The moment therefore $= \frac{1}{3} \times 31 \cdot 25 \times h^3 = 10 \cdot 42\ h^3$, which is a convenient formula to remember for the fluid *moment* about any point in the base.

The vertical component will be the fluid pressure on the horizontal projection of the inclined face (*i.e.* on the width of the batter) × distance of its C. G. below the water surface. It acts at one-third the inclined face from the base, and its moment about the outer extremity of the base is easily found and deducted from the moment of the horizontal component. Usually the pressure due to the vertical component is entirely neglected.

The moment of resistance is the moment of the weight of the dam acting through its centre of gravity. The factor of safety will be $\frac{\text{moment of resistance}}{\text{overturning moment}}$, and it is usual to allow a factor of safety of $2\frac{1}{2}$ to $3\frac{1}{2}$ in large dams. The graphic solution can be used when a cross-section of the structure has been *assumed* to start with.

Stability against sliding.

The sliding force = height × pressure × $\frac{\text{height}}{2}$ which in the case of water $= 31 \cdot 25\ h^2$. If water flows over the top of the structure to a depth K, then the sliding force $= 31 \cdot 25\ h^2 + 62 \cdot 5h \times K$. *The resisting forces* are the weight of the structure + the vertical component of the fluid pressure if the face of the wall is inclined. If there is risk of fluid pressure under the base of the structure, the weight of the structure will be reduced proportionately. Also, if the rear of the structure is submerged, a deduction must be made for buoyancy.

The sliding force must be less than the co-efficient of friction × (weight of structure + vertical component of fluid pressure) minus any pressure under the base of the structure, minus the buoyant effect of the submergence in the rear (if any). The co-efficients of friction are :—

Brickwork	·65 to ·75
Limestone masonry	·65 to ·75
Concrete	·65
Masonry resting upon moist clay ...	·33

If the problem is to be solved graphically, then the tangent of the angle which the resultant makes with the vertical load line must be less than the co-efficient of friction for the materials concerned.

The factor of safety against sliding = $\frac{\text{co-efficient of friction}}{\text{Tangent of angle made by resultant with vertical load line.}}$

Earthen Dams.

Strange gives the following information :—

The cross-section of the dam depends on—

(*i*) Repose angle of soil when wet.

(*ii*) Nature of foundations.

(*iii*) Height of dam.

The table below gives dimensions recommended :—

Height.	Freeboard.	Upstream slope.	Down-stream slope.	Width at H. F. L.
Height 15′ and under ...	4′ to 5′	2 to 1	$1\frac{1}{2}$ to 1	20′ to $23\frac{1}{2}$′
„ 15′ to 25′ ...	5′ to 6′	$2\frac{1}{2}$ to 1	2 to 1	$28\frac{1}{2}$′ to 33′
„ 25′ to 50′ ...	6′	3 to 1	2 to 1	38′
„ 50′ to 75′ ...	6′ to 7′	3 to 1	2 to 1	40′ to 45′

For dams over 75 feet special precautions are required. In the Bombay presidency the maximum height is 80 feet. In England the maximum heights are 80, 100 and 125 feet. More clayey materials should be used in the water slope than in the downstream slope. Soils which become slush or which will not bind should be rejected. Pure sand has been used for dams ; it settles into a compact mass when wet, and its particles resist sliding. Sand and clay mixed is not good (except for puddle) as it will not drain, and the frictional resistance of the sand is neutralised by the wet clay. Rich clayey earths become greasy when wet, and are dangerous ; they are also very retentive of water. The *best material* to use is that which contains enough clay to bind and enough gritty matter to drain, so that the mass never soddens. Gravel, if used, must be so intimately mixed with finer material that there are no voids. As no subsoils are absolutely water-tight, except unfissured rock, a puddle trench is necessary (in porous foundations) carried down to as water-tight a sub-stratum as possible.

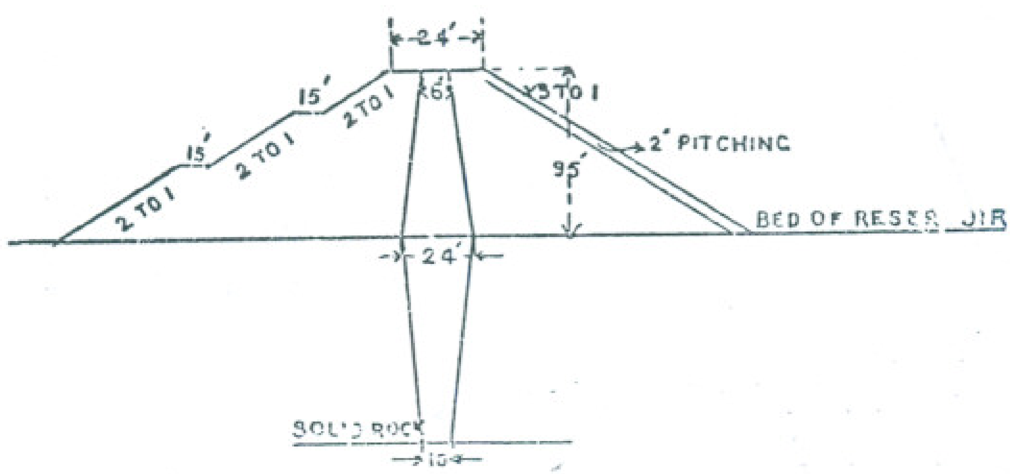
A dry stone toe at the base of the outer slope is advisable for drainage in high dams. The facing on the water slope of all earthen dams must be strong enough to resist wave action and pressure. Sarwar grass is good for low dams. The down-stream slope should be strong enough to resist guttering.

The following information is abstracted from "Rainfall Reservoirs and water supply (1913)" by Sir A. Binnie and from "Earth Dams" (1904) by Burr Bassell.

Site conditions favourable for earth dams are usually unfavourable for masonry structures, and *vice versa*. The wasteway for earth dams must be capable of discharging the maximum flood reaching the reservoir, and the crest of the spillway is usually ten feet below the crest of the dam. Open pits (trial pits) give the best information regarding subsoil ; but when a suitable stratum for foundations has been found by pits, borings will show its thickness.

The dimensions of the Yarrow dam are as follows :—

Top width 24', height from bed to water surface 90', freeboard 5', water slope 3 to 1, rear slope 2 to 1 with two berms 15' wide. A puddle core started 97 feet below the bed of the reservoir on solid rock. It was 10 feet wide at rock level and twenty-four feet wide at reservoir bed, it then diminished to six feet in width at the top of the dam as sketched below.



Many earthen dams exceed ninety feet in height, while hydraulic-fill dams of more than twice that height have been nstructed, but it is not often in India that a supply of water five to twenty cusecs is available for the sluicing of materials.

Strange recommends three parts of clay and two parts of nd as puddle for a core. There should be sufficient clay to entirely fill the interstices of the sand, and the mixture should be stiff enough to remain in a pail when the pail is turned upside down.

Drainage of the interior of the dam, especially the outer toe, is vital. This is secured by using pervious materials, such as coarse and fine gravel, on the downstream side of the crest. A heavy roller (five to ten tons, or heavier if possible) is necessary for consolidation.

Sir A. Binnie gives the following information in a slightly modified form :—

(*i*) Puddle trench for bank	20' high	2' wide	×	$1\frac{1}{2}$' deep.
,, ,,	30' ,,	3' ,,	×	2' ,,
,, ,,	40' ,,	3' ,,	×	3' ,,
,, ,,	50' ,,	4' ,,	×	4' ,,
,, ,,	60' ,,	5' ,,	×	4' ,,

the puddle being trodden in by men or by animals.

(*ii*) Puddle core should be carried up one to three feet above the water surface according to the height of the dam. The width of the core at the base should be 1/3rd the height of the dam, and diminished by a batter of 1/12 on each side.

(*iii*) Alongside the puddle core, put the same material (dry) as in the core, in six inch layers, watered and rolled ; using side slopes 1 to 1 for this material.

(*iv*) Outside this, put lighter material, but reserve all shale and gravel for the outer toe of the slope, to ensure good drainage.

(*v*) The top width of the dam to be at least 1/3rd of the height.

(*vi*) The water-slope of the completed dam should be as follows :—

2 to 1 for first 20 feet from crest.

3 to 1 next 10 ,, ,,

4 to 1 ,, 10 ,, ,,

5 to 1 ,, 10 ,, ,,

Rear slope 2 to 1.

In high canal embankments, as there is not room for so long a slope on the side towards the canal, the outer or rear slope is made long, and the inner or water slope as long as circumstances will permit. One of the main points is to secure a safe percolation gradient (hydraulic or saturation gradient) as required by the nature of soil employed.

(*vii*) Tramway tips should not be permitted within the area to be occupied by the embankment, especially where a very heavy roller for consolidation cannot be used.

(*viii*) Freeboard should be given as follows :—

For banks 20' high		..	$2\frac{1}{2}$ feet.
,, dams 30'	,,	...	3 feet.
,, ,, 40'	,,	...	3 feet.
,, ,, 50'	,,	...	4 feet.
,, ,, 60'	,,	...	4 feet.

WEIRS.

Bligh—

On sand with a direct overfall, Bligh makes the length of the *impervious* apron downstream

$$= 4 \times \text{co-efficient of percolation for class of bed} \times \sqrt{\frac{\text{Overfall, } i.e. \text{ height from L. W. L. to top of shutters}}{13}}$$

If there is no direct overfall, but a long slope, the height of the crest over L. W. L. should be taken as the "overfall" in the above formula.

The combined depths of the *crest wall and curtain walls* should be 2 × co-efficient of percolation for the class of river bed ; usually there are two walls, so that the depth of each is equal to the co-efficient of percolation, and the second wall is at the end of the impervious section. As the total width of the impervious weir (*i. e.* excluding pervious part) will be : overfall × by co-efficient of percolation for class of river bed ; there will remain overfall × by co-efficient $-4c$-$2c$ as width upstream : where c = co-efficient of percolation, *i. e.* length of ...orced percolation gradient. If overfall be ten feet, the width ... impervious apron upstream will be $10 \times c - 4c - 2c = 4c$. ...ostream, stone pitching and clay puddle of a total thickness ... should be used. The length of the upstream apron ...sured from the downstream toe of the overfall-wall. ... the *pervious* apron + length of impervious dry ...ld be 10 × co-efficient of percolation.

...*rest wall* =

... wall + depth on crest which brings L. W L. to crest line

...f material in wall

This formula can be used for the thickness of all submerged walls.

A general formula for the total width of the downstream apron is as follows :—

Total width of the apron + talus = 10 × co-efficient of percolation

$$\times \sqrt{\frac{\text{Height of weir crest above L. W. L.}}{10}}$$

$$\times \sqrt[3]{\frac{\text{flood discharge per linear foot of weir}}{75}}$$

For sloping aprons, take $10\frac{1}{2}$ to 11 × co-efficient in above formula instead of 10.

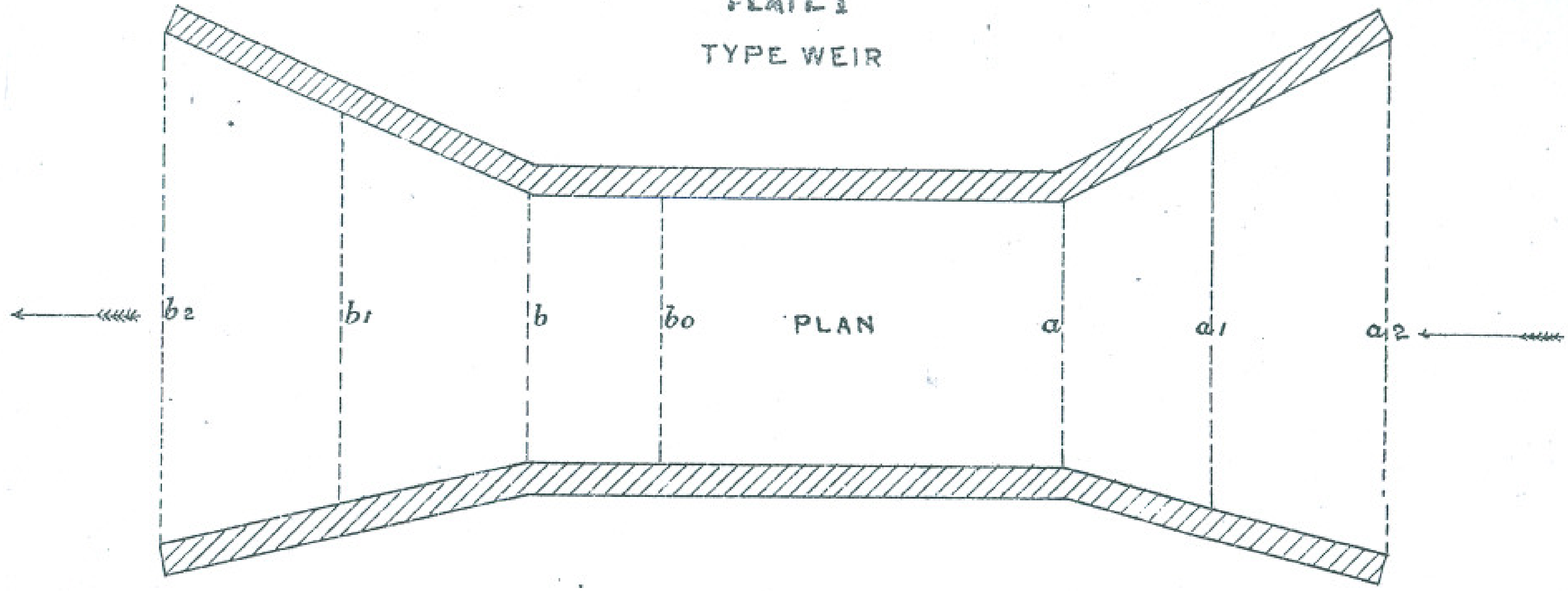
A sloping apron avoids construction in the wet, but by constricting the waterway, the velocity of the overfall is carried beyond the crest, and there is strong action on the talus and river bed. A direct overfall design affords a churning cistern below the overfall, which checks velocity, but puts strong action on the cistern. Bligh recommends reinforced concrete sheet piling (sunk by water jets in sand) in preference to wells or thick walls. Shutters diminish obstruction offered by a weir ; and also prevent formation of shoals upstream.

Mr. R. Egerton Purves, late Chief Engineer, Irrigation Branch, used an adaptation of Kennedy's formula (which is velocity = $\cdot 84 \text{ depth}^{\cdot 64}$) to determine the length of protection required upstream and downstream of a weir. The method is applicable also to the protection necessary below culverts, escapes, etc. Primarily intended for canal works to be constructed upon sand or light soil, its application could be extended to works built on gravel or shingle, once the equilibrium velocities for these materials have been ascertained. Information hitherto obtained points to $3V_0$ as being suitable for gravel, and $5V_0$ for shingle ; but the evidence is not yet conclusive. Coarse sand has an equilibrium velocity of $1{\cdot}3V_0$.

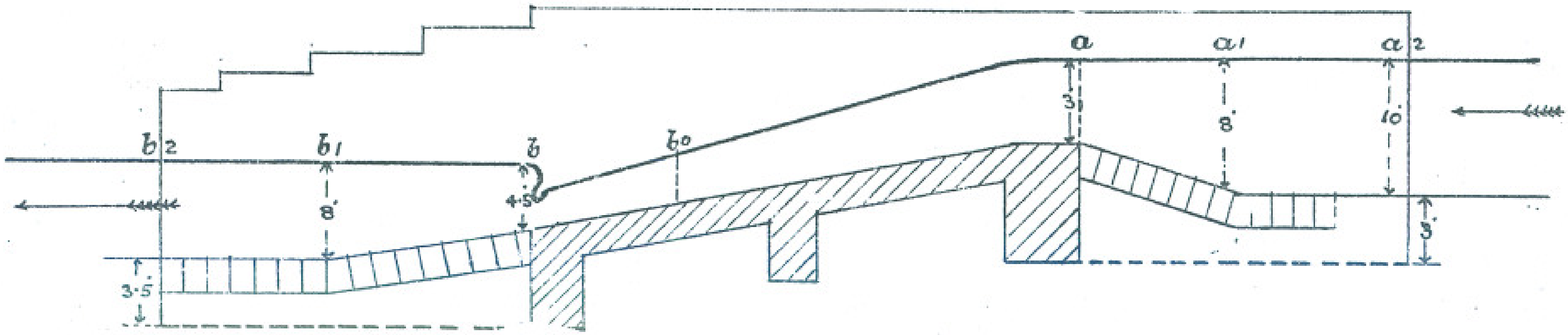
Merriman in his "Elements of Hydraulics" shows that the diameters of submerged materials moved by water, vary as the square of the velocity, and the weight of the bodies moved vary as the sixth power of the velocity. A round pebble about 1·1″ diameter has a weight equal to 10,571 grains of coarse sand,—in other words, the weights are as 1 : 10,571 and the velocities required to move them would be as $\sqrt[6]{1} : \sqrt[6]{10{,}571}$, that is as 1 : 4·69. Hence as $1{\cdot}3\ V_0$ is the equilibrium velocity for coarse sand, $6{\cdot}1V_0$ would be the equilibrium velocity for shingle 1·1″ diameter.

PLATE I

TYPE WEIR



SECTION



Attached are two tables with details of some experiments with sand and rounded pebbles made by the writer. Both pebbles and sand were obtained from the same tract. The pebbles were rounded down to spheres by means of chisels and files. The sand was graded into sizes by means of wire screens. The weights tabulated were taken in still air.

In the case of ordinary soils Kennedy limits the equilibrium principles to $3\frac{1}{2}$ f. s. and depth 9 ft. because $3\frac{1}{2}$ f. s. is the maximum velocity which ordinary soils can stand. Similarly in the case of shingle about 1·1″ diameter there must be a limiting depth and velocity. These appear to be depth 2 ft. and velocity $6\frac{1}{2}$ f. s. Experiments made upon isolated particles, placed in rough wooden troughs, do not approximate to natural condition, hence the results given in Buckley's *Pocket Book** are found to be too low in practice.

To find the depth to which protection of a bed is necessary, on sand or light soil, upstream and downstream of a weir, Kennedy's equation is :—Equilibrium velocity $= \cdot 84$ depth $^{\cdot 64}$

The first expression ·84 varies as follows :—

·82 for light silt.
·90 for coarser light silt.
·99 for sandy loam.
1·07 for coarse silt.

This last gives values nearly equivalent to $1{\cdot}3\ V_0$.

Taking a weir of shape shewn in Plate 1 with narrowing approach, and expanding outfall, calculate the sectional areas at each of the cross-sections a, a_2, b, b_1, b_2, and divide them into the known discharge. This will give the approximate mean velocity (neglecting co-efficients for simplicity). For instance suppose that at cross-section a, the mean velocity $= 3{\cdot}5$ f. s. for a depth of seven feet. We see from Kennedy's table of minimum velocities for equilibrium that there will be scour. Try a depth of eight feet ; if this gives a velocity of three feet, there will be no scour, but a short length of dry pitching may be given there for safety. This point of no-scour should be at a suitable distance from the weir crest *i. e.* such as will give an upstream slope which is not too abrupt.

Below weir.—Similarly by calculating mean velocities at b, b_1, and b_2 the nature of the protection required can be adjudged and the position of the no-scour point determined.

Flank walls.—These are founded well below the no-scour line, usually 3′ to 5′ below it, unless the nature of the sub-soil demands well foundations.

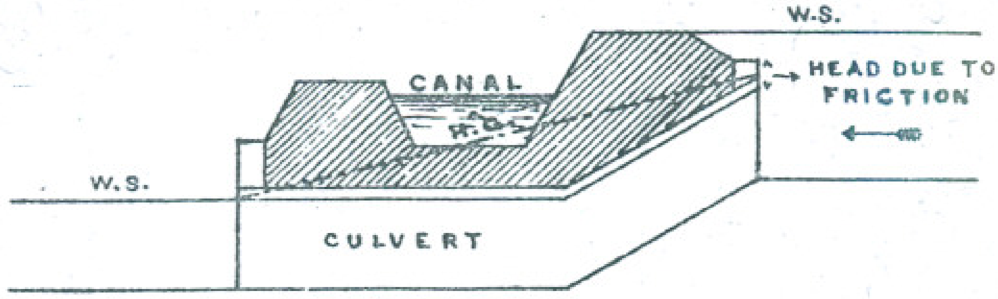
* Page 20.

TABLE I.

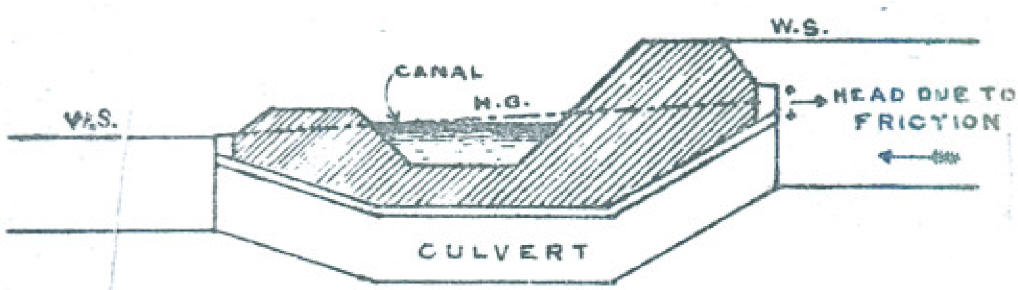
RESULT OF AUTHOR'S EXPERIMENTS WITH PEBBLES.

Diameter of pebble.	Kind of stone.	Weight in troy grains.	Equivalent number of grains of sand to equal weight of pebble.	Equilibrium velocity by the V^6 rule.
$1\,{}^{1}/_{32}'' = 1{\cdot}03125''$ ...	Ordinary blue pebble ...	386 grs. ...		Average equilibrium velocity $6{\cdot}1\ V_o$ if equilibrium velocity of sand No. III below be taken as V_o.
$1\,{}^{1}/_{32}'' = 1{\cdot}03125''$ ...	Ordinary white pebble	495 grs. ...		
$1\,{}^{5}/_{32}'' = 1{\cdot}15625''$ ...	Ordinary white pebble	554 grs. ...		
$1\,{}^{6}/_{32}'' = 1{\cdot}18750''$ ...	A brown pebble ...	637 grs. ...		
Average 1 10156″ ...	Average of mixed pebbles	518 grs. ...	2,910 of No. 1 of Table II 10,571 of No. 2 ,, ... 51,800 of No. 3 ,, ...	$\sqrt[6]{1} : \sqrt{2910}$ that is as $1 : 3{\cdot}60$ $\sqrt[6]{1} : \sqrt[6]{10571}$,, $1 : 4{\cdot}69$ $\sqrt{1} : \sqrt[6]{51800}$,, $1 : 6{\cdot}10$ } $6{\cdot}1\ V_o$.

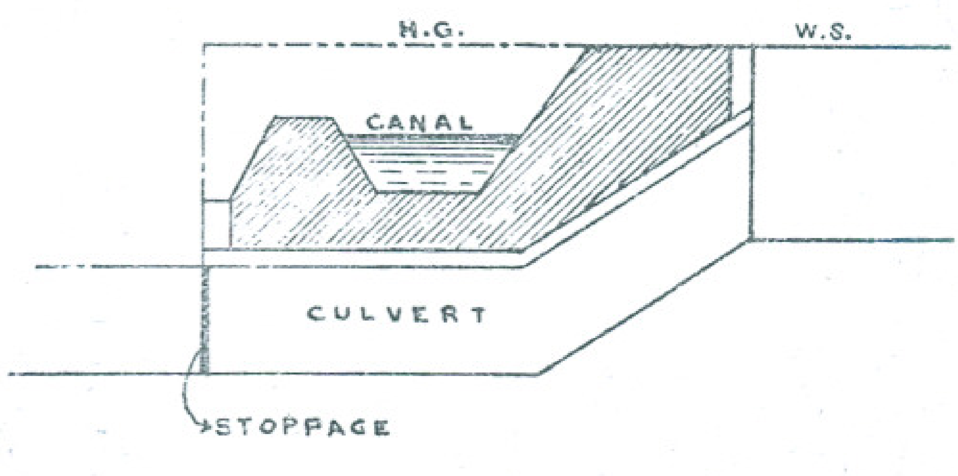
CASE I



CASE II



CASE III



MR. SCHÖNEMANN interposed a correction on a point of fact. The value of 0·031, which had been obtained for Kutter's N on the Western Jumna Canal, had been obtained in the thirty mile reach between Dadupur and Indri, where the soil was a soft sandy alluvium, and where the mean velocity did not exceed three feet per second. Mr. Wadley was mistaken in supposing that the soil was of boulders and gravel, or that the mean velocity anged from four to ten feet per second.

In the case of the Upper Bari Doab Canal the portion eferred to as having an average value of 0·027 for Kutter's N as the twenty-five mile reach between Tibri and Aliwal, where ie bed was of coarse sand in perfectly good regime, though the sides of the channel were eroded.

It was absurd to compare these portions of these canals, (with their mean velocities of three or four feet per second, and with gradients flatter than 1 in 4000,) in point of irregularity and value of Kutter's N, with the mountain torrents dealt with by Mr. Wadley, whose mean velocities were alleged by him to be upwards of ten feet per second, and whose declivities ranged from about 1 in 500 to about 1 in 50.

Mr. Wadley in his rejoinder had quoted fresh figures for Kutter's N on certain canals, which ranged from 0·020 to 0·035 ; but these referred to cases in no way comparable to those of the mountain torrents which he had credited with run-offs ranging up to 2,400 cusecs per square mile of catchment area. In Kutter's original treatise many cases had been cited of channels whose value for N ranged from 0·040 to 0·060 ; and in the article on Hydraulics in the Encyclopædia Brittanica 0·050 was given as the value for "torrential streams encumbered with detritus." Mr. Wadley's hill torrents came under this description, and he had given no good reason why he had rejected this higher value for Kutter's N, and selected in preference values so low as 0·0275 and 0·030.

Mr. Wadley had said that his values of 0·0275 and 0·030 had been "deduced from observed discharges, which was the only safe method of using this co-efficient ;" but Mr. Schönemann's point was that the co-efficient had not been deduced from the discharges, but the discharges from the co-efficient! Certain flood marks had been observed, and certain channel sections measured up to the flood marks ; and then discharges had been inferred from these data through the medium of unwarrantable assumptions as to the value of Kutter's N.

Before the engineering profession could be asked to believe in these extraordinary allegations of run-off per square mile of catchment, the data of the alleged observations should be displayed and submitted to criticism.

Mr. WADLEY in reply to Mr. Schönemann's further objection with regard to the value of Kutter's N, which had been suggested as suitable for sandy torrents, felt the subject was so complex that it would be unsafe to take a very high value of N for general use, even if in one or two isolated instances a high value had been obtained by actual observation. Actual observation of discharges was the only safe course. A perusal of Parker's "Control of Water," pages 471 to 478, was invited.

Where low velocities were concerned, the value of N was more influenced by weeds, slime, and jungle, than by the nature of the surface forming the channel. Thus surfaces of brick, stone, cement, and iron were apt to give the same value for N if there was a growth of slime. In channels in earth, and in all channels where the velocity was such as permitted silt to line the bed and slopes, the silt had the effect of reducing Kutter's N, provided the velocity had not been sufficient to produce silt waves in the bed.

Where high velocities were concerned, the value of N was influenced in the long run by the susceptibility of the containing surface to erosion. If the surface did not remain jagged, but was worn down to smoothness, the value of N would not be nearly so high as if a hard, rugged surface persisted, defying erosion. In the Central Provinces some of the rocks wore down to a smoothness resembling polishing.

If the velocity was sufficient to cause waves of silt along the bed, the smoothening effect of silt was lost, and it became an obstruction, which instead of reducing the value of N, tended to increase it; while these silt waves, if large, also caused waves in the surface of the water, thus testifying to their obstructing effect. Parker* mentions 0·020 as the highest value likely to be obtained in a regular channel having its bed and sides completely covered with a smooth lining of silt, and states that N may rise to 0·027 if the deposit of silt is copious enough to form into silt waves.

*Control of Water, page 478.

Mr. Schönemann had obtained ·031 as the value of N. on the main line of the Western Jumna Canal, and had now made it clear that he was referring to the reach from Dadupur to Indri. A reference to his own remarks, regarding velocities of four to ten feet a second with a bed varying from boulders to gravel, would show that he was writing of the first eleven miles of the canal, *viz.*, from Tajawala to Jaidhri. He could not agree with Mr. Schönemann that the reach of the canal upon which he experimented was in equable flow. Its sectional area varied considerably, and it received several streams of natural drainage during the rains. The silt brought down by these, and the volumes of silt passed into the canal from the Somb torrent at Dadupur, or brought down from the Jumna, must produce results which were quite out of keeping with the usual conditions of equable flow, and must bring things more into line with the conditions prevailing in sandy torrents. Silt waves or terraces two feet deep could be seen in places in the canal bed at certain times of the year during closures and, as stated by Parker, these would increase the value of N. The extremely *local* character of the value of N obtained from observations in a channel of variable flow had also to be taken into account.

Parker gives the following values of Kutter's N :—

	Value of 'N'.	Authority.
Channels in order, below the average ...	·0275	Jackson.
Channels in bad order ...	·03	Kutter.
Channels in very bad order ...	·035	Kutter.
Channels of worst possible character with turbulent flow and large obstructions ...	·04	Jackson.

Gibson at page 294 of "Hydraulics and its Applications" marks "when badly choked with weeds the value of N in utter's formula might become much greater than ·035." onsequently it was evident that slime, weeds, grasses, silt waves nd rugged surfaces produced more effect in increasing

N than moderate quantities of sand or silt alone.

The torrents on the Upper Swat demonstrated that they were able to cause many of the culverts, which had been designed for them, to run full bore, and that was the best test of the correctness of the fixed values of run-off taken.

In the Central Provinces, the run-off used for catchments up to one square mile, varied from $1\frac{1}{2}$ to 3 cusecs per acre, according to the slope of the catchment, and the extent to which it was covered with jungle. These values gave a run-off varying from 960 to 1,920 cusecs per square mile. For larger catchments, the run-off was taken as $1,400\ M^{\frac{3}{4}}$. Thus for a catchment of eight square miles, the run-off would be 6,660 cusecs, while on the Upper Swat Canal it would have been $\frac{2}{3} \times 8 \times 1,500 = 8,000$ cusecs*. There was thus no great disparity between the results.

In fixing values for the run-off from catchments, it was necessary to allow a good margin for safety, if the values were to be applied to a series of works (as was generally the case), and to take into account the records of torrential bursts of rain. It was mentioned that eleven inches of rain had fallen at Hamzakote in five hours. During 1915, a burst of fourteen inches of rain in twenty-four hours was registered at Jubbulpore in the Central Provinces, and the records show even heavier showers at other towns during the last forty years. In the face of downpours of this kind it would be rash not to allow a good margin for safety. In 1910 a rainfall of fifteen inches was recorded in twenty-four hours at the Kurud tank in the Central Provinces (the maximum ever recorded being 16·62 inches). There was an automatic gauge on the waste weir, and from the diagram of the gauge a maximum discharge of 5,520 cusecs was obtained.† The catchment area was 5·7 square miles, the slopes of the catchment were moderate, and bare. A steeper catchment would have given a higher flood. He had seen the catchment, and it corresponded in flatness, and distance from the hills, to those for which half values were taken on the Upper Swat Canal. A maximum discharge of 4,988 cusecs would have been allowed for on the Upper Swat Canal. The discharge, by Dickens' formula, with a co-efficient of 1,400 as used in the Central Provinces, would have been 5,161 cusecs, which was very close to the Upper Swat Canal result. Although

* *Vide* page 170.

† *Vide* Appendix II, pages 32 and 33 of "A General Theory of the Storage Capacity and Flood Regulation of Reservoirs" by Captain Garrett, R. E.

the total annual rainfall in the Central Provinces was greater than that in the North-West Frontier, the intensity of showers was not in excess of those experienced on the North-West Frontier, and it is upon the maximum rainfall in twenty-four hours, or upon the intensity of maximum showers (whichever was the greater) that designs for masonry works are based. The total annual rainfall did not help.

As stated in his paper, the high values of 2,400 and 2,CCO, &c. cusecs per square mile were merely applied to very steep and small catchments in the hills. The discharges ed, after torrential rain, in the "Cusecs Nala" and the of ela Nala first gave warning that allowance must be made d phenominal run-offs where such catchments were s erned. The results were subsequently confirmed by the discharges observed in other nalas, and a margin of about twenty-five per cent. was wisely added by the Chief Engineer for safety. The hills are very steep and very bare. The corresponding value in use in the Central Provinces where the hills are wooded is 1,920 cusecs per square mile. For catchments not in the hills, two-thirds and half the higher values were taken, according to the distance from the hills at which the catchments were situated.

It was recognised that in addition to the intensity and distribution of showers, the width and length of a catchment, its slope, the extent to which it was covered with foliage, the nature of the materials composing its surface, and the extent to which these were saturated at the time, were factors affecting the run-off. For accurate designing, a series of co-efficients for each of these factors would need to be scientifically determined, and applied with proper discrimination; but as there is no research section in the Public Works Department, a table of observed run-off values, which would approximately embrace all conditions and be reasonably safe, had to be found and applied to all cases. Some of these it fitted closely, in others there was a margin. The conditions, which affect the run-off, varied so greatly, however, that there was no other course.

On page 202 of his "Irrigation Pocket Book" Buckley uotes a flow off of 1,290 cusecs per square mile from a catchment f 4·4 square miles on the Koregaon tank. This is in excess f that arrived at by the formula 1,400 $M^{\frac{3}{4}}$ which gives 4,249 usecs, or 966 cusecs per square mile. Amongst the examples iven by Buckley, this was the smallest catchment mentioned, and the result points in the same direction as those obtained on the Upper Swat Canal, *viz.*, that as the catchment decreased the flow-off per square mile increased and needed specia treatment.

As a matter of fact Kutter's N was very little used on the Upper Swat Canal. The values for the run-offs from catchments were fixed by actual observation plus a margin for safety. But such values of N as ·0275 or ·03 approximated closely, in most cases, to the discharges fixed, where the beds of nalas were not rocky, but composed of sand.

The following interesting information on the subject Kutter's N and the run-off from hilly catchments is abstracte from Engineering News, dated 10th February 1916, page. 272 to 275 :—

	Nature of channel.	Value of 'N' obtained.
San Gabriel River.	Banks composed of granite boulders 4" to 24" diameter. Bed of sand and gravel with rocks up to 3" diameter, small eddies in surface of water.	·028, approximate value ·03.
	West bank of boulders 2" to 12" diameter, average diameter 4". East bank of boulders 3" to 36" diameter, average 12". Bed of boulders 9" to 24" diameter. Waves in surface of water.	·0354, approximate value ·035.
	Both banks of granite boulders 4" to 24" diameter, numerous large rocks in water. Bed of boulders 6" to 24" diameter. Surface of water full of eddies and waves.	·0416, approximate ·04.

Nature of channel.	Width in feet.	W. P.	Surface slope.	C.	Discharge in cusecs.	Cross sectional area in square feet.	H. M. R.	Kutter's 'N'.	Velocity in feet per second.	Catchment area in square miles.	Run-off in cusecs per square mile.	Location of catchment.
Los Angeles River. Sandy with piles of debris in stream.	267	275	·0035	59	24,420	2,380	8·65	·0375	10·27	334·8	73·2	242 sq. miles in mountains, 92·8 miles in valley.
Arroyo Seco. River. Bed rough, with coarse sand and boulders.	163	169	·009	45	7,610	810	4·84	·045	9·4	42·7	178	30 sq. miles in mountains, 12·7 miles in valley.
Big Tejunga River. Channel rough, strewn with boulders; one bank rock.	240	246·5	·0124	52·5	13,600	1,068	4·74	·0375	12·73	118	115	All in mountains.
Santa Anita Wash.— Banks low and lined with brush. Bed rough with large boulders.	78·5	79·5	·0142	42·0	3,168	312·2	4·002	·045	10·02	18·17	174	All in mountains.
San Gabriel River. Bed rough, with gravel and boulders, 12″ to 18″ diameter. Banks of light soil.	383	386	·0079	58	*26,680	2,178·2	5·64	·035	12 25	228·68	117	All in mountains.

*47,000 in 1884 from 220 square miles.

The average rainfall at Los Angeles for thirty-eight yea is 15·81". Most of it falls in December to March, *viz.*, 3·16" i January, and 3·17" in February; but the rainfall goes up to 38·18 and 34·84". In 1914, a rainfall of 23" produced great floods, wit 10·35" of rain in January and 7·04" in February. The Lo Angeles rain gauge, however, gives no indication of rainfall in th hills. The smallest streams showed run-offs of 300 to over 700 cusecs per square mile. If twenty-five per cent. be added t the run-offs for safety, we get 375 to over 875 cusecs per squar mile, and a reference to the catchment areas shown in the statemer indicates that the engineers were not dealing with such sma catchments as 0 to $2\frac{1}{2}$ square miles or even 0 to 5 square miles The smallest catchment mentioned is one of 18·17 square miles, and none of the figures for run-off, etc., relate to the maximum floods, when the engineers were not able to get near the rivers.

Applying a run-off value of 800 cusecs per square mile to a catchment of ten square miles we get 8,000 cusecs ; applying Dickens' formula $1,400 \times 10^{\frac{3}{4}}$ we get 7,874 cusecs ; applying the $\frac{2}{3}$rds values for a catchment of 10 to 15 square miles, as given in the Upper Swat Canal table, we get 8,340 cusecs.

The engineers at work on the San Gabriel and Los Angeles rivers thus appear to be arriving at approximately the same values for Kutter's N, and for the run-offs from steep catchments, as those obtained in the Punjab and Central Provinces under similar conditions. They had not yet given run-off values for very small catchments however, nor Kutter's 'N' for sandy torrents.

Erratum.

Page 165, lines 12 and 13 from bottom. *Read* :—

Dickens' formula.

Discharge = co-efficient × (square miles of catchment)$^{\frac{3}{4}}$

TABLE II.
RESULTS OF AUTHOR'S EXPERIMENTS WITH SAND.

Grade of sand.	Size of sand compared with common seeds.	Weight of 500 grains of the sand.	Equivalent number of grains of sand.	Equilibrium velocity by the V^6 rule.
(I) Passed by mesh 8 and rejected by mesh 12.	Larger than bajra but smaller than mot seeds.	89 grs. ...	17·8 grains of No. III below weigh as much as one grain of No. I.	$\sqrt{1} : \sqrt{17{\cdot}8}$ about 1·6 V_o.
(II) Passed by mesh 12 and rejected by mesh 20.	The size of sarson seeds	24½ grs. ...	4·9 grains of No. III below weigh as much as one grain of No. II.	$\sqrt[6]{1} : \sqrt{4{\cdot}9}$ about 1·3 V_o.
(III) Passed by mesh 20.	The size of mustard or poppy seeds.	5 grs. ...	Standard for comparison.	V_o.

FALLS.

Bligh.—

In canals strict economy, with efficiency, is necessary ow to the number of falls in rapidly sloping ground. Former id of the destructive effect of a mass of falling water have be proved to be erroneous. The ogee fall was designed to div destructive action in a horizontal direction, and this it does, b with most injurious effects on the bed and banks below the fall.

Crest walls are raised above the upstream bed to restra the velocity of an overfall. Using the formula for free overf

Discharge $= 3{\cdot}33\, d^{\frac{3}{2}}$ where $d =$ depth of film on crest ; but the notch is a better device, as it suits all levels of supply. The length of such a crest should be seven-eighths, if not equal to, the width of the upstream bed. The top width of a notch may vary from three-quarters to the full depth of the water passing through it, while the thickness of the notch piers should not be less than half the depth, and more if a superstructure is carried. The top length of these piers may be half the height. The proper shape for a notch to discharge the proper amount at every change of water level would be ovoidal or egg-shaped.

The ordinary formula for the base width of a fall dam is:

$$\text{Base width} = \frac{\text{Height of dam} + \text{depth on crest}}{\sqrt{\text{S. G. of material}}}$$

and if the weir is on sand, this base width must be taken ; but in canal falls, tank escape weirs, and such works, subjected to fixed and moderate depths, and founded in solid clay and backed by puddle, a less width will suffice, as below—

$$\text{Base width} = \frac{\text{Height of dam} + \text{depth of crest}}{\text{S. G. of material}}$$

$$\text{Top width} = \text{width required for notches, } i.e., \frac{\text{depth on crest}}{2}$$

Floor.—

When a sunk water cushion is not provided the thickness of the floor should be $\sqrt{\text{Height of drop wall} + \text{depth of film}}$, and this is considered sufficient with clay foundations. The length of the floor may equal twice the height of the drop wall plus twice the depth of the film, and terminate in a shallow curtain wall, and be protected by pitching beyond. The floor should be somewhat wider than the bed downstream.

Bligh advocates wingless fal' ..ithout cisterns, if there are hes. He also thinks it is false economy to combine bridges falls.

When a fall is on *sand*, use a co-efficient of 15, and take equal to the drop. Thus if the drop be 7 ft. the total gth of the floor will be 15 × 7 ft. = 105 ft. (*i. e.*, including stream protection, if impervious.)

he thickness of the floor would be $\frac{4}{3} \cdot \frac{\text{Head—hea}\ldots\text{st in creep}}{\text{S. G. of material—1}}$

For a drop wall on sand, take the base width equal to

$$\frac{\text{Drop} + \text{depth of film}}{\sqrt{\text{S. G. of material}}}$$

Small falls are more economical than large ones.

Cisterns.—

An empirical rule for the depth of a cistern is

$$\text{Depth} = \frac{\text{Depth on crest} + \text{height of fall}}{3}$$

If the fall is incomplete, a low level arch and diaphragm wall, constructed downstream, will allay surging.

Another rule for the depth of the cistern in canal falls is given in Love's *Hydraulics*.

$$\text{Depth of cistern} = 1{\cdot}5\sqrt{\text{height of fall}} \times \sqrt[3]{\text{depth on crest}}$$

The height of the fall is the difference of level between water surfaces, and by the depth on crest is meant the F. S. D. upstream.

The minimum length of a cistern determined from the ordinate to the curve of falling water is :—

$$2 \times \sqrt{\text{depth on crest (including approach velocity head)} \times \text{surface fall}} \quad \text{for small falls.}$$

$$2\tfrac{1}{2} \times \sqrt{\text{depth on crest (including approach velocity head)} \times \text{surface fall}} \quad \text{for large falls.}$$

Dyas' formula.

Total depth of water cushion = F. S. D. upstream $+ \sqrt[3]{\text{F. S. D. upstream}} \times \sqrt{\text{fall between water surfaces}}$.

The depth of F. S. depth downstream is deducted the result to find the actual depth of the cistern required.

When the F. S. depth downstream is the same as the F depth upstream, the depth of the cistern should be $\sqrt[3]{F. S} \times \sqrt{fall}$. The length of the cistern will then be $2\frac{1}{2}$ to 3 t the depth in small falls, and 3 to 4 times the depth in large fa

Cascade ordinate, i.e., horizontal distance from face wal curve of falling water $= \frac{4}{3}\sqrt{F. S. D. \text{ upstream} \times \text{height of fall}}$

RETAINING WALLS.

The horizontal thrust of earth at one-third the height a retaining wall from the base is given by Coulomb's form

$$\text{Horizontal thrust} = \frac{\text{Unit weight of earth} \times \text{height}^2}{2}$$

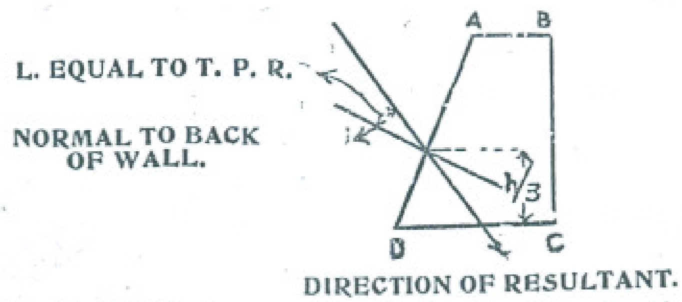
$$\times \tan^2\left(45^\circ - \frac{\text{angle of repose.}}{2}\right)$$

This gives the same result as Rankine's formula.

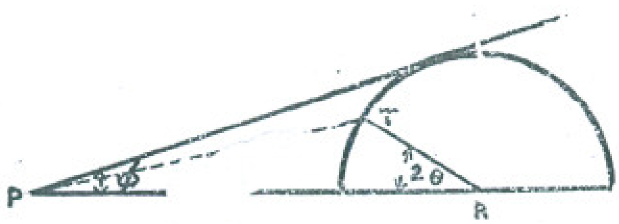
Horizontal thrust $= \frac{1}{2}$ unit of earth weight $\times$ height^2 $\times$

$$\frac{1 - \sin \angle \text{ of repose,}}{1 + \sin \angle \text{ of repose.}}$$

Prof. Reilly's graphic method takes both the vertical an the horizontal thrust of the earth into consideration, and is follows :—



A. B. C. D. is a wall with inclined back, the angle of inclination of back with vertical being Θ. Set out ϕ the (L. of

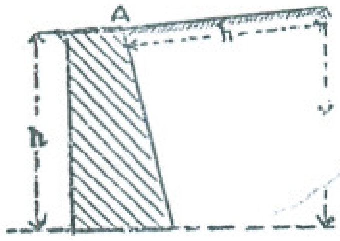


repose of the material behind the wall) and describe a semicircle to touch the upper line ; then set off an angle of 2Θ at the centre of the circle and join P. T.

The resultant pressure $= \frac{1}{2}$ unit weight of earth $\times \frac{\text{P. T.}}{\text{P. R.}} \times h^2$

T. P. R. = the angle which the resultant pressure makes the normal to the back of the wall.

Molesworth's rule for a surcharged retaining wall is to lay off



om A, a length along the surcharging slope equal to h. Then ing Y as the height in the $\frac{h}{3}$ rule, we get the following values :

Y = 1·71 h	for	1 to 7	slope of surcharge.
,, = 1·55 h	,,	$1\frac{1}{2}$ to 1	,, ,,
,, = 1·45 h	,,	2 to 1	,, ,,
,, = 1·31 h	,,	3 to 1	,, ,,
,, = 1·24 h	,,	4 to 1	,, ,,

Love adopts the same rule.

Woods, in his "Theory of Structures" gives the following formula for surcharged walls.

If the inclination of the surcharge to the horizontal equals the angle of repose of the soil, as is usually the case, then the resultant pressure

$$= \frac{\text{unit weight of earth} \times \text{height of wall}^2 \times \text{cos. L. of repose}}{2}.$$

If the inclination of the surcharge is not the same, but less than the angle of repose, then the resultant pressure $= \frac{\text{unit weight earth} \times \text{height}^2}{2} \times$ cos. L of surcharge

$$\times \frac{\text{cos. L of surcharge} - \sqrt{\text{cos.}^2 \text{ L of surcharge} - \text{cos.}^2 \text{ L of repose}}}{\text{cos. L of surcharge} + \sqrt{\text{cos.}^2 \text{ L of surcharge} - \text{cos.}^2 \text{ L of repose}}}$$

Molesworth uses the same formula.*

* Pocket Book, 27th edition, page 86.

Weep holes, with suitable filters of shingle in rear of them are sometimes forgotten. Their omission renders the wall liable to hydrostatic pressure, which is much greater than that of the earth.

An interesting case of a long retaining wall founded upon wet sand occurred in the design of the canal headworks of the Lower Bari Doab Canal at Balloki. An angle of repose of 15° was taken for the earth behind the foundations of the wall, and the problem was to find the depth and width of foundations required under such circumstances. The earth above the foundations was taken as having a repose angle of 30°. The danger to be averted was the rotation of the whole wall and its foundation in such an unresisting medium. The abutting power of the wet sand in front of the wall, the pressure on the base of the foundations, and the earth pressure behind the wall were considered. Skin friction was also taken into account. The depth of foundations required was ascertained by Rankine's rule.

Maximum intensity of pressure = Unit weight of earth × height from founds to top of wall × $\left(\frac{1 - \text{sin. L. of repose}}{1 + \text{sin. L. of repose.}}\right)^2$

The results gave foundations 18 feet deep and a base 16 feet wide. Wells were used and the outer toe of the retaining wall proper was corbelled out to distribute the pressure over the wide base.

REGULATORS.

Bligh.—Regulators must be capable of withstanding the hydrostatic pressure of the maximum river flood with the canal closed. The waterway should be sufficient to supply the canal without any increase of velocity at the entry, except when the sill is raised. The gates should be made as large as the high water pressure will admit, and the sill raised to exclude sand, and to tap the surface water. A still pool and scouring pocket should be provided in front of the regulator, and the pier noses projected as little as possible.

When foundations are on *solid clay* the hydrostatic pressure does not go much beyond the beginning of the floor, as there is no outlet for percolation.

When the head is great, as for example the 38 feet at the Betwa Canal head, an arched screen-wall, perforated with sets of vents at different levels, should be used in front and closed by gates ; or else " Stoney " gates should be used, or the vents can be put in the regulator itself in series one above the other. An angle of 60° is recommended as most suitable for skew head regulators. If the floor is on clay, the thickness will equal $\sqrt{\text{full static head}}$, and the length of the floor will equal twice the full static head. Piers in regulators should be ·4 span as a rule, if the head of water equals or exceeds twenty feet, but ·3 span if the head is less.

SLUICES.

Bligh.—In scouring sluices the sill should, if possible, be placed at low water level and should be capable of discharging more than the average dry season discharge of the river. The canal sill should be several feet higher. The divide wall should be advanced well beyond the canal head, and finished in a rounded end.

The arches and platforms of sluices should be built clear of the maximum flood, and the openings made as large and unobstructed as possible to afford high silt-scouring velocity. The end sluice should be in line with the canal regulator face. Sluices are not intended to hold up the full flood of a river, but only sufficiently to force the supply into the canal. If the velocity is so great as to overstrain the flooring, the lower gates should be kept down.* The thickness of the floor can be reduced if on boulders and clay, as the co-efficient of percolation may be 9. The critical point for the thickness of a floor is just below the bridgeway. A floor should be thick at the root and tapering to the end as the hydrostatic pressure diminishes.

A work built on boulder formation cannot be considered free from hydrostatic pressure, as the co-efficient of percolation varies from 5 to 9.

The thickness of the floors of scouring sluices founded on *sand* must be designed to withstand dynamic as well as hydrostatic action : for the hydrostatic pressure a thickness of

$$\frac{4}{3}\ \frac{\text{Head to H. G. line}}{\text{S. G.} - 1}\ \text{would apply,}$$

but as a heavy floor is necessary for dynamic action the following empirical formula is used

$$\text{Thickness of floor} = \sqrt{\frac{3 \times \text{total static head}}{2}}$$

* This sounds risky unless lower gates are of moderate height.—A. J. W.

FLOORS.

Bligh.

The thickness of floors in *weirs on sand* depends on the hydrostatic pressure, and the latter depends on the upstream head and the nature of the river bed material. Floors are liable to failure by :—

(*i*) dynamic action of floods, due to velocity and debris ;

(*ii*) piping beneath foundations, due to upstream head and blowing up pressure.

A heavy impervious weight is required on sand up to a certain point, but the hydrostatic pressure must have a free outlet *below* the impervious part, and the exit velocity below this must not be sufficient to wash out the particles of sand composing the foundations, the enforced length of percolation must be sufficient to prevent this.

The following relation must at least obtain between the upstream head and the length of enforced percolation :—

For light silt (Nile silt)	1/18
For fine sand as in Himalayan rivers ...	1/15
For coarse sand as in C. P. rivers ...	1/12
For boulders, shingle and gravel, mixed with sand	1/5 to 1/9

To the above lengths, measured to include sheets of piling and curtain walls, add fifty per cent. for safety, but make the added length *pervious*. To count in the percolation length, curtains should be at least twice their depth apart.

The end thickness of a floor should not be less than $3\frac{1}{2}$ ft., at other places the thickness should be determined by the amount of upward (hydrostatic) pressure to be resisted. The weight of masonry should everywhere be one-third more than the upward pressure, and the masonry should be regarded as under flotation (*i. e.*, immersed) unless low water level is clear of the floor ; the surface of the floor should be strong enough to withstand dynamic effects.

A certain length of solid apron is necessary upstream to protect the curtain from erosion, and this gives the hydraulic gradient a more advantageous starting point. It must however be impervious and have a watertight connection with the crest wall.

REGULATING BRIDGES AND ESCAPES.

Bligh.

A regulating bridge is usually necessary below escapes. The maximum head of water dealt with is usually moderate. The springing should be placed just above F. S. L., and ten foot spans* used.

The length of the masonry floor should equal three times the head or F. S. D. upstream (excluding the upstream floor as far as the grooves).

The thickness of the floor $= \sqrt{\text{Head}} + \frac{\text{span}}{10}$, if the founds are firm clay. One-tenth of the span has been added in this formula to give the floor sufficient thickness to distribute the weight of the piers. In pitching, two feet free board should be given, and the pitching of the slope finished at an angle of 45°. Berms should be provided at the toe where the side pitching extends beyond the bed pitching. Vertical wings are the least economical.

Escapes.

All escapes should be combined with a free fall if possible in order to reduce the width of the waterway, and the escape put as close as possible to a drainage to diminish the length of the channel. Aqueducts can be used as escapes, but usually a siphon is required beneath to form a cistern for escapage.

If the escape is intended to scour the canal bed free of silt, it should be designed to cause a velocity of about five feet a second—the depth on the crest being calculated for that velocity. The crest of an escape should be flush with the canal bed, and the bed and sides of the canal above the escape should be pitched, large blocks being used near the escape.

OUTFALLS.

Depth of scour in outfalls.

When C. V. R. $= 1{\cdot}3\,V_o$ the total depth of flow "D" required to secure "no-scour" conditions is given by the formula :—

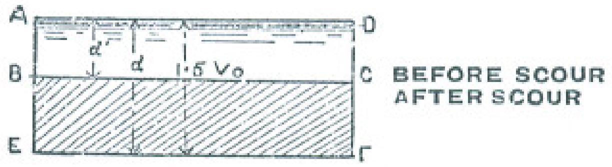
$$D^{1\cdot64} = \frac{\text{Velocity at exit of barrels} \times \text{normal depth of flow in outfall}}{\cdot84 \times 1\cdot3}$$

* Twenty foot spans have been used in recent works.

If C. V. R. be taken as equal to $1{\cdot}5\ V_o$ the formula for "D" will become :—

$$D^{1\cdot64} = \frac{\text{Normal depth of flow} \times \text{exit velocity}}{\cdot84 \times 1\cdot5}$$

as shown below :—



Let A B C D be the sectional area (a rectangular section has been taken for the sake of simplicity) *before* scour, and A E F D the sectional area *after* scour. Then since the discharge remains the *same* before and *after* scour, the equation will be as follows :—

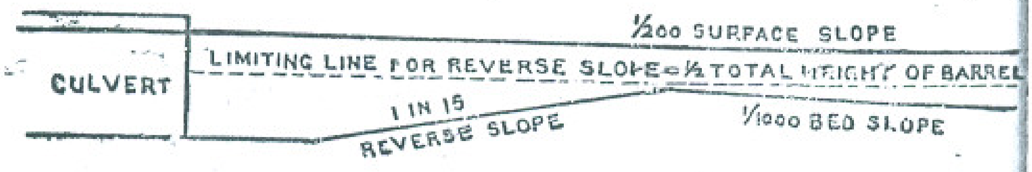
A B C D $\times\ d^1 \times$ V (velocity of exit) = A E F D $\times$ depth $\times\ 1{\cdot}5\ V_o$

To simplify sectional areas, take the bed width as one foot, then $1 \times d^1 \times V = 1 \times d \times 1{\cdot}5\ V_o$, or $d^1 \times V = d \times 1{\cdot}5\ V_o$. But $1{\cdot}5\ V_o = 1{\cdot}5 \times {\cdot}84\ d^{\cdot64}$ (Kennedy). Substituting this value in above equation we get

$$d^1 \times V = d \times 1{\cdot}5 \times {\cdot}84\ d^{\cdot64} = 1{\cdot}5 \times {\cdot}84\ d^{1\cdot64}$$

$$\therefore\ d^{1\cdot64} = \frac{d^1 \times V}{\cdot84 \times 1\cdot5} = \frac{\text{normal depth of flow} \times \text{exit velocity}}{\cdot84 \times 1\cdot5}$$

Raising bed of outfall at exit.—When it is desired to avoid excavating a very deep and expensive outfall, a reverse slope of 1 in 15 may be given in the outfall bed at the exit of the syphon, up to a limit of half the total height of the barrels as sketched below.



Example of designing bed width of outfall and pitching for the same.

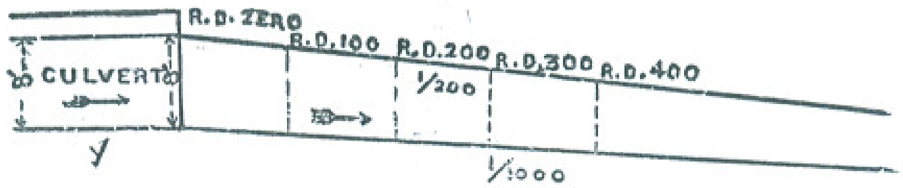
Assume a discharge of 5,000 cusecs.

,, velocity at end of wing walls = 12 F. S.

,, Normal depth of flow at exit = 8 ft.

,, surface slope $\frac{1}{200}$

,, bed slope $\frac{1}{1000}$



Sectional area at exit will be $\frac{5000}{12} = 416{\cdot}6$ square feet and as the depth is eight feet, the bed width will be $\frac{416{\cdot}6}{8} - 10 = 42$ feet.

Supposing it is desired to reduce the velocity to $1{\cdot}5\ V_o$ without either increasing the bed width too suddenly, or reducing the velocity too suddenly.—

Consulting the table of critical velocities in Garrett's Diagrams*, values of V_o are found for various depths, and if these values be multiplied by $1{\cdot}5$, the velocities which correspond to $1{\cdot}5\ V_o$ are obtained.

Thus for a depth of six feet we get $1{\cdot}5\ V_o = 2{\cdot}64 \times 1{\cdot}5 = 3{\cdot}96$ F. S. say 4 feet per second.

Try a mean width of 206 feet and a depth of 6 feet, then the sectional area $= 206 \times 6 = 1{,}236$ square feet and multiplying this by $1{\cdot}5\ V_o$ or 4 F. S. we get $1{,}236 \times 4 = 4{,}944$ cusecs, which is near enough to the desired discharge of 5,000 cusecs.

Using a surface slope of $\frac{1}{200}$ (and neglecting the bed slope as it is small in this case) a decrease in depth from 8 feet to 6 feet will carry the outfall to a length of 400 feet. In that length the velocity will be reduced from 12 feet to 4 feet (which is $1{\cdot}5\ V_o$) or at the rate of 1 F. S. for every $\frac{400}{8} = 50$ feet, which is

* Page 6.

not excessive. The bed width would increase in that length from 42 feet to 200 ft., = 158 ft., or one foot for every $\frac{400}{158} = 2{\cdot}5$ feet and this splay would not be too rapid in a pitched channel.

The depth of scour and thickness of pitching required in the above 400 feet will be as follows :—

At R. D. Zero (*i.e.*, at end of wing walls)

Velocity = 12 feet. Depth = 8 feet.

$$d^{1\cdot64} = \frac{12 \times 8}{\cdot84 \times 1\cdot5} = 76{\cdot}2$$

$$\therefore d = \sqrt[1\cdot64]{76{\cdot}2}$$

$$\log \underline{1{\cdot}64/76{\cdot}2} = \frac{1{\cdot}881816}{1{\cdot}64} = 1{\cdot}1408 \; \therefore \; d = 14 \text{ feet nearly.}$$

As the normal depth of flow equals eight feet the depth of scour which may be expected equals $14 - 8 = 6$ feet.

The width of the berm at the toe of the pitching would be $6' \times 2{\cdot}5' = 15$ feet, and this is $2\frac{1}{2}$ times the depth of the maximum scour ; but in order to comply with the rule for the protection of a curtain wall at the end of a siphon, *viz.*, that the pitching at the exit must be sufficient to cover a slope of 5 to 1 drawn from the end of the curtain to the depth of the scour, it will be necessary to have pitching on the bed of the outfall for a distance of $5 \times 6 = 30$ feet.

At R. D. 100 Velocity = $12 - 2 = 10$ F. S. (the rate o diminution, being 1 F. S. in 50 feet).

Depth of full supply = $8 - \frac{1}{2} = 7{\cdot}5$ (the rate of diminution being two feet in 400 feet).

$$\text{Then } 6^{1\cdot64} = \frac{7{\cdot}5 \times 10}{\cdot84 \times 1\cdot5} = 59{\cdot}5$$

$$d = \sqrt[1\cdot64]{59{\cdot}5}$$

$$\log \underline{1{\cdot}64/59{\cdot}5} = \frac{1{\cdot}774501}{1{\cdot}64} = 1{\cdot}08201$$

$$\therefore d = 12{\cdot}25 \text{ feet.}$$

The normal depth of flow being 7·5 feet the depth of scour at R. D. 100 will be $12{\cdot}25 - 7{\cdot}5 = 4{\cdot}75$ feet, and the width of the berm at the toe of the pitching should be $4{\cdot}75 \times 2{\cdot}5 = 11{\cdot}87$, say 12 feet.