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HYDRAULIC GRADIENTS IN SUBSOIL WATER FLOW IN RELATION TO STABILITY OF STRUCTURES RESTING ON SATURATED SOILS.

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Introduction.

The author has had to face, during the last three and a half years, serious trouble in connection with the upstream and downstream floor of some of the drainage siphons on the Main Line of the Upper Chenab Canal, more specially the Dugri No. 1 and Jauryan siphons.

Dugri 1 Siphon.

The trouble on the Dugri No. 1 (R. D. 35,800) siphon downstream, however, occurred as early as 1915 when high supplies were run in the canal for the first time. Springs appeared beyond the drop wall. In 1917 the floor was extended and strengthened. In 1919 the right wing wall settled as much as one foot at the end due to blowing out of foundation sand. In 1919-20 the wing wall was rebuilt and grouting and stock ramming done. In 1920-21 a cunette was built opposite the two central bays for draining water at a lower level. Relief pipes were inserted in the floor. In 1921 the springs became extremely active blowing out sand freely. The right wing wall settled out of plumb. The trouble finally extended to the face wall which cracked due to the settlement of the barrel lips. The barrels cracked inside. In 1923 a line of sheet piles 20 feet deep below bed (see plate VI) was put in 7·0 feet away from the face. Springs continued to blow out sand between the face wall and the sheet piles and in the floor outside. Porous tiles with ballast packing round, were tried running at the back of the face wall, over crown of the barrels and along back of both right and left wing walls, discharging through small openings at the end, in the wing walls.

This proved equally ineffective. A proposal was put up of puddling the canal bed and side slopes for a length of 100 feet upstream and downstream of the siphon to kill the hydraulic head under the floor. This however was never carried out.

In the meantime trouble had extended to the upstream floor. The barrels, face wall, wing walls, and floor all showed cracks and signs of failure. In 1923-25 the upstream floor was rebuilt in cement concrete and fresh side walls were constructed. The downstream wing walls were also remodelled and founded on wooden piles and the downstream floor was further strengthened and repaired.

All these however proved ineffective as springs continued to blow out sand at the end of the floors both upstream and downstream. Settlement cracks started appearing and finally the situation became as dangerous in 1928 as ever it was before.

All the repairs and extensions mentioned above were based on a hydraulic gradient of 1 : 10 taking no account of the high spring level all round the work both upstream and downstream.

This was based on the Hydraulic Gradient Theory as set forth by **Mr. Bligh** in Chapter VI of his book. The object of this paper is to show, how **very materially** this theory can be **at variance with facts.**

Jauryan Siphon.

The author took charge of the Marala Division in April 1926. The Jauryan siphon (R. D. 63,800) had been giving cause for anxiety for over two years and dangerous springs were active just outside the pacca floor. In October 1926 wide cracks appeared in the wing walls and the face wall and the barrels lip settled down. The siphon was saved from further damage by heading up supply on top of the floors by putting up bunds upstream and downstream so that there was no chance of sand blowing out. A number of proposals were under consideration at the time for rendering the structure safe. One of these was to line the bed and slopes 125 feet upstream and 125 feet downstream of centre line of siphon with cement concrete, reinforced brick or puddle and another was to extend the floors as in the case of Dugri No. 1. It was decided to put in 2 feet thickness of puddle in the bed and side slopes with a covering of dry bricks and protected at the ends by toe walls followed by 20 feet width of stone pitching. This work was executed in the January closure of 1927. The upstream floor, wings and face walls were also rebuilt. Pressure pipes were inserted in the floor and behind the puddle to study the effect of puddle and the uplifting pressures under the floor. The above work was designed as previously on the 1:10 Hydraulic Gradient. The results of pressure pipes observations soon disclosed that whereas the **puddle** was **absolutely watertight** it was **absolutely ineffective in reducing the upward pressures** under the floor which **were governed by the subsoil level** (not the level of the flowing water in the cunette or drain) **in the vicinity of the work.** The puddle cut off the local percolation but this was going on above and below this length thus keeping the subsoil water level unaltered even in front of the puddled length. The history of these two siphons clearly demonstrates how **works originally safe can become unsafe with a rise in the spring level.**

The upstream and downstream floors of the Jaurian siphon were remodelled in May 1927 on the basis of the latest information about pressures under floors of this nature. The trouble however did not end. Six months after construction, a small crack was seen in the middle walls, both upstream and downstream and small springs were noticed at the end of the pacca floor blowing out minute quantities of sand

and clay. The cracks being far away from the face of the barrels, there was no immediate danger, but the design was by no means the last word. After careful study of the local conditions it was believed that **the fault lay with the shallow end curtain walls,** only 2½ below top of floor which permitted sand blowing under pressure. Subsequent study, which forms the bulk of the paper, confirmed this belief.

After the summer of 1928 benefitting from the mistakes of the earlier designs, the remodelling of the Dugri floor which had become dangerous by them, was designed on the lines of the Jaurian siphon, so far as upward pressures under the floors were concerned with a line of sheet piles 8 fe t deep at the ends as a drop wall to prevent sand blowing out from underneath. The deep sheet piles were objected to in view of the fear of "piling up" of pressure behind them and underneath the pacca floor. The alternative of putting in relief strainers, instead, was favoured. The point was discussed on the 10th January 1929 in the Chief Engineer's room at Lahore and very fortunately, for the siphon and for the important experimental data it has furnished, the deep sheet piles proposed were permitted to be put in and the relief strainers in addition. This work was executed in January 1929 and the experimental work carried out so far shows that the design is sound. It is now proposed to add a line of sheet piles each to the Jauryan floors to make them safe.

The Origin of Experimental Work.

The above narrative of the troubles at the siphons, the various remedial measures employed and of the results achieved, brings out in clear relief, the fact that our existing conception of the laws of flow of water through subsoils as effecting the stability of structures resting on saturated soils, is far from being complete or correct. To get at the correct theory of flow and the principles of design of such structures it was decided at the instance of Mr. J. B. G. Smith, Chief Engineer, to carry out careful experiments on scientific lines, at the siphons and elsewhere and to study the other existing data in relation thereto. A series of pressure pipes were accordingly put in at suitable depths and distances at the Dugri and Jauryan siphons. The method of conducting the experiments and the conclusions derived, therefrom, are described in the pages that follow. Some of these conclusions differ very materially from accepted theory and established practice and as the principles involved are of wide practical application, being of vital importance, the author has ventured to present these to the Congress in the hope of inviting criticism and thus deriving benefit from the experience of other engineers.

From Colonel Clibborn's experiments of 1896 at Roorkee, on the Passage of Water Through Sand and from the discussions on these printed in G. of I. Technical paper No. 97 of 1902 as also from record of observations at the Narora Weir, the Upper Chenab Canal siphons and other places, it may be safely concluded that :—

(1) As in pipe line flow, so in subsoil water flow, the Hydraulic Pressure Gradient is represented as nearly as possible by a straight line

joining the upstream and downstream free water surfaces provided that the medium is uniform and that there are no obstructions of any sort in the region of flow.

In figure 1, Plate 1, AB represents the Hydraulic Gradient line for the pipe PP when A_1 and B are free surfaces at either end, A_1 A being loss of head at entry. The water level in a stand pipe, inserted at point X, will rise up to the point Y where the vertical through X will cut the H. G. line.

The above sketch exactly represents the pressure gradient, if the pipe is substituted by sand or any other soil, although, the loss of head at entry will be problematic owing to the extremely small velocity of water flowing through subsoil. There is, however, a very important factor involved, which has so far always been ignored in subsoil water flow and it is believed that it is being put forward for the first time in the paper. In the case of free water moving through a pipe line, BB. represents the true water level at exit as it is assumed that there is no velocity head in the reservoir BB. If the pipe were discharging free with a velocity of exit at P_1 the head due to that velocity of exit will have to be plotted as BP to get the true free water surface for purposes of obtaining the true H. G. line.

In figure 2, Plate I, A_1 A is the free water surface upstream and B_1 B_1 the **apparent** free water surface downstream. A_1 P_1 is a vertical wall and P P_1 an impervious floor formed on sand or other soil. Beyond P_1 the foundation soil is exposed. According to the commonly accepted theory A B_1 will represent the H. G. line. This will be so only if the flow at the level B_1 B_1 has no velocity or in other words there is no flow. But if the subsoil water coming out of the exposed surface of soil P_1 P_2 has a velocity, the head equivalent to that must be added above B_1 B_1 to get the true free water surface BB. The line joining A B will then be the true H. G. line. This phenomenon of the **true free water surface** being very much **higher than the apparent free water surface** is clearly demonstrated by the experimental work at the siphons and by the analysis of the available data of weirs. This will be fully discussed later on, but it may be mentioned in passing, that with the exit free water level at B_1 B_1 and the true free water level at B B unless there is a curtain wall of proper depth at P_1 the H .G. over the floor will be a curve AY_2 B, tangential to A B at A and not the straight line. A B_1 of Bligh (see photographs No. 6 Jauryan Siphon). The exact significance of this curve and of the influence of curtain walls on the H. G. Line will be explained later.

Experiments with Pressure Pipes at Dugri No. 1.

The general arrangements of these pressure pipes is shown on photographs No. 4, 5 and 6 and in plan on Plate VII. There is a row of pressure pipes fixed in the middle wall spaced about 10 feet apart. These pipes (No. 1 to 5) $1\frac{1}{2}''$ or $1\frac{1}{4}''$ diameter, have filter points at the bottom, 2′ to $2\frac{1}{2}'$ long. There are three 10′ long Tej

strainers 5″ internal diameter, one in the middle wall and one each in the wing walls. These are meant to relieve the upward pressure under the floor. Temporary pressure pipes with filter points at the bottom ends, were fixed in line with the central wall, Nos. 13 and 14 between the canal and the face wall and Nos. 6 to 12 outside the pacca floor.

Pressure pipe observations had been made on the Jauryan floor for the past two years and advantage was taken of the experience gained there, in arranging the pipes in the new Dugri floors. The relief strainers added considerably to the scope of the experiments.

Object of Experiments.

The object of these experiments was to determine :—

i. The pressures under the floor under varying conditions of canal supply and spring level.

ii. The correct location of relief strainers and the extent to which these are useful in relieving strain on the floor from upward pressure or from the blowing out of foundation soil.

iii. The true free water level in the open bed as against the apparent free water level or the level of the water flowing over the cunette floor.

Procedure.

With these objects in view, pressure pipes Nos. 6 to 12 were driven to varying depths stepping up from No. 6 to No. 10 (plate V). Observations were made of all the 14 pipes with strainers open and closed in various combinations. The results plotted are only for all strainers open or all closed. To begin with, these observations were taken every half hour when the canal was opening and for a few days afterwards. But as the pressure pipes recorded the full effect of the closing or opening of strainers within half an hour, so these half hourly observations were discontinued.

The pressure pipes 6 to 12 were driven down by one foot or two feet intervals after 48 hours and observations were recorded morning, noon and evening. Statement I gives the details of filter point levels for all pipes on different dates.

It was soon found out that the water in the pressure pipes rose as the well points were sunk further down and that there was a definite relationship between the depth of the well point and the pressure recorded in the pipe. Four cases are plotted on the plate No. V (*i*) with well point levels stepping up as on first day, (*ii*) intermediate stage, (*iii*) all well points fully down, (*iv*) as per (*iii*) with the cunette level gone up from R. L. 789·5 to 790·5 owing to a flood in the nallah. Nos. 6 and 7 were then gradually pulled up and the pressures in these fell according to the height pulled up.

Similar observations were carried out for six weeks at the Jauryan siphon with similar results.

Analysis of Experimental Data.

Some of the apparently eccentric results really bring out certain important points. The plots of No. 10 show a distinctly greater drop from No. 9 than those of No. 9 do from No. 8 although the distance between the two sets is the same. This is due to the fact that the bed level at No. 10 is 0·7′ to 0·8′ lower than the bed levels at No. 9 and No. 8 so that to get the same pressures, the well point of No. 10 has to go down about one foot deeper. This point is dealt with later on where it is shown that **scour immediately below a pacca floor may endanger an otherwise sound floor** by increasing the velocity of discharge at the end in proportion to the depth of scour and thus augmenting the tendency for "piping" or dislocation of soil particles from beneath the foundation. Some of the eccentric results of Nos. 10 and 9 are due to the presence of springs close to them and also due to leakage vertically along the pipe due to the surrounding soil not having had sufficient time to get compact. These leaks when stopped at once produced normal results in the pipes. A large range of results was thus obtained covering depths below floor up to 23 feet.

To get at a general law these results (with strainers closed) were arranged so as to show the difference of head between the normal spring line joining the indicated pressure levels on pipes 2, 3 and 4 and the indicated pressure in each pipe for each depth of strainer (filter point). This method of obtaining the normal spring line called (N. S. L.) is not quite correct as the H. G. somewhat flattens out as it recedes from the end of the pacca floor and the observations in No. 11, 120 feet away from the face confirm this. This, however, was a close enough first approximation of obtaining loss of head at each depth. These results, it will be noticed, are independent of the canal level as the N. S. L. is taken from observations of the pipes 2, 3 and 4 on the same date and time.

Loss of Head Curve.

Separate curves like the one shown in plate VII were then drawn for each pipe showing the loss of head on the horizontal and depth of strainer below floor on the vertical axis.

In the curve of plate VII T.R. is the normal spring level or the **True Free Water Surface** referred to in the earlier pages, that is a level to which water will rise in the subsoil due to the static head if there were no flow occurring in a vertical direction. This can be possible in two ways: (*i*) if the area under consideration be covered with an impervious slab at any level, or (*ii*) if the subsoil extends above the normal spring level line so that the spring level line is beyond the effect of evaporation. In this later cases if a pit were dug in the soil the subsoil water would rise up to the normal spring level. If a pipe with a length of strainer at bottom were inserted in this soil the water in this pipe would rise to the normal spring level irrespective of the depth to which the strainer has been driven. This will apply to both of the above cases. Now, suppose the soil is removed up to line LH., (bed of drain)

flow will start through the subsoil in vertical direction due to the difference in static head between the normal spring level and the water level CQ′ in the drain bed LH. This loss of head or shortly called "head" which produces this vertical flow is plotted as LH. at the bed level of the drain.

Suppose a pressure pipe were inserted in the soil such that the top of the strainer portion is at level P, then the water in this pressure pipe will rise to R, or up to the spring level line if the ground level is much above this line but it will rise only to Q, if the ground level is at LH. permitting vertical flow. At this depth therefore the loss of head caused by this flow is RQ. RQ has been plotted as PP′. The plot of all such points is called the "**Loss of Head Curve.**" These curves of loss of head for the various pipes closely resembled each other. After making correction for difference of bed levels from the normal bed level of 788·0 and for N. S. L. (this was very slight) the points of all these curves were reproduced on one common curve of plate VII, the observations of various pipes being given distinctive conventions. A smooth curve was run through these points and the equation of the curve was determined as explained below.

The slopes $\frac{dh}{dy}$ of the curve were measured at each 2′ depth and plotted against depth "Y" to a suitable scale. These showed a marked similarity to the original curve. A second curve was plotted with "h" or loss of head on the horizontal axis and the slopes $\frac{dh}{dy}$ at that point on the vertical axis. This was a straight line giving the relation

$$h = k\frac{dh}{dy} \quad .. \quad .. \quad .. \quad .. \quad .. \text{ (i)}$$

Integrating we get for the equation of the original curve

$$y = k \log h - c \quad .. \quad .. \quad .. \quad .. \quad .. \text{ (ii)}$$

where k and c are constant whose value work out as :—

$k = 15{\cdot}5$

$c = 9{\cdot}0$

Thus giving $y - 15{\cdot}5 \log h = 9$ .. .. .. (iii)

The Loss of Head Curve, it might be noted, does not quite pass through all points in the region of clayey loam showing that the constants vary there. But this is ignored as it does not effect the main discussion. This, it is believed, is an important contribution to the present knowledge of flow of water through subsoils as effecting stability and designs of structures.

The properties and practical applications of this important curve are examined in detail below.

Velocity of Subsoil Water Flow.

According to Darcy the velocity of water flowing in subsoil channels varies directly as the head and inversely as length and may be expressed by the equation $v = ki = \frac{kh}{y}$ where v is the velocity of the flow, k a constant (called the transmission constant) and i is the fall (**h**) in a given distance (y) divided by that distance (**y**). If the Hydraulic Gradient is a straight line then v = constant as i is constant but if it is a curve as in the present case then $v = k\frac{dh}{dy}$ .. .. **(iii)**

but from equation **(i)** $h = k'\frac{dh}{dy}$

$\therefore v = \frac{ki\,h}{k} = K\,h$ where K = constant.

Hence the velocity of flow at any point in the subsoil is directly proportional to the loss of head from the normal S. L. at that point.

As explained below this relationship is of the utmost importance in a study of the foundations in water-logged soils and also in a study of the yield from tube well strainer. This will not apply where the Hydraulic Gradient follows a straight line as the velocity in that case is constant.

The convixity of the loss of head curve indicates that the rate of loss of head $\frac{dh}{dy}$ increases as the depth below the surface decreases, Since the velocity of flow is proportional to $\frac{dh}{dy}$, $(v = k\frac{dy}{dh})$ to right

Hence velocity increases as the depth decreases.

This has two very important applications.

Critical velocity critical head and dislocation of foundation soil particles.

1. The dislocation of subsoil particles is a function of the velocity of flow of subsoil water through them, **hence the tendency for dislocation of subsoil increases as the depth below the bed decreases.** It will be nothing at infinity and maximum close to bed level. There is, however, a **critical velocity** for each type of soil particles. If the subsoil water flows with a velocity less than this there will be no disturbance or dislocation of particles but if the velocity is above the critical the soil particles will be dislocated in proportion to the increase. This velocity of flow being highest at the surface naturally the soil dislocation starts vigorously followed by decreasing dislocation as we go lower down till where the critical velocity is reached this disclocation will stop. The depth at which this occurs may be called the **Critical Depth** and the head or the loss of head at this point from the Normal Spring Level (or the true free water surface), the **Critical Head.**

The existence of heads, much in excess of the critical, leads to degradation of bed and sides. In this connection the formation of springs just outside the inverted filters of the siphon floors are noteworthy and also the fact that all these filters have settled at a slope from the pacca floor outwards, the settlement being maximum at the outer ends. This process of degradation of bed if not checked will lead to a **lowering of the bed** level with the **result** that the loss of head curve will fall from AB to ' A'B' ' (figure 3, Plate II) and the point of critical flow and with it the depth of **dislocation** will work further down. If this flow is occurring adjacent to a permanent floor the **soil particles** from underneath the floor will be drawn out causing **settlement** and final **collapse.**

2. The second application is in considering the discharge through strainers. Other factors remaining the same, **the rate of inflow per foot length of strainer tube will increase as the depth below ground decreases.** This is illustrated in Plates No. XI and XII (reproduced from Mr. Leggette's note on tube wells of March 1st, 1928). As strainers, for discharge purposes, are not within the scope of this paper, this discussion is not carried any further except in so far as they serve as relief pipes.

Looked at in another way a better conception of this flow is possible (*see* Plate II, figures 4 and 5). Imagine a vertical plane V V' the soil on the left of it being boxed. The flow at a point S towards right across the vertical plane V V' and then to the top will be due to a head = H, less the head required to overcome frictional resistance in a length of subsoil = ' y ' Call this **Friction Head** ϕ (y). As the velocity of flow at any point is proportional to head the flow at S will be proportional to H ϕ (y). As the depth y increases H—ϕ (y) increases and therefore the working head H—ϕ (y) decreases and with it the velocity and discharge. At infinity, theoretically speaking, there will be no velocity and no discharge.

In the above we have considered a vertical column of soil through which the water has to pass in order to discharge at the surface. If a Tej or some similar strainer were to replace this soil column the working head will be :—

$H - (h_e - \phi (y)$. (*See* figure 5).

Where h_e is the loss at entrance at the slits and ϕ (y) is the loss due to friction through the strainer pipe. This quantity decreases as y' increases showing that the velocity and discharge (inflow) of the strainer decreases as the depth increases. There are other factors besides the above but this is just to explain the broad issue. ϕ (y) will of course be much greater than ϕ (y) so that the loss of head or (working head) curve will give higher values for a strainer than for a sand column.

Again ϕ (y) will be very much less for strainers of the Tej or Cook type compared to the Leggette or Ashford type as the former are comparatively smooth inside.

We are now in a position to discuss as to how the foundations are undermined. (*See* fig. 6).

Undermining of Foundations.

Imagine a floor with bottom F F′ at the same level as the ground level outside and adjoining it, which is H feet below the normal spring level. The flow from underneath the floor at F′ into the open area F′ G will occur with a velocity=K H (equation iv) where K is a constant. The sand wall or column F′ M will act as a strainer and the depression curve, showing the pressures under the floor, will take the shape $\gamma_o\ \gamma_o'$ the depth of this curve below the normal spring level = h_e indicating the head which produces the flow at any point P_o under the floor. This curve of depression $\gamma_o\ \gamma_o'$ will be very nearly the same as the curve of loss of head and will be a symplotic to the normal spring line. The length F′ P_o along F F′ from F′ as zero being approximately the same for any depression h_c as the depth F′ P′ for a loss of head h_c. This should be very nearly correct as the loss of head at each point is a function $\phi(y)$ of depth or length of floor, being H— (y).

If V_c is the critical velocity the soil particles from underneath the floor will be disturbed to a distance F_o.

Where $V = V_c$ and $h = h_c$ the critical head so that a crack will occur near but beyond this point of critical velocity. This process will gradually go on till by stages of degradation the settlement will reach the face wall of the siphon or other work and endanger the main structure, (mark the first crack on Jauryan siphon central wall which has occurred where h_e = 13″).

Effect of End Curtain Walls.

Next put in a line of sheet piles or a continuous toe wall to a depth y_1. The flow from underneath the floor will now occur with velocity Kh_1, and the depression curve will be $y_1\ y_1'$ with a critical head h_c occurring at P_1. As Kh_1 is greater than V_c, the dislocation of particles will still continue and a crack will occur near P_1.

If the sheet piles are driven to y_2 or y_3 the depression curves over the floor will be $y_2\ y_2'$ and $y_3\ y_3'$ and the cracks will occur near P_2 and P_3 The structure will become therefore safer as the toe walls or sheet piles are carried further down. When this sheet pile line goes to y_4 such that $kh_4 = vc$, the flow from underneath the piles will occur with a velocity less than the critical and therefore there will be no dislocation of soil particles and the structure will be safe. The problems is to determine the critical velocity and critical head for different types of foundation soils. That known, the design of the depth of toe wall or the piles, is a simple matter. With the help of the loss of head curve, h_c can be easily determined for each site by a series of observations with pressure pipes keeping (H) the depth of discharge water below normal spring level, as near the maximum as is likely to occur under the worst conditions. If the piles are carried a little lower than the calculated depth it will add to the stability of the work by giving a factor of safety.

That the above is in keeping with actual facts is clearly borne out by the relative pressure diagrams, under the pacca floors of the Dugri and the Jauryan siphons. The Dugri has a curtain wall of sheet piles 8′ below floor and the Jauryan has a curtain wall only 2′ 5′ below the bed consisting of wells. Compare photographs No. 5 and 6. In the Dugri the pressure gradient under the floor follows the normal gradient till the end reach where it curves down to the sheet pile line. The fact of pipe No. 5 always recording pressures below the normal gradient line was thus clearly explained. In the Jauryan the depression from the normal gradient line starts from near the face of the barrel. This is due to the shallow toe wall under which the velocities are higher. The dotted lines in both pictures give the normal gradients.

Under the Dugri floor, the uplift or "**blowing up**" pressures are about the maximum for the given normal spring level but the "**blowing out**" pressure, which undermines the floor by blowing out sand from underneath is little. Under the Jauryan floor on the other hand, "the blowing up" pressures have been reduced but the blowing out pressures have been dangerously increased. If H be the drop below the normal Gradient line to the apparent water level or the cunette water level at any point H_s the "**blowing up**" pressure or static Head and H_v the "blowing out" pressures or kinetic Head at that point then,

$H = H_s + H_v$ Since H is constant for any set of given conditions, H_s will increase or decrease by the amount of decrease or increase respectively, of H_v. In syphon floors H_s can be cheaply provided for by giving extra reinforcement in the floor slab but in Weirs, where gravity sections are necessary, the relative weights of H_s and H_v become very important and must be carefully considered. Increase of H_s means great expense and increase of H_v endangers the structures by undermining.

A safe mean can be obtained by somewhat reducing the thickness of floor and correspondingly adding to the depth and length of the inverted filter, depending on the depth of the toe wall which will govern both H_s and H_v.

It must, however, be clearly understood that the normal gradient line is not materially effected by the apparent water level on the bed. Pressure pipes driven to the necessary depth in the open bed outside the Dugri and Jauryan floors have indicated pressures in line with the normal gradient points. The Narora Weir observations show that the principle is of universal application.

It might be mentioned here that **Bernoullie's Law holds in case of subsoil water flow** just as much as in the case of free water flow, only the interpretation is more complicated.

Wells Versus Sheet Piles.

Attention is invited at this stage to the faulty nature of the wells as curtain wells. They act as efficient anchors—much better than sheet

piles—to the end of the floor but as to protection of soil from dislocation they are inefficient because the gaps between the different wells cannot be suitably plugged below a very small depth and, however well the blanking off is done by wooden or steel piles there always remains a vertical slit of varying width below the point to which open plugging of this gap is possible.

Thus in Fig. 7, plate III, if a gap is efficiently plugged up to b, the only portions of wells which is fully effective in keeping boxed the foundation soil, is to a depth of a b. Below that the soil movement will occur if bb' is greater than the critical head. However small the slit may be below b, this dislocation will continue to occur and the actual failure of the floor will be only a question of time. In works of this nature, therefore, wells are inefficient for protection of foundation soil, Sheet Piles are better in this respect though they are not self-supporting to the same extent as wells.

The use of the water-jet, however, in clearing the earth from in between the adjacent wells, enables the plugging to be done effectively to any required depth : where this is possible, the wells will be distinctly superior to the sheet piles.

Piling up of Water Pressure against Steel Piles.

Considerable anxiety has been expressed by eminent engineers regarding piling up of pressure against steel sheet piles at the end of a floor. A reference to plate III figure 6 will give a clear conception. If the piles go down very deep the pressure under the floor behind the steel piles will rise very close to the normal gradient line. For steel sheet piles of depths y_1 y_2 y_3 y_4, the pressure gradients (or depression curves) are clearly indicated. The floor can be made strong enough in reinforced concrete to resist this maximum pressure (**i.e.,** the difference between the N. S. L. and the floor top) and no fear need be entertained for this piling up of pressure. Plate VI showing pressure line along centre wall of D. S. floor of Dugri I is a good illustration, in this respect, as there is a line of steel piles 20 feet deep under this floor 7 feet from the face.

Relief Strainers.

Plate VIII shows the effect of inserting under the floor, strainers discharging water out in the cunette or drain. The upper pressure line is the normal spring line with all strainers closed, the only relief to the percolation water from the canal being in the subsoil and at the outer end of the sheet piles, through the inverted filter or the open drain bed. It might be noticed how the line is steep near the canal and then flattens out to almost 1 : 50 along the normal spring line (also *see* photograph No. 5).

The lower line indicates pressures if the strainers are permitted to run. The section through the cone of depression is of the same family as the depression curve yo yo' in figure 6.

This depression curve of the strainer shows that the strainers are located in the wrong position, for the end of the floor, which it is desired to protect from the dislocation of soil particles, gets practically no relief or at this point the curve has again risen close to the normal spring line the net relief being 0·3 feet. If the sheet piles were non-existent this floor will fail inspite of the relief strainers.

The correct position of the strainer would be immediately below the sheet pile line. For this it will give the maximum relief at the end of the floor and the depth of the sheet piles could be safely cut down to the extent explained below (*see* plate IV, figures 8 and 9).

At the strainer the sheet piles could be theoretically cut down to nothing, as all the pressure will be relieved by the strainer but this is risky, as, if for any reason the strainer is closed or gets choked the full pressure will be brought into play and render the structure unsafe. The controlling feature will, however, be the spacing of the strainers along the line of sheet piles.

Let the strainers be spread x feet apart. Taking the section through those along the sheet pile line we get the series of depression curves of figure 10. At the centre of the strainers we shall still have to provide for a pressure of y feet which will be a function of x the spacing. On the loss of head curve find out L where loss of head is y.

If M is the point of crtitical head and critical velocity, then the length of sheet piles required will be LM. Any depth below this will be a factor of safety.

Besides relieving pressure from underneath the floor and its end and thus preserving its foundation soil from dislocation a strainer, at this position, also relieves pressure under the open bed outside the pacca floor. This open floor may be an invested filter or pitching. Since water is flowing towards the strainer there can be no tendency for soil particles to move except towards the strainer. This they can not do, as it is all compact soil and the strainer is supposed to be proof against silt intrusion. Naturally those soil particles cannot move towards the open floor within the radius of influence of strainers. This helps to preserve the soil outside the pacca floor adjoining the end and therefore adds considerably to its safety. The extent of relief afforded to the open floor is indicated by the shaded area LMN., (figure 8). Thus at any section P, the pressure tending to blow up soil particles will be P Q and not P R=H.

Apparently the correct policy would be to take the sheet piles up to the critical depth and then give, in addition, strainers for relief.

In case of strainers the very finest slits must be used so as to preclude any possibility of the fine soil particles being carried away, thus causing

cavitation under the floor, and defeating the very object which the strainers are meant to serve. Also the strainers should be blanked off in clay strata, as no matter how fine the silts are, clay will get into them and cause cavitation under the floors in time. If there is doubt about the strainer being absolutely proof against silt intrusion, their insertion must be carefully considered.

Critical Head.

From the foregoing investigation it is clear that the critical head for any class of foundation soil will determine the design of structure on the soil. The question is what is the **Critical Head.**

Colonel Clibborn in his Roorkee experiments found that with Khanki sands the blowing of sand occurs under heads of 1½ to 3 feet, but the experiments not being conclusive no definite law could be established. (Technical Paper Government of India No. 97 of 1902.) (Plate IV, figure 1.) Regarding pressure lines of figure 3, Plate II, he records on page 5, para. 11 that sand was carried up the pipe with a head of 19 feet, etc. The normal H gradient line when produced cuts the line of vertical pipe about 6″ above the exit level of water, giving a head of 6″ which dislocated sand. Again in plate IV, figure 1, the sand blowing occurred at 16 feet head. The normal H. G. line, in this case, cuts the vertical through the glass pipe 9″ above water exit level showing that the head to dislocate sand was only 9″. From these it appears that the **Critical Head for Khanki sand is 9″.**

Mr. Beresford on page 31 of the same paper gives his opinion based on tests that "water passed through without moving particles of sand, provided the **head** of **water** pressure did not exceed one-third of the depth or thickness of sand. Increased head produced turbidity and a further increase to half depth or over caused the sand to blow. The tendency was completely stopped by the addition of 3″ to 6″ of fine ballast on the surface of sand which was 45″ deep." The last sentence contains the **germs of the idea of adding inverted filters** of graded ballast at the end of a pacca floor.

Applying the test of Mr. Beresford to the curve of plate VII the head being 3·8 feet, three times head=11·4 feet. The loss of head at that depth is 0·7 feet or 8½″.

The class of sand at Dugri I being very nearly the same as the Khanki sand the agreement between 8½″ and 9″ is satisfactory.

Parker in his control of water (page 299) gives L = 3·6h for sands of the Khanki type. This from the curve will give a still lower value of critical head.

The critical head for the Punjab sands may therefore be accepted from 6 inches to one foot.

This Critical Head will obviously be greater for coarse sand, gravels etc.

Critical Gradient.

Like the critical head, there is a critical gradient for each class of soil. If the hydraulic or pressure gradient of flow exceeds this critical gradient at any point the velocity will be higher than the critical and the soil particles will start moving beyond that point. In the case of the curve of plate VII this critical gradient for a depth of 11·4′ (which has been assumed to be the critical depth) is very nearly 1 in 9. With Parker's ratio of 3·6, this gradient is 1 : 13.

Further Scientific Research.

It has not been possible so far to carry out any experimental work on scientific lines regarding the critical head and critical gradient for various classes of soils with various degrees of packing. This is a problem which can be best investigated in the Scientific Research Laboratory at Lahore, and the results will be well worth the labour spent. The investigation of critical velocity may be of use but has no practical application in the design of structures.

Surface Water effecting Spring Level.

Dr. Bose in memoir on the flow of liquid in saturated soil (April 1929) and Mr. Wilsdon in the introduction to the memoir, have ably brought out certain important points regarding laminar flow and the effect on stream lines, of adding water on the surface or of withdrawing water by infilteration galleries. A complete study of the stream lines of flow and lines of equal velocity potential is of the greatest importance for a correct appreciation of the effect on uplift and on soil dislocation, of the depth and location of curtain walls, under floors of weirs, siphons, etc., resting on saturated soils. Experiments were carried out at the Dugri and Jaurian siphons to determine the effect of raising and lowering the water level in the cunette without changing the canal level with remarkable results. These results are confirmed by the available datas of pressure pipes observations at the Narora and Sulemanki weirs.

Plate V shows that with the canal supply level remaining unchanged, a rise in cunette level from 89·4 to 90·6 has at once given a corresponding rise in the pressure pipes under the floor and in the open bed. Sufficient and conclusive observations are not available to establish the general law for the amount of increase at various depths in the vertical direction or distances in the horizontal direction, but it appears that the rise becomes

smaller as the bottom strainer of the pressure pipe goes down or as the distance increases from the toe wall toward the canal (*see* table below).

	Dugri Upstream.				Dugri Downstream.				
S. No.	Bed Level of pipe.	21-7-29	2-8-29	Diff.	Bed level of pipe.	21-7-29	2-8-29	Diff.	
Canal ..	W. L.	95.52	95.52	0.0	..	95.52	95.52	0.0	
Cunette ..	,,	89.54	90.60	1.06	..	88.35	89.9	1.55	
1 ..	778.75	94.12	94.72	0.62	96.3	93.72	94.02	0.28	
2 ..	778.24	94.01	94.59	0.58	70.3	92.94	93.52	0.58	
3 ..	780.20	93.91	94.32	0.41	78.8	93.10	93.53	0.43	
4 ..	779.20	93.79	94.44	0.65	78.1	93.01	93.47	0.46	B
Strainer ..	771.70	93.30	93.98	0.68	78.5	90.36	90.95	0.59	
5 ..	*780.80	93.10	93.55	0.45	74.6	92.86	93.38	0.52	
6 ..	*780.782	91.50	91.69	0.19	82.2	92.57	93.12	0.55	
7 ..	*778.780	92.40	92.90	0.50					
8 ..	763	93.11	93·89	0.78					
9 ..	768	92.98	93.80	0.82	A B=pipes under floor.				
11 ..	767	93.52	94.09	0.57	A=pipes in open drain bed.				
12 ..	768	93.07	93.86	0.79					
Mean ..		93·23	93·82	0·59	Mean	92·65	93·14	0·49	

* For pipes 6 & 7, upstream, the filter levels are different on the two dates which accounts for the low differences.

A possible explanation for this phenomenon is, that a rise in the cunette water level exerts a back pressure against the general direction of flow, thus reducing the velocity of flow under the floor. From Bernoullies law, assuming that it holds, the static head or the upward pressure increases by the amount that the kinetic head is reduced due to the reduced velocities. The practical application of the experimented results and phenomena outlined in this paper, has been dealt with in a separate paper.

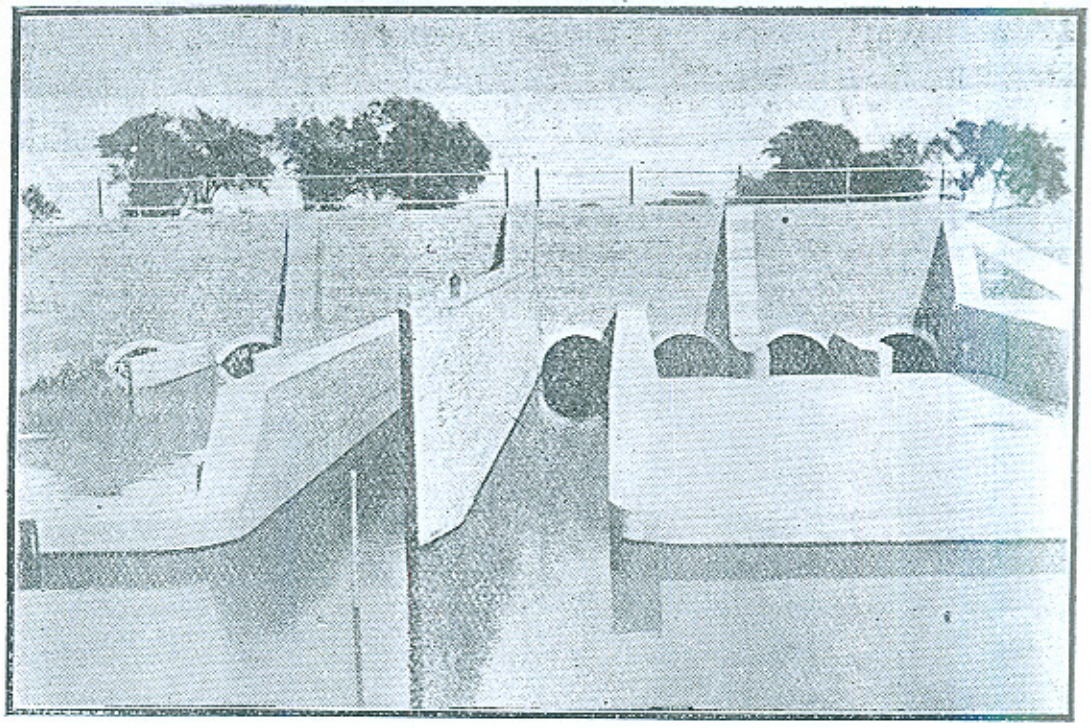


Photo No. 1.
Dugri 1 siphon Downstream showing finished structure.
April 1929.

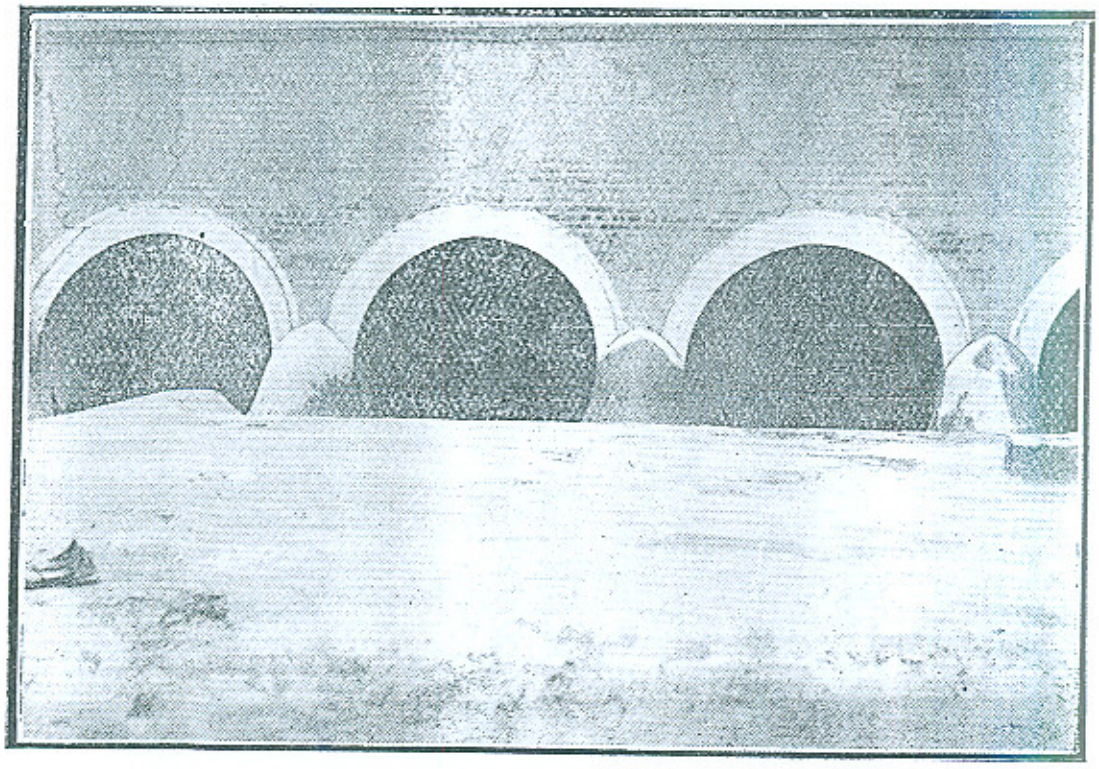


Photo No. 2.
Dugri 1 siphon Downstream. Before remodelling January 1929.
Showing cracks in face wall and round barrel rings.

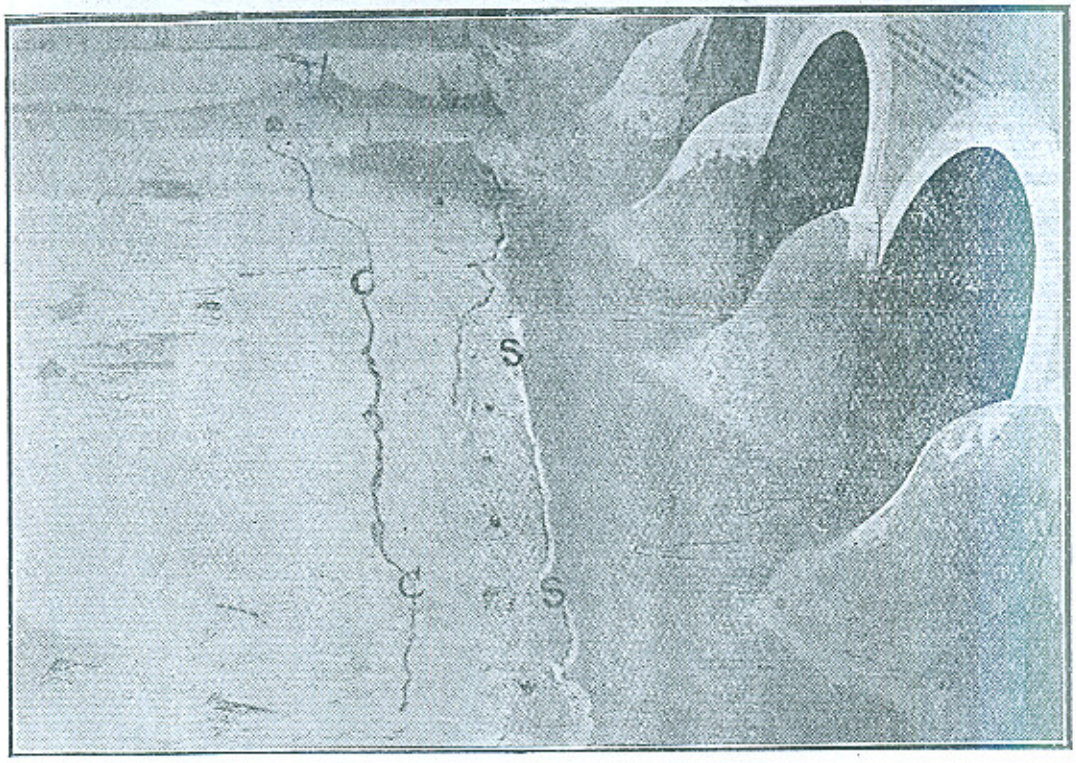


Photo No. 3.
Dugri 1 siphon Downstream. Before remodelling January 1929.
Showing cracks in floor SS at sheet pile line and CC Downstream. Note the numerous pipes let in for grouting during previous years.

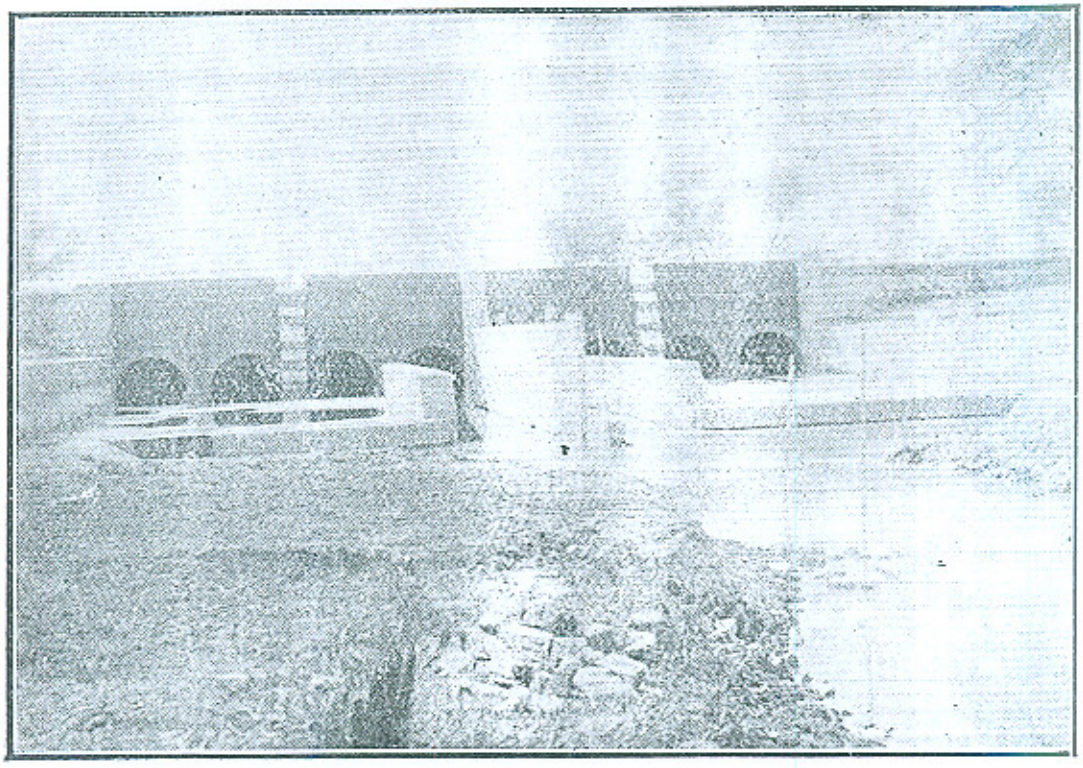


Photo No. 4.
Dugri 1 siphon Upstream completed in January 1929 showing arrangement of pressure pipes for experiments.

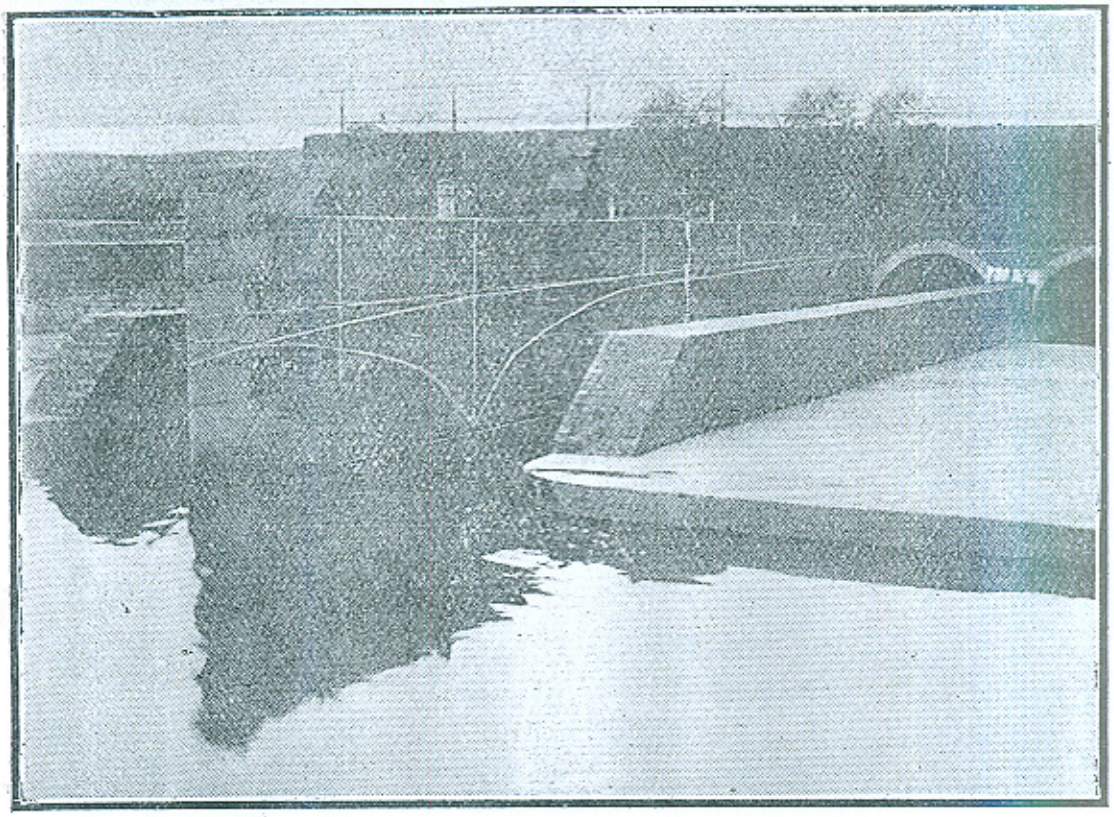


Photo No. 5.
Dugri 1 Upstream (September 1929) showing normal H. G. Line (8′ deep toe wall of sheet piles at the end) also effect of a relief strainer (Tej type).

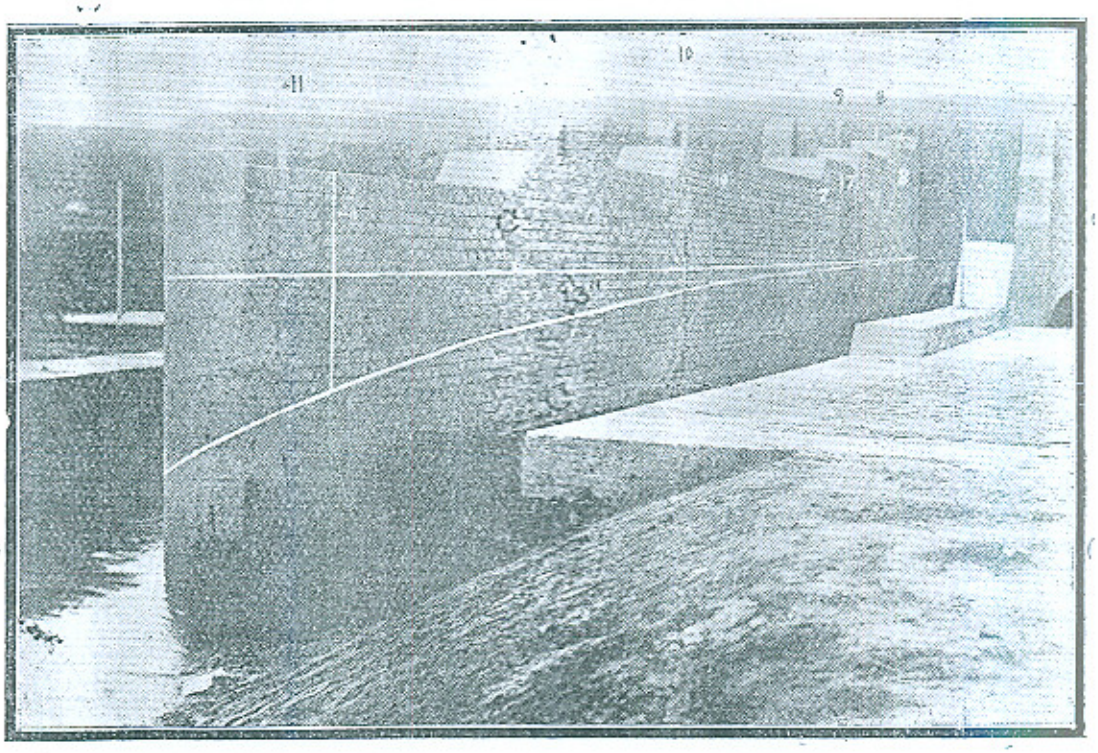


Photo No. 6.
Jaurian siphon Upstream (September 1929) showing normal Hydraulic Gradient line and actual H. G. line. Note the steepening gradient towards toe wall which is shallow. Note also crack C C which has occurred with $H_c = 13''$ at projection of crack where it cuts base.

PLATE I.

FIG 1

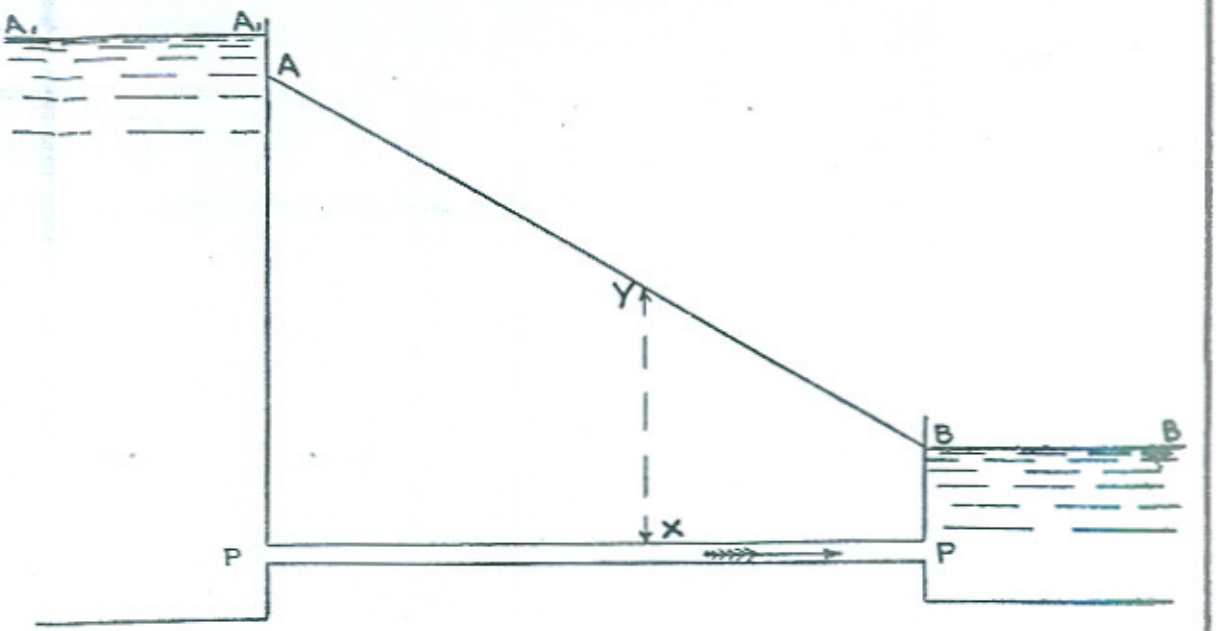
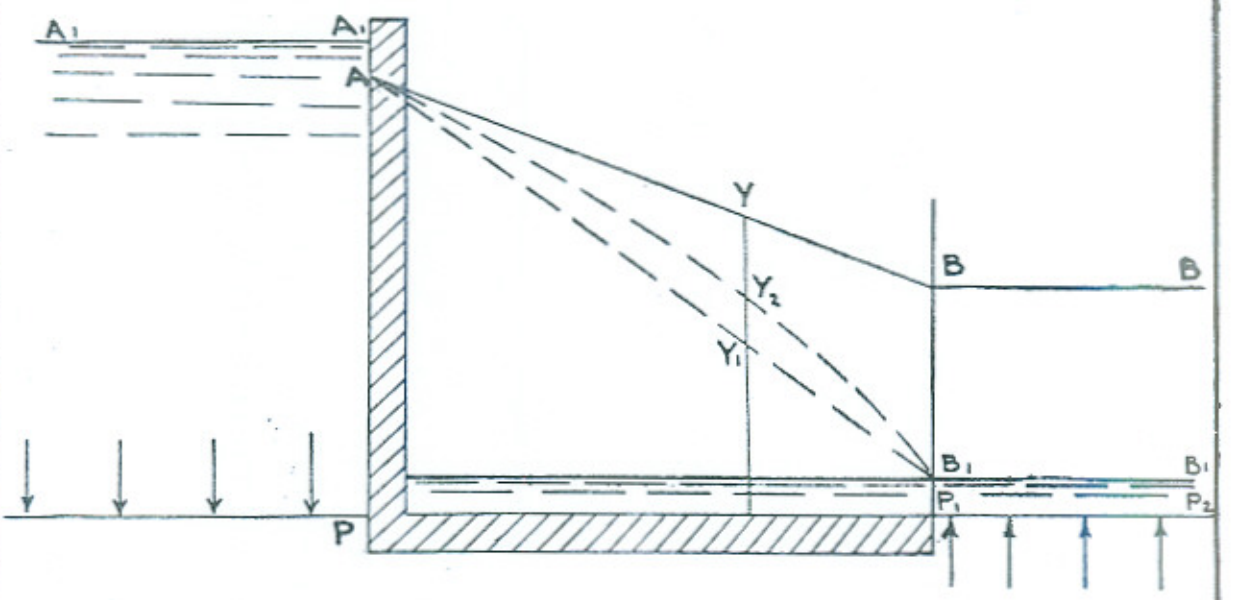


FIG 2



Ankhosla

PLATE II.

FIG. 3

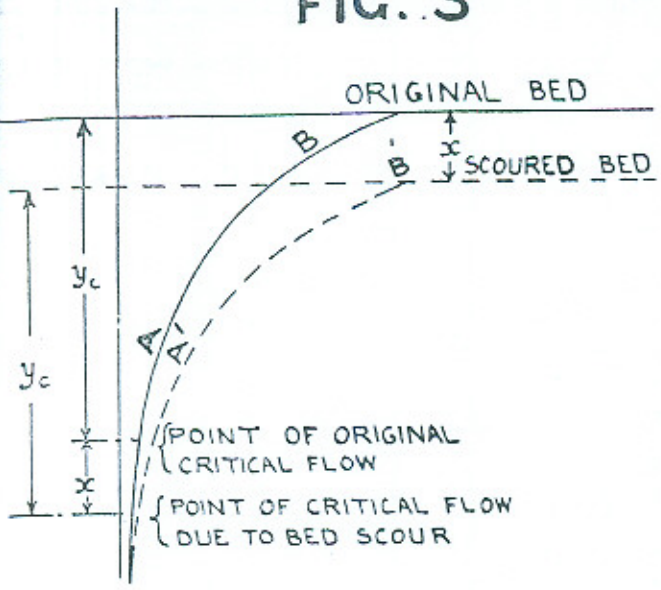


FIG 5

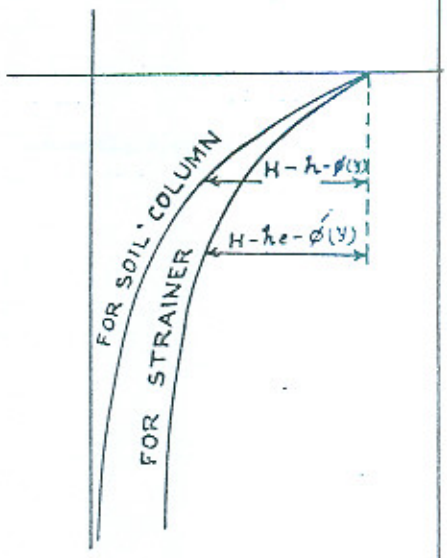
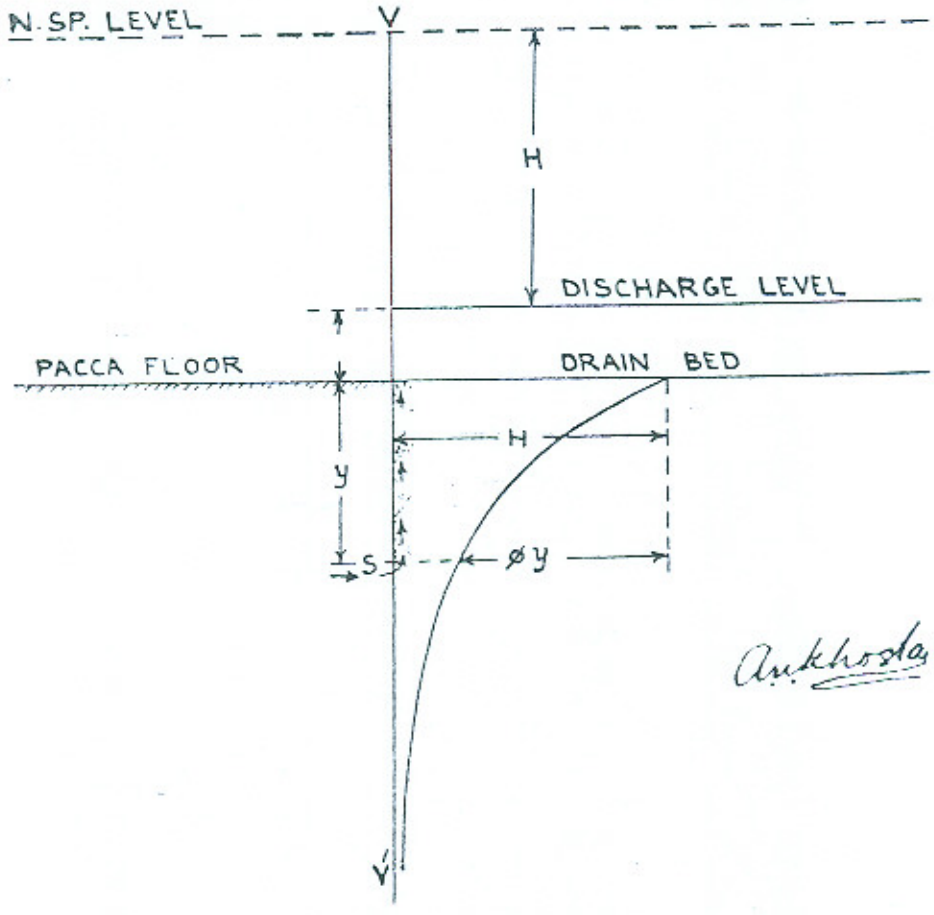


FIG. 4



Ankhosla

PLATE III

FIG. 6

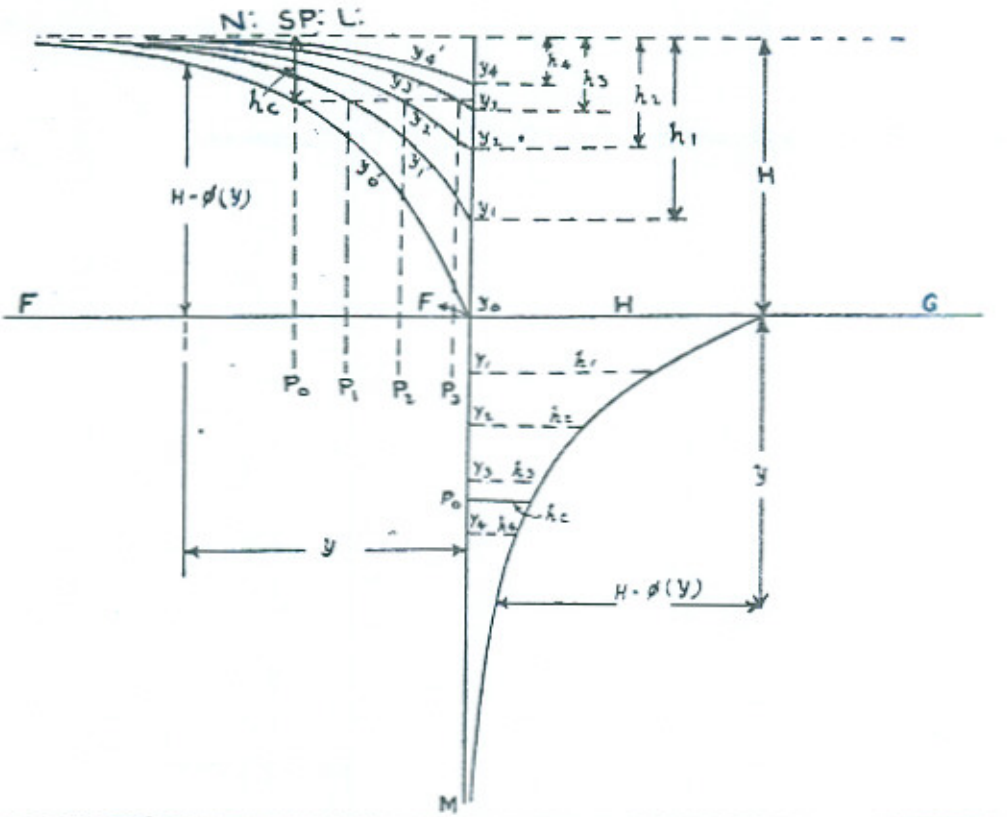


FIG. 7

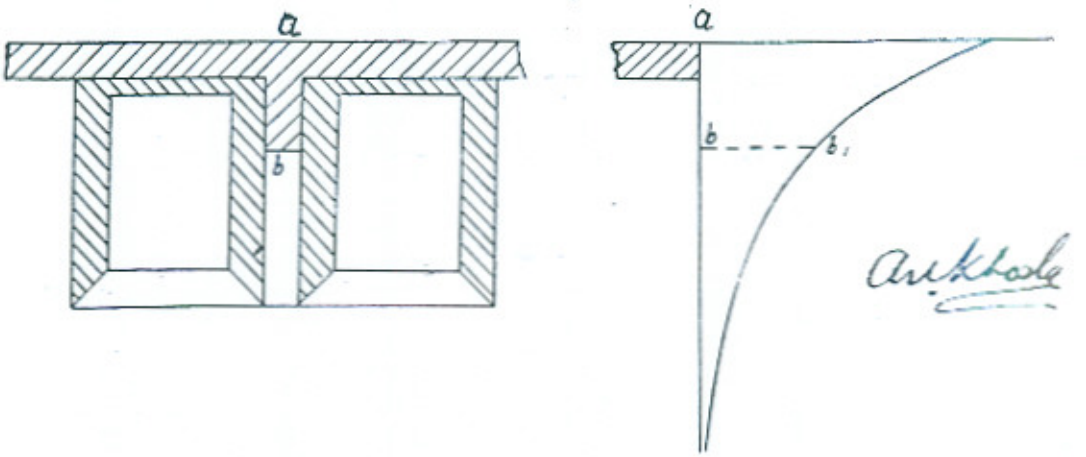


PLATE IV.

FIG. 8

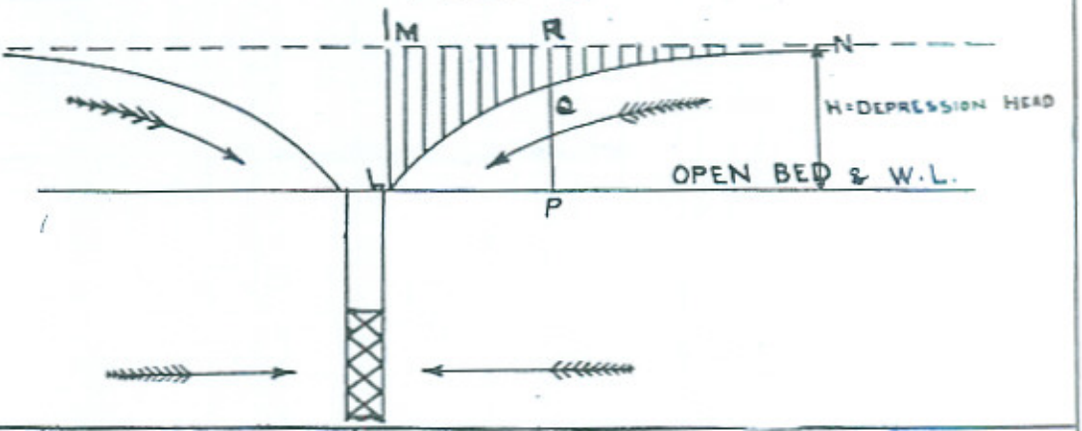


FIG. 9

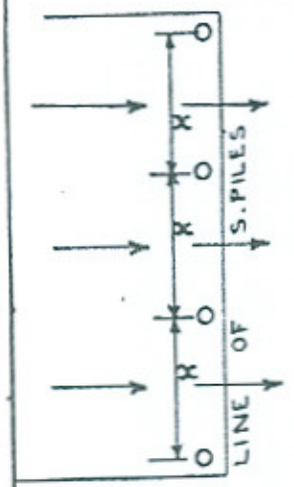


FIG. 11

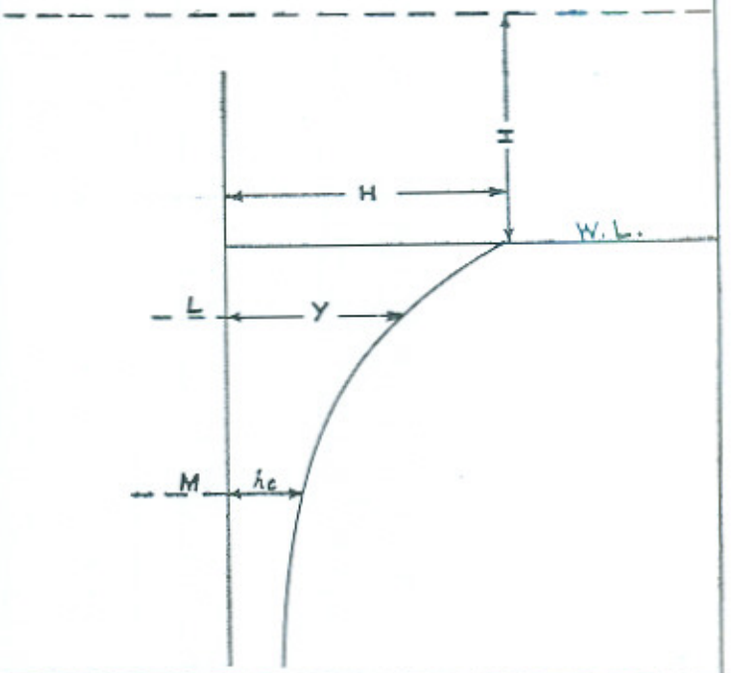
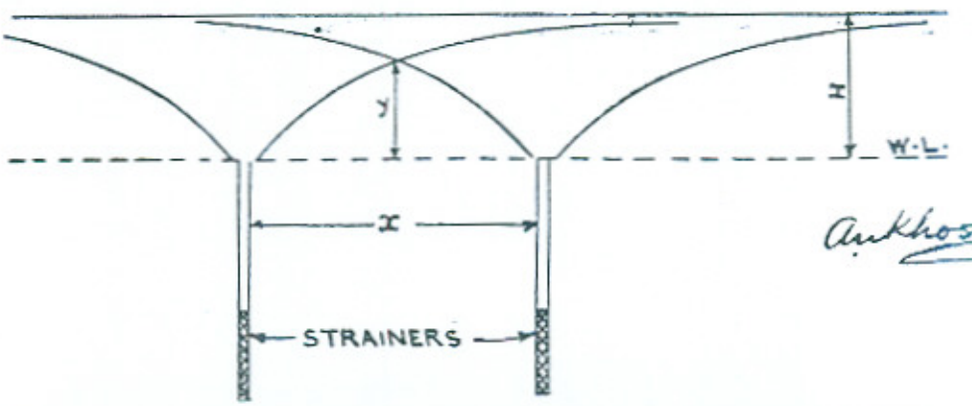


FIG. 10



Ankhosle

PLATE V.

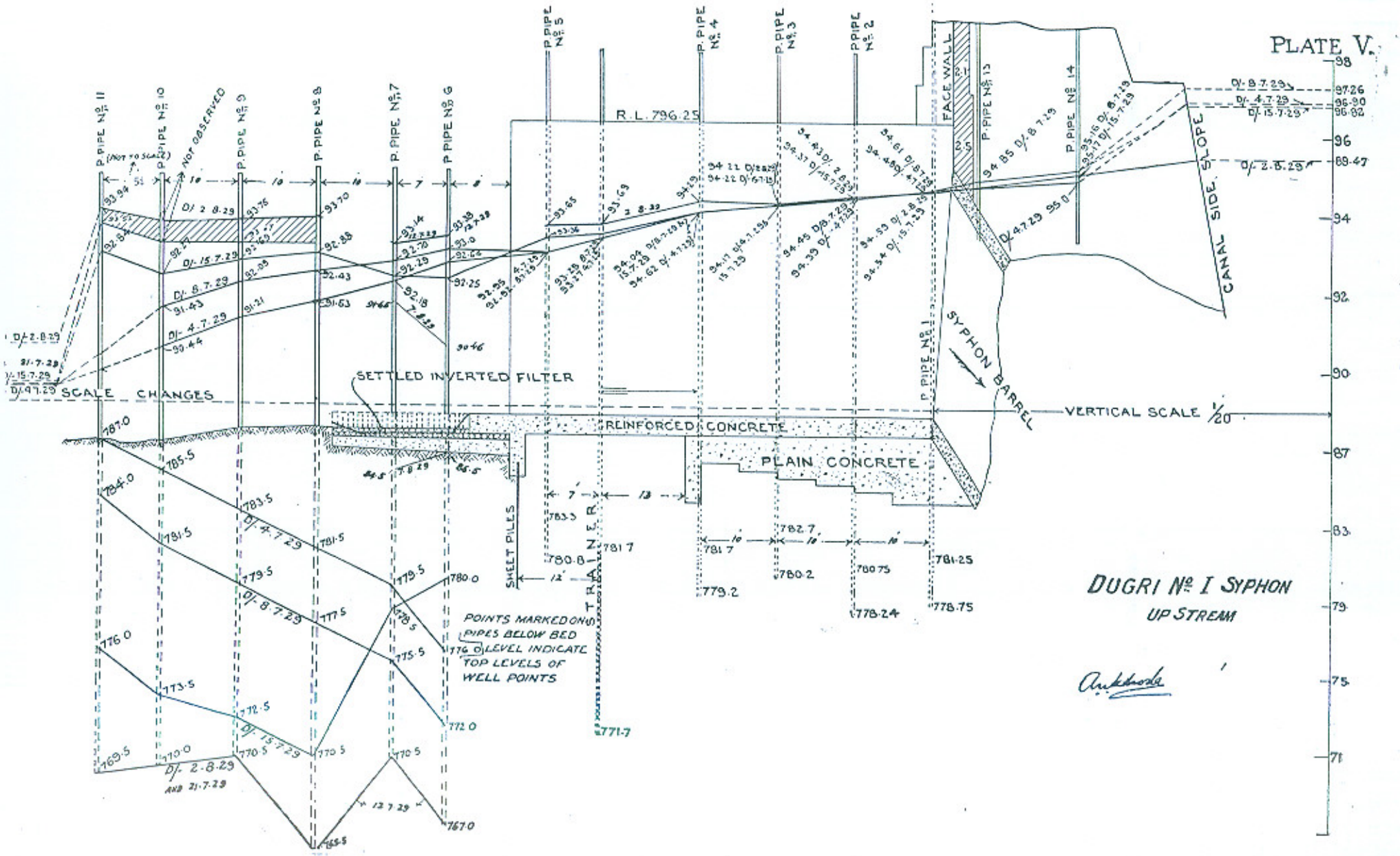

P. PIPE No. 11
P. PIPE No. 10
P. PIPE No. 9
P. PIPE No. 8
P. PIPE No. 7
P. PIPE No. 6
P. PIPE No. 5
P. PIPE No. 4
P. PIPE No. 3
P. PIPE No. 2
P. PIPE No. 1
P. PIPE No. 13
P. PIPE No. 14
FACE WALL
CANAL SIDE SLOPE
SYPHON BARREL
R.L. 796.25
SETTLED INVERTED FILTER
REINFORCED CONCRETE
PLAIN CONCRETE
SHEET PILES
TRAINER
SCALE CHANGES
VERTICAL SCALE 1/20
POINTS MARKED ON PIPES BELOW BED LEVEL INDICATE TOP LEVELS OF WELL POINTS
DUGRI No. I SYPHON
UP STREAM
D/- 2.8.29
D/- 15.7.29
D/- 8.7.29
D/- 4.7.29
AND 21.7.29
12.7.29

PLATE VI.

DUGRI No. I SYPHON

DOWNSTREAM

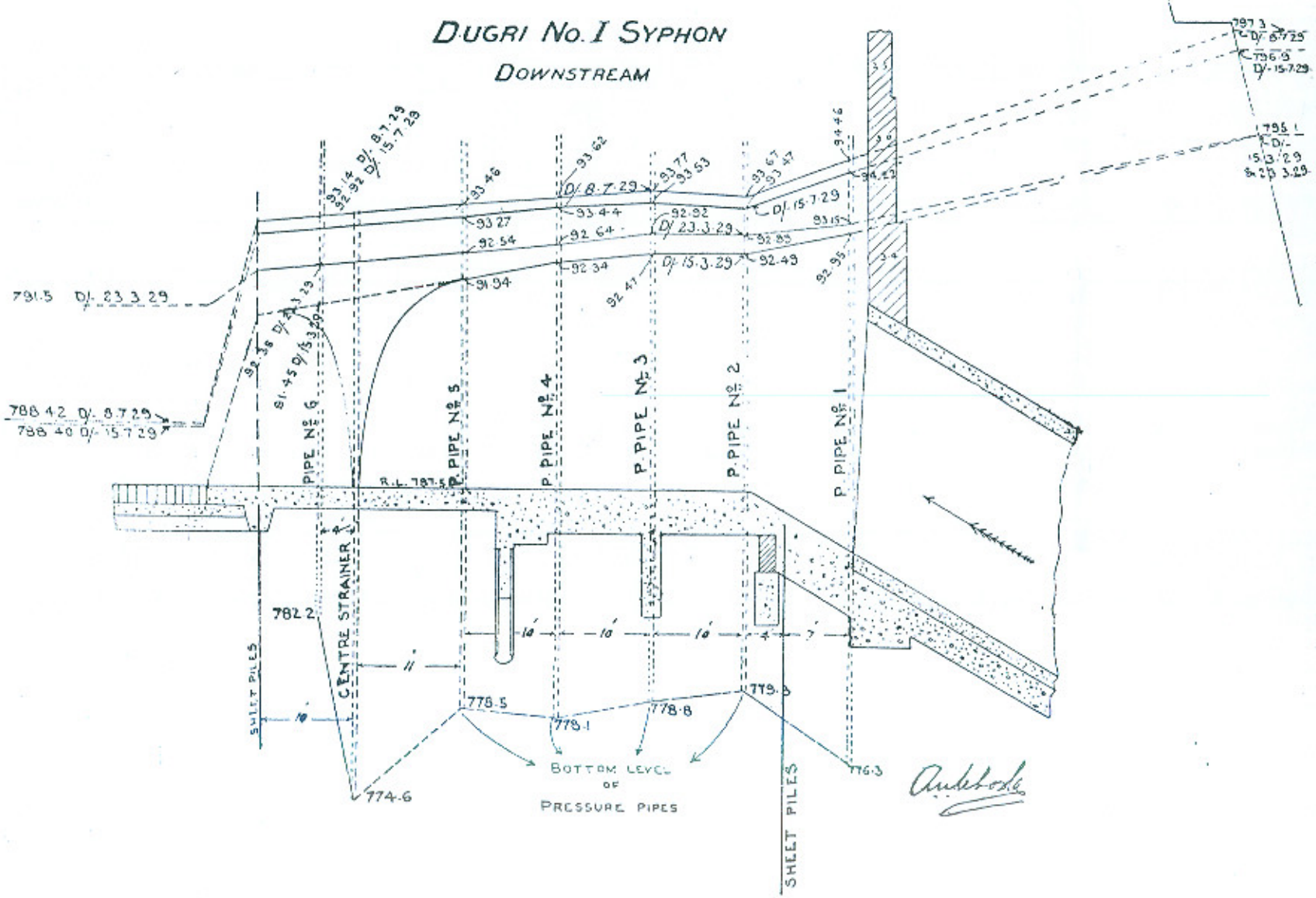
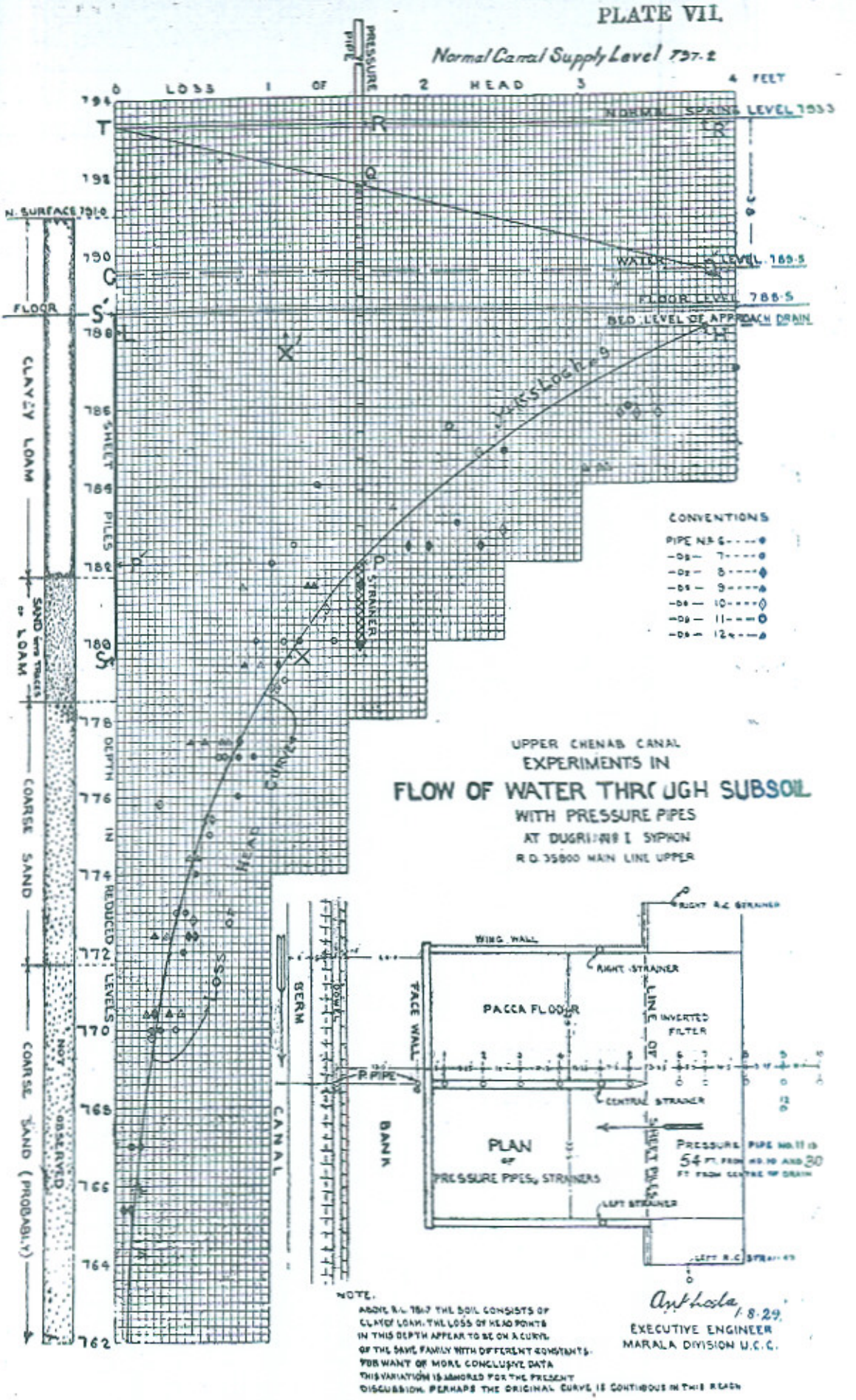


PLATE VII.



UPPER CHENAB CANAL

EXPERIMENTS IN

FLOW OF WATER THROUGH SUBSOIL

WITH PRESSURE PIPES

AT DUGRI SIPHON I

R.D. 35800 MAIN LINE UPPER

NOTE.

ABOVE R.L. 784.7 THE SOIL CONSISTS OF CLAYEY LOAM. THE LOSS OF HEAD POINTS IN THIS DEPTH APPEAR TO BE ON A CURVE OF THE SAME FAMILY WITH DIFFERENT CONSTANTS. FOR WANT OF MORE CONCLUSIVE DATA THIS VARIATION IS IGNORED FOR THE PRESENT DISCUSSION. PERHAPS THE ORIGINAL CURVE IS CONTINUOUS IN THIS REACH

Anthala 1.8.29.

EXECUTIVE ENGINEER

MARALA DIVISION U.C.C.

PLATE VIII.

DUGRI No. I. SYPHON UPSTREAM

SHOWING EFFECT OF STRAINER WORKING & CLOSED

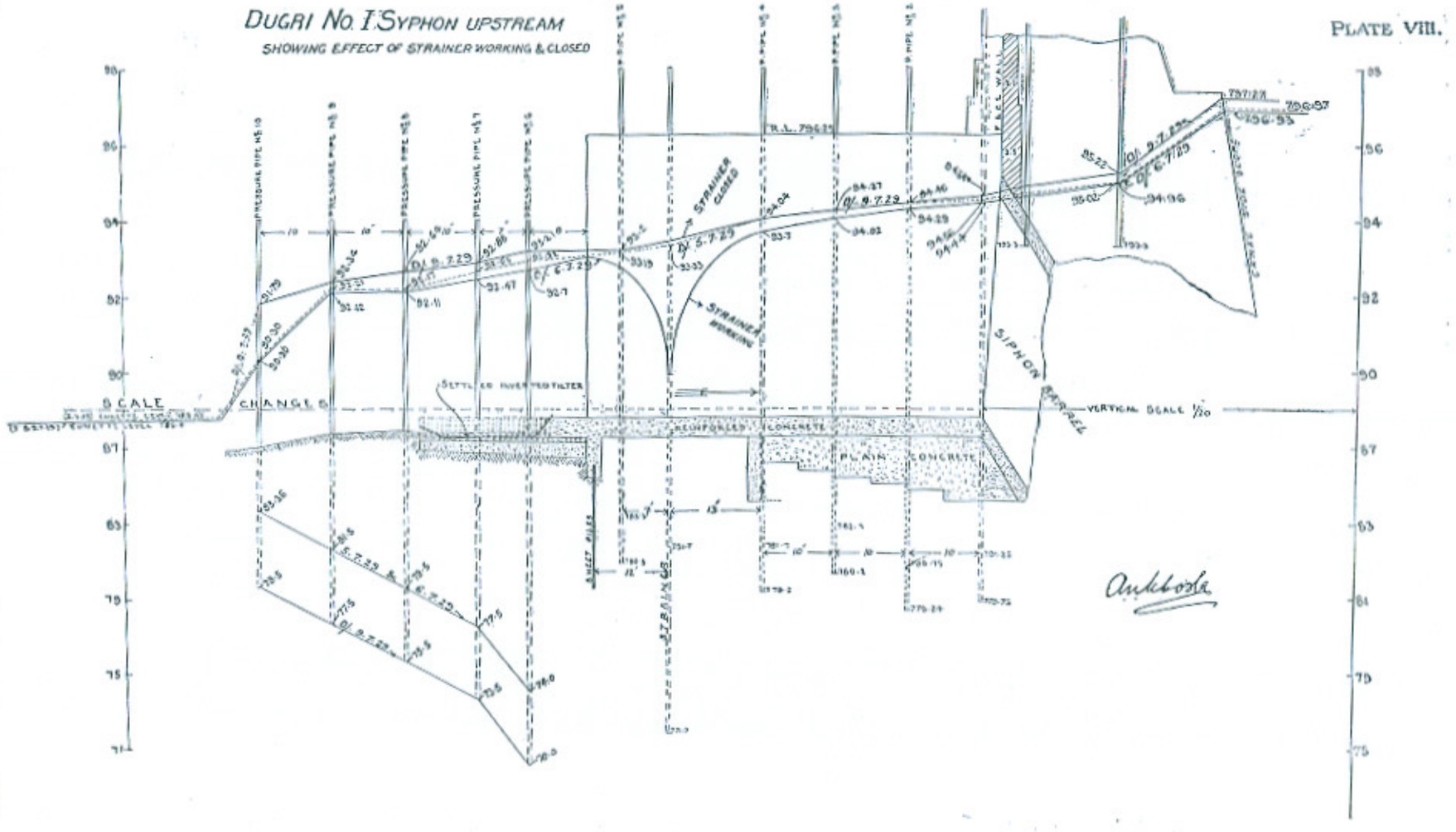
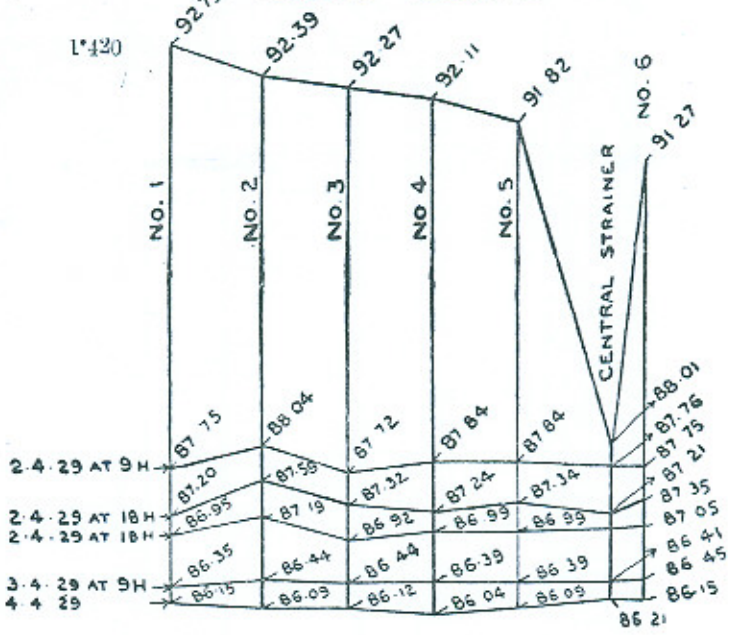


PLATE IX.

DUGRI NO. 1 SYPHON

DOWNSTREAM

CANAL CLOSING



JAURIAN SYPHON U.S.

PRESSURE PIPES BEHIND PUDDLE

CANAL OPENING

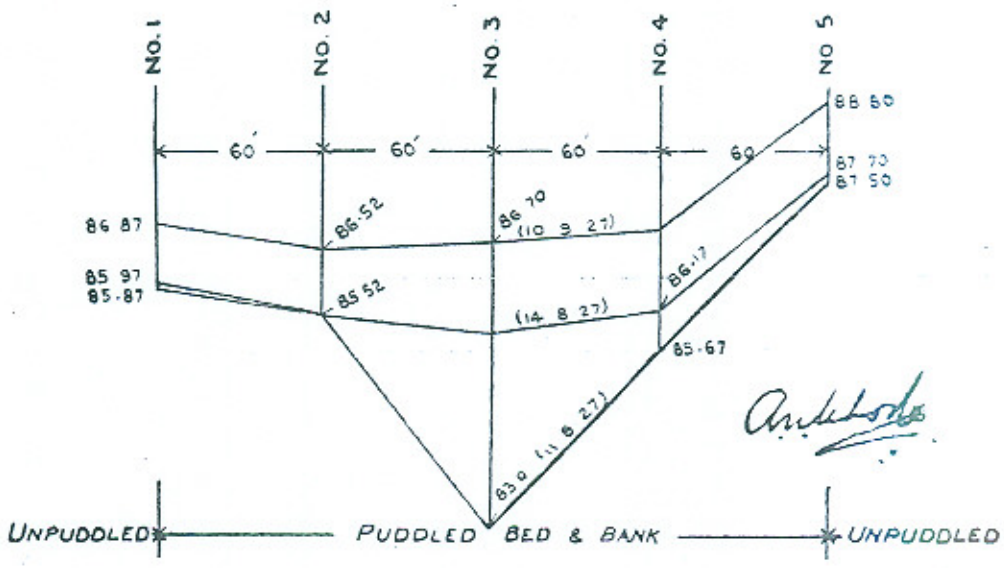
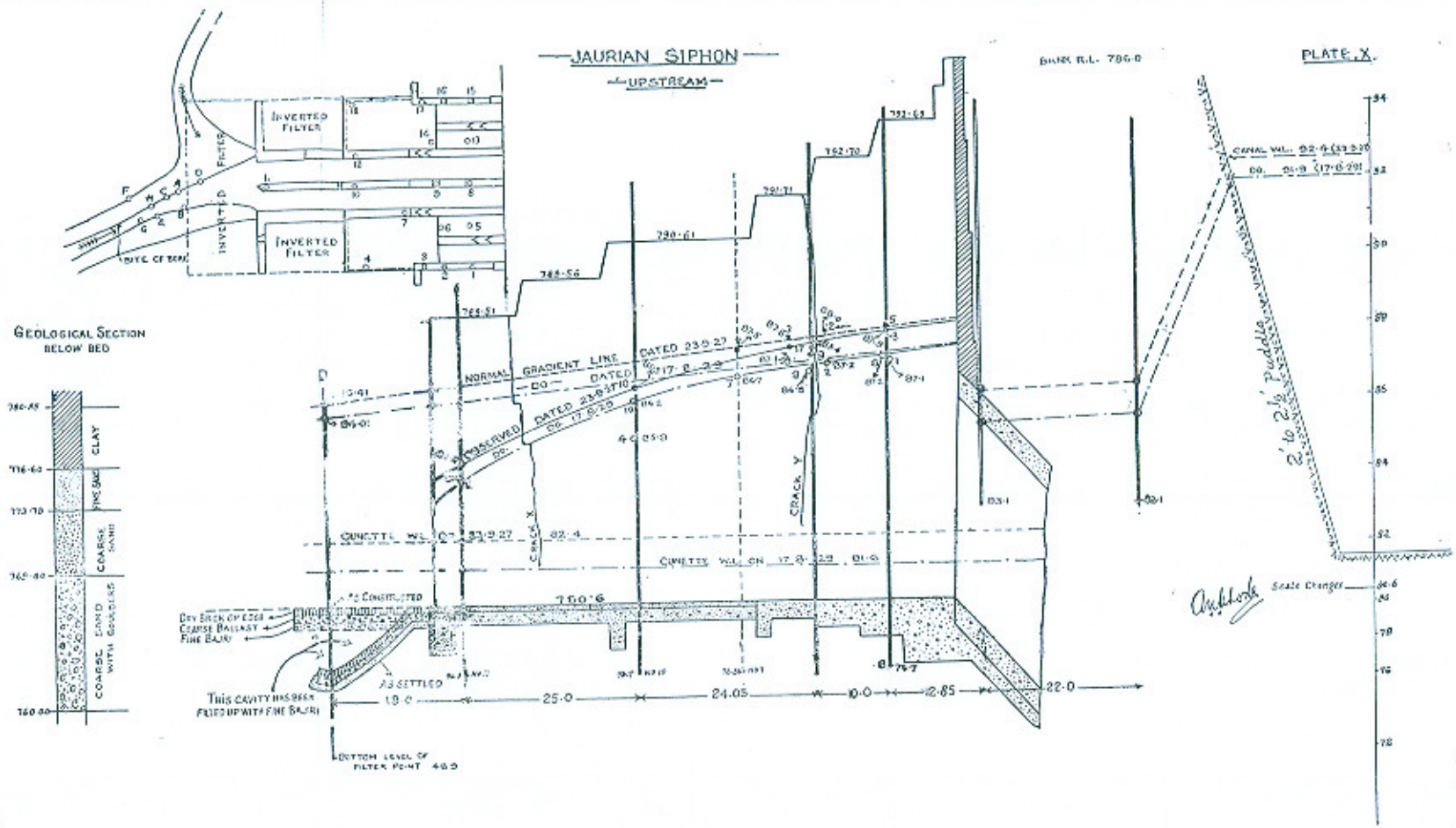
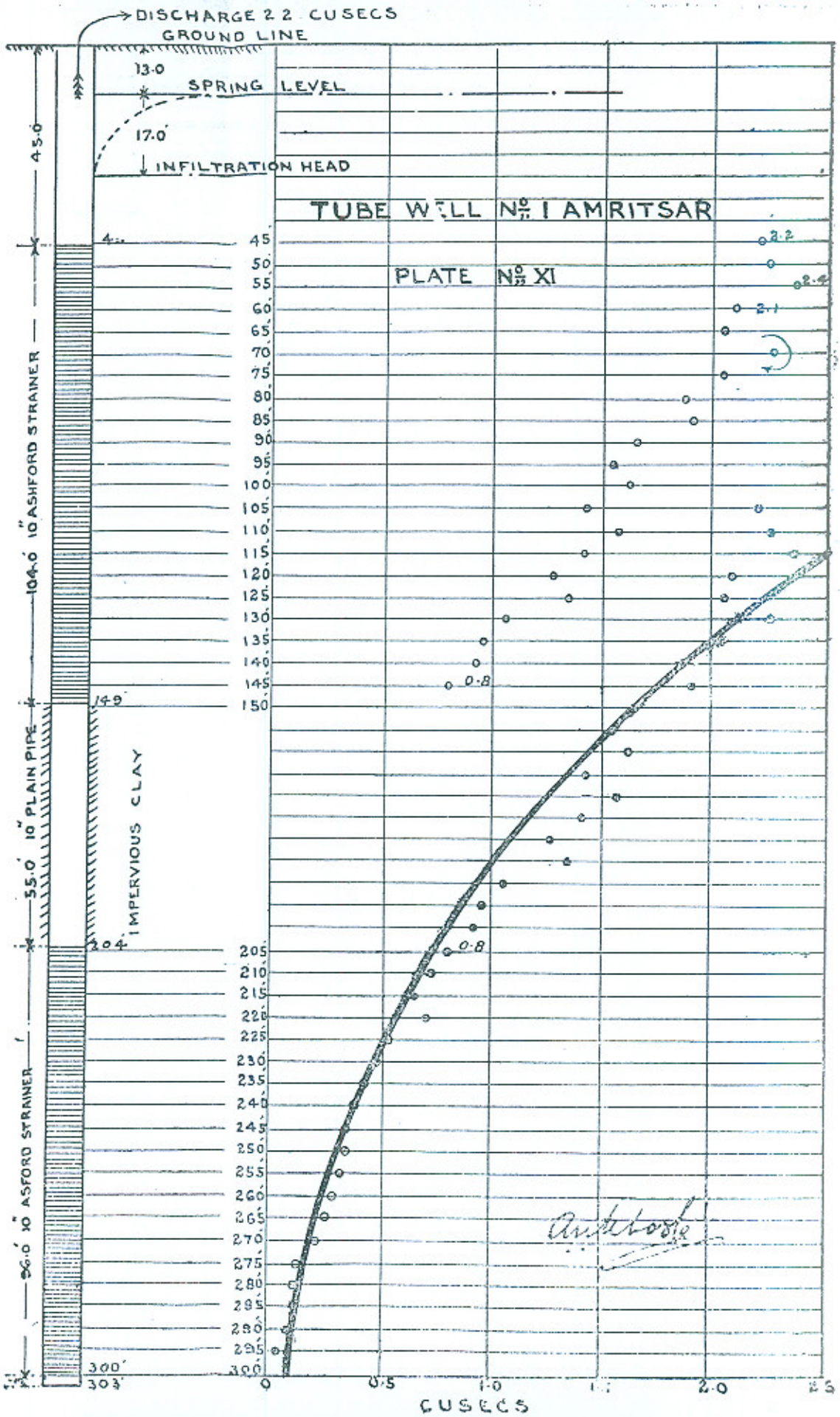
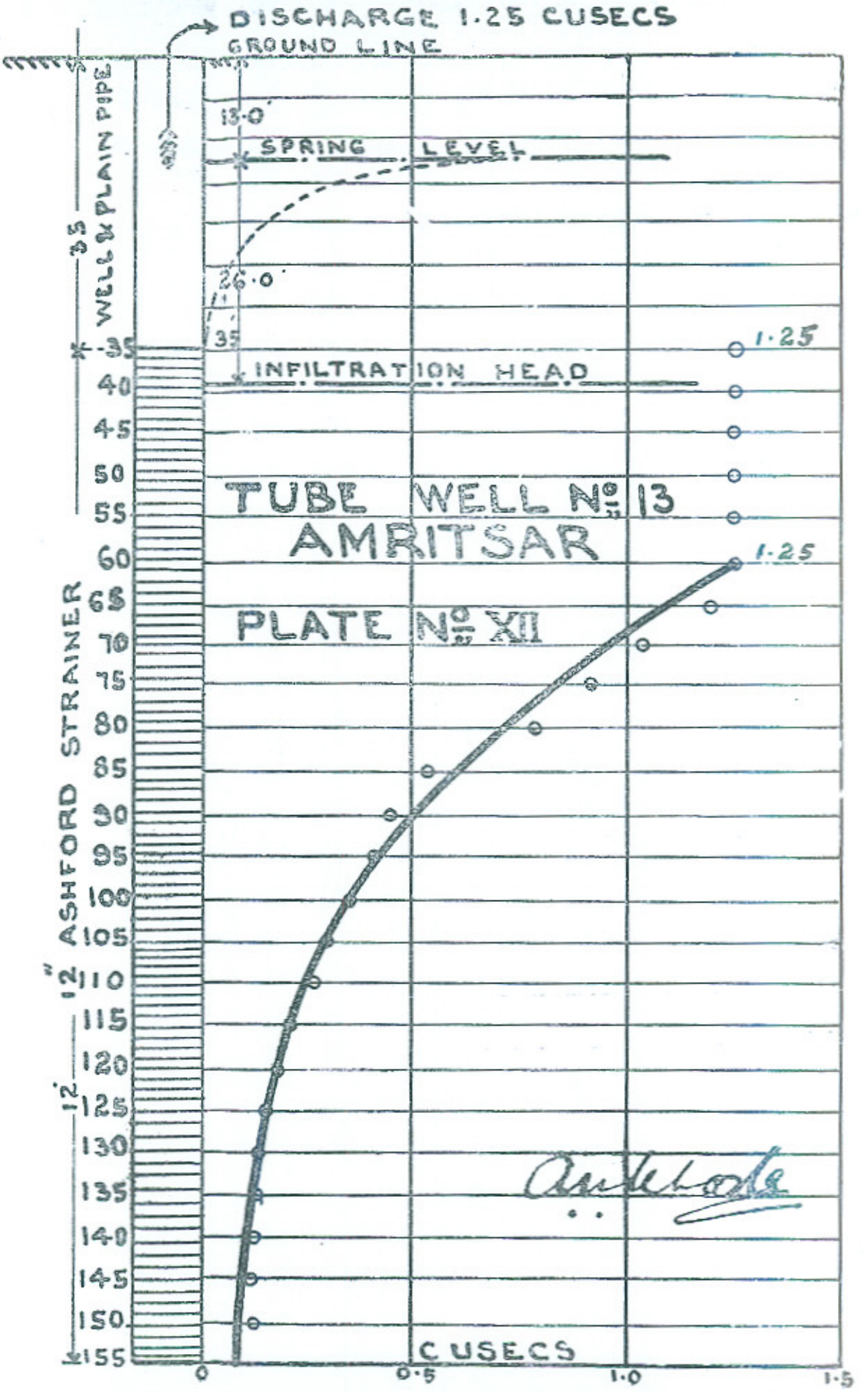


DIAGRAM SHOWING THAT PUDDLE IN JAURIAN BED IS ABSOLUTELY WATER TIGHT AS THE RISE IN PIPES FROM ENDS INWARDS TAKES INCREASING TIME TO COME TO NORMAL AS A RESULT OF FLOW THROUGH THE BANK FROM UPSTREAM AND DOWNSTREAM ENDS OF PUDDLED BED & SLOPES AND NOT A RESULT OF DIRECT FLOW THROUGH THE PUDDLE.


JAURIAN SIPHON
UPSTREAM
PLATE X.
INVERTED FILTER
INVERTED FILTER
INVERTED FILTER
BANK R.L. 786·0
GEOLOGICAL SECTION BELOW BED
CLAY
FINE SAND
COARSE SAND
COARSE SAND WITH GOOD STONE
NORMAL GRADIENT LINE
OBSERVED
CUNETTE W.L. ON
CRACK X
CRACK Y
Fe CONSOLIDATED
DRY RUBBLE PITCHING ON EDGE
COARSE BALLAST
FINE BALLAST
AS SETTLED
This CAVITY HAS BEEN FILLED UP WITH FINE BALLAST
BOTTOM LEVEL OF FILTER POINT 46·0
18·0
25·0
24·05
10·0
12·85
22·0
CANAL W.L. 92·4
2' to 2½' Puddle
Scale Changes


DISCHARGE 2.2 CUSECS
GROUND LINE
13.0
SPRING LEVEL
17.0
INFILTRATION HEAD
TUBE WELL No. I AMRITSAR
PLATE No. XI
45.0
104.0' 10" ASHFORD STRAINER
55.0' 10" PLAIN PIPE
IMPERVIOUS CLAY
96.0' 10" ASFORD STRAINER
149'
204'
300'
303'
2.2
2.1
0.8
0.8
0
0.5
1.0
2.0
CUSECS


DISCHARGE 1·25 CUSECS
GROUND LINE
13·0
SPRING LEVEL
26·0
35
INFILTRATION HEAD
1·25
TUBE WELL No. 13
AMRITSAR
PLATE No. XII
1·25
35' WELL & PLAIN PIPE
12" ASHFORD STRAINER
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
115
120
125
130
135
140
145
150
155
0
0·5
1·0
1·5
CUSECS

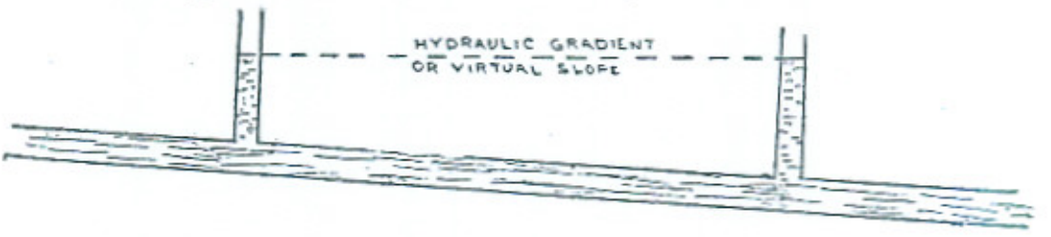
DISCUSSION.

Mr. Khosla in introducing his paper drew attention to the essential features of his theory of subsoil flow. He laid special emphasis on the fact that the equation for the "Loss of Head Curve" proposed by him was of universal application in a relatively wide channel and had received confirmation from Dr. Bose's mathematical treatment of the subject in his paper No. 140.

Mr. W. P. Thompson in a written communication said that the statement on page 140 that "according to the commonly accepted theory" A. B. 1 in figure 2, plate 1, would represent the hydraulic gradient was not fair to the service generally as there were some Engineers who did not subscribe to it.

All the elementary books on hydraulics described the "hydraulic gradient" to which also was given the name "virtual slope" by means of an illustration here reproduced.

The hydraulic gradient as the illustration shows was measured by the difference in the water levels in the columns connected to the flowing water.



The height of the water level in the column was an indication of the combined static and kinetic energy of the water at each point.

It evidently did not matter how near to the column the water enters or leaves the pipe.

The column applied to the flow of water in a closed medium, as a pipe, gave an accurate enough indication of the state of affairs.

Applied to soil flow which was quite a different thing from a closed medium, the indications might have their value but were only an approximation and in fact the statement on page 141 that the water in the pipes rose and fell with the depth of insertion of the pipe indicated that in the medium sounded there were different streams or, expressed differently, a variable stream and consequently the statement made on page 147 that Bernonill's law holds in case of subsoil water flow must be taken with a considerable number of provisos.

Bernsuilli's law was essentially the law of flow of streams where stream lines can be reasonably assumed.

Dr. Bose said that Mr. Khosla had dealt with this subject of subsoil water movement in a way which would appeal directly to the engineering instinct of the congress. He would like to add a few remarks bearing on the theoretical side of the problem. In a memoir of the Panjab Irrigation

Research Laboratory, it had been shown that the movement of water in a porous medium like the soil can for all intents and purposes be replaced by the motion of a very viscous fluid, whose movement on the free surface follows Darcy's Law of subsoil water movement. It was further shewn in the Memoir that stream-lines and lines of equal hydraulic-head can be represented by two functions ψ and ϕ which satisfy Laplace's Equation *viz*:

$$\nabla^2\psi = 0\,,\ \nabla^2\phi = 0.$$

As their names suggested ψ, the stream-lines, *i.e.*, lines of flow in the medium and ϕ the lines on which the hydraulic head was the same, cut each other at right angles.

Mr. Khosla's experiments on the effect on the hydraulic gradient produced by a line of sheet-piles would be used to verify the soundness of the above theory. Mr. Khosla had not supplied any data of his pressure pipe observations for the above case of curtain walls, but the speaker had calculated the probable effect of such a line of sheet-piles of depth c in a medium shown in the diagram Fig. 1.

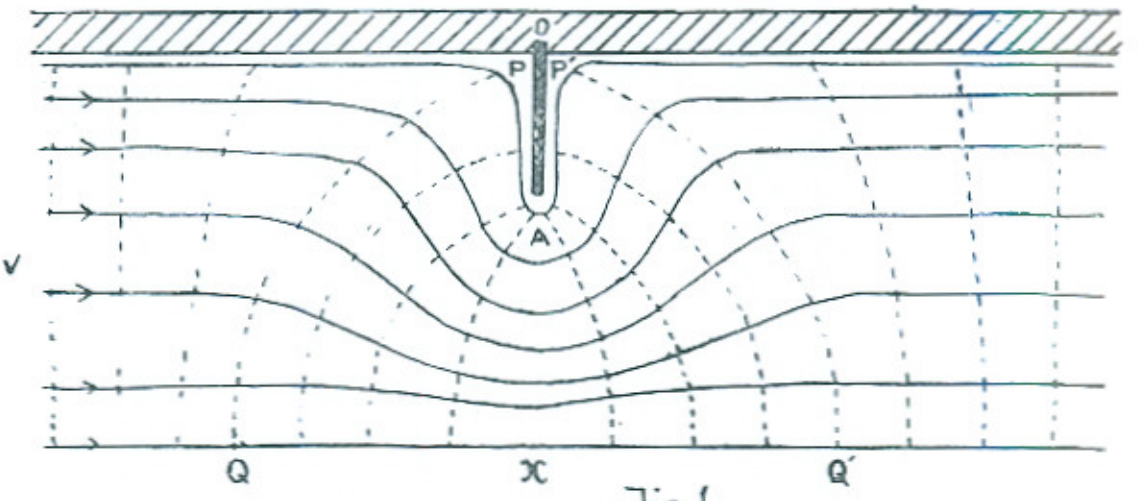


Fig 1.

In the diagram Fig 1. the full lines were the stream lines on which ψ is constant and the dotted lines were the lines of equal hydraulic-head on which ϕ was constant, *i.e.*, the hydraulic pressure on them is constant.

There lines were given by

$$\psi = Vc\ \overline{e}^{\xi} \cos\zeta - V_x.$$

$$\phi = Vc\ \overline{e}^{\xi} \sin\zeta + Vy.$$

where V is the velocity in the stream, at a considerable distance from the sheet piles, and c the depth of the sheet piles OA. These values of ϕ and ψ satisfied the equations of Laplace, and also the conditions imposed by the above boundary. The quantities ξ and ζ are known as Elliptic co-ordinates and are connected to X and Y by the following equations.

$$X = c \cosh\xi \cos\zeta$$
$$Y = c \sinh\xi \sin\zeta$$

If the hydraulic pressure on the two points P and P′ on two sides of the sheet piles were considered the following results were obtained

(1) for P where $\zeta = \frac{\pi}{2}$

$$\phi p = Vy + Vc\, e^{-\xi}$$

and (2) for P′ where $\zeta = \frac{3\pi}{2}$

$$\phi p' = Vy - Vc\, e^{-\xi}$$

so that the difference of pressure between P and P′ would be given by

$$\delta p = \phi p - \phi p' = 2\, Vc\, e^{-\xi}$$

Now ξ determined the distance of the point P from the sheet-pile along the Y-axis. Taking two points PP′ quite close to the sheet-pile and taking ξ as being very small then.

$$\delta P = 2\, Vc = 2\, cki.$$

where k. was the transmission contant and i the slope of H. G ; so that, increasing c, the length of the sheet-piles the drop can be increased. Of course this could not go on indefinitely. If c become very long, the effect could be to screen off the flow from the left altogether.

Mr. Khosla had mentioned another fact that there was a piling up of pressure in the upstream side. This was quite apparent from the curvature of the ϕ lines on the left-hand side of the sheet-piles. The pressure line PQ in the diagram would not have been curved if OA were absent (see next diagram) so that the pressure at P was really equivalent to the pressure at a point Q_1 upstream vertically over Q which would have a higher pressure than that at P if OA were absent. The stream lines and ϕ lines would be as in the absence of the sheet piles shewn in Fig. 2.

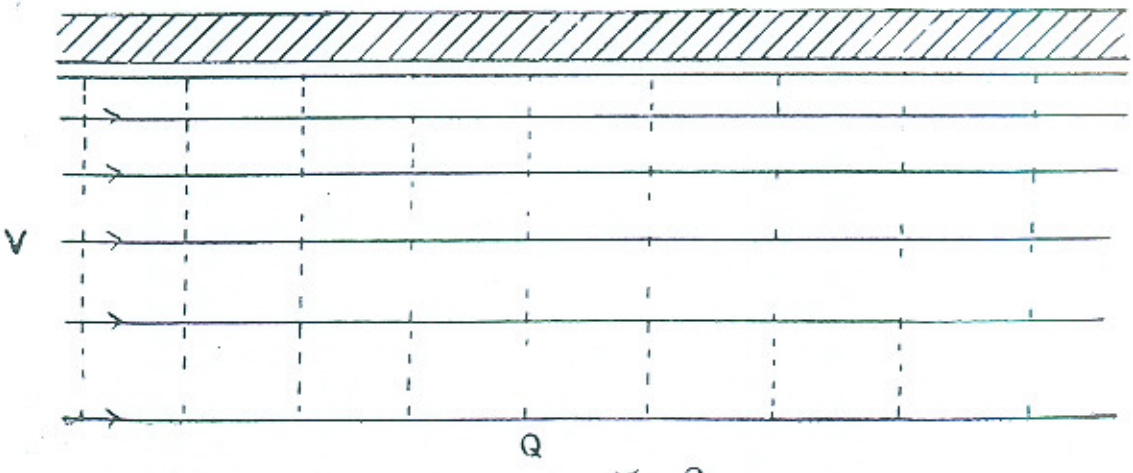


Fig 2

Another interesting deduction would be made from the above diagrams. The effect of putting in sheet piles was to disturb the ϕ lines in their neighbourhood and to bend the lines of higher pressure on the upstream side and lines of lower pressure on the downstream side towards the sheet piles. This produced the local piling up of pressure on the upstream side, and the draw-down in the pressure curve on the downstream side. This disturbance, it was apparent, is only local and the ϕ lines as they recede from the sheet piling resume their normal uniform spacing. The following diagram showed the pressure-lines in two cases; with and without sheet piling. The firm line was the normal

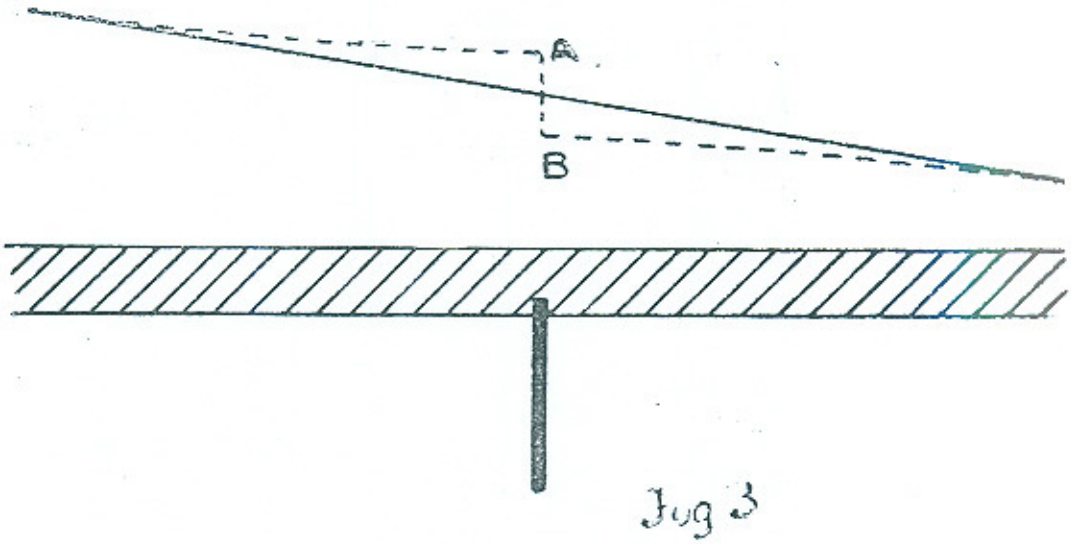


Fig 3

pressure line while the dotted lines were the pressure lines in the presence of sheet piling at A. B. so that the drop in the pressure due to the sheet piling was only local.

Mr. Montagu began by expressing the view that the paper dealt with a problem, ever present to the irrigation engineer, in a manner not less than fundamental. If therefore he, the speaker, has any criticisms to offer they would be directed towards too rapid an advance rather than to the basic theory of the paper, in which he, for one, was a firm believer.

2. Mr. Montagu then went on to sketch the growth of weir design as it was presented to him as a junior officer just joining the Department. He indicated the necessity for the upstream floor, the downstream floor and the balance between these two.

Subsequent experience, also largely at Marala, had convinced him of the necessity of wells on the downstream edge of the weir glacis and the consequent increase of pressures resulting.

Whereas Mr. Montagu had felt the logic of the facts sketched, he had been unable to produce a theory or a quantitative demonstration on which to found one. This Mr. Khosla had done. He, the speaker, thought he was not overstating the case if he held that prior to Mr. Khosla's paper, engineers had been working in the dark. Now they had the light of coherent, substantiated theory to guide them.

3. Mr. Montagu felt entitled to make a few small corrections in some of the statements recorded by Mr. Khosla. The wing walls at Dugri Syphon were not founded on wooden piles, but on a reinforced concrete raft.

The wooden piles were driven to consolidate the slush and to permit of the concrete being laid. The high spring level was perfectly well known and hence the decision to re-build and extend the wing walls.

4. The speaker suggested that the use of relief strainers in works perpetually exposed to hydr ulic pressure (p. 139 and elsewhere) was fundamentally unsound. It appeared to him to be sound engineering and u'timately economical, to accept pressures as they existed and design against them. At empts to relieve pressure by means of strainers invariably had one of two results.

(*i*) The strainer functioned and brought away fine silt and sand, and ended by piping the structures.

(*ii*) The strainer choked up and pressures higher than the design were obtained, to the danger of the work.

Moreover the effect of relief pipes whether fitted with strainers or not was only local. He was pleased to see this so well brought out.

5. The author had brought out the fact that the pressure beneath the floor of a masonry work depended on the height of the Local Spring Water Level (L. S. W. L).

That the emergent velocity depended on this pressure available:

That the soil particles are transported (not dislocated) by the moving water, and that there is a lower limit of velocity beyond which soil particles are no longer moved.

Surely this was a sufficiently large proposition to prove, and would have been sufficient for one paper. Unfortunately Mr. Khosla presented his proposition, his conclusions and his evidence in a way which was extremely difficult to follow.

6. For instance Plate V offers many puzzles

(*i*) Pipes 13 and 14 were driven, apparently to any unspecified depth and left.

(*ii*) Nos. 2, 3, 4, 5 were each given two positions only but three results are given as a pressure curve.

(*iii*) Nos. 6, 11, were the "pressure" lines corrected for depth of point. Otherwise how would they give any indication of the pressures under the floor?

(*iv*) See level 781·5 dated 8th July 1929.

(*v*) See level 781·5 dated 4th july 1929.

of two pipe points.

The corresponding water levels were 791·43 and 791·63 respectively, the difference being due to a difference in bed level of the drain.

Surely this was the way they were supposed to be read.

7. The speaker felt that Mr. Khosla had failed to present his "loss of head" curve in a clear manner, and suggested the following simple treatment.

Plot the actual pressure (*i.e.*, head acting) against the depth of the point the former horizontally to a logarithmic scale and the latter to a natural scale. This gave the desired law at once, but he (speaker) ventured to emphasize that the one set of observations presented by the author would only give a particular equation and not a general one. Far more investigations in the field were required to establish a general law.

Referring to p. 143 he said there seemed to be no reason for the expression of the law derived in terms of logs to base 10. The natural base "e" would indicate similarities far more easily and the problem should certainly be examined on this basis.

8. On p. 144 the author proceeded to apply Darcy's law to subsoil flow. Dr. Bose in his paper had proved that it applied only to the surface. To apply it to the body of the water was only the roughest approximation and depended for its accuracy on a very low velocity. But the Author postulated velocities of a high order to move sand particles.

If it was postulated that

$$V = K h.$$

and that the Total Energy Line was straight,
then V was constant
i was constant
h was fixed in value all along the path.

In the limit this was absurd.

It followed that either a T. E. line could not be straight, or that the Darcy approximation was not true or that Mr. Khosla's classification of the curve connecting depth and pressure was wrong.

Whichever it may be (and it may well be all three) clearly Mr. Khosla's deduction that

$$V \propto h$$

was fundamentally untrue and at the best was only a rough approximation. Further deductions on this basis therefore were unjustified.

9. Mr. Khosla's observations on the point of the minimum velocity required to move soil particles (p. 144) might be linked up with the limiting silt moving velocity in Mr. Lacey's Rational Silt Formulae (Paper No. 4736 Inst. Civil Engineer) and Thrupp's experiments on sand ripples: also the aniline dye experiment on viscous flow in the hydraulic laboratory for 1st year students. Mr. Lacey shewed that it was 0·882 f. p. s.

10. The demonstration on p. 146 as to the location of the first crack to indicate failure of floor was obscurely presented but of itself extremely neat, and deserving of careful examination. It appeared clear

that fineness of the subsoil is of more importance than any other quality and therefore that the minimum

silt, soil, sand } moving velocity V_s was determined for each site in terms of the smallest " fineness " present.

This was another argument for frankly accepting the minimum velocity V_s (*i.e.*, the maximum permissible) at Mr. Lacey's figure.

11. Mr. Khosla's contention that Bernouilli's theorem holds for subsoil flow was extremely interesting (p. 147).

If Mr. Khosla accepted Bernouilli, then he was guilty of an inconsistency in making use of Darcy's approximation.

12. The speaker could not pass over Mr. Khosla's calculation for V_s on p. 150 without one more remark on this point.

If H_s were 0·75 feet

Then V_s would be 7 f. p. s. which was obviously far too high.

If Mr. Lacey's value for V_s were accepted *viz.* 0·882 f.p.s. then H_s = ·01 feet, a very different figure.

13. Mr. Khosla had made free use of the terms hydraulic gradient, critical velocity, critical gradient, etc. Mr. Montagu suggested that hydraulic gradient should be barred in favour of Total Energy line and that a more definite term than " critical " be applied to the minimum velocity with which he was dealing. He (the speaker) used " Minimum

sand, soil, silt } moving velocity " and designated it V_s

Mr. Haigh said he would like to express his appreciation of the service to engineering that Mr. Khosla had done in preparing this paper. The subject was one of great importance to canal engineers and one that was largely shrouded in mystery to the average man.

2. Mr. Khosla's experimental work which to his knowledge represented a great deal of his personal time was of great value to the study of the subject, and while he could not entirely accept the theory Mr. Khosla had based on his results, it represented at any rate an advance on the generally accepted treatment of the subject.

3. Mr. Khosla's paper chiefly centred round the formula:

$$y = k \log h - c$$

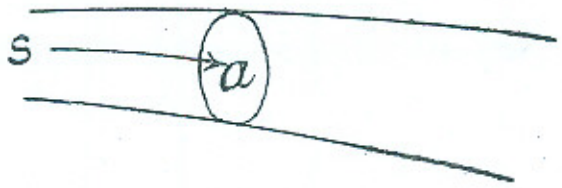
where y was the depth

h, which was termed the loss of head was the pressure in excess of static at the depth y, k and c being constants to which he had assigned values.

4. This formula Mr. Khosla had developed from experiments at one or two particular sites and he (Mr. Haigh) was not clear to what extent he considered this formula applicable to other sites.

5. The formula, however, merely represented the condition of converging subsoil flow.

If, in any stream tube of subsoil flow



a, p and v represented the sectional area, the pressure in excess of static and the velocity at a distance from any origin from Darcey's law.

$$v = \frac{k\ d\ h}{ds}$$

and the discharge which is, of course, constant along the stream tube

$$q = va = \frac{k_1\ a\ d\ h}{ds}$$

Now if the area vary uniformly as s,

$$a = k_2\ s$$

$$\therefore\ q = k_3 \frac{s\ d\ h}{ds}$$

$$\text{or } d\ h = k_4 \frac{ds}{s}$$

when e, integrating $h = k_5 \log s + k_6$

$$= k_5 \log \frac{s}{so}$$

6. This was, of course, the same formula as Mr. Khosla's —though his y was not necessarily measured along the line of flow—and if the conditions at the site where the experiments were carried out were examined it would be found that there concentration of stream lines obtained. Considering it in plan they had flow into the bed of the drain from both sides and also from the end under the sypho . floor.

A particular case of this formula was that for the discharge of a tube well

$$c\ h = q \log \frac{R_1}{R_2}$$

where R_1 was the radius of influence of the well and R_2 the radius of the effective strainer. In this case

$$c = \frac{2\ \pi\ k\ m}{n}$$

where k was the transmission constant.

m was the length of strainer.

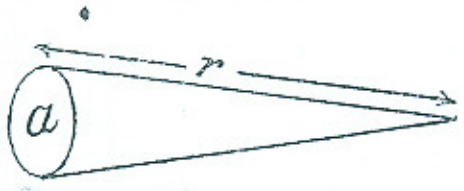
n was 2·3026 the factor for the conversion of ommon to hyperbolic logarithms.

The same formula would also give us the discharge of a drain. In this case

$$c = \frac{\pi k m}{n}$$

when m was the length of the drain c = ·004 per foot when k = ·003 an average value. In this case R_2 may be taken as half the surface width.

7. As a matter of fact in the case of a syphon outfall he would expect conditions to be more analogous to flow towards a point. Considering conditions of flow in both the vertical and horizontal planes they would see there was convergence in both cases, *i.e.*, flow would be somewhat similar to the theoretical case of flow towards a point. In that case if a were the area of flow at radius r they had



$$v = k\frac{d h}{d r}$$

$$q = \frac{k a d h}{d r}$$

Now $a = c r^2$ $\quad \therefore \; q = c r^2 \frac{k d h}{d r}$

$$d h = \frac{q d r}{c k r^2}$$

$$h = \frac{q}{c k r} + c$$

$$h = \frac{q}{c k}\left(\frac{1}{R_2} - \frac{1}{R_1}\right)$$

$C = 4\pi$ for flow in all directions or might be taken as π for the quarter spherical flow of a syphon outfall.

For k = 0·003, c k = 0·00928 = ·01 say.

$$\therefore \; \cdot 01\, h = q\left(\frac{1}{R_2} - \frac{1}{R_1}\right)$$

The second term of the bracket being small might be taken as Zero or all practical purposes hence $\cdot 01\, h = \frac{q}{R_2}$

8. Mr. Khosla's results plotted to the logarithmic form, however, indicated that the conditions at site approximated more to concentration in one plane. Probably the bed puddle or the presence of clay in the upper strata interfered considerably with flow from the canal.

9. Converging flow was always accompanied by a convex pressure gradient which was very steep at the downstream end indicating the necessity for careful consideration of conditions there.

10. Similarly with diverging flow they had a concave pressure gradient, which sloped rapidly at the upstream end. This, however, did not matter as the soil was well contained in that neighbourhood. Such conditions occurred when the subsoil level was low and accounted for the lack of trouble given by some canal works designed with a very steep pressure gradient.

11. Mr. Khosla, on page 144, states that there was a critical velocity which marked the point beyond which movement of the soil—or piping—would take place. That was a generally accepted theory but it did not seem to be able to bear a detailed examination.

They might consider, for instance, what was the actual velocity which was supposed to cause the break down of the soil. Taking a pressure gradient as high as 1 in 1, a transmission factor of ·003 and assuming that the interstices on any section occupy 25 per cent. of the volume or 16 per cent. of the area the velocity would be

$$\frac{\cdot 003 \times 100}{16} = \cdot 019 \text{ ft/sec.}$$

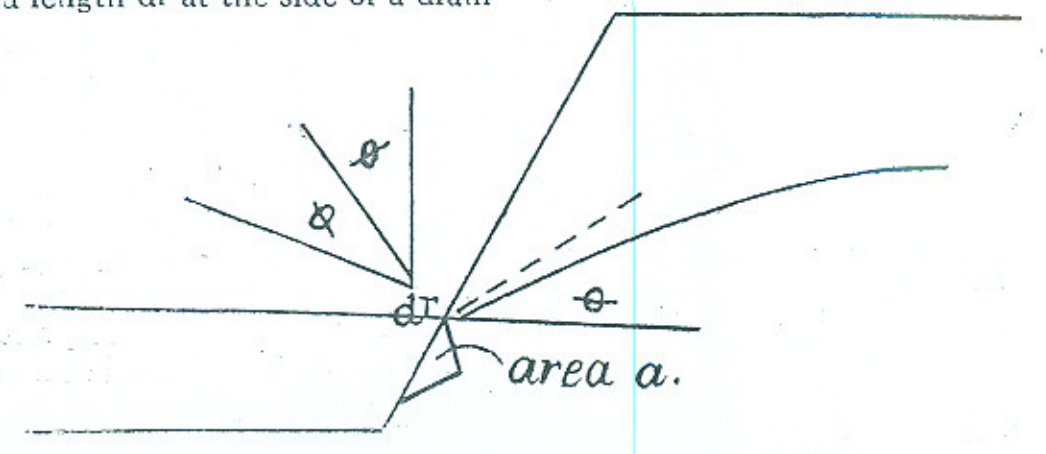
say 1/50 of a foot per second.

Obviously such a velocity could not transport the finest silt in an open stream let alone particles mechanically contained by those surrounding them.

12. The essential condition for movement of the soil was that the difference of pressure across it was in excess of the containing force. When, of course, soil was mechanically contained as in the case of flow towards a tube well strainer no movement could take place unless the pressure were sufficient to break the strainer. In the case of a uniform rate of change of pressure in a vertical stream tube movement would take place if the rate of change exceeded the weight in water of unit length of the soil.

13. In other words if the specific gravity of the soil were 2, under the most favourable conditions it could support a pressure gradient of 1 in 1. When the stream tube were inclined the rate of change of pressure must exceed the component of the weight in the direction of flow—a fact which explained sloughing from the sides of drains where the rate of change of pressure was the same as under the bed, but the component of the weight of the soil resisting motion was much less.

14. Consider the case of a small portion of a stream tube of area a and length dr at the side of a drain



The forces acting on it were—

gravity $= (m-1)\, a\, d\, r \dfrac{d\, p}{d\, r}$ where m is the specific gravity of the soil.

friction $= -(m-1)\, a\, d\, r \cos \theta \tan \phi$ where ϕ was the angle of repose and θ was the angle of inclination of the stream tube to the horizontal.

and pressure $= +a\, d\, p$

For equilibrium

$$(m-1)\, d\, p + d\, p = (m-1) \tan \phi \cos \theta \; d\, r$$

$$m\, d\, p = (m-1) \tan \phi \cos \theta \; d\, r$$

$$\frac{d\ p}{d\ r} = \frac{m-1}{m} \tan \phi \cos \theta$$

also

$$\frac{d\ p}{d\ r} = \frac{q}{\pi\, k\, r}$$

and

$$q = \frac{\pi\, k}{n} \quad \frac{h}{\log \dfrac{R_1}{R_2}}$$

$$\therefore \frac{m-1}{m} \tan \phi \cos \theta = \frac{h}{n\ R_2 \log \dfrac{R_1}{R_2}}$$

$$h = \frac{m-1,}{m}\ n,\ R_2 \log \frac{R_1}{R_2} \tan \phi \cos \theta$$

For a drain of bed width 2′ taking

$R_1 = 100$

$m = 2$

$\tan \phi \cos \theta = \cdot 3$

Then $h = 0 \cdot 7$

For bed width 20′

$R_1 = 1{,}000.$

$h = 7 \cdot 0.$

Results which were not inconsistent with actual experience.

15. If the rate of change of pressure were not uniform they had either converging or diverging flow. In the case of the former the rate of change of pressure increased as they passed along the stream line and consequently motion would take place from the point where it exceeded the containing force and, as Mr. Khosla pointed out, the motion would be accompanied by a change of pressure distribution tending to progressive disruption of the soil. Where, however, flow was divergent they might have a locally excessive rate of change of pressure contained by the weight of the soil in front of it.

16. An example of this might be found in the conditions existing near the spaces between wells which had not been properly sealed.

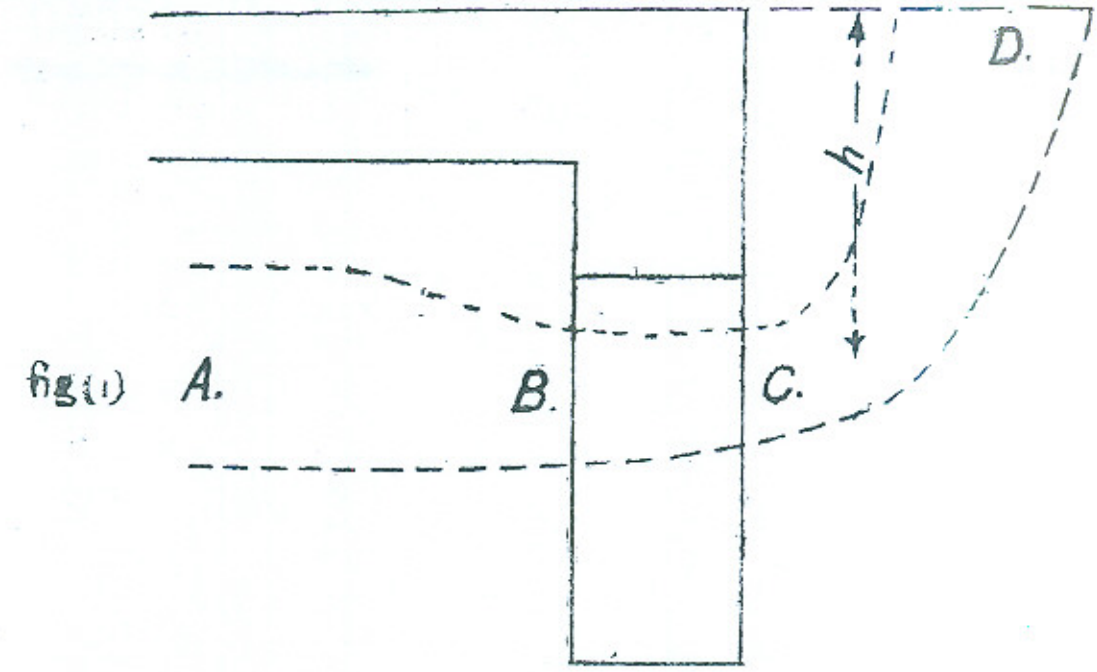


fig (i)

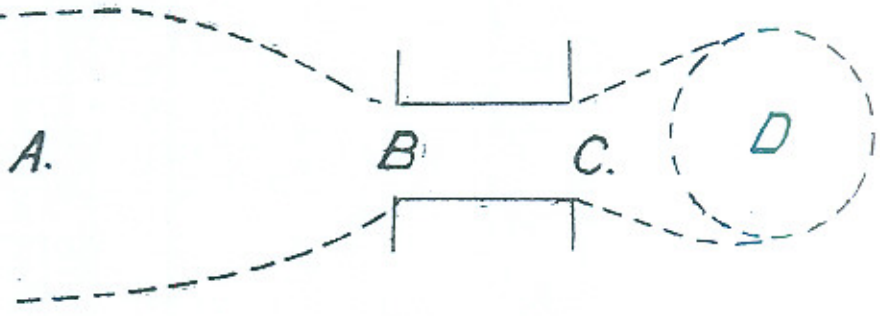


fig (ii

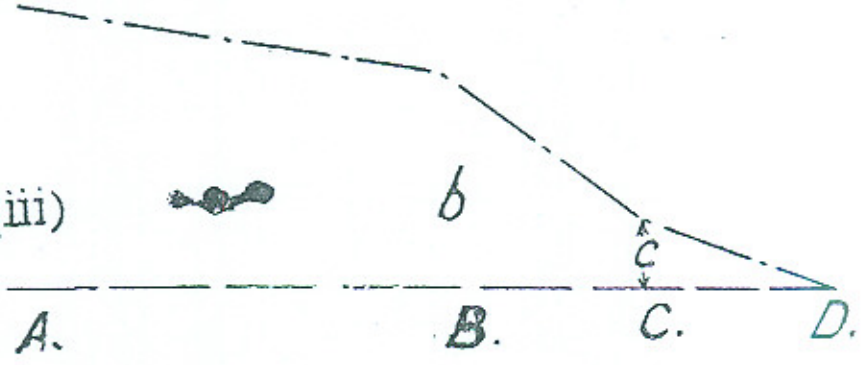


fig (iii)

For instance, the pressure gradient along the stream tube A, B, C, D in fig. I, would be somewhat as shown in fig (iii). Owing, chiefly, to the lateral convergence the slope from B to C might be excessive. At c, however, they had the weight of the soil capable of exerting a considerablee

horizontal pressure to resist movement. The weight of the soil there would be $(m - 1)\ h - c$ and the horizontal pressure it could exert would be

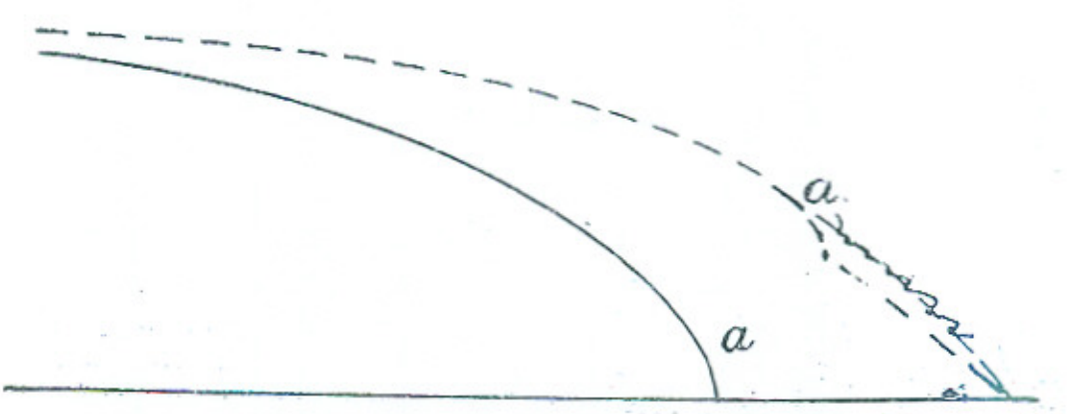
$$\left[(m - 1)\ h - c \right] \frac{1 + \sin \phi}{1 - \sin \phi}$$

where ϕ was the angle of repose.

If h were small, i.e., if the sealing course were shallow or if erosion had taken place downstream they might get dangerous conditions. Danger might also occur if the filling at D were so course that the fine material under the wells could pass through it without restraint.

17. Any change of direction in a stream tube was accompanied by a concentration of flow and consequent increased rate of change of pressure on the inside of the curve. Water flowing under a work had invariably to change its direction from horizontal to vertical at the downstream end which was the danger point to be carefully considered in design. If, by means of a curtain wall the point where the change of direction occurred were depressed they could secure adequate containing weight, but as in the case of spaces between wells it was necessary to see that the weight was suitable and was not removed by scour.

18. The case of rapidly converging flow such as occurred downstream of a syphon when spring level was high required very careful consideration. As has been shown there was for every depression head into a drain a minimum bed width necessary for stability. Similarly at a syphon outfall there was a minimum radius necessary to secure the earthern sides of the drain against sloughing, and on account of the more rapid convergence at a syphon outfall (horizontal as well as vertical) this radius was greater than in the case of an open drain. The use of piling in such cases lay in the fact that it increased the length of the hydraulic gradient, provided a region of uniform change of pressure at the end thereof, and also earth pressure to contain the local high rate of change round the bottom of the piles. Such piling was best arranged in a semicrcle, i.e, normal to the line of flow. The pressure gradients obtaining without and with piling would be similar to the full and dotted curves below.



The sudden steepening at a in the curves was due to the change of direction from horizontal to vertical flow.

19. With reference to Mr. Khosla's para. 2 or p. 145, the rate of inflow per foot length of strainer would only increase with the depth when the Sub-soil water surface was free in the vicinity of the well and vertical flow took place. This occurred, of course, in the case of seepage drain relief pipes, but not in the ordinary tube well.

20. While on the subject of this para, he would like to ask Mr. Khosla if he had any data on which to base his statement that the loss of head in a Leggett or Ashford Strainer is much greater than in a Tej or Cook strainer? Such figures might also show whether the loss of head in either case was sufficiently large to have any appreciable effect on the flow.

Bhai Bakhshi Singh Sidhu said that Mr. Khosla deserved to be congratulated for boldly declaring that "our existing conception of the law of flow of water through subsoil as affecting the stability of structures resting on saturated soil is far from being correct or complete." The statement had, however, not been printed in bold type like so many other statements to which the author wished to draw attention more prominently. He (Mr. Khosla) had proved from observational data that "accepted theory, established practice and the principles of both are not quite convincing." Mr. Bligh's theory of Hydraulic gradient had been accepted for quite a long time as furnishing an explanation of certain failures and affording a basis for calculations for new designs and repairs to damaged or threatened structures.

Mr. Khosla, had pointed out that due to a rise in the spring level originally safe structures might become unsafe. This factor he did not appear to have taken fully into account in his observations and deductions therefrom. He had omitted to mention and plot the spring level round the structures say about a couple of hundred feet away from them and had confined his attention solely to the W. S. Level in the canal and the pipes. Near one of the syphons on the L. B. D. Canal water surface in a well 200 feet away from the canal and the approach channel was only 0·25 feet below the water surface in the canal whereas in the outfall and approach channels of the syphon it was well over 4·0 feet below the canal water surface level. This indicated that the spring level in the outfall or approach channels might be considerably lowered by sub-soil flow into the drains leaving the static pressure under the various parts of the structures unaffected appreciably.

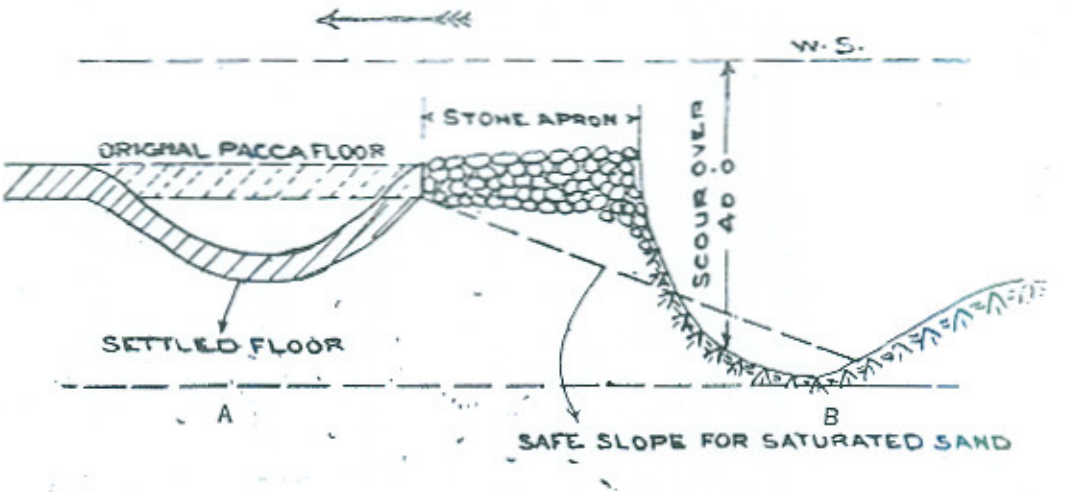
In order to take into account the effect of spring level it would have been very instructive if variations in water surface level in the pressure pipes caused by rise and fall of the spring level round the structures had been analysed and commented on.

The speaker was fortunately not a believer in the hydraulic theory to which Mr. Khosla seemed to cling while exploding its exposition by Mr. Bligh. In support of his view he mentioned the following :—

(*a*) Early that month a portion of the upstream impervious floor of the Balloki Barrage was laid bare and fine cracks in the mortar joints between stones continuing for a length of several feet were noticed. There were many small holes also. Through these cracks and holes water was oozing out when water surface level outside the ring bund was only 4 feet above the floor level. Normally for months and months together the difference in water surface level above and below the barrage was over 15 feet. The downstream floor was free from any cracks at all, although there was quite a large number of pressure relieving pipes fixed there.

(*b*) Another kind of damage discovered was subsidence of a portion of the upstream pacca floor at its upstream end next to the loose stone apron which apron was almost undisturbed.

It would serve no useful purpose to theorise as to the cause of this on incomplete data but the fact remains that Mr. Khosla's new theory like Mr. Bligh's time honoured theory did not afford any convincing explanation for the above described classes of damage. The diagram which is reproduced below may help to point out one of the directions in which



to look for the cause. (Notice the side slope of saturated fine sand under water and calculate horizontal pressure at A. and B.)

Heavy expenditure had been incurred on several masonry works designed blindly on Mr. Bligh's theory without attaining any appreciable increase in their stability. They must, therefore, exercise a good deal of caution in considering Mr. Khosla's theory which was apparently based on observations at two works in a particular locality and dealt with only a particular aspect of the problem.

Mr. Nand Gopal said that the subject of the paper was interesting and important to any engineer who has anything to do with waterlogging. He was afraid that the heading of the paper was the only thing that he

had understood thoroughly and his remarks were based on that understanding. He said that Mr. Khosla called his paper "HYDRAULIC GRADIENTS IN SUBSOIL WATER FLOW IN RELATION TO STABILITY OF STRUCTURES RESTING ON SATURATED SOILS." He did not think this was quite correct. Mr. Khosla's experiments were all carried out near a large Canal where there was direct cause of pressure available to give him the results he had obtained, but these conditions did not represent all cases of saturated soils. Mr. Khosla did not take into consideration the cases of saturated soils some distance away from such big channels. He asked Mr. Khosla to say whether his observations related to such soils also. He had no occasion to make such observations himself except in regard to a few wells made in water-logged areas where the subsoil water level had already risen to about 3 or 4 feet of the surface. He asked Mr. Khosla if the latter had anything to say about this.

Mr. Colyer asked whether the author thought that the puddling mentioned on page 137 would have been effective, had it been executed.

Similar puddling work at Khambi Culvert on the Upper Jhelum Canal had proved effective at the time (1928), and perhaps Mr. Dhody would say if this result had been sustained.

He enquired whether the term 'Hydraulic Gradient' as used on page 138, meant the difference in upstream and downstream water-levels divided by the length of enforced creep along the line of contact of the soil with the structure, or if it had some other significance.

After commenting on this valuable addition to Hydraulic Engineering knowledge, he compared the author's theory with that of Bligh; he said that Bligh's seemed to be based *entirely* on the prevention of the transportation of sand particles *along the line of enforced creep; viz*, along the line of contract between the structure and the soil, whereas the author appeared to have considered primarily—perhaps exclusively—the whole percolation area of the soil beneath the structure and to have omitted any consideration of the line of contact between the soil and the structure, which Bligh evidently held to be the line of least resistance and the source of failures. The speaker suggested that a complete theory should comprise both these considerations, and illustrated this need by the case of the wing-walls of a distributary 'head' built in a Main Canal of which the banks were of sand. If the creep-line, or rather the area of contact between the masonary wings and the canal bank was made only of the same section as would just prevent seepage through the bank itself, it was most likely that the 'head' would fail by 'piping' along the plane of contact of the wing-wall and the bank.

Bligh, on the other hand, appears not to consider the effect of what the author has called the 'free-water-surface,' and which he shows to be of the greatest importance.

The speaker had discussed with Mr. Wilsdon the probable effect on the Sutlej Valley and other weirs, of an almost inevitable rise in the

subsoil water-level, and had come to the conclusion that many works which so far had given no trouble would be subject to very different conditions of pressure when equilibrium of the subsoil water level in their vicinity had been reached.

He would be obliged if the author would briefly describe the use of the water-jet, and the method of plugging the spaces between wells mentioned on page 148.

With reference to page 140 of the paper **Mr. A.A. Musto** said that High Velocity at exit could only exist if *piping* had already started, or if the creep co-efficient taken for design were too low.

With reference to page 147-148 he said that wells had little or no advantage over piling, as foundations or anchorages for a masonry or concrete block floor, since once piping started and the subsoil was removed form one edge, they would settle and fall as easily as piling, if not more so, as their great weight would tend to over-turn them ; whereas the piling would hang on to the masonry above it and in which it was embeded. As watertight curtains, wells were most difficult to make effective, and were unlikely ever to be as good as well-driven piling, while the wells were infinitely more expensive.

The first essential for protection of the floor was a *long* and *thick* apron of pitching stone which would fall, *at a long distance* from the work and thus prevent retrogression of levels near the solid structure. This was needed both upstream and downstream. but, of course, more so on the downstream side.

In *replying* to *criticisms*, **Mr. Khosla** first dealt with Mr. Nand Gopal's remarks that in saturated soil the pressure indicated in a stand pipe did not show any variation, no matter how deep a pipe was driven in the ground. This would naturally be so unless the deep soil was excavated some depth below the normal spring level, which would create a difference of pressure and then give varying pressures for varying depths to which the pipes might be sunk.

Dealing with Mr. Colyer's remarks, he said that puddling would only stop local leakage, but would not at all affect the subsoil level all round.

Regarding, 'creep' he said the question was receiving attention, and practical tests were to be carried out in the laboratory, also the mathematical treatment was being undertaken by Dr. Bose.

Replying to Mr. Thompson, Mr. Khosla agreed that other engineers in the past had treated the Bligh theory with suspicion. He did not mean

to be unfair to the service. All he meant was that to his knowledge a fair proportion of the profession did place implicit faith in the Bligh theory without regard to its limitation. The application of Bernoilli's law or rather its extension to subsoil water flow would be discussed later on in reply to Mr. Montagu's, criticism.

In reply to Dr. Bose, the author expressed his indebtedness for the mathematical proof produced by Dr. Bose for the physical phenomenon of local drop in pressure at the sheet-piles, the piling up upstream and the drawdown downstream, which was apparent at all intermediate toe walls. The pressures above and below the 20 feet deep piles of Plate VI were a clear demonstration of this. The pressures at the crest walls of the Narora weir (Plate II of paper 142) were another example.

Dr. Bose had evolved an equation

$$\delta p = 2\, vc \qquad (B)$$

$$= 2\, k \frac{dh}{dy} \times 2\, c \text{ since } v = k \frac{dh}{dy}$$

If $\frac{dh}{dy}$ were constant, which would be the case, as nearly as possible, with a deep toe wall at the end, then p or drop of pressure at the piles would equal Ki × 2c. This was of the form given by Bligh with the great difference that the multiplier varied with the slope 'i' and that the drop in pressure was local. The fact that the effect of the intermediate piles was local was worth remembering. From this, equation, given the drop at the sheet piles and the length of piles, the velocity of subsoil flow can be found, as

$$v = \frac{\delta p}{2 c}$$

Unfortunately Dr. Bose has not had the time to work out mathematically the effect of sheet piles at the upstream or the downstream ends. This, when done, would be extremely valuable.

The author reserved for the second paper his discussion of the effect of end piles with the help of ψ lines.

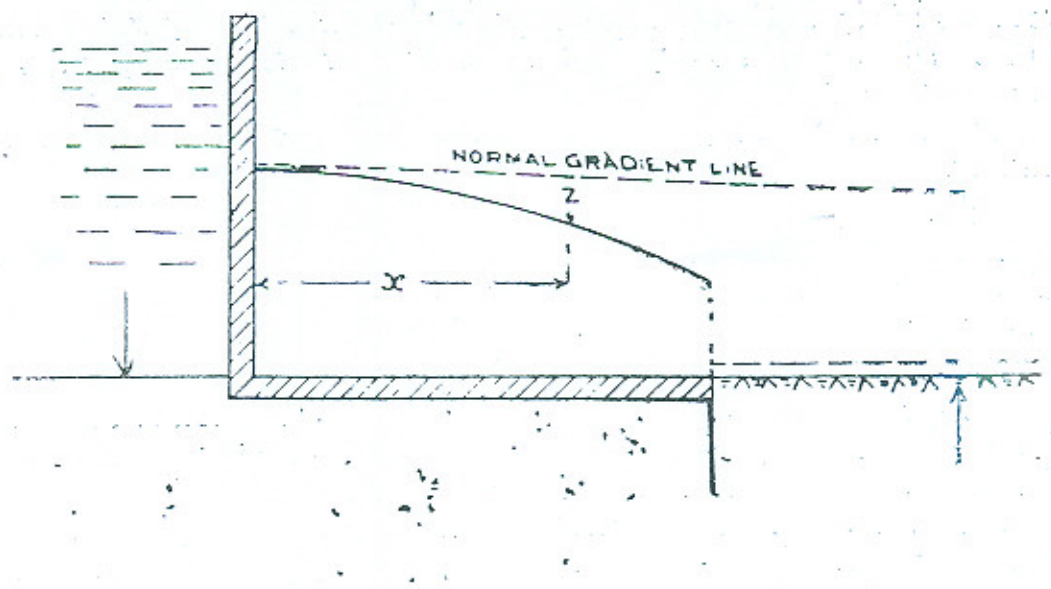
Dr. Bose in his paper No. 140 on the Exponential Law of Subsoil Flow, had given the general equation.

$$u = \frac{1}{a^2 \mu} \frac{\delta}{\delta x} (P - g \rho z) \ldots\ldots\ldots (C)$$

For the free water surface, p = constant

$$\text{hence } u = K \frac{dz}{dx} \ldots\ldots\ldots\ldots (D).$$

Which corresponded to the horizontal flow under a flat floor and agreed with what the author had stated on page 147 of his paper.



Next taking the case of vertical flow along axis of Z, p would be changing and the flow would correspond to the experimental one of Dugri I. The equation would then reduce to

$$w = \frac{1}{a^2\mu}\frac{\delta}{\delta z}(p - g\rho z) = \frac{1}{a^2\mu}\frac{\delta p}{\delta z} - C$$

$$= k\frac{dp}{dz} - C \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (E)$$

This equation furnished the justification for the assumption $V = K\frac{dh}{dy}$ *viz*, equation (*iii*) of page 144. Darcy's law applied, with all its limitations to horizontal flow only. Bose's law was therefore a more appropriate authority, for such assumption in the vertical direction both upward and downward, as also for general application in the horizontal direction as further explained in reply to Mr. Montagu's criticism.

Replying to Mr. Montagu's criticism, the author thanked him for his appreciative remarks. It was a great satisfaction to find Mr. Montagu a believer in the basic theory of this paper and in general agreement with the main conclusions. He accepted the correction regarding foundations of Dugri wing walls with thanks. Mr. Montagu having constructed these, knew better.

The Author fully agreed with Mr. Montagu regarding the risk involved in the use of relief strainers and had already given reasons for that in the text.

Regarding Plate V, the pipes 13 and 14 were driven to 4 feet below spring level in the bank and were not moved. It was, however, found that the occurrence of the face wall and the masonry floor underneath the barrels prevented any direct connection between pipes 13, 14 and 1, 2, 3, etc. Strictly speaking a correction had to be applied to the

readings in pipes 13 and 14 to allow for the fact that they were located in the middle of a masonry floor and above it whereas the pipes 1, 2, 3, etc., were located in the subsoil below the floor. On account of this uncertain factor, this part of the investigation was not pursued any further.

The two positions of the pressure pipes 1, 2, 3, etc., indicated the bottom and top of the filter lengths which varied between 2 and 2·5 feet. These pipes were fixtures. The three pressure curves related to different water levels upstream and to different dates in order to connect these results with the observations on pipes 6 to 12, in the open bed.

Regarding Nos. 6 to 11, the lines below the bed indicated the bottom levels of pipes and the lines above the bed indicated the pressures recorded for those depths. Each individual pipe had been separately treated for loss of head curve. The pressure lines of plate 5 were just to indicate the general features. Thus on 4th July 1929, pipe No. 8 had its filter point at R. L. 781·5 and the pressure recorded was 701·63; on 8th July 1929 the point was at R. L. 777·5 and the pressure rose to 792·43; on 15th July, 1929 the point level was 770·5 and the pressure rose to 792·88 and on 12th July, 1929 the filter point level was 765·5 and the pressure recorded was 793·1.

The author agreed with Mr. Montagu's suggestion of trying logarithmic plotting. This would have saved the author a lot of time if he had attempted it in the first instance. The equation given in the paper applies only to the site of experiments and for other places, the constants would vary but the logarithmic law would remain unchanged so long as the bed was wide. For a narrow bed the effect of flow from the sides would come in and the law would then become hyperbolic instead of logarithmic. This was dealt with at length in reply to Mr. Haigh's criticism. The Naperian base 'e' was certainly better and more logical but the base 10 was taken as the engineer was generally more familiar with the latter. The logarithmic equation could as well be expressed as

$h = Ke^{y}$ where h = loss of head
y = depth of filter point and
K = constant.

Darcy's law had the limitations given below. His experimental work was restricted to flow through horizontal pipes of uniform section or between two horizontal planes. The flow was horizontal. The pressure indicated, related to the points at the top of the soil in contact with the underside of the upper plane.

His law was naturally linear with a uniform rate of change of pressure. No mathematical solution existed, to the author's knowledge, to justify its application to a varying rate of change of pressure. The relationship $V = k\frac{dh}{dl}$ was therefore an extra-polation of the Darcy's form $V = k\frac{h}{l}$. The author was here open to correction, but if his surmise

as above was correct, then the credit for this advance of a fundamental character would naturally go to Dr. Bose who in his paper No. 140 on the " Exponential Law of Subsoil Flow " had evolved a universal equation for a varying rate of change of pressure, which held both on the free surface and for any point in the body mass and for all directions. This law was experimentally and mathematically correct. The further application of this law had already been discussed. Mr. Montagu had evidently not followed the arguments of page 147 which dealt with the two component parts of the total energy. The total energy line was represented everywhere by the normal spring level. This was a straight line and with normal, uninterrupted flow, would remain a straight line indicating constant velocity and a uniform rate of change of pressure. In this case the Darcy law $V = k \frac{h}{l}$ would STRICTLY hold. But directly the flow was altered by pumping out from a strainer or in-filteration gallery or by digging a drain anywhere below the normal spring level the conditions of flow would change. There would be concentration of flow towards such strainer, gallery or drain. The rate of change of pressure would become non-uniform and would obey, not the Darcy linear law but the more general *Bose Law*. Whereas the total energy line still followed the linear law, the free surface as indicated by pressure pipes' water levels followed a curve such that the intercept between the total energy line and this curve represented the kinetic or velocity head and the intercept below this curve to the flow level represented the static or pressure head. This in other words was the extension of the Bernoulli's Law to subsoil Hydraulics for which the Author might reasonably claim credit. Mr. Montagu's suggestion of inconsistency in this respect was not correct. The above exposition proved the contention. Whereas Darcy's Law would certainly give it the appearance of being inconsistent, the Bose's Law explained and established it beyond doubt.

The calculations of Mr. Montagu in para. 12 were not clear to the author.

The suggestion of Mr. Montagu in para. 13 were valuable and would receive due consideration in a further study and presentation of the subject if future experimental work showed a new presentation desirable.

In reply to Mr. Haigh, the author expressed his sincere thanks to him for appreciating the author's service to the profession in presenting his paper. Mr. Haigh's criticism was in itself an advance and a new exposition of the subject. He had approached the problem from a different aspect altogether and had obtained two sets of equations, one logarithmic and the other hyperbolic. The logarithmic formula applied to convergent flow towards a plane and as such was a case of two dimensional flow. The hyperbolic formula applied to convergent flow towards a point which represented a three dimensional flow.

Dr. Bose in his paper had stated that in subsoil flow, the ϕ curves were identical with the curves of equi-hydraulic head. In his criticism of this paper he had said that the ψ and ϕ functions satisfied Laplace's equations, *viz* :—

$$\nabla^2 \phi = 0$$
$$\nabla^2 \psi = 0$$

There were two solutions to the equation $\nabla^2\phi = 0$, *viz.*,

$$\phi = k e^{-r} \text{ or } A \log \phi = B - r \text{ or } r + A \log \phi = B$$

for two dimensional co-ordinates,

and $\phi = \frac{k}{r}$

for three dimensional co-ordinates. In both the equations r was the vector. Taking r in the horizontal direction, both the logarithmic and hyperbolic equations of Mr. Haigh were satisfied. Taking r in the vertical direction, the logarithmic and hyperbolic equations for upward or downward flow were obtained.

In this connection attention was invited to the loss of head curve for the Jaurian Syphon (Plate A). As would be seen from the plan of Plate X, the pipes were in the bed of a narrow drain and the effect of flow from the sides came in. A cross section of the drain with location of pipes A and B was also shown for reference. This case approximated to convergent flow towards a point. The plot of the loss of head on a logarithmic scale approximated to two hyperbolae, which meet somewhere at R. L. 74, *i.e.*, at the junction of coarse and fine sane strata. The actual flow possibly followed a curve which was a cross between a hyperbolic and a logarithmic curve, the former predominating. The results were not sufficiently exhaustive for generalisation. It might also be noted that the bed of the drain at the Dugri was over 50 feet wide so that flow from the sides was comparatively very small and the conditions approximated to the logarithmic.

Mr. Haigh's treatment of the actual process of soil dislocation with reference to specific gravity of soil was highly interesting and instructive. The disruption of soils was a process of progressive degradation from top down or from exposed slopes inwards. Whether it was the high velocity of exit at the exit end that did it or the difference between weight of soil and upward pressure at the point, the result was quite clear. There was spring formation all over the area with pockets of no flow in between. The minute springs could be seen distinctly bringing out soil particles with them which they kept suspended above the drain bed according to the conditions of pressure difference at the site. Reference was at this stage invited to a paper by Charles Terzaghi on " Effect of minor geological details on the safety of Dams " (American Institute of Mining and Metal : Engineers Inc. 1929 wherein he ascribed the disruption of soil at the toe of a dam to the upward pull of outflowing water overbalancing the downward pull exerted by the force of gravity. On page 42 of his

paper, dealing with the possibility of erosion and deposition along geological boundaries within the subsoil, he remarks that "the failure of weirs resting on stratified materials sometimes occurred several years after the water was stored up whereas, according to the theory for uniform materials and according to laboratory evidence obtained from uniform materials, failure occurred either at once or not at all." Whatever the relative weight of these remarks may be, the author considered that the curve of loss of head was a definite messure of the disruptive tendency in the soil at each point. The physical explanation of it might be sought either in velocities or in the relative magnitudes of the upward and downward pulls of water and soil respectively.

Regarding the statement about the Leggett and the Tej strainers, the Author had no data to support it and as such did not suggest any great importance to be attached.

In reply to Mr. Bakhshi Singh Sidhu, the author referred him to pressure pipe No. 11 of Dugri Siphon which as stated on page 142 was 120 feet away from the siphon face and recorded much the same normal spring level as elsewhere near the siphon (see plate V). As to the upstream floor of Balloki weir, with a head of 4 feet outside the ring bund, water would naturally ooze out from the cracks in the floor enclosed inside the ring bund.

The damage to the "pacca" floor adjoining the loose protection appeared to have been due to sloughing of foundation soil underneath the loose protection which remained arched over, into a local scour hole outside such protection. Local knowledge would perhaps clear this point.

The author had not attempted to lay down definite laws. He had simply invited attention to fundamentals and had stressed the need for further experimental work. The deductions of the paper should be accepted only so far as actually proved in the text. Mr. Sidhu was quite correct in drawing attention to this aspect.

The author thanked Mr. Musto for his taking part in the discussion and associated himself with Mr. Mursto's remarks regarding long and thick aprons of stone upstream and downstream of a pacca work.

The President in winding up the discussion on Mr. Khosla's paper, said that he believed that all present would agree with him that they were much indebted to Mr. Khosla for his introducing this very interesting and important paper, which had evoked the interest and ingenuity of the mathematicians and the engineers and he hoped that the discussions which had taken place would help greatly in throwing further light on the subject.

STATEMENT A.

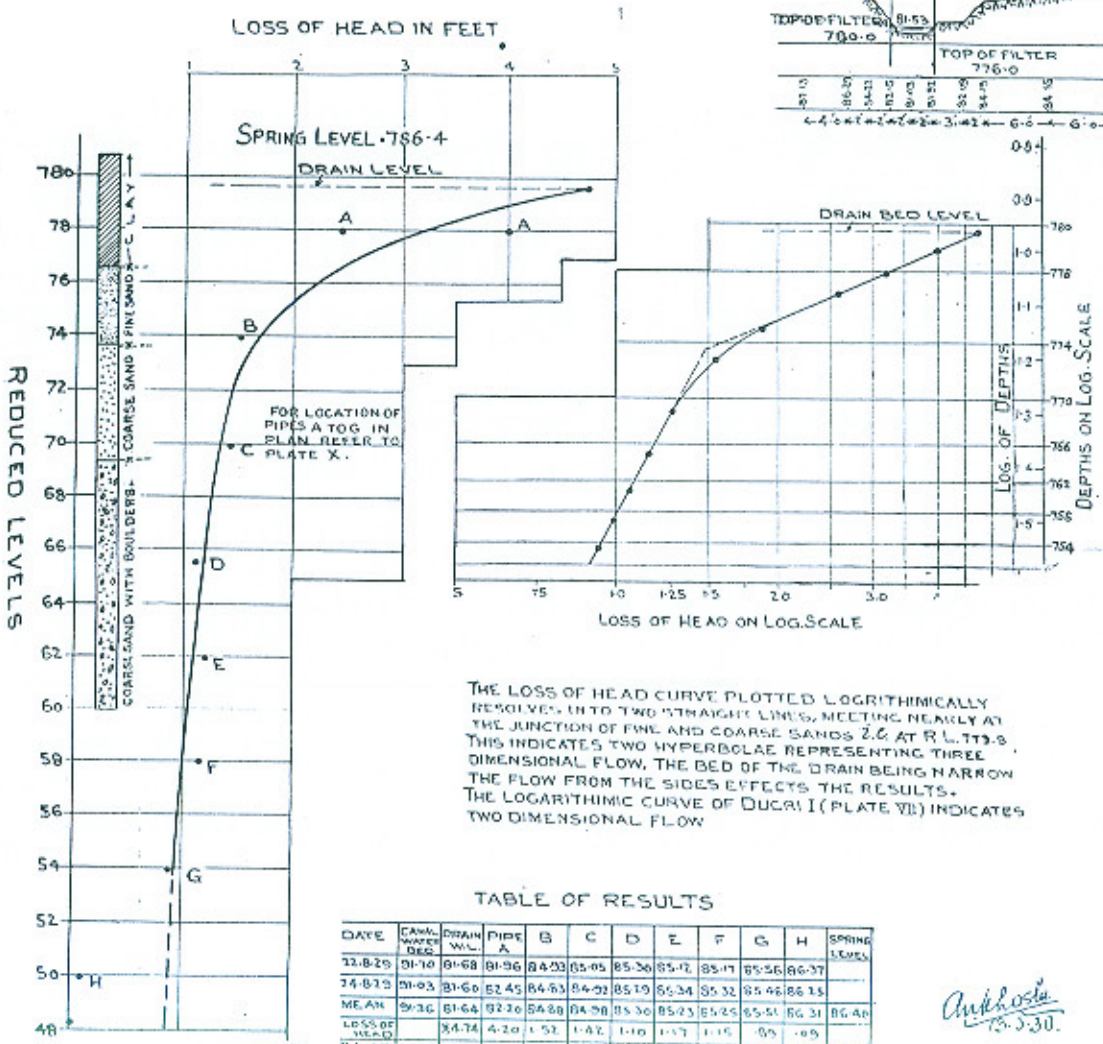
Statement showing bottom levels of filter points of pressure pipes at Dugri I Syphon.

Date.	Pipe Numbers.						
	6	7	8	9	12	10	11
	R. L. of bottom of filter point.						
	775	778	780	782	..	784	785·5
2-7-29 ..	774	777	779	781	779	783	784·5
4-7-29 ..	772	775	777	779	779	780·86	783·5
6-7-29 ..	770	773	775	777	777	779	781·5
8-7-29 ..	768	771	773	775	775	777	779·5
11-7-29 ..	765	768	770	772	772	774	776·5
13-7-29 ..	778	776	768	770	770	771	773·5
15-7-29 ..	780	778	768	768	768	768	770·5
18-7-29 ..	780	778	763	768	768	768	767
1-8-29 ..	782	780	763	768	768	..	767
6-8-29 ..	784	783	763	768	768	..	767

PLATE A

LOSS OF HEAD CURVE

JAURIAN SYPHON



TYPICAL SECTION OF DRAIN

THE LOSS OF HEAD CURVE PLOTTED LOGRITHIMICALLY RESOLVES IN TO TWO STRAIGHT LINES, MEETING NEARLY AT THE JUNCTION OF FINE AND COARSE SANDS 2.6 AT R.L. 773·8 THIS INDICATES TWO HYPERBOLAE REPRESENTING THREE DIMENSIONAL FLOW, THE BED OF THE DRAIN BEING NARROW THE FLOW FROM THE SIDES EFFECTS THE RESULTS. THE LOGARITHIMIC CURVE OF DUCRI I (PLATE VII) INDICATES TWO DIMENSIONAL FLOW

TABLE OF RESULTS

DATE	CANAL WATER DEP.	DRAIN W.L.	PIPE A	B	C	D	E	F	G	H	SPRING LEVEL
12-8-25	91·10	81·68	81·96	84·03	85·05	85·36	85·42	85·47	85·56	86·37	
14-8-25	91·03	81·60	82·45	84·63	84·92	85·19	85·34	85·32	85·46	86·35	
MEAN	91·06	81·64	82·20	84·88	84·98	85·30	85·23	85·25	85·51	86·31	86·46
LOSS OF HEAD		84·74	4·20	1·52	1·42	1·10	1·17	1·15	·09	·89	
R.L. OF FILTER			71·6	74	70	66	62	56	54	50	

BORING COULD NOT BE TAKEN BELOW R.L. 80 DUE TO BOULDERS THE RAPID INCREASE IN LOSS OF HEAD BETWEEN R.L.'S 48 & 54 IS APPARENTLY DUE TO THIS BOULDER STRATA OR SOME OTHER HEAVILY POROUS GEOLOGICAL FORMATION IN WHICH THE CONDITIONS OF DISCHARGE MAY BE A CROSS BETWEEN THE SUBSOIL WATER & THE OPEN WATER FLOW

'X' ON 17.8.25 WITH SPRING LEVEL 86·5 AND FILTER POINT AT R.L. 78·0 PIPE A RECORDED 84·04, WHENCE LOSS OF HEAD = 2.46

Authosta 15.5.30.