WATER-LOGGING.

(A Hydro Geological Problem.)

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Introduction—Cause of water-logging—Cones of saturation—First traces of water-logging—Direction of percolation flow—Stratigraphy of alluvial soils—Theory of springs—Clay, silt and sand—Remedial measures—Sheet piling—Surface drains—Tube wells.

Introduction.

Water-logging is not a subject requiring many words for its introduction. It has been recognized as an evil in the Punjab since 1841, although little practical progress has been made towards either its prevention or its cure. The reasons for water-logging have been known for a long time, but a study of the earlier records will shew that the literature dealing with the subject tends to move in a circle ; what is now required is the practical application of the theories which have time after time been fully expounded.

Cause of Water-Logging.

Water, entering the ground, percolates downwards until it is absorbed in the water-table, unless its course is checked by meeting an impervious stratum along which it is forced to travel till it finds an outlet. Should, however, hydrostatic pressure be superimposed, in excess of that which can be absorbed by the class and depth of the soil above the impervious stratum, the percolating water will again rise to the surface and ooze out, thereby causing water-logging. Sand, being much less retentive than clay, more readily permits the transmission of any pressure, and consequently subterranean flow follows sandy strata, the greatest flow being along the top of the underlying impervious stratum. If the pervious strata are able to pass the percolating water freely from the point where hydrostatic pressure is superimposed to the outlet, the saturated level should not rise, and water-logging can only take place under such circumstances by the subsoil flow being forced up, either by meeting a saturated clayey belt under a water-course, or by a reduction in the cross sectional area of the pervious strata. Should the medium above an impervious stratum not be able to pass the percolating water, then, since its capacity cannot be increased, percolation must be checked if water-logging is to be prevented.

Cones of Saturation.

Alluvial soils absorb water in different degrees. Channels with three or four feet of clay under their beds will have no appreciable cones of saturation, while a cone of saturation will quickly appear where the soil is sandy and the depth of the stratum is limited.

Two sections on the Upper Jhelum Canal may be taken as examples. The first, at R. D. 359,000, consists of 4·5 feet of black cotton soil over 2·5 feet of sand, 13 feet of clay and 18 feet of sand overlaying another belt of clay. The canal excavation here was 3·16 feet, and as the bed scoured two feet the water was brought into direct connection with the upper stratum of sand, with the result that the water level in a well in the neighbourhood of the canal, which used to be sixty feet below ground, rose to within two feet of the surface after the canal had been running a couple of months. At R. D. 367,000 there was 5·5 feet of soil overlying 6·3 feet of sand, which in turn overlay 15 feet of sandy soil and 6 feet of clay. Digging here was 10·92 feet, and the water-level in a neighbouring well, which in June 1916 was 35 feet below ground, has risen to within 26 feet of the surface and is still rising. These are examples of false and true cones respectively, and indicate how the strata influence the percolation.

The dotted line marking the level of the water-table from R. D. 102,500 to 120,000, along the geological section of the Upper Chenab Canal, reproduced as Plate No. 1, is an interesting illustration of the cascades in a false water-table due to the stratification of the soil.

First Traces of Water-Logging.

On the Upper Jhelum Canal, and between R. D. 10,000 and R. D. 115,000 of the Upper Chenab Canal (Plate No. 1), water-logging has invariably first made itself felt along the drainage lines, and, while the ridges remain dry, the rest of the area traversed by these canals is inclined to become swampy. Water-logging is more intense where alluviation is still in progress, that is to say, where drainages have recently been established ; and is less marked along old consolidated drainage lines, but while the accepted theory that water-logging is caused by interference with the sub-surface flow along the natural drainages of the country is correct, yet, as sometimes further developed, this theory is misleading, since the presence of a saturation cone does not block the passage up to the water-level of the channel to which it is due, but only to the additional

height of the water table caused by the extra percolation from the channel.

A geological consideration of alluvial soils, that is, of their stratigraphy, affords much information about the blocking theory as generally expounded. If the subterranean flow is confined to a water bearing bed between impervious strata, percolation from a canal above the upper stratum will not affect the subterranean flow and there will be no blocking. An overflowing well is another instance. The overflow is due to the source of supply to the water bearing stratum being at a higher level than the ground round the well, while this stratum, from which the well draws its supply, is confined between impervious beds of clay. The result is that the water is forced over the top by hydrostatic pressure.

The explanation as to why low-lying lines of drainage in clayey soils should first shew signs of water-logging, appears to be that the water from below, moistening the clay, causes it to expand, and thereby opening the interstices, permits upward percolation under the influence of the hydrostatic head. Once saturated, the clay, being retentive, refuses to part with its excessive moisture, and lines of drainage being naturally alluvial, with beds of sand underlying the clay, the sand, if connected with a water channel, will continue to feed the clay. The blocking of surface drainage by this means has much to do with causing further water-logging due to pondage, though the idea that cones of saturation are rigid blocks in the way of subterranean flow may be ignored.

Direction of Percolation Flow.

The flow of percolation water is limited and controlled, not only by the porosity of the strata, but also by their inclination, curvature and continuity, so that it does not necessarily follow that the direction of the subterranean flow is in accordance with the surface slopes and flow. It may be entirely different, though generally the underground layers follow the surface slopes.

Stratigraphy of Alluvial Soils.

Water from melting snow, or from cloud bursts in mountain ranges, impoverishes the mountains by scouring away the elements of which they are composed. On reaching the plains this water flows off in different directions, making beds for itself, and also building ridges along its routes. Thé heavier detritus is deposited along the foot hills, and, as the gradients flatten out,

the lighter material is deposited in layers in the plains. As these layers consolidate, rain and sun introduce physical changes. The sand is separated from the clay as it contracts, while the clay expands under the influence of moisture and the soluble clay floats off while the sand settles down. This is the behaviour of the surface alluvium, but as the alternating rain and heat only reach down to a limited depth from the surface, the whole of the alluvium does not entirely rearrange itself, rains being periodical ; but the case where only the layers are directly in contact with the continuous percolation flow is different, change here being more or less complete. A thick clay stratum is invariably found in wells above the Greensand, which is in itself of firm texture, and, owing to the load of alluvium above it, is capable of standing very high upward hydrostatic pressure. Layers, under process of precipitation, during their incomplete condition, grow slushy when wet, and these are the conditions prevailing in the plains with which we are mostly concerned. Adjoining areas in the plains may have a very different stratigraphy if they are fed by different channels, since each channel has its separate zone of influence where the strata may differ in thickness, porosity, dip and soundness, from the strata of the next drainage zone, so that the water-bearing strata in different drainage zones may be at different elevations.

Theory of Springs.

Springs are of two kinds, land springs and deep seated springs. If the surface soil be loose and overlie a retentive substratum, water soaking through from above to the retentive stratum is forced to change its direction and run along this substratum till it reaches a depression in the ground, where it bursts out, forming a land spring. The supply of deep seated springs on the other hand is derived from a saturated, porous soil, enclosed by impervious strata, and fed by rainfall or a river or channel flowing at a higher level.

The distinction between the two is that the one runs above the impervious stratum, and the other below it, though perfect, deep-seated springs ordinarily emerge from a depth of many hundred feet. At Lucknow, water under pressure was struck at 158 feet below ground and the water rose to within 61 feet of the top of the bore hole, while an artesian supply was only got at a depth of 1,189 feet.

Clay, Silt and Sand.

Strange found that a good, natural, specimen of clay, when dried, lost twenty-five per cent. of its weight and ten per cent. of

its bulk, becoming extremely compact, and, if not allowed to expand, offered great resistance to the passage of water. A dried specimen of this clay, reduced to a fine powder, absorbed about seventy-five per cent. of its weight of water, and then allowed free percolation, but when the powdered clay was pressed into a tube, eight feet long and three inches in diameter, it absorbed thirty-five per cent. of its weight of water, and there were no traces of filteration through the tube. The compressed particles of clay, in absorbing the water, expanded so as to become watertight, while the greater the pressure of the water the more satisfactory were the results. This was probably due to the boxing.

Silt is a mixture of sand and clay in which wetting produces very little change. A two-inch cube filled with silt-puddle, when dried, only showed a diminution of 0·05 of an inch. Silt becomes fairly water-tight under high pressure. In a tank experiment made by the author, where the tank was partitioned into two chambers by means of a piece of old corrugated iron sheet, with two large cracks in the centre, and loose side joints, a foot of silt was placed in one chamber : when the depth of water in this chamber was only three inches there was leakage, but when the depth of water was increased to nine inches, this stopped. Percolation in silt, however, cannot be regular, as the material is not homogeneous, while in clay it is either through cracks or spaces between the undulating laminations, or at the surface through the interstices of the expanded clay, but the flow is sinuous. Silt retards flow, and since it absorbs water, it is a very defective agency for percolation. Silt particles also have a tendency to move along with any flow and close the outlets.

Sand is a water-bearing medium, permitting uninterrupted easy flow, which is not sinuous but direct. Under the influence of moisture there is a contraction of about eight per cent. in a dry sand containing thirty-five per cent. of voids. A tank one cubic foot in capacity, filled with dry sand, sank an inch when moistened. There is a continuity in the lines of flow through sand, which is not found in clay, while in silt it is very doubtful. It is probable that sand movement does not take place along with percolation flow, but as sand alone does not work as a cementing material this is immaterial for the author's purpose. Sand in contact with a channel will, however, undoubtedly greatly increase percolation losses.

Remedial Measures.

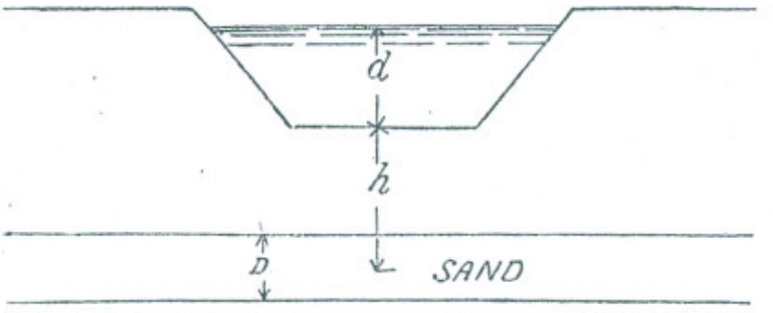
Lining channels is a perfect preventive treatment, the usefulness of which no one can question, but, unfortunately, it is generally accepted by engineers that lining must be considered a hopeless task, since neither can the main lines of canals be closed for a sufficiently long period, nor can the necessary arrangements for collecting the huge requisite labour force be economically made, nor the large amount of good clay, fit for puddle, be had all over a canal tract. This is borne out by the fact that the engineers on the Lower Chenab Canal have only succeeded in lining one mile of the bed, while another mile between R. D. 117,000 and 122,000 was left incomplete, only thirty-two per cent. of the bed width having been puddled ; a sheer waste of money.

The materials so far used for water proofing are clay used as puddle, and cement as plaster. Clay puddle has fulfilled anticipations to some extent, but cement plaster does not hold out much hope. Cement concrete lining is a method by itself which has been successfully used in America, where the canals, however, are much smaller than in this country, but whatever material is used on the surface, it must suffer from erosion, and there will consequently be recurring expenditure. Till the whole system is completely lined, a full measure of success cannot be expected, as water-logging at a particular point may be due to percolation taking place at a point at a higher level some miles away, though every mile lined will be so much to the good in reducing percolation.

Sheet Piling.

The author considers that a system of sheet piling, if not as efficacious as lining, would at any rate be little inferior as a preventive measure, and well worth a trial. In order to better understand this system, an attempt has been made in the previous pages to explain the geological conditions of alluvial soils ; the direction of percolation ; the media which permit filteration ; the behaviour of clay, silt and sand strata ; the relative position of water bearing media in adjacent drainage zones ; and causes which lead to water-logging. It is clear that the percolation velocity cannot be increased indefinitely without causing some movement in the sand ; there must be a limiting value to it. In the approximate percolation formula Q (amount of percolation per foot run) $= K. D. (d + h)$ the velocity is directly proportional to $(d+h)$, but, on the analogy of pumping from wells in alluvial soils, where the draw off cannot be increased above the critical

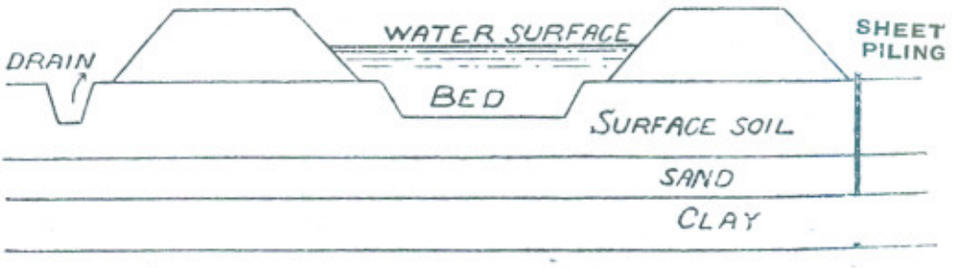
velocity, without disturbing the sand, the percolation velocity



will also be governed by the critical pressure head. The bed width is not a factor in this rough and ready formula, the discharge being directly proportional to the depth of the water bearing stratum. Any filteration taking place in the soil above this stratum is also ignored, owing to its slushy nature.

To reduce percolation losses, either $(d+h)$ or D must be reduced, but $(d+h)$ can only be affected by means of an impervious lining, which has hitherto been found a hopeless task, so that the only other feasible measure is to stop the flow in D, which the author claims can be successfully done by means of sheet piling.

The Upper Jhelum Canal, between R. D. 399,000 and 413,000, is surrounded by a water-logged area with a continuous layer of clay, about thirteen feet below the ground (Plate II), and this would be an ideal place to experiment with sheet piling, with a view to stopping percolation into the water-logged ground on the right of the canal. When piling was originally proposed, the objection was raised that it would interfere with the subterranean flow, but this objection cannot stand in a case where there is a clay stratum above the permanent water-table, since, by stopping the piles short at the clay stratum, there is no interference with the flow beneath it, and any temporary



flow caused by periodical rains in the stratum above the impervious layer can be taken up by a drain (*vide* sketch) on the

left of the canal. Such a drain, with the slope of the country favourable to a side-long canal, would not permit water to rise so as to interfere in any way with the ordinary cultivation on the side of the canal.

When the permanent water-table is nearer to the surface and a clay stratum a few feet below it, as for example in the head reaches of the Upper Chenab Canal, it is evident that sheet piling would totally stop any lateral percolation, and the only way by which this water could escape would be from beneath the piling owing to its increased velocity, and the piling would have to be taken down into the permanent water-table stratum to such a depth that the increased velocity and reduced sectional area would pass the discharge which was passing before the advent of the canal, *i.e.* sufficiently to create a counter pressure from the subsoil-water to deaden the pressure caused by the channel above. Moreover boxing has its own curious effect on slush, causing it to attain, under pressure, water tightening and cementing properties.

In order to show that a reduction of the cross sectional area reduces the percolation discharge, the author carried out an experiment in a masonry tank 3′—3″ by 2′—2″, partitioned into three chambers, two feet, one foot, and three inches long, respectively, by a sheet iron shutter and a wire screen covered with muslin cloth. The first chamber contained 1′—8″ of saturated sand, with four inches of water on the top, (a depth which was maintained throughout the experiment), while there was a foot of wet sand in the second chamber. The opening between these two compartments was manipulated by means of a shutter. The percolated water collected in the third chamber, whence it was drawn off, and the author found that the time taken to fill a tumbler, with various heights of opening was as follows :—

11″	opening	300 secs.
10″	,,	320 ,,
9″	,,	345 ,,
8″	,,	370 ,,
7″	,,	390 ,,

Unfortunately, owing to a rush of work, the experiment could not be completed, but a greatly reduced percolation might obviously be looked for, if the shutter were lowered down still more, since the maximum rate of flow is at the bottom.

The conclusion to be drawn is that sheet piling would certainly prevent percolation. There might be some difficulty

in driving piles vertically in line, so as to have perfect water-tight joints, but this is not absolutely essential and the difficulty has been overcome. Most of the driving would be through slush, but the sand should present little difficulty. The cost per foot run, with an average depth of thirteen feet in length of planking, 0·15 foot thick, comes to Rs. 5 per foot run of the canal, the details being as follows :—

Sinking, one anna per foot of depth,	0	13	0
1·95 c. feet of wood at Rs. 2 per foot	3	14	0
Coal tarring, ...	0	5	0

as compared with puddle and cement lining on the Lower Chenab Canal which cost on

the main line, near Muradian ... Rs. 35 per foot run.
the Jhang Branch (cement plaster) ... Rs. 13 per foot run.
the Kot Nikka Branch (cement plaster.) Rs. 7 per foot run.

The advantages of the sheet piling are that the life with deodar wood will not be less than fifty years while it may be much more ; also there will be no supervision, no repairs, no wear and tear, and therefore no recurring expenditure. It is also a water saving system, and the drain can control the capacity of the cana

Surface Drains.

Surface drains do not constitute a preventive system and any curing properties they may have are very limited, as they only intercept percolation flow to a very small extent. Drains averaging five feet deep, along the main line of the Upper Jhelum Canal between R. D. 399,000 and 413,000 only pass 0·75 cusec. The Chak Qazi drain has proved to be a failu e, and water-logging exists as before. This drain is fifty feet away from the toe of the right bank of the canal and runs parallel to it. While the drain is working, the condition of the subsoil water is as follows :—

Canal water	R. L. 760·8	depth 5·1 feet in canal.
Drain water	R. L. 755·3	
Water level on the right side of the drain 50 feet away	R. L. 757·3	
Water level on the right side of the drain 150 feet away	R. L. 757·1	

The reason why, in alluvial soils, surface drains cannot serve the purpose, has already been mentioned. Even in the immediate vicinity of drains the water level may be about four feet below ground, and eight thousand acres on the Lower

Chenab Canal are under cultivation under these conditions. Water so near the surface, however, must interfere with the growth of crops where the roots go down more than four feet, and Mr. Ball of Manchester has pointed out that fluctuation in sub-surface water is one of the most important causes of Egyptian cotton deterioration, as the temperature required for the growth cannot be controlled. Short and long staple cotton go down eight and ten feet, and probably more in a free soil, but Mr. Roberts, Principal of the Punjab Agricultural College, writes that cotton can grow when the water level is four feet from the surface, provided the level does not fluctuate, otherwise the bolls drop and the crop is sometimes killed.

Another disadvantage in crop-growing in water-logged areas, with the water running only four feet below the surface, is that *dub* grass, which goes down more than fourteen feet into the ground, cannot be easily uprooted.

Surface drains are also considered defective in dam embankments. Strange writes that shallow drains in clayey soils are of little use for draining the rear of a dam, as they do not tap the deep seated percolation water. Deep drains, extending if possible to the bed rock, are required, but if these drains are left as open trenches, they are liable to slips, to becoming choked with silt brought into them from the surface, and to being blocked by the growth of weeds, so that in a few years, and possibly even in a few months as happened in the Chak Qazi water-logged tract, they are likely to lose most of their efficiency. In the Chak Qazi drain, percolation was only perceptible where sand strata were intercepted, till slips occurred and blocked the flow. Weed clearance was done, but three months later had to be done again.

Tube Wells.

The only other cure for water-logging which has been tried has been a system of tube wells, but the principles mentioned in this paper do not hold out much hope of success. The advantage claimed for this method is that water is drawn from the ground, but if the water bearing medium is connected with the canal, the bed is again replenished by percolating water, so that the percolation losses are increased. A statement shewing the water levels in wells Nos. 13, 19, 20 and 25, situated round pump No. 7 at Amritsar is attached.* The statement covers

* Appendix Page 140.

observations for a $2\frac{1}{2}$ years period, but there is nowhere any sign of a reduction in the water table, nor can such be expected, for the tube well only draws water from the deeper strata, with which we are not concerned, instead of from the top stratum, which is not touched in Amritsar, owing to the head of the strainer being fixed some thirty feet below ground level. A disadvantage of a tube well installation is that by pumping out water from the low lying water-bearing media, well irrigation may be affected miles away from the tube well site.

APPENDIX.

Water surface levels in wells round experimental tube well No. 7 of the Amritsar Hydro-Electrical Installation.

Month and Years.	Well No. 13.	Well No. 9.	Well No. 21.	Well No. 14.
October 1915	748·3	744·1	743·7	747·1
November 1915	47·7	44·2	43·1	46·9
December 1915	47·3	44·0	43·0	46·1
January 1916	47.0	44 3	43·2	45·7
February 1916	47·0	44·5	43·3	45·9
March 1916	46 8	44·7	43·5	45·5
April 1916	47·2	44·0	43·1	46·6
May 1916	47·0	43 7	42·3	46·1
June 1916	46·4	43·6	42·2	45·2
July 1916	47·8	43·2	43·2	46·5
August 1916	52·7	46·9	45·7	51·7
September 1916	52·2	49·4	46·7	52·1
October 1916	51·0	48·3	46·2	50·4
November 1916	49·7	47·6	44·4	49·6
December 1916	49·0	45·5	42·6	49·8
January 1917	49·0	45·5	42·6	49·6
February 1917	48·2	45·1	41·4	41·5
March 1917	48·1	45·0	40·6	41·6
April 1917	46·9	44·7	39·4	47·0
May 1917	46·8	44·7	39·5	46·1
June 1917	45·7	44·6	39·6	46·0
July 1917	45·9	44·4	43·7	45·6
August 1917	42·2	43·3	43 0	44 2
September 1917	51·8	50·7	50·4	51·5
October 1917	53·0	51·7	53·7	51·9
November 1917	50·5	47·7	50·5	47·8
December 1917	53·3	49·8	49·2	50·8
January 1918	51·4	48·6	48·4	49·7
February 1918	50·6	48·2	39·4	48·8
March 1918	49·9	47·5	46·6	47·9
April 1918 ...	...	...	44·3	47·4
May 1918	...	...	43·7	44·5
June 1918	..	42·2	42·5	42·5
July 1918	...	41·6	41·6	41·8
August 1918	...	41·4	41·8	42·1

FIG 13

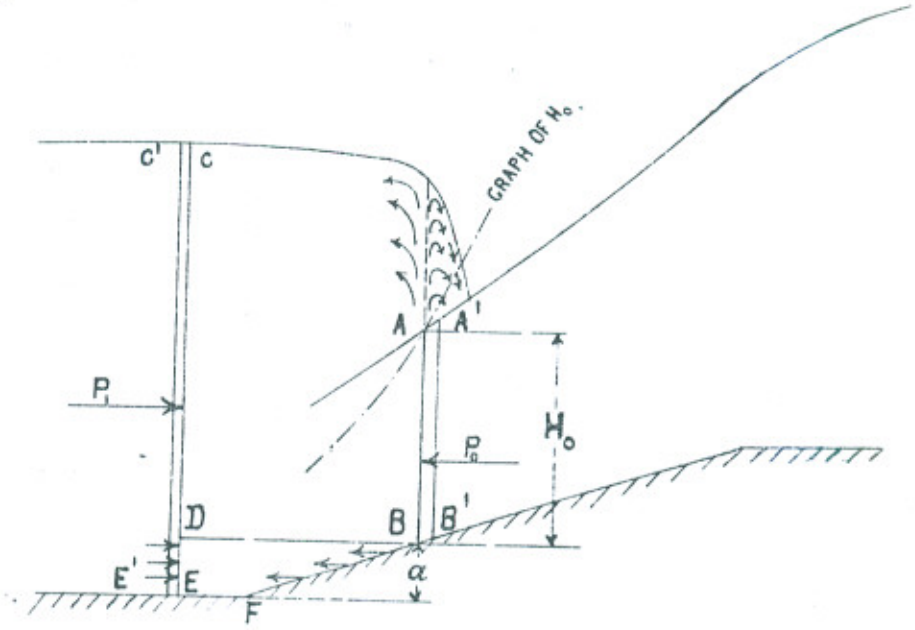
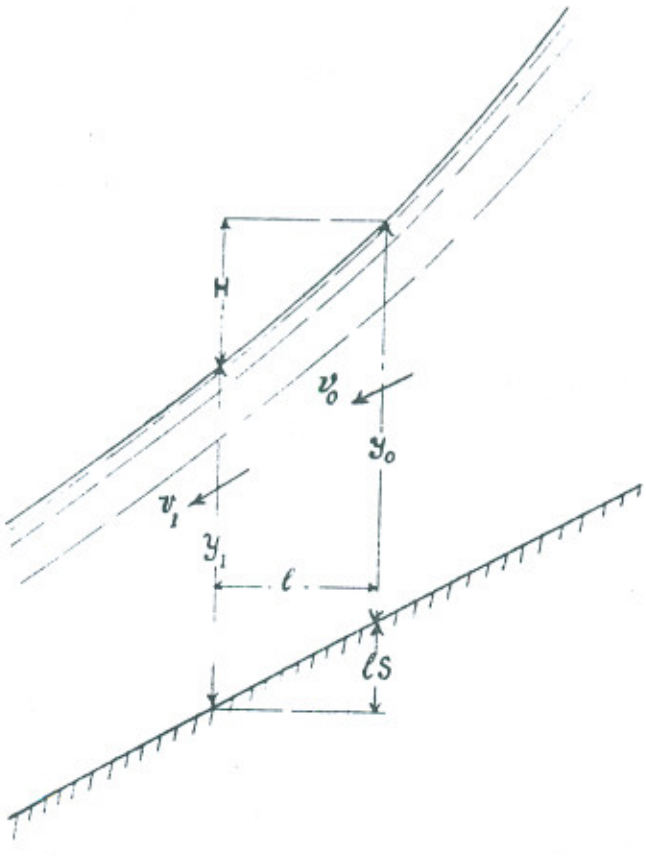


FIG 14



DISCUSSION.

MR. BURKITT briefly introduced his paper, and explained more fully on the black board, for the information of members, his sketch illustrating the experiments about which there appeared to be some misconception.

MR. E. S. LINDLEY said that in May 1915 his superior had suggested to him a problem in standing waves ; after reading the little that had then been written on the subject, he submitted an estimate for experiments, which however only found a place in office records. Mr. Burkitt had been more fortunate in his opportunities.

The standing wave problem was of more than merely academic interest, and in asking for means to experiment, he had mentioned the following practical applications that he foresaw and wished to investigate :—

1. Two forms of gauge-outlet.
2. A " Venturi " flume.
3. An improvement for cross-drainage works.

One form of the first was now in use as the " Harvey Outlet," which was only a small Venturi flume ; the other form was the Stoddard-Harvey module, that had not yet been perfected (as far as he knew). The Venturi flume had been introduced in America in the very small and imperfect form, described in the Journal for Agricultural Research of April 23rd, 1917 ; possibly a contracted flume, built on the Lower Bari Doab Canal main line, was intended to be the same, while Mr. Harvey, in his paper before the present Congress, suggested that what he proposed was again the same thing.

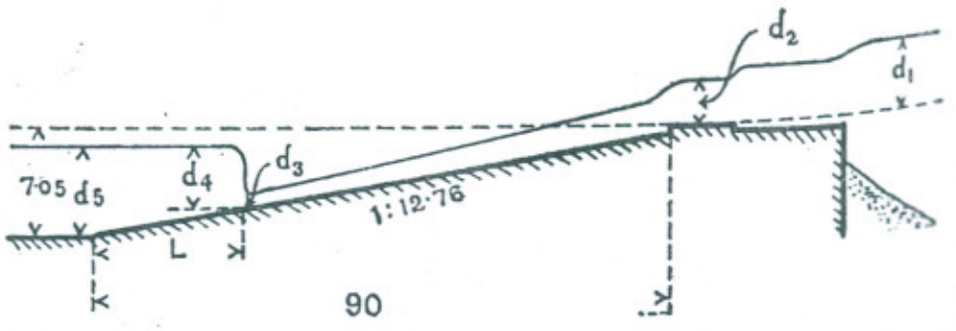
The third was what Mr. Burkitt had described on page 108, where the heading up due to a syphon was diminished by recovering the velocity head at the exit.

About 1915 an experimental flume was built at Sargodha, but no standing wave would form. It would have been more useful to build only the side-walls, and to have had a set of slabs that could be formed into weirs of different shapes and at different levels, using also a submerged orifice to give the conditions of flow obtaining near the toe of a glacis, as under a fairly wide range of conditions a wave would jump to four-fifths of the original depth. The ninety cusecs and three feet afflux available for the experiments which he wished to make, would with such a flume have given data of value compared with any recorded up to date.

The theory of standing waves had already been exhaustively discussed in Paper 1355 (1916) of the American Society of Civil Engineers, by Mr. Karl Kennison; and in Paper 4081 (1913-14) of the Institution of Civil Engineers, by Professor Gibson. These showed conclusively that the correct formula was obtained by equating momenta, as Mr. Burkitt did, and not by equating energies, as had been done in the formula long accepted as correct, and ascribed to Bidone. For waves on a sloping glacis, Professor Gibson introduced a term for the fall of the glacis, but, in testing by it some experiments of D'Arcy and Bazin, did not state how he had fixed the length in which to take the fall into account. Mr. Burkitt took this fall down to the toe of the glacis, and his experiments confirmed this, because the wave appeared always to have extended so far. In practice the wave was made to form on the glacis, and its correct theoretical position was higher up than that given by Mr. Burkitt's formula, and lower than that obtained without allowing for the slope; but the difference between the two extreme positions was not large compared with the length in which there was great turbulence, and the balance of advantage lay with the simplicity for calculation of the formula that made no allowance for slope; a greater difference would be made by an ordinary error in assuming a coefficient of rugosity.

Table I*, which he had prepared, gave the data of some observations he had made on the Rasul weir, with such accuracy as was possible, while Fig. 1 gave the details and the notation.

Rasul Weir



The weir was closed, except for about a quarter of the bay in which the observations were made, so that the

* Page 126:

flow was practically from one still-pond to another, and not with high velocities of approach such as occurred in floods: d_1 and d_5 were measured on open gauges at the closed sides of the bay, d_2 was measured at the middle of the crest, where the surface became distinctly more horizontal at small depths: when d_2 exceeded two-thirds of d_1 d_3 was measured just above the wave, and was difficult to get accurately, in very high velocities, on hammer-dressed masonry. The standing wave had an almost vertical face with only a little curl of foam in the trough, and rose to slightly above the level of the standing water downstream, boiling all over without any marked central spout-up, so that the face of the wave was taken as giving its position. It was not possible to determine the discharge with any degree of accuracy, but the data had been used to calculate the discharge per foot of width in various ways. Q_1 was the discharge over a broad crest with a free run-off, Q_2 the same taking the actual depth and afflux, Q_3 was calculated from the data of the wave without any allowance for fall on the glacis, by a convenient adaptation of the Unwin formula (momentum), and Q_4 by Mr. Burkitt's formula.

$$Q_1 = 3{\cdot}09 \times d_1^{1{\cdot}5}$$

$$Q_2 = d_2 \times \sqrt{2g(d_1 - d_2)}$$

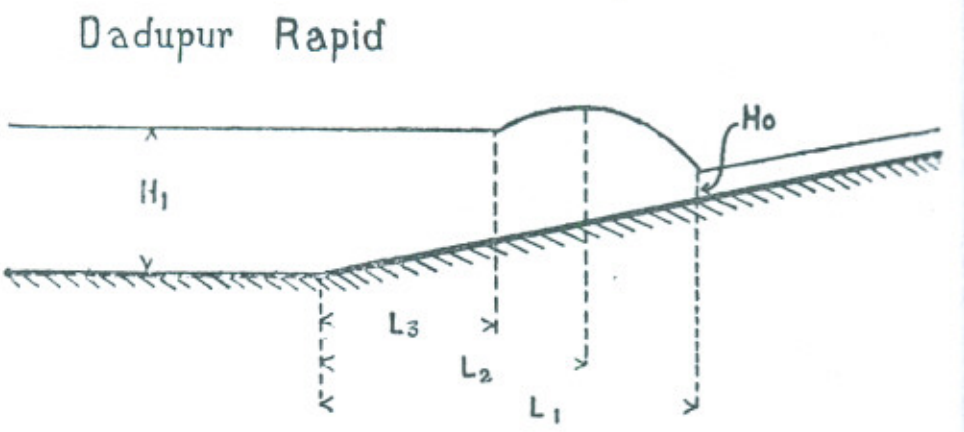
$$Q_3 = \sqrt{g/2 \times d_3 \times d_4 \times (d_3 + d_4)}$$

$$Q_4 = \sqrt{g/2 \times d_3 \times d_5 \, 2\frac{(d_4^2 - d}{d_5 - d^3}}$$

In these observations $V/\sqrt{gd}$ varied from 2·3 to 2·6. The discharges and depths of flow on the glacis gave a value of $N = 0{\cdot}026$ in the Manning formula.

It will be seen that when d_1 exceeds three feet the flow would be free, and the formula Q_1 applied; with lesser depths the discharge was damped by the width of the crest, and a little grass that was found entangled in links lying on the face of the shutters forming the crest, and Q_2 applied; the fact that Q_3 agreed with these, while Q_4 was always appreciably less, showed that it was wrong to take the fall to the foo of the glacis.

Table II* gave a few data of standing waves on the Dadupur rapid, while Fig. 2 gave details and the notation.



In these observations $V/\sqrt{gd}$ was from 3·9 to 4·5, and $N = 0{\cdot}025$; the wave was much more violent, rising in ten feet from the beginning to the marked centre of spouting up, which continued for another ten feet. In an observation made by him the position of all three points was noted, but in the others made for him only the first was measured. H_0 was measured as before, just before the wave, and H_1 on a gauge downstream. Data of the crest were not given, as it was neither a broad nor a sharp crest. The total discharge passing was taken from a curve which seemed accurate, referred to a gauge almost on the crest. There was a main rapid, and a rafting bay with a crest and glacis 0·75 ft. lower, and the total discharge passing had been distributed according to the level measured in the approach channel clear of any draw-down to the weir, with an allowance for the velocity of approach there. A trial was made with velocities of approach due to the different rates of discharge, but gave results that were obviously absurd. The most doubtful point here being the position of the wave, this was calculated from the other data, both by the Unwin formula (momentum) without allowance for slope, and by Burkitt's formula, and again the former seemed to be nearer the mark. The observations showed the great increase in turbulence as compared with the Rasul wave.

Coming to the practical applications referred to above both were the securing of a constant relation between the discharge passing and the gauge-level above a work, indepen-

*Page 126*j*.

dently of variations of level below that work. Mr. Burkitt's experiments went a good way towards showing the practical possibility of the Venturi flume, which was an open Venturi meter, with contraction sufficient to give the stage $V^2 = g \times d$ at the throat, so that only one gauge need be read since there was this constant relation. The flume had to consist of a throat long enough to allow this condition to be fully established; a bell-mouth to fill the whole throat and convert as much as possible of the head into velocity, (that was, coefficients of contraction and of velocity, both as near as possible to unity) and according to the afflux available, an expanding flume to recover the velocity head. Mr. Burkitt made the point that contraction could be effected, either by raising the bed or by narrowing the sides of a channel, and also gave data for the condition of 'no afflux,' though it would be more correct to speak of afflux that was too small to be appreciable in his small-scale tests. Further tests were wanted to show the minimum afflux required for modularity.

In the discussion of a recent paper before the American Society of Civil Engineers by Mr. Floyd Nagler, the originator of the Venturi meter Mr. Clemens Hershell pointed out that a sharp-crested weir could not be considered a really accurate measuring device unless the conditions under which it was calibrated were accurately reproduced, and advocated a certain standard rounding of the crest, to get a standard suppression of contraction. A bettea course, however, would be to suppress entirely all contractions but the natural surface contraction, and make a broad-crest weir or Venturi flume. This had the added advantage that, instead of requiring a free fall, a small afflux was sufficient.

The Stoddard-Harvey module, involving the formation of a standing wave below a submerged orifice, was outside the subject of the present paper; but the following data were worth placing on record. On the Rasul undersluices he had formed a standing wave, with depths of 10·3 and 8·5 ft. up and down-stream with a depth of flow roughly measured as 4 ft. from a gate-opening of 5½ ft., while theoretically a depth of flow of 0·4 of the up-stream depth would jump to 0·8 of it, calling for a working head of only a fifth, without any attempt to diminish the loss of head in the standing wave.

MR. W. C. CHOPRA instanced the case of the very broadcrested weirs in the form of super-passages on the Raya

branch of the Upper Chenab Canal, where the calculation of the discharge became very difficult, because, on the one hand, the crest was not wide enough to allow of a steady stream in the flume to admit of direct observation or of calculation by Kutter's formula, and on the other hand, the crest was so broad that the ratio of the head or the depth of water to the width of the weir became so small as to defy even the low values of coefficient given by Unwin, and he enquired what coefficient the author would recommend in such cases, there being no standing wave to speak of, on account of the small head. A coefficient of 0·577 was taken, but it was doubtful whether it applied to the case. He also instanced the case of the weir across the Bhimber diversion on the Upper Jhelum Canal, where over a length of 850 feet of crest wall, the water-level sloped from right to left, so that while there was a fall of several feet at one end, it was almost *nil* on the other, and where two standing waves were produced (the first just below the raised crest-wall, on the horizontal part of the glacis, and the other lower down, on the sloping part of the glacis), and inquired what co-efficient should be employed in such cases, and what, in the author's opinion, should, for purposes of calculation, be considered (*a*) the water-level upstream of the crest, (*b*) the water-level downstream of the weir.

He had been taking the mean values (for downstream the mean of the stream immediately below the lower standing wave), but was doubtful if that was quite correct, and would be glad to have the author's views on the matter.

MR. C. A. COLYER drew attention to Fig. 4 of Mr. Burkitt's paper, and described a double wave of the kind noted by the author, which he had seen formed on the Merala weir in a big flood.

MR. F. W. WOODS remarked on the statement that on the Swat river, large freshets passed over the Abazai weir "without appreciable afflux, the difference in the water-levels against the upstream and downstream sides of the *bund*, that obstructed half the waterway of the river, being approximately that caused by the natural slope of the river." It would have been more satisfactory if a plan, with longitudinal and cross sections of the river and the recorded water-levels of the flood, had been given, to enable them to judge of the real conditions at the site, and of the merits of the author's inferences therefrom.

The author went on to state that " eventually a flood, *about twice as big* as the maximum recorded for the winter season was *safely passed*, though *calculations based on* any of the *accepted formulae showed that* the afflux should have been so great as to have overtopped the *bund*." The words in italics were va gue. One would have liked to know the estimated magnitude of the flood in cusecs, and how the estimate had been arrived at. Also, the recorded afflux of this flood, that is to say, the difference of the water surface levels upstream and downstream respectively of the weir, or of the *bund*. It was not enough to say that it was *safely passed*, as that might have happened even if the afflux had been very considerable. One would also have liked to see the figures of the calculations " by accepted formulæ," which Mr. Burkitt claimed to have been so erroneous in this case ; for it was, *prima facie*, improbable that formulae, which had held the field for so long, should have done so without earlier detection of their worthlessness.

Mr. Burkitt's main thesis might very possibly be correct, but he would bring conviction to their minds more readily if he treated them to precise facts and figures which they could analyze and check to their own satisfaction.

Mr. Burkitt's proposition that, when a standing wave occurred below a waterfall over a weir or flume, the discharge was the same as it would have been if the overfall were free, with the tail-water level below the level of the crest of the weir or the floor of the flume, and that neither the discharge nor the upstream water-level wasa ffected by artificial variations of the level of the tail-water surface downstream of the standing wave, was probably true. As long ago as the year 1893, Mr. Herbert, Executive Engineer of the Lower Chenab Canal at Khanki, stated that it had been found, by experience of the weirs and rapids on the Upper Bari Doab Canal, that the discharge over a weir could be estimated accurately by the " free fall " formula, so long as a standing wave existed below the weir. Colonel Ottley, Chief Engineer, and Mr. Preston, Superintending Engineer, corroborated this opinion ; though at a later date Mr. Beresford and Mr. Benton, Chief Engineers, expressed opinions to the contrary. It would simplify their calculations of the discharge of streams, and especially of streams of great magnitude in flood, if their discharge could be calculated by the simple free-fall formula

$$Q = \tfrac{2}{3} c a \sqrt{2gh}$$

in cases where a and h were known from the recorded water-

level of the flood upstream of the weir, and the known level of the crest. The difficulty of applying it to the design of particular works lay in the fact that one could not predict beforehand whether, and to what extent, standing waves would occur downstream of a designed weir, under any particular conditions of flood. It had been a matter of common observation of floods passing over various river weirs in the Punjab, that standing waves might occur below some parts of a weir and not below other parts of it ; and it could not be predicted with confidence that any given flood would produce standing waves throughout the length of the overfall.

When a flood of great magnitude careered down the sandy alluvial bed of a Punjab river, it flowed not in a single well-formed channel, but in several different channels, between shoals, and in shallow spillways over shoals. It swept away shoals in some places, or deposited them in others, and was subject to unforeseen deflections from its normal course by those shoals, and by obstructions of various kinds ; so that in its passage over a masonry weir at any point of its course, its hydraulic conditions did not always lend themselves easily to a precise estimation of the discharge, and this was the main drawback to the application, in practice, of the results of Mr. Burkitt's useful investigations.

MR. BURKITT in his reply at the Congress to the criticisms put forward by the members, said that he could not reply off hand to Mr. Lindley. It was not his formula for the position of a standing wave at a weir. He had got it from Mr. Lindley, but the formula did not agree at all until Mr. Harvey had given him his suggestions.

The two-thirds depth referred to in his paper would occur at the downstream end of a super-passage, as long as there was a free fall or a standing wave, and the downstream water-level would then not affect the discharge. He regretted that his information with regard to the construction of the Abazai weir was not more complete, but during the flood all the engineers were busily engaged in saving their *bund*. Most of the American papers had been writing a great deal about making weirs that would cause no afflux, and to avoid having an afflux, during a big flood, when the depth of water was great, the weir should be designed in such a manner that there would be no standing wave. The main thing which he wanted to prove was that a weir could be built that would not cause any afflux after the depth had reached a certain point.

Unfortunately it has been impossible to obtain any reply from Mr. Burkitt, beyond that taken down at the Congress, to the points raised by certain members, otherwise he might have pointed out that Mr. Lindley's conclusions that his formula for ascertaining the position of the standing wave was wrong, appeared to depend on the accuracy of the discharges given by certain formulæ, the applicability of which to the conditions of the case was itself open to doubt. Instancing that the formula $Q_2 = d_2 \times \sqrt{2g\left(d_1 - d_2\right)}$ gave the full theoretical discharge for a water-level of the height d_2 on the crest, without regard to frictional losses upstream of the point where the water became horizontal at this level, which under the conditions of the case were not apparently negligible, with the result that the actual discharge might well be less than that given thereby.

It is also likely that he would have qualified his reply to Mr. W. C. Chopra, that "the two-thirds depth, referred to in his paper, would occur at the downstream edge of the superpassage....." by adding "provided there was no loss due to friction in traversing the superpassage."

Data of Standing-wave Observations on Works.

TABLE I.

RASUL WEIR (Bay No. 8).

Observed.					Calculated.				
d_1	d_2	d_3	L	d_5	d_4	Q_1	Q_2	Q_3	Q_4
3·85	2·70	1 60	40	7·00	3·86	23·3	23·2	23·4	20·4
3·65	2·50	*1·25	29	6·10	3·82	21·6	21·5	19·8	18·2
3·60	2·30	†1·25	25	5·60	3·64	21·1	21·0	19·0	17·5
3·45	*2·70	1·25	29½	5·80	3·48	19·8	18·8	18·3	16·5
3·15	2·15	1·25	39	6·40	3·34	17·3	17 2	17·8	15 6
3·00	2·00	1·10	38	6·20	3·22	16·1	16·0	15·8	14·1
2·80	2·00	0·84	30	5·30	2·95	14·5	14·4	12·35	11·35
2·45	1·90	0·88	30½	5·20	2·80	11·85	11·30	12·10	11·00
2·15	1·71	0·71	32	4·70	2·19	9·75	9·10	8·55	7·64
2·15	1·72	0·75	30	4·60	2·25	9·75	9·05	9·05	8·10
2·00	1·60	0·67	29	4·55	2·27	8·75	8·12	8·50	7·74
1·65	1·42	0·57	30	4·10	1·75	6·65	5·46	6·13	5·43
1·65	1·30	0·52	30	3·95	1·60	6·55	6·17	5 35	4·71
1·35	1·15	0 40	30	3·70	1·35	4·85	4·12	3·91	3·47

$V/\sqrt{gd}$ is from 2·3 to 2 6.

TABLE II.

DADUPUR RAPID.

Observed.							Calculated.	
Date.	Q	H_0	H_1	L_1	L_2	L_3	L_0	L_B
17-7-17 ...	48·0	2·09	9 21	42	†30	...	$28\frac{3}{4}$	$24\frac{1}{2}$
18-11-16 ...	42·9	1·80	9·12	$39\frac{3}{4}$	$28\frac{3}{4}$	$18\frac{3}{4}$	$28\frac{1}{2}$	25
17-7-17 ...	39·4	1·81	9·27	53	43	...	$42\frac{1}{4}$	39
18-11-16 ...	34·5	1·55	9·08	50	†41	30	$43\frac{3}{4}$	$40\frac{1}{4}$
8-7-17 ...	25·55	1·38	7·13	$44\frac{1}{4}$	$\dagger 37\frac{1}{4}$	...	$35\frac{1}{2}$	31
8-7-17 ...	17·85	1·06	7·11	$52\frac{1}{4}$	$\dagger 47\frac{1}{4}$	...	$49\frac{1}{4}$	45

$V/\sqrt{gd}$ is from 3·9 to 4·5.

† Not observed. * Doubtful.