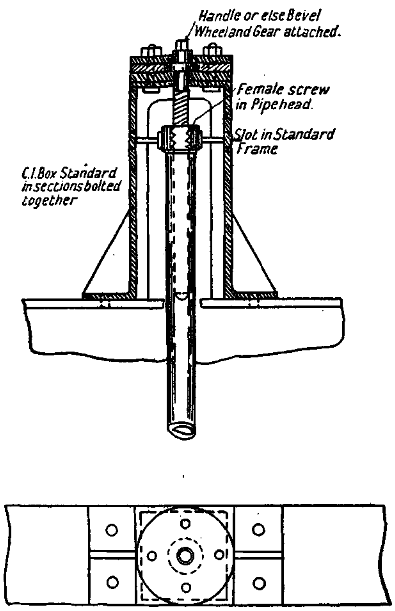


Fig. 7.

(7) Fig. 8 represents a group of three screws which operate the pipe valves in the Waghad Tank outlet culvert which are shown in Fig. 1, Chap. XII. These are on the principle of Fig. 7. At the top is a thrust box containing brasses with hollow annular ways in which similar annular projections on the steel screw rod work. Below is a hollow square pillar in the head of which is a female screw, engaged by the screw of the lifting rod. To the base of the pillar, which is moved up and down through a square bracket bearing, the lifting rod of the gate is attached.

(8) Fig. 9 is on the same good principle, but in an improved style. Here the lifting screw works against a step at its base and is provided with a thrust collar working in a brass thrust box at the top. It is therefore practically independent of any deviation from the exact alignment of the gate lifting rod, and is thus not subjected to other than legitimate stresses. The

lifting screw is enclosed in a round cast-iron standard, in the hollow pipe of which a circular brass stud is free to move. In this is cut the female screw, and it is prevented from revolving by projecting arms which run in two slots cut in the side of the box as in Fig. 7. To these projections two links are attached, which latter below the frame are joined by a cross bar, or pin on which the gate lifting rod is keyed. Instead of the clumsy capstan bar, a bevel wheel could be keyed on the top of the screw rod, and a bracket fitted to the frame, or cast on it, would carry a pinion and hand wheel. The

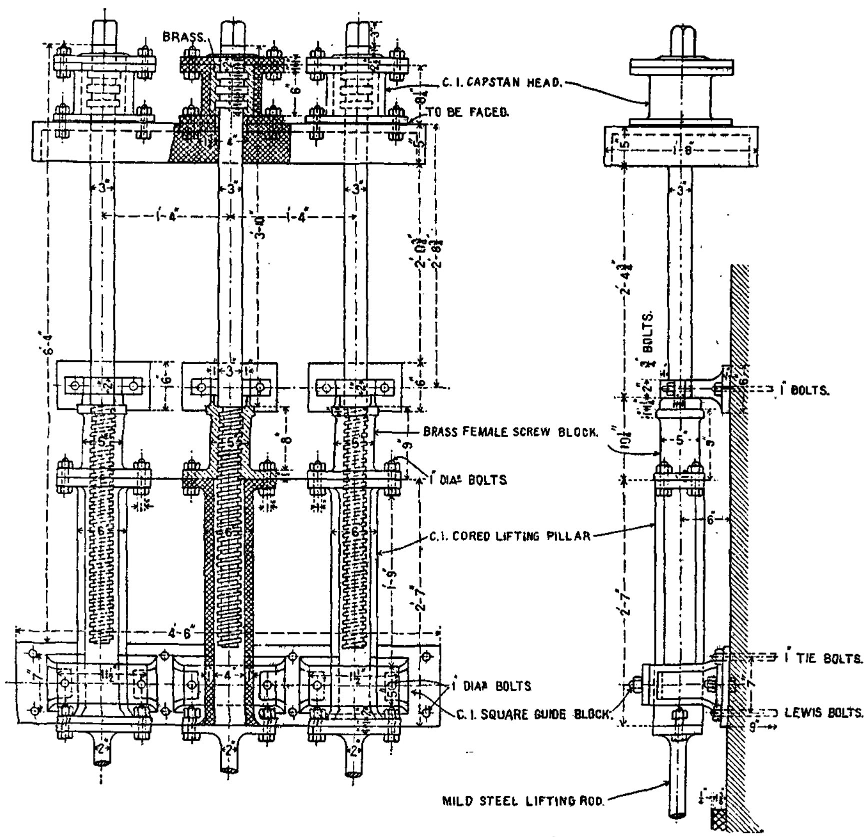


FIG. 8.—Lift Screw Gear of Waghad Tank.

hand wheel could be made removable to prevent unauthorised tampering with the sluices. Ball bearings should also be arranged for at the thrust collars.

(9) An example of American lifting gear taken from Wilson's "Irrigation Engineering" is given in Fig. 10. Here a bevel wheel is keyed on the female screw block, which is held in the jaws of the cast-iron frame carrying the pinion wheel and hand wheel.

Steel ball-bearing discs are fitted at the base of the female screw block, where it bears against the frame. The lifting screw rod is square in section below the screw and passes through a square hole in a projection at the base of the cast-iron frame.

This arrangement is obviously inferior to that of the two last examples in that the screw is exposed in the open to dust and rain, and when the sluice gates are open, sticks up in an absurd manner as if to invite damage; whereas in the other case the screw is protected from injury of any kind, being

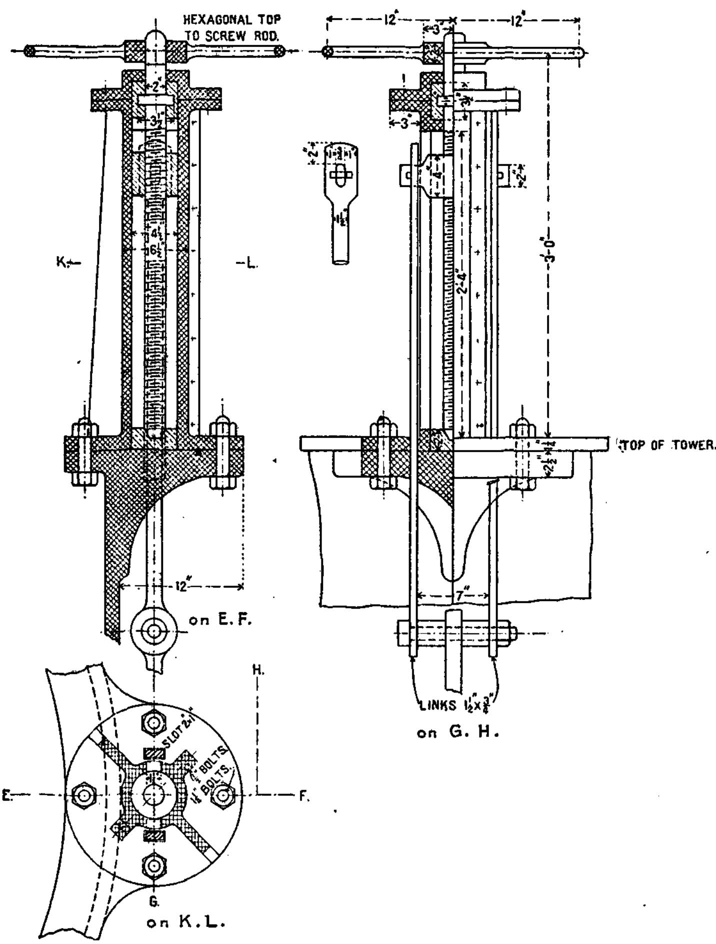


FIG. 9.—" Battman's " Screw Lift Gear.

enclosed in a box out of sight. Length of rod is also saved. This apparatus or one in which a pinion engages a rack appears to be universal in the States. The adoption of ball or roller bearings at the points of thrust is worthy of imitation.

(10) A very instructive example is given in Figs. 11, 11a and 11b of the screw lifting gear of the Bhatgarh Dam undersluices, a section of which is given in Fig. 15, Chap. II., and for which we are indebted to " The Irrigation

Works of India." These sluice gates have a lift of 8 feet under a mean head of 80 feet, the area of the sluice ways being 8 feet by 4 feet; the pressure on each is therefore $whA = \frac{1}{36} \times 80 \times 4 \times 8 = 71$ tons. The screw rod is $5\frac{1}{2}$ inches diameter for the screwed upper length, the remainder being $5\frac{1}{2}$ inches square. These heavy, expensive forged rods are about 87 feet in total length, though made up in $12\frac{1}{2}$ feet lengths, and are supported at intervals against buckling by passing through square holes in brackets projecting from the rear face of the dam.

The motive power is, as usual, applied to the brass female screw, which is provided outside with two collars which work in corresponding grooves in a thrust box. This latter, which is of cast-iron, is in two pieces, bolted

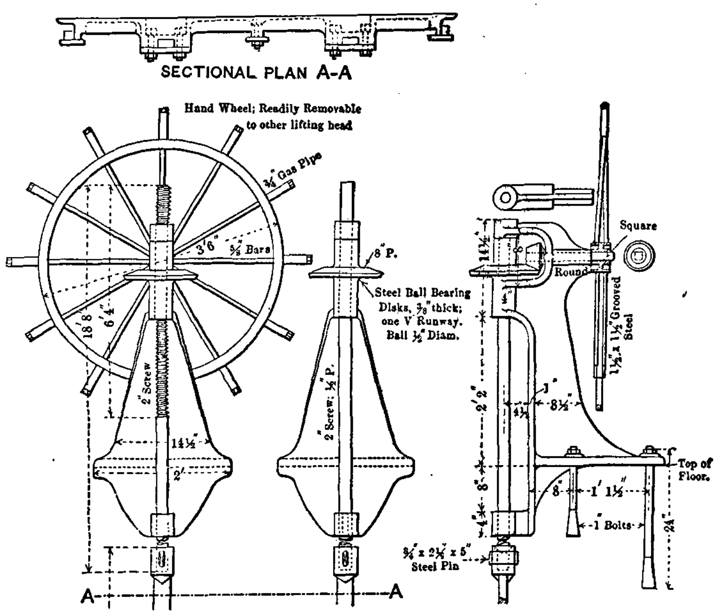


FIG. 10.

together horizontally and further provided with the usual vertical holding down bolts (*vide* plan of lifting nut collar, etc., in Fig. 11a). The squared part of the rod commences at the termination of the screwed upper part, which has a run of 8 feet, and this passes through a square brass guide which prevents the rod revolving, and relieves it of any torsional strain below this point. The screw lifting rod rises through the capstan nut, and when the gate is fully open must stick out some 9 feet above the platform level on top of the dam.

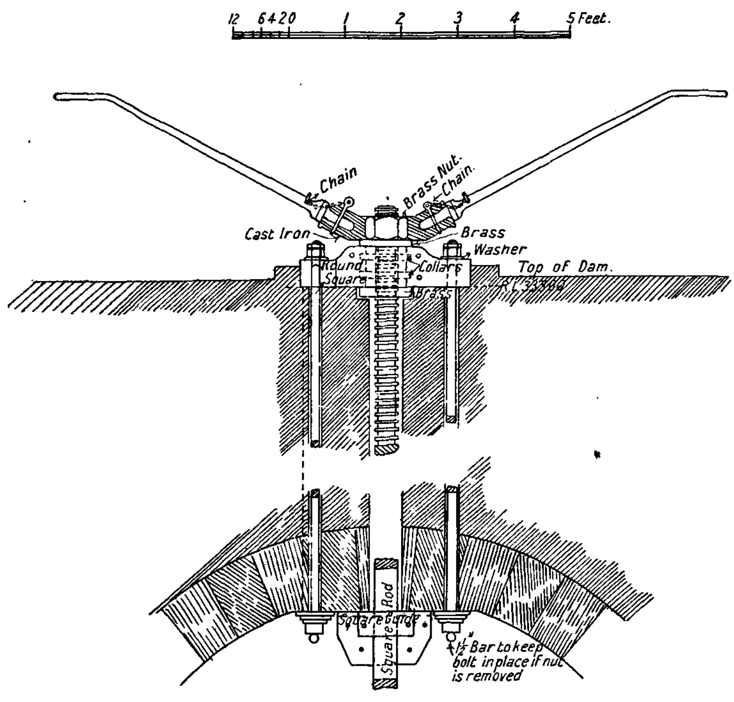
(11) This manifestly unsuitable arrangement could easily be avoided by adopting the general design of Fig. 7, 8 or 9, viz., of applying the motive power to the male screw, which latter, being provided with a collar working in a thrust box, is prevented from vertical movement but not from revolving. The female screw would be placed in the head of a long cast-iron pipe, which is fastened to the gate, and which is prevented from revolving by

attached arms, the extremities of which slide in vertical grooves, and is, of course, free to move vertically in either sense. It is quite evident that this arrangement would be much less expensive and equally effective as that portrayed in Fig. 11.

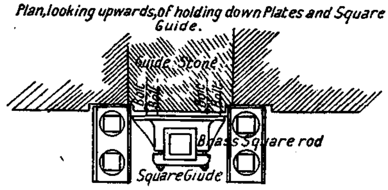
A further improvement would be to apply bevel gearing to the screw

BHATGARH RESERVOIR

LIFTING GEAR TO UNDER SLUICES.



Plan, looking upwards, of holding down Plates and Square Guide.



(11a.)

FIGS. 11, 11a.

head in lieu of the capstan, which has to be worked by several men. A bevel spur wheel could be fixed horizontally on the rod head, worked by one or two bevelled pinions turned by wheel handles revolving vertically, and antifriction rollers above and below the thrust collars.

(12) The principle of the application of free rollers and roller wheels fixed on axles to draw gates, together with the devices employed in rendering the gate watertight, will now be briefly noticed.

When a gate or shutter slides in a groove, the pressure of the water forces it against the inner side of the vertical grooves, as also against the lintel at the top and the sill at the bottom, thus effectually preventing any leakage on all four sides of the shutter. The resistance of the friction of the end side of the gate against that of the grooves is very considerable, and as the parts in contact cannot be lubricated, and the gate often remains unmoved for a long time, the parts become rusty, and the friction is thus increased greatly in excess of what would normally be the case. Now the coefficient of friction of smooth metal surfaces in contact unlubricated is about ·2. Thus if a gate were subjected to a horizontal water pressure of 1,000 lbs., it could be lifted, neglecting its own weight, by a force of $1,000 \times {\cdot}2 = 200$ lbs. On the other hand, when the surfaces are not smooth, the coefficient of friction may possibly become as great as unity. Hence a gate without rollers requires power not only to lift it, but to force it down, as its own submerged weight is insufficient to overcome the friction induced by the pressure of the water. This is the reason why screw power has hitherto been so largely used as lifting apparatus.

By the simple device of mounting the gate on wheels with axle bearings, the friction is greatly diminished, so that the requisite lifting power is considerably reduced, and none is required to lower the gate, that being effected by its own weight.

The combined axle and roller friction now induced, which is similar to that of a railway truck, can be taken as one-eightieth of the water pressure plus the weight of the gate itself in the process of lifting. Thus the net lifting power required, supposing the water pressure to be 1,000 lbs., would be $\frac{1,000}{80} = (12\frac{1}{2} + W)$ lbs.

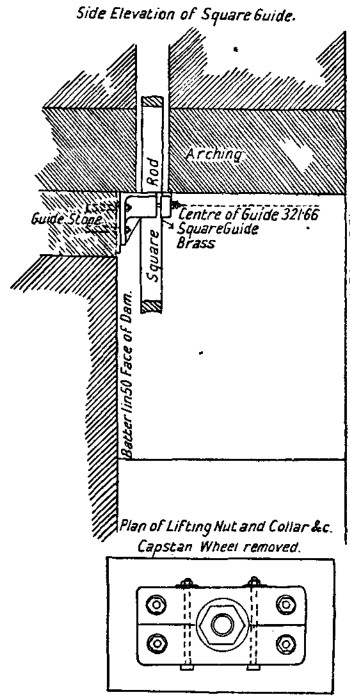


FIG. 11b.

If the axle bearings are not lubricated the coefficient of friction will naturally be much greater. Within certain limits a roller wheel of large diameter would give better results than one of smaller size. The rolling friction induced between the wheel rim and the surface would be less in the former case, and in addition the leverage of the greater wheel radius acts in favour of the motive power.

(13) When a roller or wheel is used with a gate there is naturally no close contact between the side of the gate and that of the sluice opening or of the table of the groove, the gate being raised off it. In order to prevent leakage through this opening the device of stanching rods is usually adopted, which consists generally of a round solid rod or else gas piping which is fastened to the gate at the top only the rest hanging free. The pressure of the water forces this against the opening slit existing between the gate and the jamb, or shoulder of the masonry, forming a perfect watertight closure. When the gate is pulled up or down the stanching rod goes with it, scraping against the jamb; the friction thus set up is quite negligible.

With Indian roller gates, stanching rods do not appear to be employed. A watertight connection is formed by either slightly inclining the roller path or else the gate, so that when the gate is fully down close connection is formed. This, however, allows of water passing the ends of the gate the moment it is raised off the final position, which is objectionable as straw and other detritus may accumulate and foul the rollers, whereas where a stanching strip is employed no leakage can at any time occur at the sides.

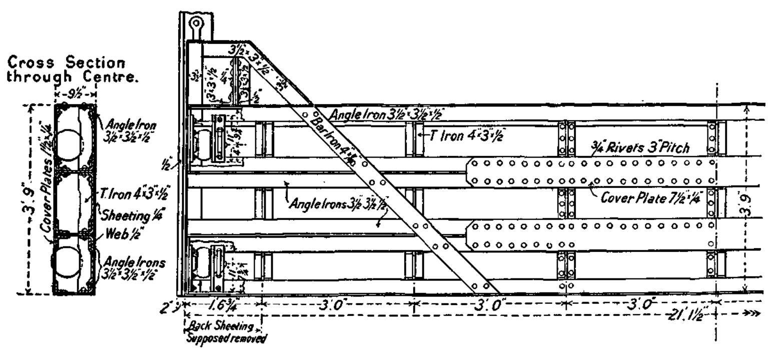


FIG. 12.—Middle Gate of Chenab Weir Sluices.

(**14**) Gate rollers or wheels consist either of small solid rollers mounted on axles which pass through axle boxes fixed to the gate. The method used in the Chenab weir-sluice gates is exhibited in Fig. 12.

In other sluice gates the rollers consist of a pair of trolley wheels with long axle bars reaching across with the wheels keyed on, as in the case of a railway truck. The large regulating gates across the Rhone at Geneva which are 25 × 10 or × 15, are mounted on quite large wheels keyed on shafts, the wheels being double flanged and working on an ordinary vertical steel rail fixed upright against the jambs or shoulders of the sluice opening.

In Indian and Egyptian works, draw gates always work in cast-iron grooves; the grooves are an essential feature as they protect the lifting chains from the current and afford a recess in which the chains hang safely when the gates are in place, and are disconnected from the travelling winch.

A great improvement would be to adopt forced-axle lubrication as is done to "Stoney" gate-rollers by flexible tubes connecting with the surface. The journals could then be packed in watertight stuffing boxes like the piston of a steam cylinder.

(15) We have hitherto considered the combined axle and rolling friction which is induced by small wheels fixed on axles running in bearings bolted to the gate. When, however, the pressure is very considerable, due to a great head of water or the exceptional dimensions of the opening, another arrangement, viz., that of free rollers, is made use of. These rollers run on axles, but there is no pressure induced on the latter; the axles are only used to keep the rollers the proper distance apart, and are fixed in a frame. This roller cradle is suspended in the groove unattached to the gate. As the side of the gate bears against the rollers, when the former is moved the rollers revolve, and the whole frame of rollers rises, and also falls, with the gate. The friction induced is pure roller friction, which, between smooth surfaces, is something very small indeed, so much so that its coefficient can be entirely ignored, the lifting power being only the weight of the gate plus the friction of the lifting apparatus. Again, under suitable conditions, the gate can be counterpoised by weights hanging from overhead pulleys, and in such cases gates of the largest size can be manipulated by one man. These free rollers are termed "Stoney's Patent Antifriction Rollers."

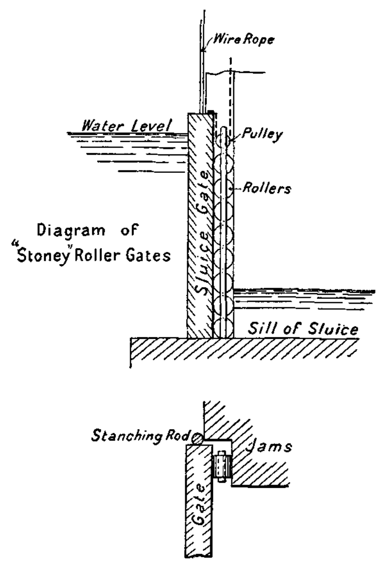


FIG. 13.

The diagrams in Fig. 13 represent a sluice gate fitted with free rollers in sectional plan and elevation. It will be seen that the frame of the gate on plan just clears the sluice wall, and projects beyond the groove. These spaces are closed by stanching rods. These are simple round rods of about 2 inches diameter, which are fastened to the gate, but have lateral play, so that when the gate is under pressure they are forced into the corner, effectually closing the aperture. On the gate being raised they are carried up with it, still bearing against the side of the gate and the sluice wall; the friction thus induced is, however, quite trifling. Owing to the suspending pulleys the roller frame moves half as fast as the gate.

(16) In Fig. 14, which is derived from Vol. CLII., "Min. Pro. Inst. C.E." the details of the roller gates used in the sluices of the Assuan Dam, viz., those at R.L. 100·00, are given. The following is the description by Mr. F. W. Scott Stokes, M. Inst. C.E., the author of the paper on the sluices and lock gates of the Assuan Dam.

(Sluices at R.L. 96 *and R.L.* 100.) (Fig. 14.)

"The culverts are 2 metres (6 feet 6¼ inches) wide by 3·5 metres (11 feet 6 inches) high. The entrance to the culvert is well mouthed and is roofed over by a casting curved on the dam face, and flat at the place where it joins the sluice lintel. The grooves, lintel, and sill of the sluice are of cast iron arranged with machine faces, and put together with turned bolts. The sluice is built up of a steel plate skin with rolled strengthening girders at the back, framed into cast-iron beams on each side, which form the roller paths. Adjustable bars are fitted on each side of the face to reduce the leakage, as wear takes place owing to the cutting action of the silt in the water. On

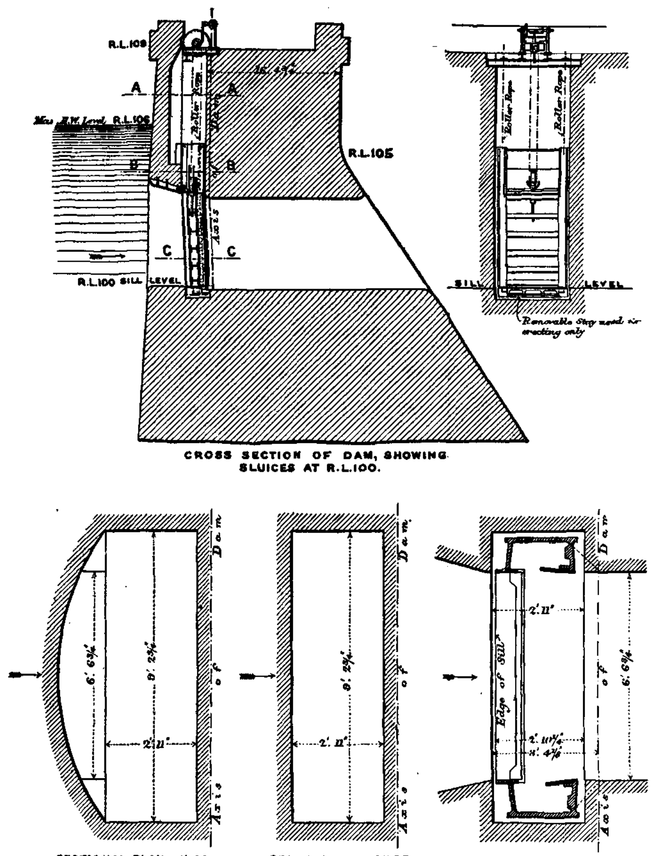


FIG. 14.—Assuan Dam. Sluices at R.L. 100·0.

the top of the sluice is also fitted an adjustable bar, which shuts down on the lintel at the same moment that the bottom of the skin lands on the sill, and thus makes a watertight joint. The rollers are arranged in cradles formed of flat bars, and are hung in position on each side of the gates by steel wire ropes. The crab has two speeds of lift, namely, for small adjustments by one man, or for quick working by four. An automatic self-sustaining gear is provided, so that the sluice (which is not counterbalanced in any way) cannot run down, the handles having to be turned when lowering in order to release the gear. The barrel shaft is worked by means of a worm wheel, and a worm fitted with a ball thrust bearing. All the bearings and pulleys are fitted with screw-down grease lubricators; this is the more advisable owing to the sand and dust."

(*Sluices with Rollers at R.L.* 92.) (Fig. 15.)

(17) "These twenty-five sluices are 7 metres (23 feet) high by 2 metres (6 feet 6¾ inches) wide, and are for the purpose of regulation when the fifty without rollers have been lowered, and before the water rises high enough for those at R.L. 96 and R.L. 100 to give the required discharge.

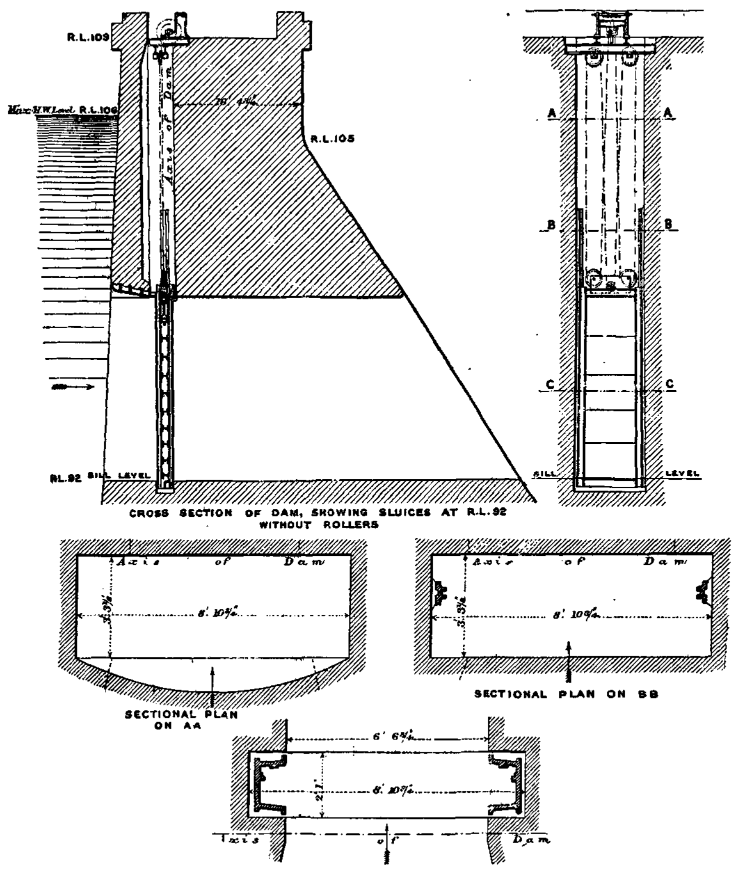


FIG. 15.—Assuan Dam. Sluices at R.L. 92·00.

"As the head above sill level is 14 metres (45 feet 11 inches), the rollers and other parts have to be of much more ample proportions, and the grooves built into the masonry of considerably larger size than those of the sluices of R.L. 100 and R.L. 96. The action of the water passing through the culvert at a high velocity has also to be avoided; hence the rollers are kept well back from the face, and an inclined shield plate is provided to protect them from a direct flow. A whirlpool motion is set up in the grooves, but it is not of sufficient force to disturb the rollers hanging below the bottom of the gate when raised. The grooves are built of four sections weighing about 2 tons each. They are bolted together with turned bolts, and are arranged with projections on each side to key into the masonry and prevent the leakage of water round the back. The sluice is built of heavy, specially rolled steel joists, with steel skin plates and cast-iron roller path guiders on each side.

"A vertical rod, which is contained in a groove bolted to the skin on each

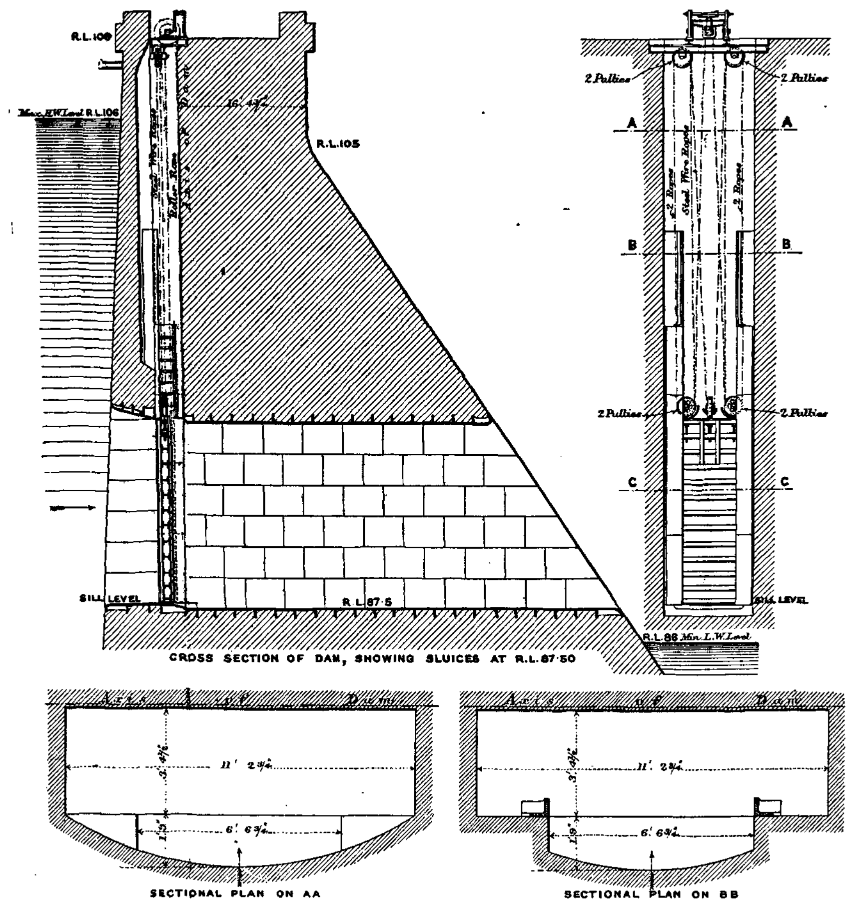


FIG. 16.—Assuan Dam. Sluices at R.L. 87·50.

side of the face, is pressed by the water into the corner formed by the face of the sluice and the face of the fixed frame, thus making a watertight joint."[3]

(*Sluices at R.L.* 87·50.) (Fig. 16.)

(18) "It was originally intended to have the lowest sluices in the dam at R.L. 84, but it was found that R.L. 87·50 would suit the surface levels and channels better. The head of water above sill level is 18·5 metres (60 feet 8 inches), giving a pressure of about 210 tons against the sluice. The

entrance to the culvert is bell-mouthed, and roofed by an arched casting. A slight drop is arranged on the down-stream side of the sluice sill, which has been found to reduce greatly the wear and tear on the culvert bottom, and to give a freer discharge under the sluice. A similar drop has been given to the other culverts. The grooves are built of four sections, bolted together and to the lintel and sill castings. Stanching rods are fitted to the sluice, as in those for R.L. 92 with rollers. The gate is built of special rolled steel joists framed into two cast roller paths. The skin is riveted to the joists in sections, which, when in position, are finally fixed together by turned bolts. The sluice gate is carried by a steel wire rope in two parts arranged round pulleys on the gate and on girders at the pit top. Forced grease lubrication is employed as in other sluices and gear. The crab is similar in design to those for the sluices at R.L. 96 and 100."

(19) Fig. 17 illustrates the case of double roller gates as used for under-sluices of wide span. The outer gate is generally the lower one. These gates require a watertight joint between them when both are lowered. The projection of the rollers and the thickness of the middle groove flanges places them somewhat far apart. This difficulty can be overcome by projecting the top plate of the lower gate inwards, so that a narrow flat plate will cover the space, or else a plate attached to and suitably hinged to the base of the inner upper gate, so that when the lower is raised it will lift the flap, which will still keep in contact with some projecting vertical bars inserted for the purpose, and thus either gate can be lowered to the floor.

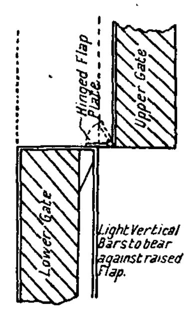


FIG. 17.

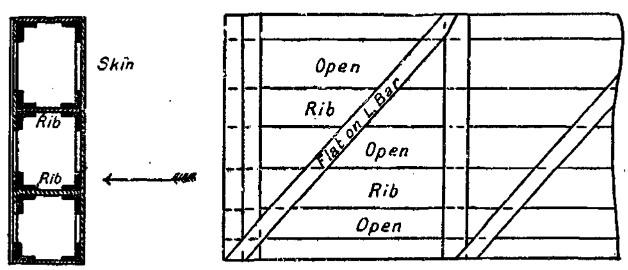
Roller gates are best designed with a shoulder, or else an angle iron can project to form an abutting line for the vertical stanching rods. If necessary, stanching rods can be applied inside the groove itself at the ends of the gates.

(20) Gates are usually built up as plate girders, but sometimes rolled beams are used for the longitudinals, as in the Assuan Dam sluices. The gate consists of the outer skin and a series of longitudinal ribs which convey the horizontal water pressure to the rollers. As the pressure is greatest at the base of the gate, the ribs should be closer together near the lower end of the gate, the spacing being gradually increased so as to cause each rib to bear approximately the same horizontal pressure. The first point to be determined is the depth of the ribs, *i.e.*, the width of the gate at the centre. This may vary from $\frac{1}{20}$ to $\frac{1}{10}$ of the span. Representing the distance apart of the lowest ribs as b, and the head of water above the middle point between the ribs as h, and the length between the piers or span as S, the pressure to be supported by each rib will be $\frac{1}{2}\, w\, (h \times S \times b) = W$ in feet tons, w being $\frac{1}{36}$ ton, the weight of a cubic foot of water.

This pressure W is a distributed load, consequently the bending moment

M will be $\frac{WL}{8}$, L being the distance between the centre of the roller supports, which is somewhat greater than S. The rib in ordinary cases will be of double **T** section formed of two angle irons, the upper tables of which are riveted to the skin and the lower to the web. If the skin plate on one side and the web be neglected from consideration, the sectional area of each of the flanges will consist of that of the angle iron plus the portion of plate enclosed between them, as shown shaded in Fig. 18.

(**21**) The effective depth of the girder can then be considered to be not the extreme depth of the double **T** beam, but the distance between the centre of gravity of the angle irons or d, which lines fall a little below the inner side of the top tables of the angle irons (*vide* Fig. 18). Tensile and compressive stress on the top and bottom flanges will be $\frac{WL}{8d}$ tons. The safe stress on iron may be taken as $4\frac{1}{2}$ tons, and that on steel as 6 tons



Rear Elevation of Gate.

Section. FIG. 18.

per square inch, termed m. The required net sectional area of the flanges, less rivet holes, will be $a = \frac{WL}{8dm}$. If rolled beams are used, the thickness of the web will have to be taken into consideration, and the moment of inertia about the neutral axis of the beam calculated (*vide* "Molesworth's Pocket Book," p. 130). Then $R = \frac{mI}{N}$, in which R is the resistance of the beam to flexure which should equal $\frac{WL}{8}$, m the safe compressive or tensile stress per square inch of the metal, I the moment of inertia, and N in this case $= \frac{D}{2}$, D being the outside depth of the section.

(**22**) No continuous plate is required on the inside of the gate, but vertical and inclined strips of angle or flat bar at intervals, so as to enable access to be obtained for painting the parts. The area of the skin plate, as also that of these strips reduced to the equivalent of a thinner plate over the whole surface, could be included in the sectional area of the flanges, or in the value obtained for I. The webs of the ribs will naturally have to be strengthened by angle or T irons at intervals, as is usual in plate girders. In large single gates, where space is not confined, the ribs are generally composed of open lattice work with a good curvature to the centre, similar to a bow-string girder placed on its side. The top and bottom of the gate should be plated over.

(23) The shearing stress on the axles of the gate roller wheels, if such be adopted, will be one-quarter the total water pressure on the whole gate, *i.e.*, if two sets of rollers be used. The position of the rollers should be fixed with reference to the pressure so as to secure the upper having the same pressure as the lower roller.

The axles, if supported on both sides of the rollers, are subjected to double shear, consequently their sectional area should be $\frac{W}{8m}$, W being the total water pressure on the whole gate $= wh \times ld$, h being the mean head and ld the area of gate, and m the safe shearing stress of the metal, viz., 5 tons for iron and $6\frac{1}{2}$ tons for cast steel.

It might be further noted that the web of the ribs is subjected to shearing stress which is greatest at the ends of the gate, where it will amount to half the total water pressure on the gate; presumably it is sufficiently supported against buckling. Its effective sectional area should equal this pressure in tons divided by m, the safe shearing stress of the metal, generally assumed as the same or less than the compressive strength. The cover plates for joints in the skin should be of T section. These are not shown in the sketch in Fig. 18. The gate is not only subject to transverse thrust, but to vertical bending moment due to its own weight. This is met by the disposition of the bars on the rear side, as shown in the elevation, and by the skin on the face of the gate.

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