PAPER No. 142.

STABILITY OF WEIRS AND CANAL WORKS.

AN APPLICATION OF THE NEW THEORY OF HYDRAULIC GRADIENTS.

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Introduction.

In a former paper, the laws governing the flow of water in subsoils as affecting the stability of structures, resting thereon, have been clearly brought out and discussed. It has been shown how the pressure gradients under the floors of weirs, etc., differ so widely from those of Bligh, which latter, have formed, so far, the basis of our designs of such structures. It has also been shown how the shape of the pressure gradient curve determines the static uplift of floors and also forms the measure of the blowing out or, undermining from beneath them. The influence of the depth of the end toe walls in modifying the shape of these curves, and consequently, the conditions of uplift and undermining of floors, has also been fully dealt with. The object of this paper is to investigate the extent to which the theories set forth in the former paper, are applicable to, or, are supported by observations at actual weirs and other canal works. With this in view, the data available at some of these structures have been collected and scrutinised and are presented in a form readily applicable to the study in hand. Among the old weirs, Narora of the Lower Ganges Canal, Rasul of the Lower Jhelum, Marala of the Upper Chenab have been discussed. Regarding Khanki, a mass of data is said to be available which, if so, shall be studied and presented at the Congress. Of the new weirs, only Sulemanki has a series of pressure pipes in the weir floor and elsewhere, which have given useful information. No pressure pipes exist either at Ferozepore or at Islam; but a series of them if inserted in these two weirs should be most helpful in a study of pressure gradients under different conditions of geological formation, etc. Data are being collected for the Jaba crossing of the Upper Jhelum Canal and Siswan Super-passage of the Sirhind Canal where the history of trouble and the record of pressure pipe observations extends over a series of years.

Some important features of the trouble, the repairs and of the structural details of general interest at the Dugri No. I, and Jauryan syphons of the Upper Chenab Canal, have also been described.

There is, perhaps, an enormous mass of useful data lying buried and lost in the oblivion of our Divisional files, and if this paper can succeed in exposing some of this to the light of the day, it shall have amply requited the labour spent on it.

In a general discussion of the stability of weirs, it is impossible to brush aside, altogether, the other factors, besides the uplift and the undermining due to pressure gradients in subsoil flow, which cause damage and destruction to the loose stone protection, the blocks and the impervious floor. But only a passing notice has been taken of these where necessary in order to keep the paper within reasonable bounds.

Cracks in Jauryan Floor.

Before proceeding with a discussion of the weirs, it is desirable to invite attention to the way cracks occurred in the remodelled Jauryan Floors (built of reinforced concrete) as this is also the way settlement is likely to occur in weir floors with shallow end curtain walls.

Plate I shows the location of cracks on these floors. The pressure gradients in both cases were above the critical just before the crack x or y occurred. There must have been a cavity underneath as a result of slow but continuous undermining and the slab must have been suspended at odd portions of the soil which were intact. These last points of support gone, the slab settled and the crack appeared. It is noteworthy that just before the settlement started on 25th September 1929 the pressure in pipes No. 10 and 11 dropped by 2·15 and 0·84 respectively, showing sudden development of a steep gradient equivalent to spring formation. The same thing happened with pipe No. 4 on 16th August, 1929. The water level dropped by 0·8 feet and the crack y developed.

Dates.	No. 10.	No. 11.	Canal.	Cunette.	Remarks.
23-9-29 ..	786·26	783·43	92·27	81·44	Crack x developed on 25-9-29. Canal and cunette level almost steady.
25-9-29 ..	784·11	782·59	92·25	81·43	
Difference ..	2·15	0·84	0·02	0·01	

Dates.	No. 4.	Canal.	Cunette.	Remarks.
16-8-29 ..	86·66	91·72	81·6	Crack y opened on 17-8-29 Canal and cunette almos steady.
17-8-29 ..	85·86	91·90	81·6	
	0·80	0·18	0·0	

The overseer incharge noticing this sudden fall in the pressure level wired that something unusual had happened, and right enough there was a crack shortly after. In more than one instance, cracks have occurred immediately on the reduction of supply in, or complete closing off of, the canal. In high supply the upward pressure under the floor kept the floor suspended although cavities existed underneath. But as soon as, with the reduction of supply, this upward pressure was withdrawn the floor settled on firm soil, the superimposed weight being greater than this reduced upwards pressure.

The settlement of the inverted filters and the scour at the end of these are noteworthy. They explain exactly how floors are undermined. This settlement happened both Upstream and Downstream and has been filled up, along with the scour holes at the end. Similar settlement has occurred to a smaller extent at the ends of the Dugri inverted filters. The structural details about the repairs and the remodelling of the Jauryan and the Dugri siphons are of considerable interest and are briefly dealt with later on.

The Narora Weir.

The Narora weir has been specially selected, as the theory of hydraulic gradients in relation to weir designs started from there.

A description of the failure of part of this weir, the repairs and the subsequent additions of a long and impervious floor on the upstream side, ending in a line of wooden sheet piles 18 to 19′ below floor, is given in paper No. 7 of the 1913 Engineering Conference at Simla, by Mr. C. H. Hutton and in the Editor's notes by Mr. F. W. Woods on Chapter VI, pages 185 to 193, of the Design of Irrigation Works by Bligh. Mention is also made of this by Mr. Beresford in the proceedings of the Institution of Civil Engineers 1904, pages 76 and 77.

Briefly, the failure of 30th March 1898 occurred due to :—

(i) Washing away of the Upstream 30 feet puddle apron and the 20 feet loose stone apron, exposing the crest wells 7·5′ to 9′ below bed level.

(ii) Retrogressions of levels Downstream.

The failure of the upstream floor and protection increased the pressure under the floor and the retrogression of levels downstream encouraged piping through the imperfectly closed interstices between the circular wells of the downstream toe wall. In this connection reference is invited to para. 1, page 149 of the paper on Hydraulic Gradients in Subsoil Flow, showing how scour holes close to the pacca work, increase the tendency to piping and hastens failure. This piping through the interstices of the toe wall blew out sand from underneath the floor and cavity formed. Borings revealed cavities under the crest wall and floor extending 50′ on either side of the point of fracture. The cavity was 8′ deep where the spring burst through the lower line of wells and out into the grouted pitching. The lower part of the floor

settled and separated from the top, which was, in turn, lifted up by the increased pressure which worked its way in between the two layers of masonry. The above description, causes and conclusions are based on the accounts given by the authorities quoted above. Attention in this connection is invited to Damage to Bay No. 4 (downstream) of Marala Weir 1929, explained later on. (Plate IV.)

It is at this weir that Mr. Beresford ordered the first pressure observation pipes to be fixed in the year 1898 which were actually fixed on 27th March 1898. Colonel Clibborn's experiments were apparently carried out in the same connection. **Mr. Bligh** evolved from these, his **theory** of the **Hydraulic Gradients** (without foundation on facts) which forms the basis of our present design of structure on saturated soils. A close analysis of the observations at this weir will, therefore, be appropriate. The Executive Engineer, Narora Division, very kindly supplied his original file of observations in September 1929, from which typical plots have been made to elucidate the discussion.

In plate II, figure 1 shows a section of the Narora weir with its pressure pipes, crest and toe walls, the original puddle floor (30′ wide) upstream, the subsequent additions and the upstream end sheet piles. Explanatory notes are given on the drawing.

Figure 2 shows the observations, with the original floor and those after the additions.

The H. G. Lines, according to Bligh, have been plotted in both cases, with c=15 (H. G.=1/15) and the line of creep assumed to hug the underside of the floor, down the sheet piles or wells and up again. (vertical = twice horizontal).

The Upstream levels in both cases are almost the same (584·2 and 584·4).

Case, 1. Downstream levels 574·9 and 574·8 respectively. The effect of the extension of floor by 80 feet of the addition of the sheet piles is that in pipe No. 2 there is a drop of (579·41—578·07=) 1·34 feet and a drop in pipe 3 of (578·73—578·07=) 0·66 feet—an insignificant achievement considering the cost.

Case, 2. Downstream levels 572·9 Upstream levels as above. In this case pipe No. 3 is actually higher by about 0·3 than in the original design and pipe No. 2 is lower by 0·5 feet, showing that in the **worst case the pressures under the pacca floor are practically uneffected by the extensions.** The only useful part of the extensions is the 3 feet high biff wall downstream and the adjoining 25′ × 1½′ concrete slab. The rest of the additions appear to have been unnecessary, altogether.

Bligh Theory and Actuals.

The figures 4, 5, 7 give a clear conception of the Bligh H. G. line in relation to actual facts. At the downstream floor, the only part of the structure subjected to upward pressures, **the Brigh Line is dangerously below the actuals.**

The question arises then, why are all these numerous structures based on the Bligh theory still standing.

A general reply is impossible but each individual design can be separately analysed. But there are two important points which allow a large margin of safety.

1. The full upward pressure never comes into play unless there is a cavity or spring line underneath the floor.

Water in the bound condition cannot exert full upward pressure on the entire area of the floor which is in contact partly with the soil particles and partly with water. The effective area on which the full hydrostatic pressure can act, is then a function of the porosity of the foundation soil. Thus the average uplift pressure under the floor will be a certain percentage (perhaps 2/3rd or 3/4th) of the indicated pressure. Immediately, however, a cavity forms and the underside of the floor comes in contact with an open sheet of water, the full hydrostatic head is exerted and were it not for the shear resistance of the masonry of the floor, which comes into play, some of the floors may be endangered.

2. The actual weight given in the floors is generally in excess of the requirements as given by the Bligh method. **The Engineer's instinct adds where theory is wanting.**

Mr. Hutton in his paper has attempted to show that the observations are in accordance with Bligh's theory if the line of creep is modified. Tested in the light of original observations, there seems to be no relation at all between the calculated and the observed.

Figures 3 to 8 show, for the three observation chains, the effect of varying the Upstream level with the Downstream kept more or less constant. The results are fully in agreement with those of the Dugri and Jauryan siphons. It is therefore clear that **the apparent water surface at the downstream of a work is no indication of the true spring level there.** It is always lower and the difference between the true spring level and the apparent water surface gives the unbalanced head which keeps up the continuous movement of the soil particles, which if not provided for, may be a serious danger to the tail portion of such works. That this unbalanced and hidden head has been a danger to our siphon is proved. That it has played a great part in the undermining of part of the Narora weir is also apparent. But a question has been asked why all our falls, etc. do not fail, if the true spring level is higher than the apparent spring level as indicated by the level of flowing water downstream, when the downstream water level falls in a closure. The following explanation is offered :—

Response of subsoil flow to application or withdrawal of outside head.

When the canal is running there is the head of that water on the floor. When the canal is closing the water level downstream also falls.

The saturation level behind the wing walls and in the bed fall simultaneously so that a more or less steady difference in head is maintained and no disturbance of foundation soil takes place. The time taken by the spring level in the vicinity of a canal or river to adjust itself to the rise or the fall of the source of supply is so small that it might be safely neglected. This fact is demonstrated by the observations of pressure pipes taken every three hours while the canal was opening and closing. The time lag is so small that it cannot have any injurious effect. The fact of this rapid adjustment in soil flow was noticed by Slichter in his sub-soil flow observations when just after rainfall his discharges increased suddenly. On page 70 of his paper on the Field Measurements of the Rate of Movement of Underground Waters (United States Geological Surveys) he states "between 9a.m. and 3 p.m., (29th June 1903) 3 " of rain fell, so that the heavy precipitation coincided with the early part of the ground water measurement." The velocity found was 6·4 feet a day. On July 3rd the experiment was repeated, there being no rain in the intervening time. The velocity found in the second trial was 5·3 feet a day. The change in velocity was undoubtedly due to the enormous rainfall during the first experiment. On page 71 he says, " Actual computation will show that in a uniform sand of diameter of grain of one half millimeter the ground water will reach within 1% of its final maximum velocity due to a sudden application of pressure or head in approximately 30 seconds of time." Mr. Wilsdon in his introduction to memoir No. 3 by Dr. Bose (page 5, fig. 2) has shown how surface irrigation changes the stream line contours of sub-surface flow. This is stated by him to be an actual case and as such is of great importance. He has not discussed the time factor. His observations are in keeping with the observations at the siphons and with those of Slichter. What is true of the application of pressure is true of the withdrawal of such pressure. The response in both cases is equally quick. The following observations are reproduced as an illustration.—

Date.	Time.	Canal gauge.	No. 8	No. 11.	Remarks.
		Jaurvan Siphon U. S.			
1-6-27 ..	10 a m.	82·44	82·30	81·54	Canal opening. These pipes No. 8 and No. 11 are shown on Plate I.
	12 a.m.	86·75	83·60	82·04	
	2 p.m.	87·75	84·10	82·54	
	4 p m.	88·90	85·00	82·74	
2-6-27 ..	10 a.m.	89·50	85·50	83·54	
	12 a.m.	89·70	86·10	83·74	
	4 p.m.	89·90	86·10	83·84	
3-6-27 ..	10 a.m.	90·00	86·81	83·94	
	12 a.m.	90·00	87·01	83·94	
	4 p.m.	90·10	87·01	84·04	
4-6-27 ..		90·40	87·11	84·14	

Dates.	Time.	Canal.	Cunette.	No. 1.	No. 5.	Remarks.
		Dugri Siphon U./S.				
15-7-29 ..		Closed				Canal opening. Pipes 1 and 5 are 35 and 85 feet respectively from water edge.
11-2-29 ..		..	..	86·33	85·76	
11-2-29 ..	M	790·05	88·80	89·53	87·16	
	N	90·71	88·90	90·03	87·26	
	E	91·44	89·04	90·73	87·76	
12-2-29 ..	M	91·74	89·82	91·03	89·06	
	N	92·14	89·98	91·33	89·36	
	E	92·34	90·05	91·63	89·66	
13-2-29 ..	M	93·00	90·50	92·13	90·66	
14-2-29 ..	M	93·2	..	92·53	89·86	
15-2-29 ..	M	93·9	..	93·13	90·66	

Dates.	Canal gauge.	Cunette. W. L.	No. 8.	No. 11.	No. 17.	No. 18.	Remarks.
27-8-29	91.56	82.54	87.03	84.26	85.73	86.41	Jauryan siphon upstream showing effect of withdrawal of external pressure Pipes No. 17 and 18 are outside the pacca floor. 120 ft. away from water edge with strainers about 50 ft. below floor. Canal closing.
28-8-29	84.08	81.60	83.75	82.43	83.36	83.75	
29-8-29	83.50	81.71	83.20	82.67	83.14	83.33	
30-8-29	82.85	81.20	82.67	81.96	82.61	83.33	
31-8-29	82.80	81.1	82.55	81.71	82.51	82.73	
1-10-27	92.40	..	87.82	84.42	..	..	
4-10-27	9130	..	87.42	84.22	..	..	

A series of observations were taken, morning and evening, to watch the effect of canal closing at Bambanwala (25 miles below Marala—Upper Chenab Canal) or spring levels in a well about 1,200 feet away in the Rest House Compound. The Canal dropped from 775·2 to 768·0 in 24 hours (April 10 to 11, 1928) and to 766·2 till 14th. The Spring level gradually dropped from R. L. 773·5 on the 10th to 770·0 till 18th and stayed at 769·7 till the canal started to rise again. This was 1,200 feet away. The drop in spring level, in the vicinity of the fall, must obviously have been much greater and quicker.

This property of quick response of sub-soil flow to external application or withdrawal of pressure is of great importance in a study of our waterlogging problem. It emphasizes the supreme necessity of efficient **surface drainage** and the evil consequences of ignoring this drainage. Drainage operations in the Marala Division substantiate this statement. The remarkable drying up of the large marshy areas on the Main Line Lower (Upper Chenab Canal) as the result of the lowering of the supply in the canal is a good illustration in this respect.

Deep curtain walls at upstream end.

The effect of shallow and deep curtain walls and the nature of flow of water and the accompanying soil movement in a vertical direction just outside a toe wall have been discussed in a previous paper. From Narora and siphon observations it is also clear that piping can occur even though the downstream water level may be high, if scour holes form near the toe wall, as **the true free water surface is higher than the apparent,** the difference determining the extent of piping. It now remains to discuss the effect of deep curtain walls or sheet piles at the upstream end of the work and at intermediate points of the floor. The pressure past the 20 feet sheet piles in the downstream floor of the Dugri siphon and those at the line of well under the crest of the Narora weir indicate that there is a definite drop at these points followed by a flattening of the gradient downstream. But the data are not sufficient or conclusive for a definite mathematical law. Regarding the effect of curtain walls at the Upstream end, the only data, it has been possible to lay hands on, are those at the Narora Weir (plate III). It might be noticed that out of a total drop between Upstream and Downstream levels almost 45 % is lost up to pipe 1A. The same phenomenon is noticeable in case of pipe No. 2 before the extensions were done.

In the original design there was the 30 feet floor of stone over puddle at top R. L. 578 and a line of wells under the crest (bottom R. L. of curbs at 564 or perhaps 563·5). Thus there was a depth of 14 feet and horizontal length of 30 feet of impervious floor. After the extensions, the top of floor is 577·5 and the sheet piles go down to R. L. 560·5 and 559·5 giving a depth of 17 to 18ft. with a puddled floor only 5 feet in front. The effective depth in both cases is therefore not different and it is not surprising to find that the loss of head from upstream to pipe No. 2 in the first case and to pipe (1 A) after extension, is very nearly the same.

Mr. Woods in his editorial note suggests **loss of head at entry** as a possible explanation. The velocity of entry is, however, so small that such a big loss appears unlikely. It perhaps does account for a small part and experimental work is needed to determine this extent.

Loss of Head Curve for downward vertical flow.

In connection with plate VII of the Paper on Hydraulic Gradients etc., the nature of flow as a result of withdrawal of head of 3·8 feet from the normal gradient line has been discussed. If instead, a 3·8 feet head is superimposed on the N. G. line, similar flow should naturally occur in the reverse direction, the velocities decreasing from top down and so also the pressures but following a curve of the same family as the loss of head curve of plate VII.

A curve has been derived from the observations at chain 32. (Pl. III). The fall between upstream and downstream levels has been plotted on the left hand horizontally as A B and the loss of head up to pressure pipe, 1A, has been plotted at a point 18 feet below as C D, leaving D E effective head at that point. A_3 B_3 is another observation of head which is exactly equal to D E. C_3 D_3 is similarly plotted 18 feet below B_3 or D. This gives three points on the curve B, B_3 and D, D_3. An approximate curve was run through these and other points were tested on this curve. With small trial and error the final curve of plate III was worked out. It will be seen that points D, D_2 D_3 D_4 and D_5 lie on the curve as close as can be expected. The equation of this curve is $y+64{\cdot}5 \log h = 64{\cdot}0$. The constants in this equation depend on the point where observations start (H = 9·8 being the starting point here).

This curve is of the same family as the loss of head curve, with the water coming vertically up, the one difference being that whereas in the latter the velocity varied directly with the loss of head, in this case they vary with the effective head A B, A_1 B_1, A_2 B_2 etc. The loss of head at any depth x being represented by x x_1 and the velocity at that point being proportional to x_1 x_2. Thus in a **downward vertical flow, under superimposed head, the rate of loss of head and the velocity, both, decrease as the depth increases.** In the example under consideration for a 9·8 feet head, an 18 feet deep sheet pile is responsible for a loss of 4·65′ and is therefore equivalent to a horizontal length of $15 \times 4{\cdot}6 = 69{\cdot}75$ feet with Bligh's c=15. A deep curtain wall at the upstream end is also useful in preventing sloughing of foundation sand in the event of a scour hole forming close to the pacca work as happened at the Narora Weir.

The Rasul Weir.

On page 197 of Bligh's book a set of observations is given regarding the Rasul Weir. Unfortunately no other observations could be got out of the Rasul Division and criticism on an isolated case cannot lead to much. The shape of the Hydraulic Gradient line is much the same as in the Narora Weir and exhibits the same general features. This figure is reproduced here for the information of the reader as plate IV.

The damage to Rasul weir during the record flood of August 1929 is of considerable professional interest and has a definite bearing on this paper. In certain bays the damage is on the upstream of the crest, in others it is downstream and in bay No. 4 the upstream floor, the crest and the downstream floor are all gone.

Apparently the oblique and cross flow during the flood damaged the upstream and downstream loose protection by swirls, surging and scour, causing deep holes close to the shallow curtain walls at the upstream and downstream ends of the weir floor. This resulted in sloughing of the sand foundations at the upstream end and piping at the downstream end. The middle portion of the weir was possibly damaged by the standing wave. At one place in bay No. 5 the top "Kharanja" has lifted up by 2½ feet above the flat masonry, the shape of the "blister" being much the same as at Marala described later on. The further details of this damage with causes that led to it, will, perhaps, be described by one of the local officers at the Congress.

Standing wave below weir.

Plate VII shows a typical standing wave during the record flood of 1st September 1928 at Marala weir showing also the uplift of the Kharanja at the trough. The standing wave in this case is eight feet above the trough but a height of nine feet has been observed during the flood of 28th August 1929. The observed profiles will be exhibited at the Congress. The upward pressure at each part of the "blister" or lifted floor is shown at right angle to this floor and it will be seen that the crack or the point of maximum uplift is very near the centre of pressure of this pressure diagram.

So long as the standing wave forms on the pacca floor, the tendency of lifting up will be spread over a big area and if the different layers of masonry or concrete can be properly keyed to each other, no damage will occur provided the overall thickness of masonry is strong enough for the maximum upward pressure at the trough. But directly the standing wave travels down due to retrogression of downstream bed, etc., and forms at the block or stone protection, a double action sets in. The back pressure of the standing wave tends to lift up blocks and stone, At the same time this pressure has a tendency to get relieved through the interstices between the blocks or stone. This second case exactly corresponds to the conditions of Plate VII of the paper on pressure gradients, etc. Piping starts with vigour from underneath the blocks or stone and finally extends to the foundation soil beneath the pacca floor causing failure.

The action of swirls and surging is exactly similar to that of a standing wave. There is an abrupt drop in head in both cases—as much as four feet has been observed by the author. This difference in head will certainly cause piping and may in an extreme case lift up blocks even, which once dislocated can be easily carried forward with the force of flood waters.

Thus, the magnitude and position of the standing wave under different conditions of supply, is an important factor in determining the shape, length and thickness of the pacca floor and the type and magnitude of loose protection.

The Marala Weir.

Some pressure pipes were inserted in the downstream floor of this weir in 1926-27, and it is hoped, more will be put in this year at suitable points. The results of these when available will be of interest.

In the record flood of 1st September, 1928, in Bay No. 4, as also in some others, the top 'Kharanja' of the downstream glacis Lifted up, the layer of stone masonry underneath remaining intact. This lift up was due to the back pressure of the standing wave (which was about 8 feet higher above the trough) having been exerted in full under the top 'Kharanja' layer which had separated from the lower layer, by means of the intrusive water finding its way in between the two layers. The crack in the 'Kharanja' naturally occurred at the point of maximum up-lift which happened very nearly at the trough, where the difference in pressure was maximum. The up-lift is shown in Photograph No. 1 and Plate VII.

In the second record flood of 28th August 1929, a similar uplift occurred at and above the B line of wells accompanied by a slight settlement of the masonry floor underneath. The well line B also appears to have settled in part of the damaged reach. This is clear from the fact that there is a hollow underneath the concrete blocks laid diagonally and true, in 1928-29 to replace the 'Kharanja' layer that had lifted. They are still at the true designed level so that the hollow underneath can only mean settlement of the wells and the flat masonry below the Kharanja. (See Plate V).

The damage apparently started with the loose stone protection getting washed down with the immense volume of water or sucked out by swirl action due to oblique flow and then carried away; very likely the latter is what happened. With this stone gone, the concrete blocks followed leaving the face of the well line C more or less exposed resulting in blowing out of sand from between the wells, in the subsiding flood. Between the Well Lines B & C, it will be noticed that the stone settled at places as much as 14 feet deep, *i.e.*, to a depth of 779·9 against a bottom level of wells of 777·8.

The blowing out having occurred to that extent, it is no wonder that some sand got blown out from above the B Line of wells and possibly some from underneath the wells causing settlement of the wells and the floor above. Were it not for the deep line of wells at the end, serious damage might have occurred. It is noteworthy that on the settled area between B and C line of wells, the probings every where indicated stone. This shows that this floor failed by undermining and settlement, not by the masonry and stone having been carried away by the volume of the flood.

Here it may be mentioned that undermining can occur even if the stone apron is in tact provided that the block protection adjoining the end of the pacca work is sucked or washed away due to the destructive agency of swirls, standing wave surging, or hydrodynamic action. At Marala, when the blocks used to be laid in smooth faced moulds and the

spaces between them were filled up with spawls and stone, this filling was usually carried away with the current, thus exposing the layer of loose stone under the blocks and finally the bottom sand. That done the undermining of the blocks started both due to suction caused by surface flow and the blowing out caused by the pressure gradient of subsoil flow. The block either settled or were bodily lifted up under suitable hydraulic head conditions and were deposited some 50 or 100 feet away intact, almost as if lifted by human hand. With the lowering of the bed, piping started in that reach (as explained in the paper on Hydraulic Gradients, etc.) and it the toe walls were shallow, failure would occur. To gaurd against this class of damage occurring every year, the new blocks were poured in between side walls of Kattals (spawls) which by their projections into the blocks on either side prevented dislocation but permitted settlement. The spawls thus wedged in, did not move out and away with the current. Repairs done in this way have proved satisfactory. By some strange process water will move wide blocks (as much as 12 feet wide block of masonry 1½ to 2 feet thick, has been seen bodily moved at Marala). *The only way to secure them effectively, without impairing the pervious and flexible nature of protection, is to stand them on end or to interlock or wedge them as explained above.* This also points to the necessity of filling up spawls in the spaces between blocks where such have been removed. This part of the repairs is as important as the replacing of or repairs to the missing or damaged blocks.

The Khanki Weir.

A large mass of pressure pipe observations is said to be available regarding the Khanki weir. If so, it will be collected, scrutinised and presented at the Congress.

The Sulemanki Weir.

Out of the Sutlej Valley Weirs, this is the only one which has pressure pipes in the weir floor and elsewhere. Unfortunately a large number of the pressure pipes are blocked since June 1926 but a sufficient number of pipes are still in working order and the representative observations have been plotted on Plate VI.

The plots of these observations bring out the following points :—

(*i*) The drop from the upstream water level to the upstream of the sheet piles varies with the total drop from upstream to downstream levels. Compare curves 1 and 2 of fig. 1 and 1 and 2 of fig. 3. In other words there is greater flow through or underneath the sheet piles in higher heads.

(*ii*) Most of the head is lost at the sheet piles and even under the worst conditions the pressure gradient at the downstream floor remain flat and safe.

(*iii*) The pressure pipe levels under the downstream floor are governed by the downstream water levels and very little by the upstream water levels.

The selection of site for this Weir has been very happy and the bedding of sheet piles in the 10 feet thick clay layer, very sound and fortunate. This clay layer underlies the weir floor and extends both upstream and downstream. How far? It is yet to find. The flow that occurs can be either through the sheet pile interlocks, or through the clay or through the sand underneath this clay. Whatever way it may be, it is clear that the sheet piles bedded as they are in clay are very effective, and as far as the present study indicates, this weir is safe against piping due to subsoil pressure gradient, inspite of the shallow downstream end toe wall, if the clay remain intact. The dislocation of blocks due to standing wave or other reasons may, however, cause trouble and it might be advisable to consider putting in a line of sheet piles at the end of block protection.

The retrogression of bed levels at this weir requires separate consideration but the proposed line of sheet piles will help.

Jaba Level Crossing Upper Jhelum Canal.

This will be an interesting study as there has been trouble, repairs, extensions and a series of pressure pipes observations at this work. As the data could not be obtained in time, their discussion will be presented at the Congress.

Structural Details of Siphon Floors.

As there are some novel features in the design of the approach and exit floors of the Dugri siphon, a brief outline of these is given here.

The approach to the middle two barrels out of 8 was lowered in the shape of a cunette, two feet below the normal floor level, with the idea of improving drainage upstream and also of passing low floods through the cunette, the high floor barrels coming into action only in case of high supplies. This provision ensures automatic clearing of the barrels, as there is either no flow or flow with good velocity.

The upward pressures under the floor slab were based on actual observations of spring level. These were between 5′ and 6′ at the Dugri siphon. For a gravity section of floor against uplift, a very heavy section of concrete would be required in view of its loss of weight in water. Such a heavy section would be inadmissible on account of the depth of excavations in slush with consequent damage to the structure which it was desired to protect. It was decided to put in a reinforced concrete slab designed as a continuous beam in the two bays of the cunette (3 bays in case of Sambarial siphon R. D. 90.275) the load of the middle and side piers being exactly equal to the reactions at these points due to the uniform upward water pressure less the weight of the slab. These reactions and the moments were calculated by the theory of 3 moments both for 2 spans and 3 spans. Thus the middle wall is about double the height of the side walls of the cunette. The high level floor is either a gravity section or consists of sand filled boxes, with bottom and top slabs of doubly reinforced concrete, the reinforcements of the top and bottom

slabs being tied by verticals. In the case of the Sambarial siphon, the height of the centre walls worked out to 22 feet and it was decided to take the two centre walls only 8 feet high, the balance of the load being given by earth filled up to necessary height over a reinforced concrete roof slab.

The reinforcement of the bottom slab was calculated both for uplift and for settlement due to cavitation underneath. The full upward pressure of water was taken for calculations although it was realized that this full pressure could not come into play unless a cavity occurred. The load of slab and piers balanced this pressure at its highest but when this pressure went down earth pressure came into play and balanced the downward load. The heavy downward load is very useful, as in case of cavitation under the floor, this weight settles the structure down and prevents further development which would surely occur in case of a light floor which could remain suspended under low pressures, permitting direct passages to water between the underside of the slab and the foundation soil. The junction of the new floor with the old floor at the face of the barrels was a work of some difficulty and required careful supervision. If this junction had been left faulty, blowing up would have started near the face endangering the structure. To secure a water tight joint, holes 2' long were drilled obliquely into the foundation concrete under the barrels lips by means of pneumatic drills (see photo 3) ; reinforcement bars were fixed in these, extending well into the inforcement of the new floor and were carefully tamped and grouted with cement. This has proved very successful.

Unless the slab of the cunette runs under the entire floor, as in the case where the high floor consists of sand filled, R. C. Boxes, the junction of the cunette floor slab with the side walls is very important. They must be rigidly fixed, as any break along the joint will start piping under great pressure and endanger the structure by undermining. This is how failure occurred after the repairs of 1923-25 at the Dugri No. 1. (Photo 2).

Another important point was the securing of sheet piles at the end to the floor as a leak between the top of the piles and the bottom of the slab would have been dangerous. This was achieved by passing reinforcement bars through holes at top of the sheet piles and bedding them at least 40 diameter in the slab concrete, which projects down $1\frac{1}{2}$ feet below the general bottom, both sides of the sheet piles, in the shape of a toe wall to give a good bedding to the top of the sheet piles.

At the end of the pacca floor **an inverted filter** has been given consisting of fine grit at the bottom covered with increasing grades of ballast and finally covered by dry brick pitching on top. This filter is thickest at the pacca floor end and diminishes further out. These filters have been of immense help in saving these siphons at some stage or the other. They have pushed the springs and sand blowing at a safer distance from the structures. In the immediate neighbourhood of the pacca floor they let

In remodelling the siphon floors some of the old work had to be dismantled where grouting under pressure had been done. This exposed regular seams C G. (photo 3) filled with neat cement grout giving a most perfect joint between the two layers of masonary which had separated by unequal settlement. These seams were as much as 1 inch or more in thickness. The grouting under pressure besides filling up cavities, impregnates the foundation sands and forms a lean cement and sand mixture, which in setting becomes very much more resistant to undermining than the original sand. It fills up all masonary joints which come in its way and fills up all pores in concrete. The latter fact was noticeable when with grouting under the floor slab of the Jauryan siphon, some air bubbles started coming out at some places through the pores of the concrete slab. They continued for a while and then automatically stopped showing that the pores have been filled up under pressure. Such operations must however be carefully watched so as to prevent damage by uplift of any weak portion of the floor. In one instance the effect of grouting was noticed through a relief pipe 45 to 50′ away from the grout pipe.

Further Observations and Laboratory Research.

Before closing the paper it is desirable to indicate lines on which further observations and laboratory research may be usefully carried out. A set of pressure pipes with filter points at the bottom, should be fitted in weirs and some selected falls at selected points so that they can indicate Loss of head

i. At entry

ii. Under the vertical drop wall of a fall or under upstream line of sheet piles or wells in a weir and also at different depths from surface down.

iii. Upstream and downstream of intermediate curtain walls both immediately under the floor and at the bottom of these walls, and at different depths from surface down on both sides.

iv. At the end curtain walls. (Sufficient data in this respect are available at the siphons dealt with in the paper on Hydraulic Gradients etc.)

v. At the junctions of wing walls and floors.

Not till such information is collected in fuller detail in the field and in the laboratory, can we hope to lay down any scientifically correct theory about Hydraulic Gradients as affecting Design of Structures on Saturated Soils.

In the laboratory experiments should be carried out with models of weirs and falls with varying depths and locations of end and intermediate curtain walls and also without any wall at all. The conditions should be as near the natural as possible. Both flat and sloping downstream floors should be tried.

Pressures should be observed at different points along the vertical both for downward and upward vertical flow. Flow line contours should be studied along with velocity heads at different points and at different depths. The intensity of flow between the foundation soil and the underside of a floor slab and down and up along the sides of the lines of wells or sheet piles should be carefully studied, as this flow along the line of creep is an important factor in determining the stability of a structure. Colonel Clibborns has exhibited one such observation in figure 5, plate, 1, as reviewed through a glass front. On page 204 of Freeman's Hydraulic Laboratory Practice the following occurs: "The tests, a continuation of those shown in figures 106 and 107 (pages 116 and 117) were conducted for the purpose of studying the phenomenon of seepage. Although the water pressures in the lower sections of a dam are greater than those in the upper sections the tests showed that the velocity of seepage water through the dam is much less in the lower section than in the upper section. In the case of dams with clay core walls the seepage water flows downwards until it is near the cut off wall, then it flows upward and around the core wall." The above remarks are illustrated by photographs on pages 490-494. These along with pages 423 to 425 are well worth perusal in this connection.

A practical study of subsoil flow in the laboratory as relating to design of structures resting on saturated soils coupled with a mathematical analysis on the lines of Dr. Bose's memoir of April 1929, will be a most useful contribution and for that reason the best place for the research is the Research Laboratory at Lahore with its glass flume and other facilities which can be modified to suit. This research, amongst other things, will include a study of the critical depths and critical gradients for various classes of soil for different degrees of packing and of the stream lines and lines of equal velocity potential as effected by obstructions of different shapes and depths as also by superimposition or withdrawal of external pressure, including a consideration of the time lag. The results of models must, of course, be tested against full size structures of which quite a large number exist and the most suitable ones can be selected for the study. Some definite laws can then be formulated to help in future designs.

Conclusions.

For want of fuller data the conclusions are necessarily provisional unless and so far as the text of the paper has already proved.

i. The accepted theory of the hydraulic gradients as applied to designs of weir floors etc., as adopted by Mr. Bligh and reduced to an empirical formula in the second edition of his book, is unsupported by facts and can be dangerous in its application as demonstrated by observations at the various siphons of the Main Line Upper of the Upper Chenab Canal and by those at the Narora weir.

ii. The structures designed on this theory stand because the instinct of the engineer put in where theory was wanting.

iii. Except in pervious weirs of the Okhla and the Sone type where pressures get released without damage all along the path of flow, it is desirable to have a deep toe wall upstream, which will, once and for all, kill a fair proportion of the head and economise considerably in the thickness of floor and to have a deep curtain wall at the downstream end to prevent undermining by piping due to the unseen head between the true free water surface and the apparent water surface, downstream of a weir or over a siphon floor. Intermediate curtain walls, specially, a deep one under the weir crest, are desirable to localize action, should damage occur due to standing wave or other causes. The deep end curtain walls also ensure against damage due to sloughing of saturated sand in case of deep scour holes forming close to the ends of the pacca floor. The exact depth of the downstream curtain wall, will depend on the Pressure Gradients under the worst conditions, (full supply upstream and water at bed level or lower, downstream) and on the critical depth for the class of foundation soil. **Shallow toe walls** specially **in a concrete monolithic weir** or a reinforced concrete floor of a siphon are risky as the structure being rigid, small cavities will not be discovered, the action will continue and perhaps **the first warning may be a collapse.**

iv. An inverted filter at the end of the downstream curtain wall is a great factor of safety as it permits passage to water and prevents sand-movement in the close vicinity of the work. The importance of the proper maintenance of the upstream and downstream stone protection, inverted filters etc. of weirs (and siphon floors) are too well known to require comments. The text has shown how the piping may commence in an otherwise sound design if the bed of the channel drops, at the end of an impervious floor by local scour or retrogression. The loss of head curve drops, giving increased pressures for the same depths below the bed, thus inducing piping.

v. Structures have failed by subsidence due to undermining more than by uplift. The uplift comes in with the full upward pressure (due to subsoil pressure gradient or the back pressure of a standing wave exerted at the trough. Marala weir floor uplift in 1917-28-29 are instances of this latter) working its way in between the top and bottom layers of masonry and concrete which may have previously separated owing to settlement of the lower layer, faulty workmanship, lack of adhesion of mortar or to the upper layer expanding or contracting more than the lower owing to changing temperature conditions. The adhesion between different layers of masonry or concrete laid at different times must receive the most careful attention, as, while a thickness of floor may be adequate, if monolithic, it will become unsafe as soon as any layer separates permitting water to enter in between and to exert the full hydrostatic pressure which anything less than the full thickness of floor cannot safely stand.

vi. The correct place to put relief strainers (which must be the approved metal strainers like the Tej with slits as fine as possible and not

the old type of pipes filled with graded ballast) is the toe wall, and the correct spacing, to be effective, is determined as explained in the text. They are useless in the body of the work unless placed very close all over, for their influence is local, and does not in the least effect the True Hydraulic Gradient Line.

vii. The pressure or hydraulic gradient under a floor is a straight line joining the Upstream with the downstream True water surface, as distinct from the level of the tail water, unless modified by curtain walls, the correct influence of which in regard to depth and location has yet to be studied.

viii. The true water level or the spring level at the tail of a weir or fall or on a siphon floor is higher than the visible water level and the difference from the true to the visible determines the extent of possible danger from piping, and therefore the depth of the downstream curtain wall. This head exists irrespective of the tail water levels but it increases as the tail water level drops, the upstream water levels remaining constant.

ix. There is a critical head for each class of soil which will start getting dislocated under heads in excess of the critical. The undermining of foundations is a slow process of degradation and not as usually understood in tube well practice, due to the fine grains of sand getting washed out of the interstices between the coarser grains.

The critical head for the Punjab sands ranges from 6 inches to one foot, but the fact requires confirmation.

x. Interlocked steel sheet piles are more efficient in boxing foundation soils than wells unless the interspaces between the latter are satisfactorily filled down to the bottom with the aid of a water jet. The wells are however, better as self-sufficient anchors. In case of sheet piles care should be taken to ensure that they are really interlocked and that they are secured at top to the concrete or masonry of the floor above. Failure in either of these respects will permit creep with high velocities and result in undermining.

xi. The flow through subsoil gives a very quick response to any superimposition or withdrawal of outside pressure.

xii. Scientific research in laboratory and in field is required in right earnest, to evolve the correct theory of pressure gradients in saturated soils in relation to length of floor and depth and location of curtain walls, to replace the existing faulty conception of it. The results expected are of sufficient importance to justify the expenditure involved.

Acknowledgments.

The author is grateful to Mr. Murphy, Superintending Engineer, whose help, encouragement and guidance at all stages made it possible for the author to initiate and prosecute this investigation during the last three years.

Acknowledgments are due to Mr. M. D. Mithal, Sub-Divisional Officer, Headworks, who has considerably helped the author in collecting and collating data and to Mr. A. B. Malik, Temporary Engineer, who did the observation work with great care, and P. Gian Chand, Overseer, who in addition to carrying out experiments in a scientific manner put up some original suggestions at different stages of investigation. The works of Mr. Wilsdon, Dr. Bose and of other authors drawn upon have already been acknowledged in the text.

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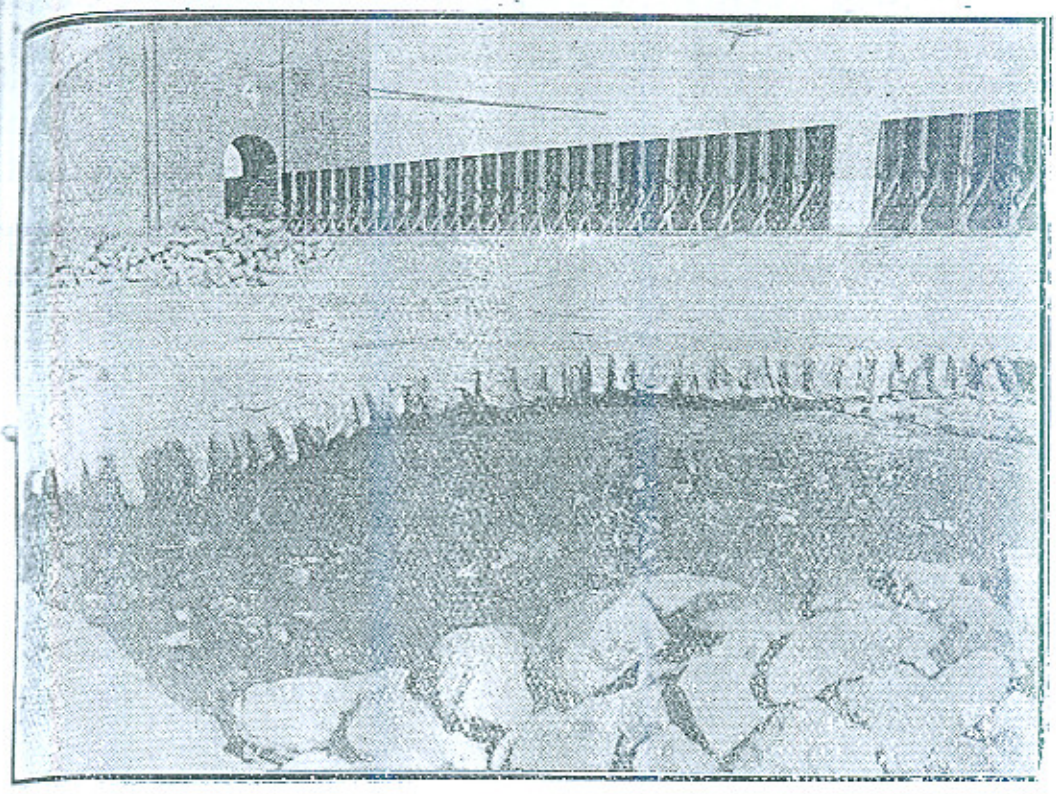


Photo No. 1.

Bay 4. Marala weir, after record flood of 1st September 1928 showing uplift of top Kharanja due to back pressure of standing wave about 8ft. high.

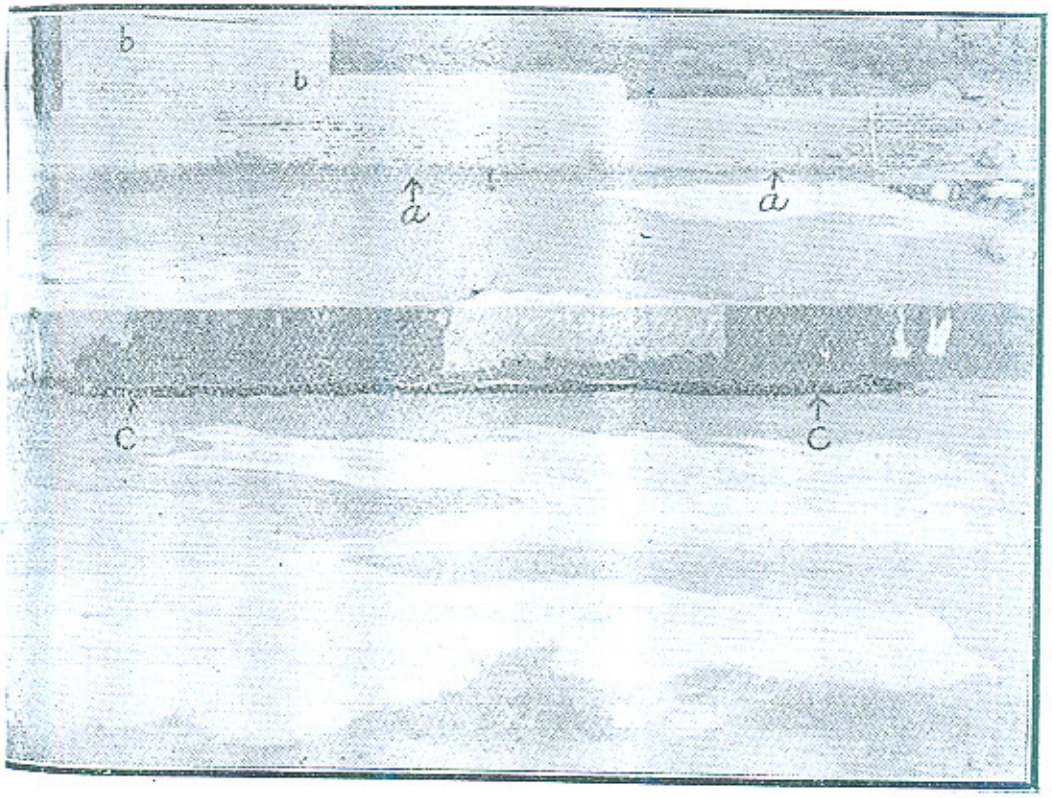


Photo No. 2.

Dugri 1 Upstream floor (January 1929, before remodelling).

Showing cunette wall separate from floor CC.

Tremendous quantities of sand continued blowing through this crack close to face of the barrels.

Note also a crack between floor and wing wall aa, and crack in wing wall bb.

Conclusion.—Side walls and floors should be rigidly connected by vertical agreement to ensure settlement of the whole mass in one.

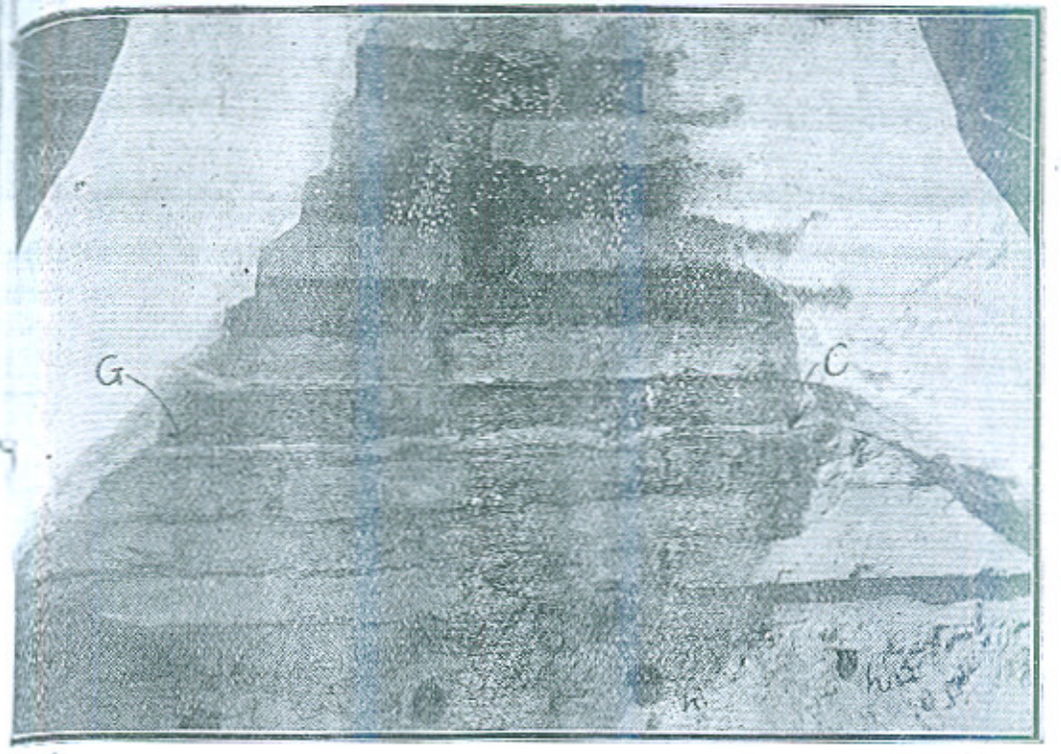


Photo No. 3.
Dugri siphon 1 Upstream (before remodelling) January 1929 showing holes (h) Drilled by compressor for letting in reinforcement to connect the old and new work.
Mark the seam of cement grout (CG) done in 1925 which has effectively filled in the big crack.

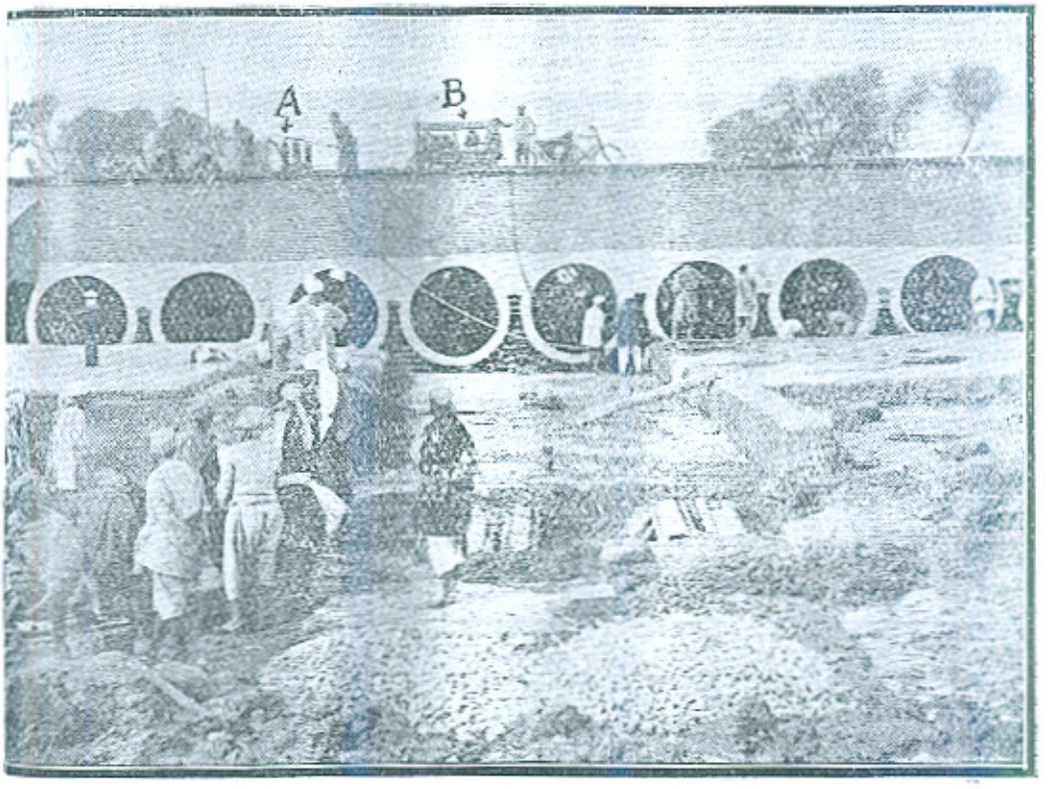


Photo No. 4.
Dugri siphon Upstream. (January 1929). General view of work in progress. Note the grouting machine A and compressor B, the latter drilling holes with pneumatic drill and former cleaning rubbish out of these holes with water under pressure and also cleaning floor. This very machine does cement grouting under pressure with results as exhibited by the seam (CG) in photograph 3.

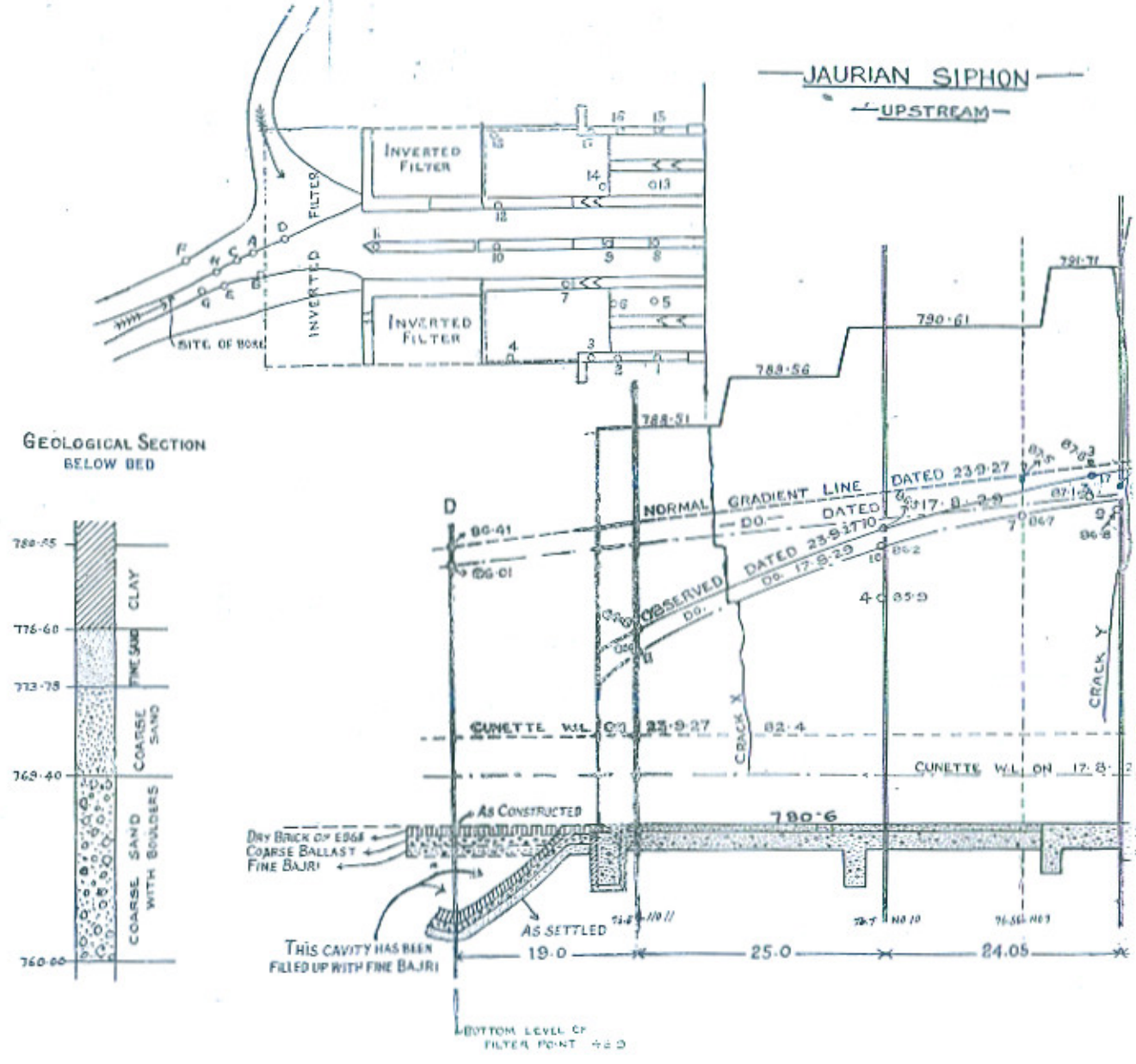
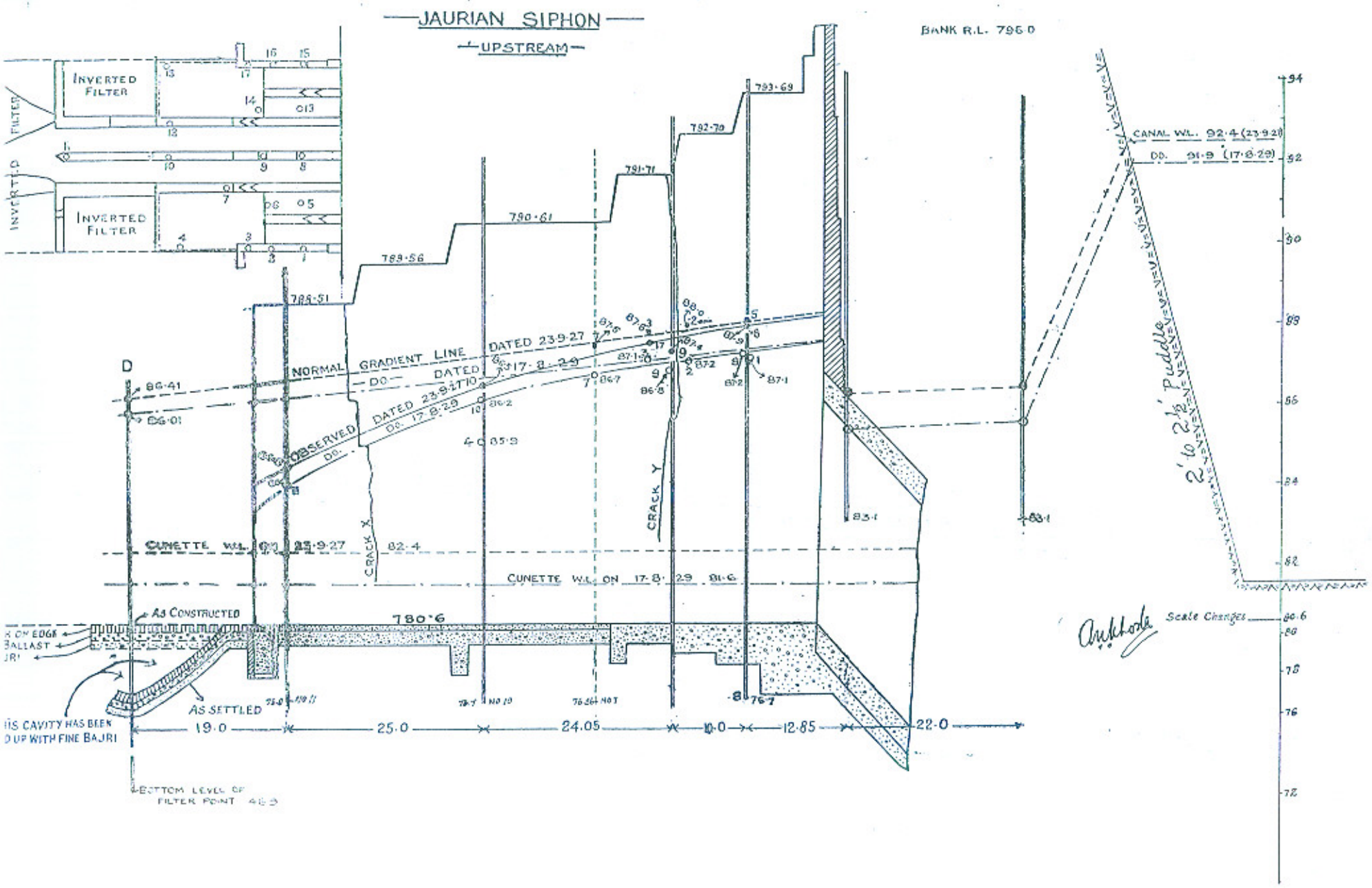
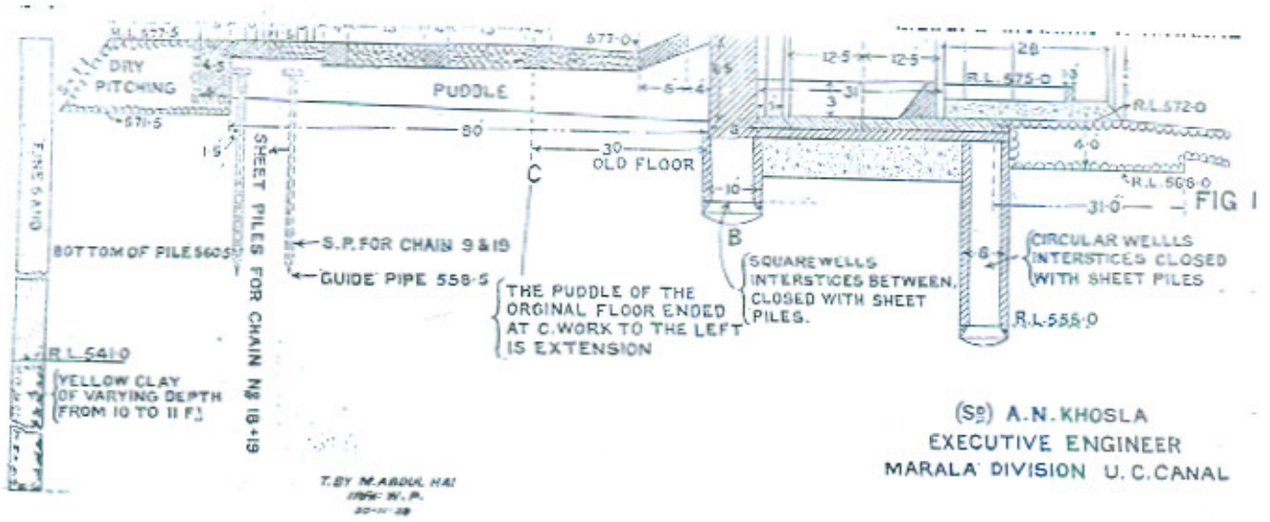
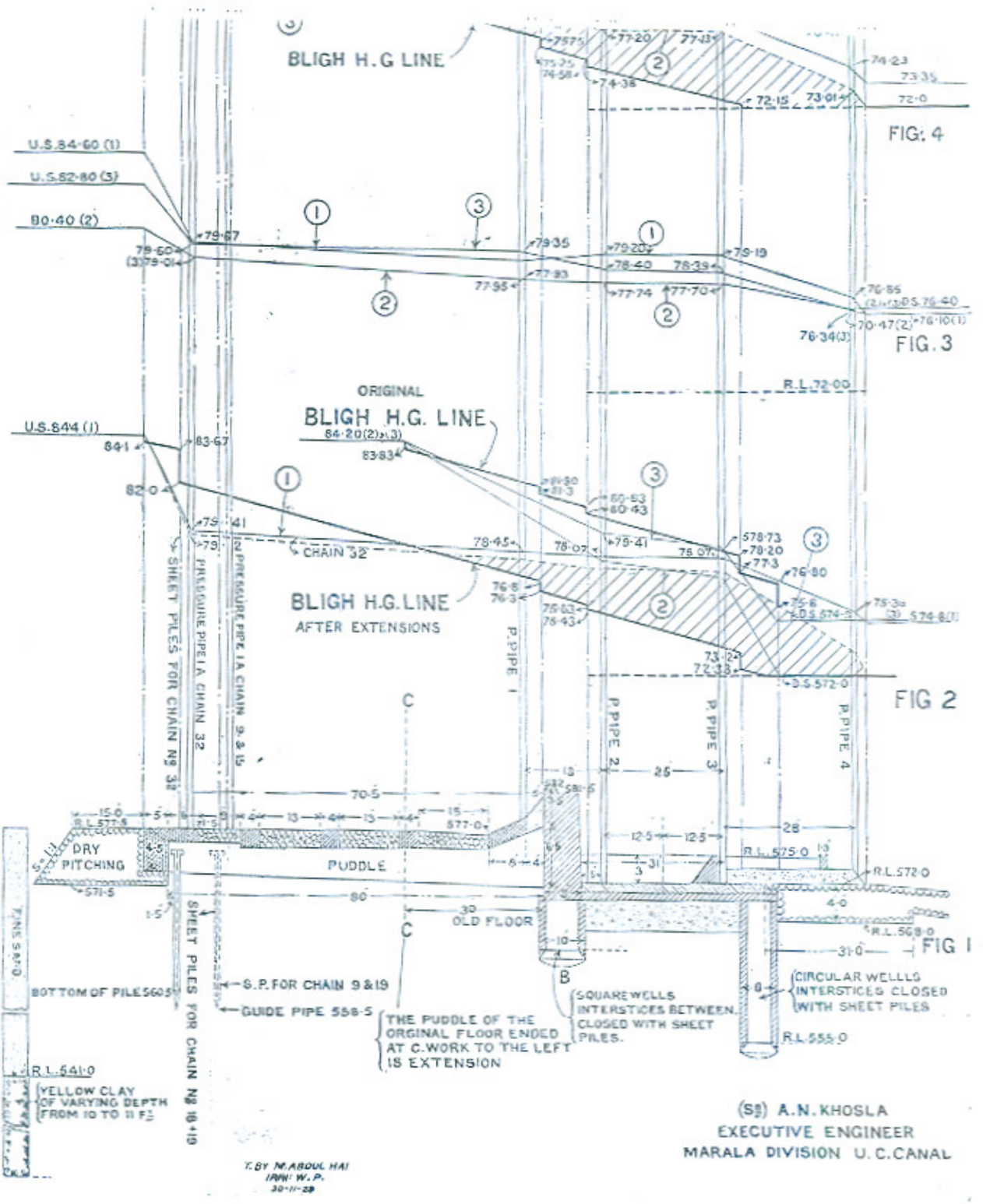

JAURIAN SIPHON
UPSTREAM
INVERTED FILTER
INVERTED FILTER
INVERTED FILTER
SITE OF BORE
GEOLOGICAL SECTION
BELOW BED
780·55
776·60
773·75
769·40
760·00
CLAY
FINE SAND
COARSE SAND
COARSE SAND WITH BOULDERS
788·56
790·61
791·71
788·51
NORMAL GRADIENT LINE
DO. DATED 23·9·27
DO. DATED 17·9·29
OBSERVED DO. DATED 23·9·27
CRACK X
CRACK Y
CUNETTE W.L. ON 23·9·27
CUNETTE W.L. ON 17·9·
AS CONSTRUCTED
AS SETTLED
790·6
DRY BRICK OR EDGE
COARSE BALLAST
FINE BAJRI
THIS CAVITY HAS BEEN FILLED UP WITH FINE BAJRI
19·0
25·0
24·05
BOTTOM LEVEL OF FILTER POINT

PLATE I.


JAURIAN SIPHON
UPSTREAM
BANK R.L. 795·0
INVERTED FILTER
INVERTED FILTER
FILTER
INVERTED
793·56
790·61
791·71
792·79
793·69
785·51
NORMAL GRADIENT LINE
DATED 23·9·27
DO DATED 17·8·29
OBSERVED DATED 23·9·27 & 10 DO. 17·8·29
CRACK X
CRACK Y
CUNETTE W.L. ON 23·9·27 82·4
CUNETTE W.L. ON 17·8·29 81·6
AS CONSTRUCTED
AS SETTLED
780·6
19·0
25·0
24·05
10·0
12·85
22·0
BOTTOM LEVEL OF FILTER POINT 46·3
CANAL W.L. 92·4 (23·9·27)
DO. 91·9 (17·8·29)
Scale Changes
94
92
90
88
86
84
82
80·6
80
78
76
72


R.L. 577-5
DRY PITCHING
4-5
4-0
PUDDLE
577-0
6-5
6-4
12-5
12-5
28
R.L. 575-0
13
3
31
R.L. 572-0
4-0
R.L. 568-0
31-0
FIG I
671-5
80
30
OLD FLOOR
C
B
10
5
1-5
SHEET PILES FOR CHAIN Ng 18+19
S.P. FOR CHAIN 9 & 19
GUIDE PIPE 558-5
BOTTOM OF PILES 605
R.L. 541-0
YELLOW CLAY OF VARYING DEPTH FROM 10 TO 11 Ft
THE PUDDLE OF THE ORGINAL FLOOR ENDED AT C. WORK TO THE LEFT IS EXTENSION
SQUARE WELLS INTERSTICES BETWEEN, CLOSED WITH SHEET PILES.
6
CIRCULAR WELLLS INTERSTICES CLOSED WITH SHEET PILES
R.L. 555-0
T. BY M. ABDUL HAI
(Sd) A.N. KHOSLA
EXECUTIVE ENGINEER
MARALA DIVISION U.C. CANAL


BLIGH H.G LINE
FIG. 4
U.S.84·60 (1)
U.S.82·80 (3)
80·40 (2)
FIG. 3
ORIGINAL
BLIGH H.G. LINE
R.L 72·00
U.S.844 (1)
BLIGH H.G. LINE
AFTER EXTENSIONS
CHAIN 32
SHEET PILES FOR CHAIN Nº 32
P.PIPE 1
P.PIPE 2
P.PIPE 3
P.PIPE 4
FIG 2
DRY PITCHING
PUDDLE
OLD FLOOR
SHEET PILES FOR CHAIN Nº 16+19
S.P. FOR CHAIN 9 &19
GUIDE PIPE 558·5
BOTTOM OF PILE 560·5
THE PUDDLE OF THE ORGINAL FLOOR ENDED AT C.WORK TO THE LEFT IS EXTENSION
SQUARE WELLS INTERSTICES BETWEEN CLOSED WITH SHEET PILES.
CIRCULAR WELLLS INTERSTICES CLOSED WITH SHEET PILES
R.L 555·0
R.L 541·0
YELLOW CLAY OF VARYING DEPTH FROM 10 TO 11 F.S.
FIG 1
(S.D.O) A.N. KHOSLA
EXECUTIVE ENGINEER
MARALA DIVISION U.C.CANAL
T. BY M. ABDUL HAI
IRRI. W. P.
30-11-28

NARORA WEIR

PRESSURE PIPE OBSERVATION

PLATE No II

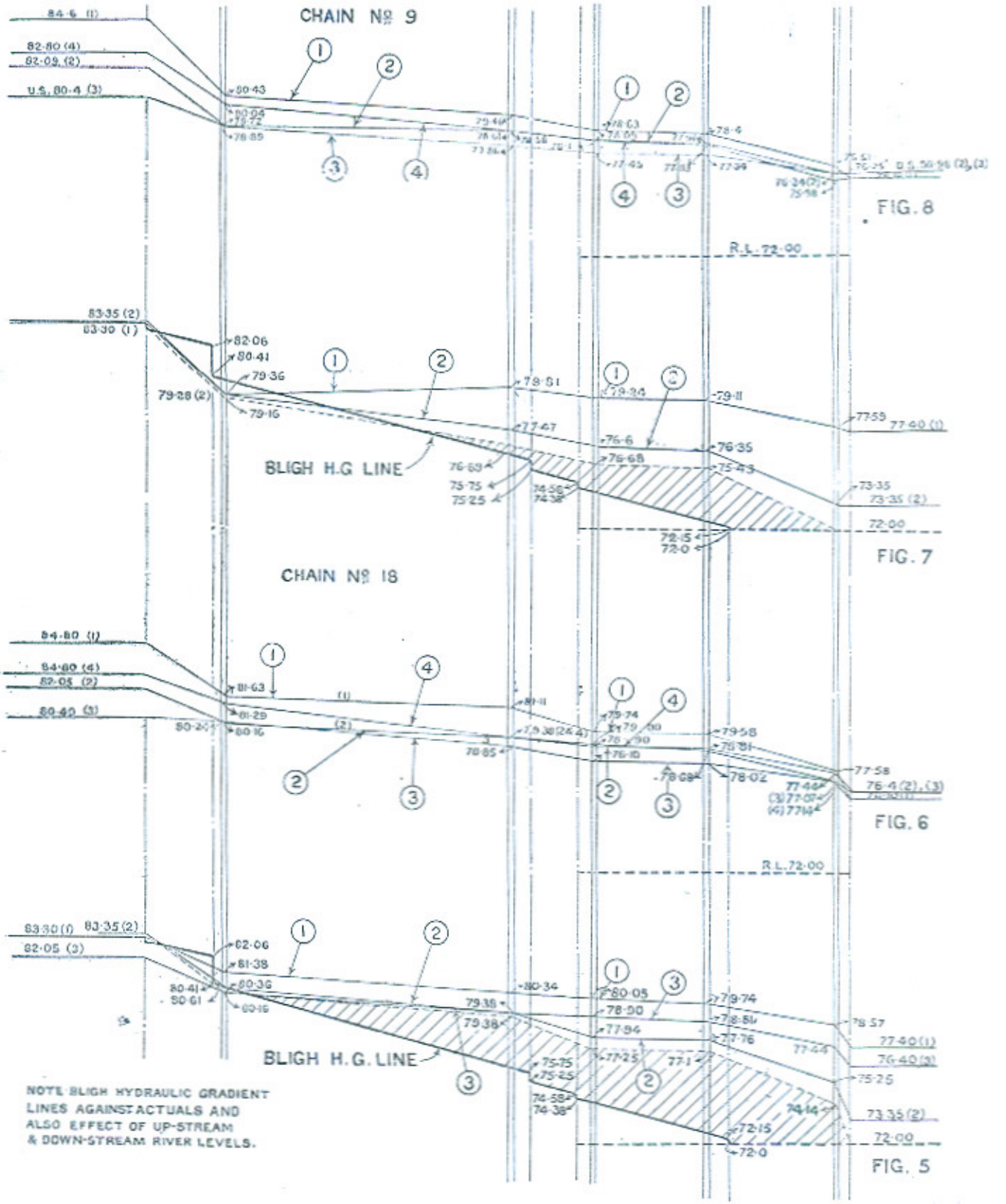
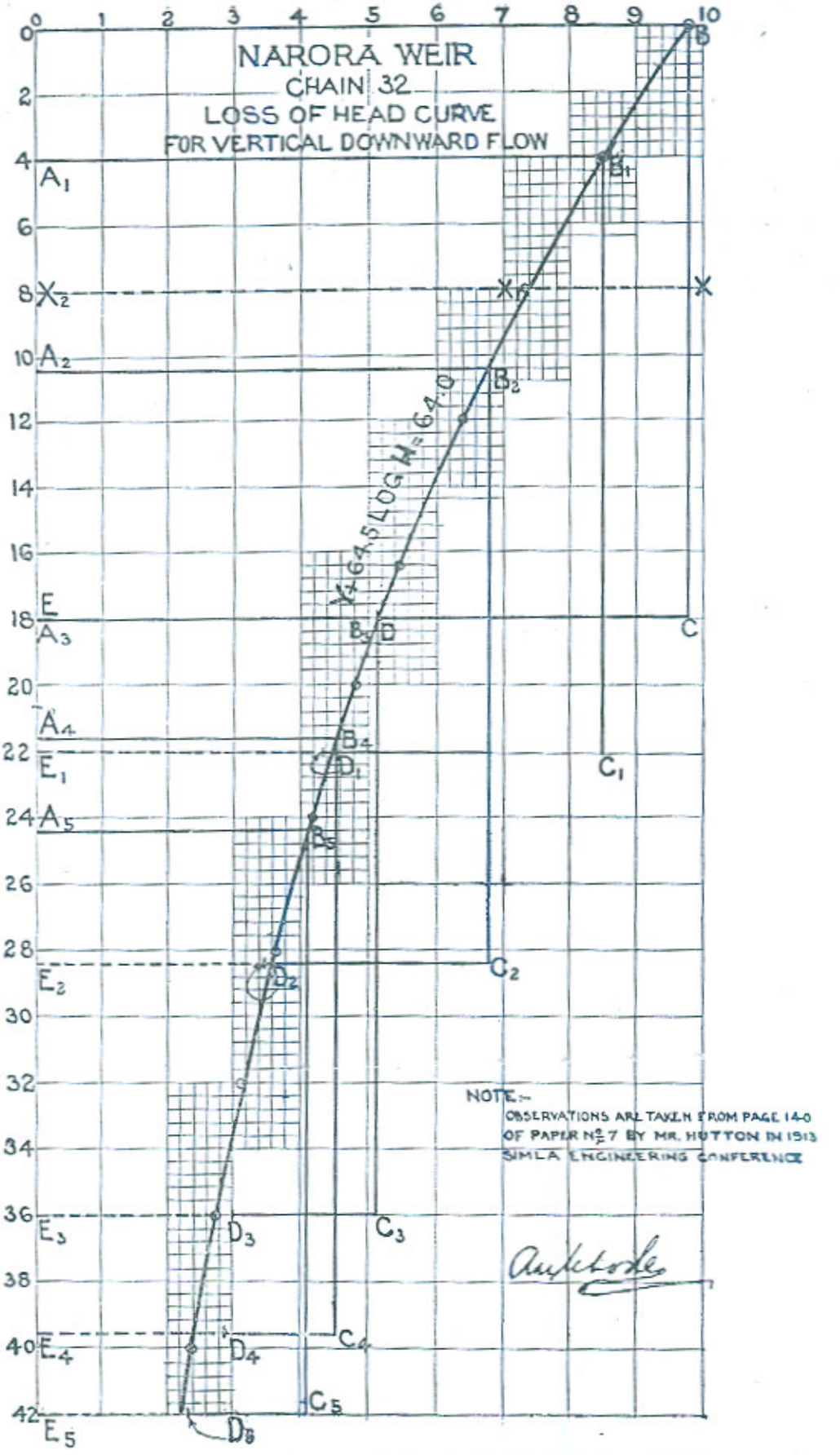
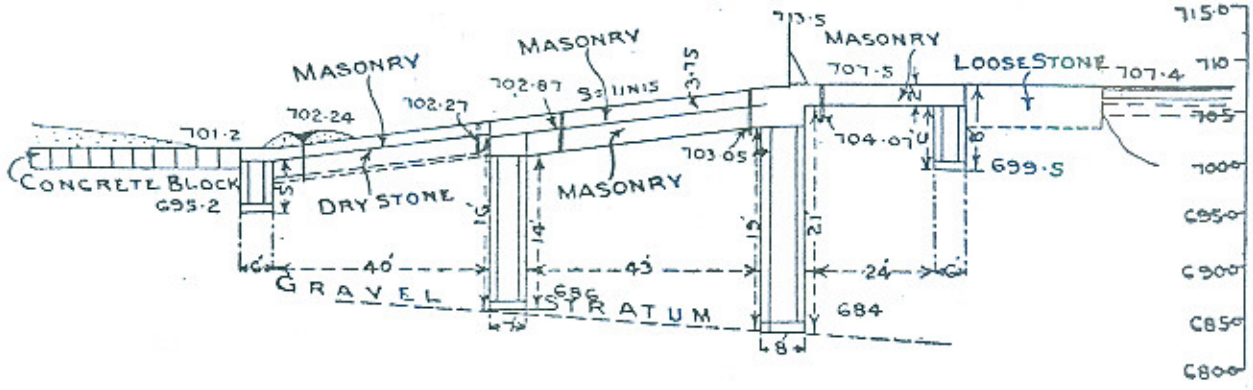


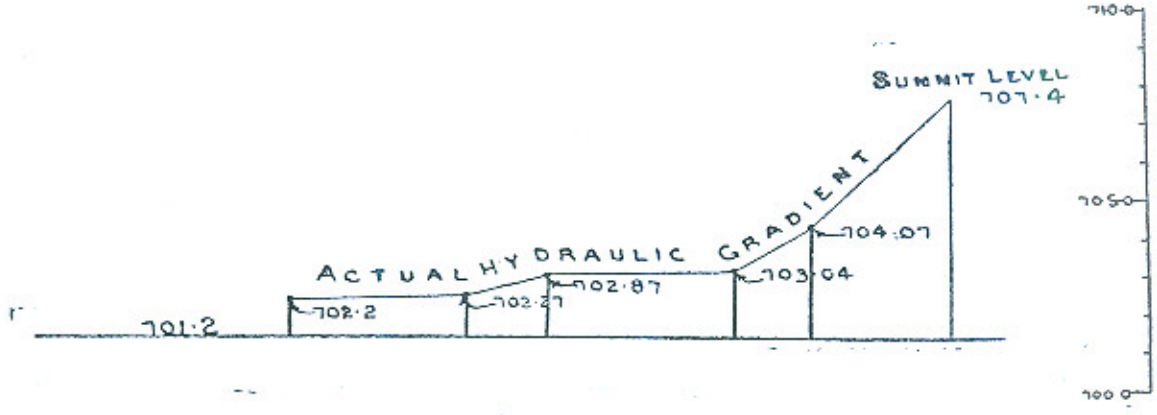
PLATE III


NARORA WEIR
CHAIN 32
LOSS OF HEAD CURVE
FOR VERTICAL DOWNWARD FLOW
0 1 2 3 4 5 6 7 8 9 10
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42
A1
X2
A2
E
A3
A4
E1
A5
E2
E3
E4
E5
B
B1
B2
B3
B4
B5
C
C1
C2
C3
C4
C5
D1
D2
D3
D4
D5
H=64.0
H=64.5 LOG
NOTE:-
OBSERVATIONS ARE TAKEN FROM PAGE 140
OF PAPER No. 7 BY MR. HUTTON IN 1913
SIMLA ENGINEERING CONFERENCE

CROSS SECTION
OF
JHELUM WEIR AT RASUL

PLATE IV.





NOTE:-

THIS IS TAKEN FROM PAGE 197 OF THE PRACTICAL DESIGN OF IRRIGATION WORKS BY Mr W G BLIGH (THIRD EDITION)

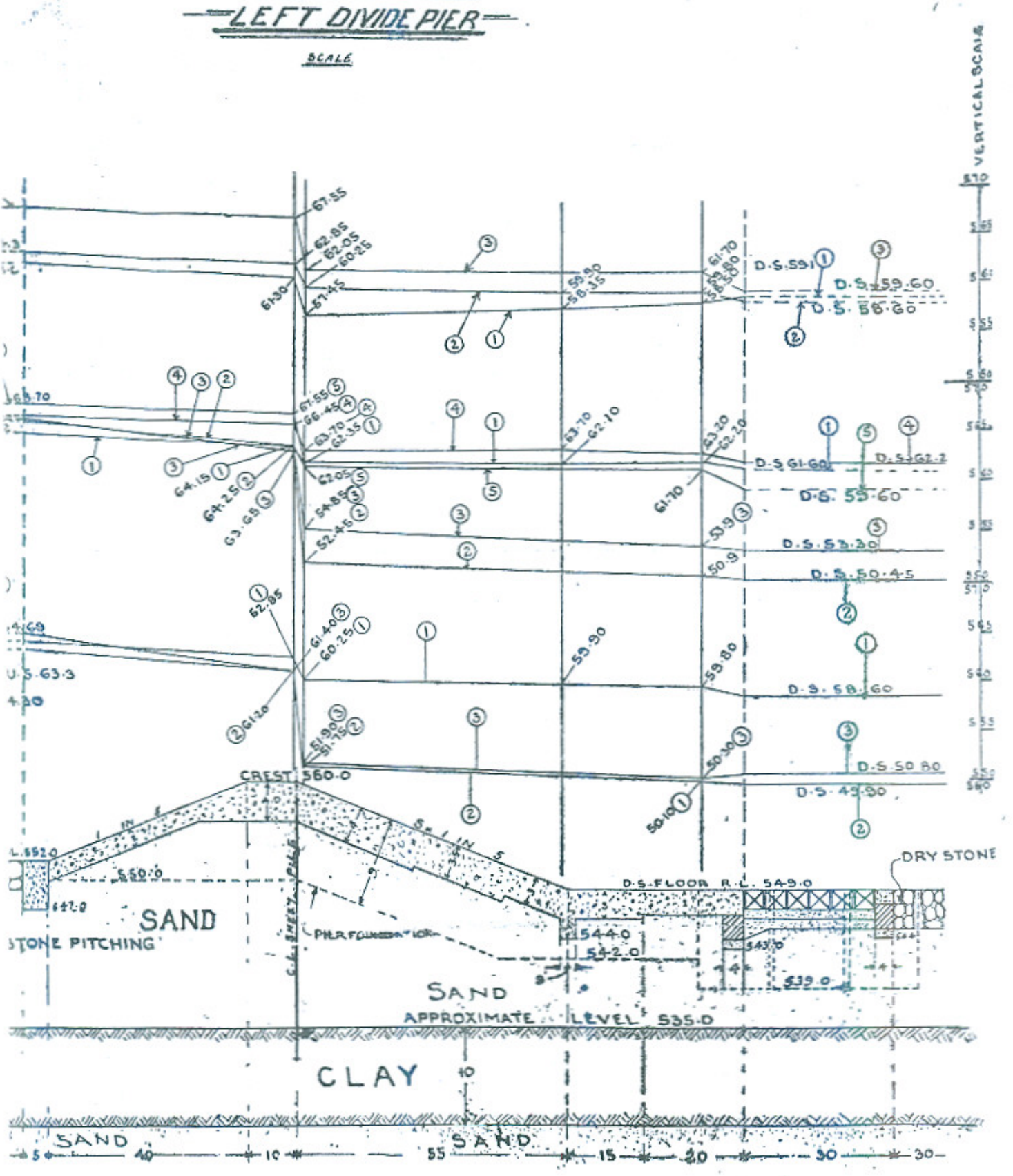
Ambhole

EXECUTIVE ENGINEER
MARALA DIVISION U C. CANAL

PLATE V.

SULEMANKI WEIR
LEFT DIVIDE PIER

SCALE



Ankhosla

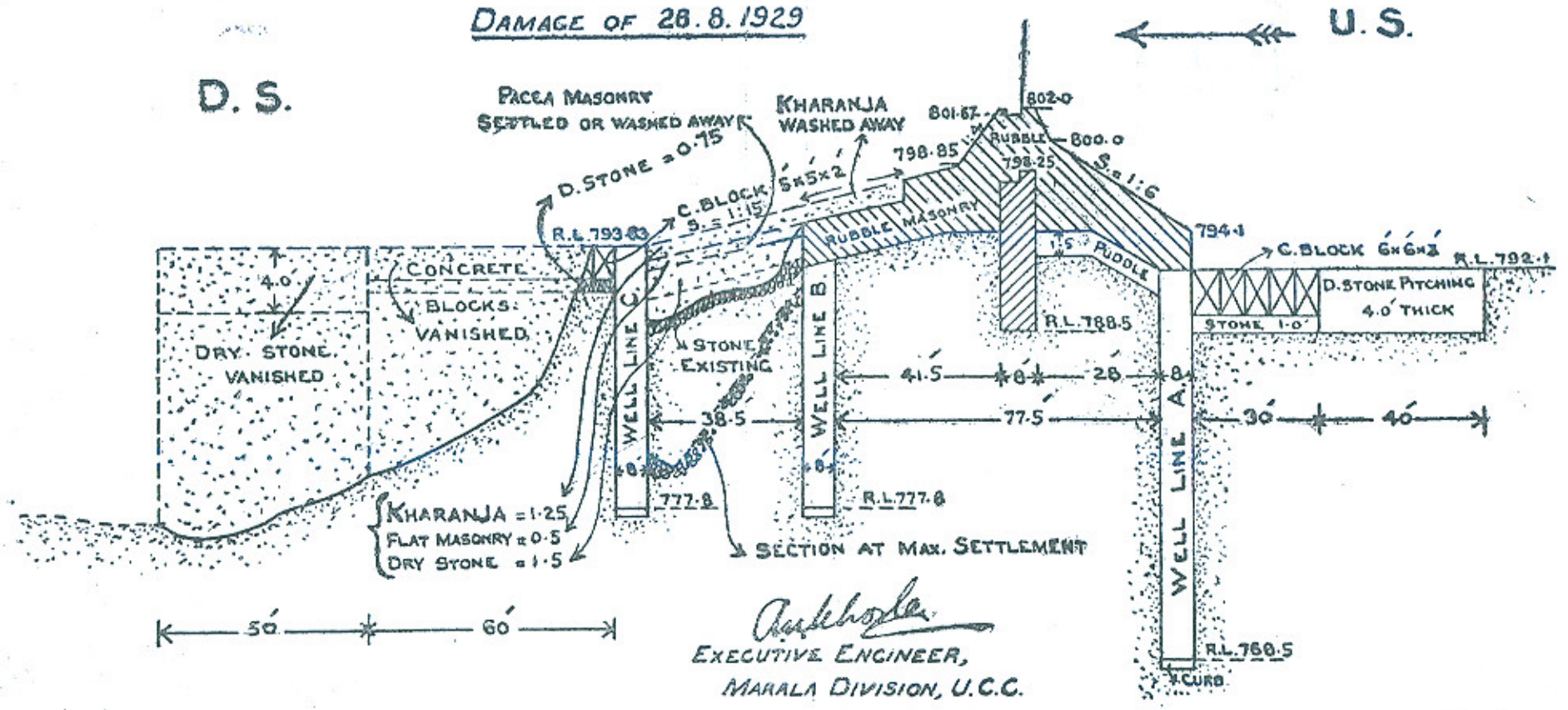
EXECUTIVE ENGINEER
MARALA DIVISION, U.C.C

MARALA WEIR, U.C.C.

PLATE VI

CROSS SECTION THROUGH BAY No. 4

DAMAGE OF 28.8.1929



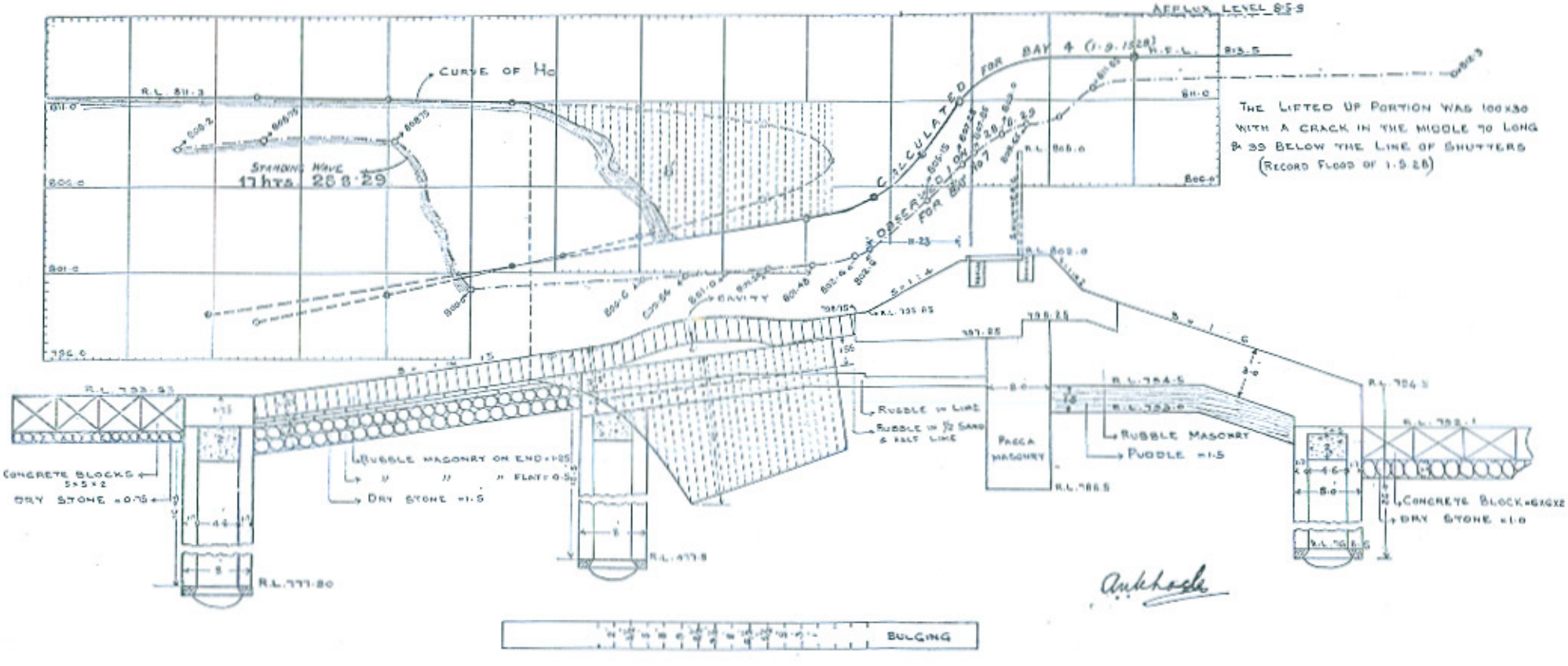
EXECUTIVE ENGINEER,
MARALA DIVISION, U.C.C.

MARALA WEIR

SECTION THROUGH BAY NO. 4

SHOWING STANDING WAVE & FLOOR UP-LIFT

PLATE VII



DISCUSSION.

Mr. Khosla in introducing his paper said that, since writing the paper he had had occasion to study the pressure pipe observations at the Balloki Weir of the L. B. D. Canal and of the Jaba Crossing of the Upper Jhelum Canal. In both the cases the indications were much the same as at the Narora Weir. The data at these two places unfortunately did not show any pressure observations above the crest line so that it was difficult to find the actual loss at the first line of wells, but at the intermediate line of wells the drop in pressure between points just upstream and just downstream was defined, coupled with a flattening of the slope both upstream and downstream as indicated by the theoretical discussion on the subject by Dr. Bose in connection with his first paper.

In 1916 some extensive observations were taken by Mr. J. B. G. Smith, now C.I.E., Chief Engineer, then Executive Engineer, Balloki, on pressure pipes in the Balloki weir floor, as a result of which it was found that the floor as existing at that time was unsafe against uplift. Various alternative proposals were put up to remedy this defect. They were :—

(*a*) Extending the upstream floor.

(*b*) Thickening the downstream floor.

(*c*) Extraction of the piles which closed the gaps between the individual wells of the downstream end toe wall in order to remove the obstruction to flow due to this impervious toe wall, which was considered to pile up the pressure under the floor upstream of it.

(*d*) Dismantling the tops of the downstream line of wells and thus permitting the passage of water between the floor and the top of wells.

(*e*) Providing additional relief pipes.

Very fortunately and wisely all the alternatives were ruled out and it was decided to thicken the downstream floor.

In view of what he had already explained in his paper of that morning the alternative (*d*) above would have been fatal.

He gave in brief a general idea of the necessity of having a deep line of piles or wells just at the upstream end, and the downstream end, the intermediate piles as had already been explained in the first paper had a local effect in so far as that they created a local drop and flattened the gradient. From the uplift point of view the advantage was not so great, but as the gradients were flattened out the velocities were decreased and, therefore, the tendency to piping was correspondingly reduced. In addition these piles localised damage by preventing it from spreading beyond the line of obstruction.

He explained, with reference to the sketch of plate B. how the upstream sheet piling at the Narora Weir cut off all lines above 79·5, and was by itself responsible for a drop of 5 feet. The downstream wells took care of another 5 feet head and the horizontal floor of over 120 feet accounted for only 2 feet. The local drop at the well line under the crest

was also diagramatically explained. The diagram also indicated the loss of head curves at the upstream and downstream pile and well lines respectively. These were reproduced from plate III of this paper and plate VII of paper No. 138.

L. Kanwar Sain said that Mr. Khosla's paper was undoubtedly a great advance on the high road of investigation into the stability of structures resting on saturated soils.

It would be remembered that in 1926 during the discussion on Mr. Foy's paper 'Weirs and Headworks on the Sutlej Valley Project' Mr. Colyer made some very significant remarks on this important question of true hydraulic gradients. Expressing his surprise at finding a lack of precise knowledge on this point at the All-India Irrigation Conference of 1913, he remarked "Even in 1926 our knowledge is very hazy and it will be regrettable if advantage is not taken of the new weirs on the Sutlej Valley Project, and elsewhere, to erect several series of stand pipes in the structures, and to record continuously the actual height of water in them."

Out of four weirs on S. V. P. pressure pipes had been put in only at Suleimanke and at Panjnad. At Suleimanke, however, the sheet piles going into the clay, these pressure pipes were only of local interest and would add nothing to our knowledge on the broad principles of this problem. The Panjnad weir had not yet started functioning. Mr. Khosla had, therefore, perhaps of necessity, to fall back mostly on the Narora Weir data in order to support his observations at the Dugri No. 1 and the Jauryan Syphons, and thus gave them an opportunity of comparing the data on which Bligh based his theory with Mr. Khosla's own more exhaustive observations.

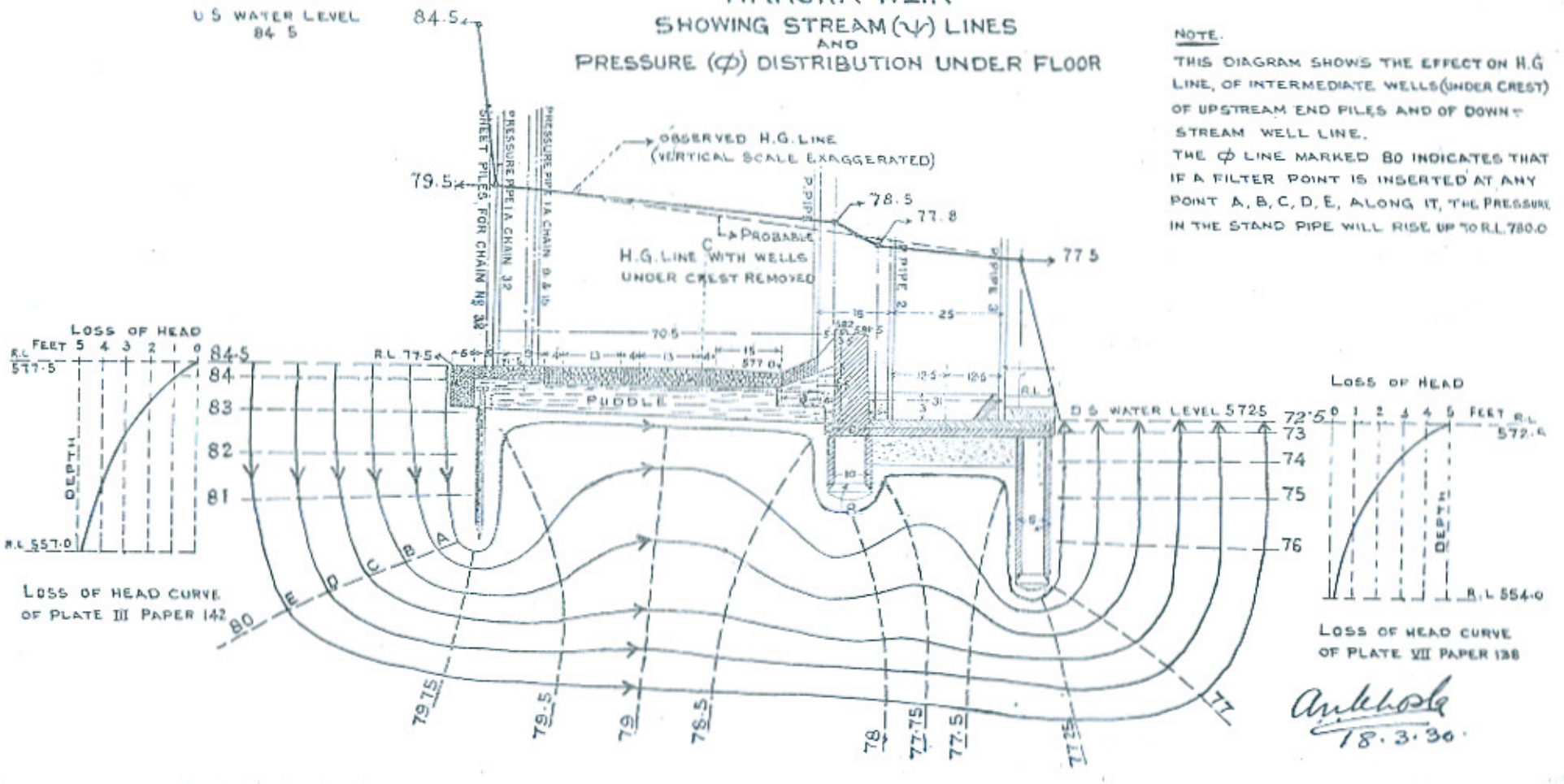
Mr. Khosla admitted frankly in his paper that the Narora Weir was standing in spite of the Bligh line being dangerously below the actuals. One of the reasons he gave for this was that water could not exert full upward pressure on the entire area of the floor which was in contact partly with the soil particles and partly with water, also that the effective area on which the full hydrostatic pressure could act was a function of the porosity of the foundation soil. Thus the average uplift pressure under the floor would be a certain percentage of the indicated pressure. Now it might be noted that Bligh's co-efficient of creep also depended on the fineness or coarseness of sand and the nature of its packing and it might be that Bligh in his farsightedness had allowed for this in his conclusions based on the Narora Weir pressure pipe observations. This also made it doubtful how far the rise of water in pressure pipes could serve as a direct measure of the hydrostatic uplift on the floor; for as long as there is no cavity underneath the floor, the full upward pressure indicated by the pressure pipe, did not come into play and the formation of a cavity showed that the failure had already started by undermining or 'piping.'

Regarding the hydraulic gradient line being straight or otherwise he would draw attention to Mr. W. B. Gordon's remarks in Technical paper No. 97, page 29, "the proposition illustrated in his Fig. 1 (*i.e.*,

NARORA WEIR

SHOWING STREAM (ψ) LINES AND PRESSURE (φ) DISTRIBUTION UNDER FLOOR

PLATE. B

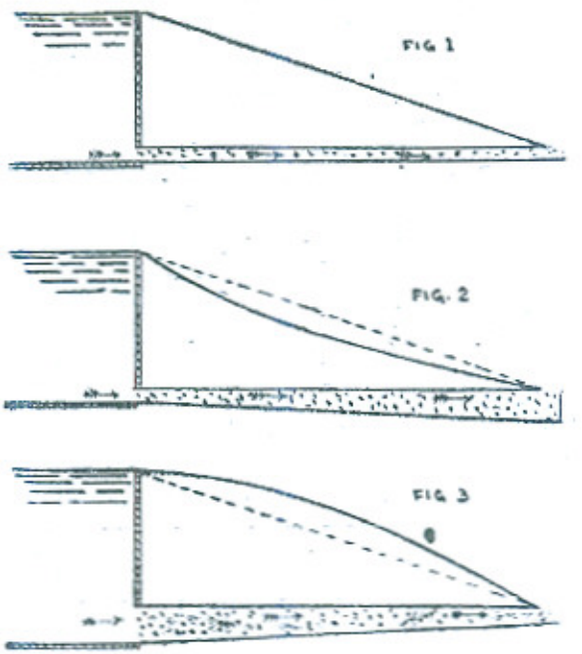


NOTE.

THIS DIAGRAM SHOWS THE EFFECT ON H.G LINE, OF INTERMEDIATE WELLS (UNDER CREST) OF UPSTREAM END PILES AND OF DOWN-STREAM WELL LINE.

THE φ LINE MARKED 80 INDICATES THAT IF A FILTER POINT IS INSERTED AT ANY POINT A, B, C, D, E, ALONG IT, THE PRESSURE IN THE STAND PIPE WILL RISE UP TO R.L. 780.0

that of a straight line) is true when the pipe is of the same diameter throughout and the sand in it is homogenous. But if the pipe increased in size towards the outlet, the hydraulic gradient will be a curve which is convex to floor line (Fig. 2) and that if the pipe decreases in size towards the outlet the hydraulic gradient will be a curve concave to floor line (Fig. 3).



Either condition might exist if there were a sloping impermeable stratum below a weir. The above remarks might be true even when the impermeable pacca floor was sloping for some length as in the case of Sutlej Valley Weir glacis.

The actual hydraulic gradient for Rasul as shown on plate IV of Mr. Khosla's paper was convex to the floor line, while those for Narora were concave to the floor line.

Mr. Khosla's findings as to the velocity of subsoil water flow and its effect on the undermining of founds were most valuable and revolutionised the entire ideas on the subject of creep and failure by piping.

A likely impression from Mr. Khosla's paper was that sufficiently deep curtain walls or sheet piles should be provided upstream and downstream of impervious floors. A central line of sheet piles was also of some use, while the upstream floor did not need to be long and the length of the downstream floor might be determined only from dynamic considerations to dissipate the energy of falling water and keep the standing wave on the pacca floor, the thickness of the downstream floor of course being made sufficient to withstand upward pressure. That piping or failure by undermining would be entirely taken care of by a proper depth of downstream curtain wa l or a line of sheet piles of sufficient length.

Would Mr. Khosla please modify this impression or confirm it ?

The top course of masonry lifting up from the rest of the floor both at Rasul and Marala showed cement concrete to be a preferable form of construction particularly if the entire thickness was put in the same day as was being done at Panjnad. This procedure did not give any great difficulty in actual working provided a working plan had been clearly thought out beforehand.

On page 206, Mr. Khosla said the reinforcement of the bottom slab was calculated both for uplift and for *settlement due to cavitation underneath.* Might he enquire what was the size of the cavity assumed in the calculations ? He doubted, if such reinforcement was necessary. He was of opinion that R. C. box construction would be cheaper and much sounder than a gravity section in the normal floor of a Weir.

Mr. Mithal thanked the author for mentioning him in the paper. He desired to associate himself with the deductions and theory brought out by the author.

Mr. G. Lacey said that the paper was a valuable contribution to the theory of subsoil flow and would do much to place it on a rational basis. So much had been said on the subject of Bligh's theory of hydraulic gradients, that it was without foundation on facts and much of the same character, that he the speaker, as one responsible for giving instruction to students on this subject, thought that a little might be said as to the position that Bligh's theory held in the engineering world before the publication of Mr. Khosla's paper. It would be improper to state that the general theory of hydraulic gradients as laid down by Bligh in his text-book had " gone by the board " and it would be a poor tribute to an engineer who had presented the profession with a useful working hypothesis of design ; it would however be perfectly correct to state that Bligh's own special theory of " creep " had long been suspect. Bligh's theory, briefly stated, was that provided the distance between two line of piles was not less than twice the depth to which the piles were sunk below the underside of the floor, twice the length of such piles might be taken into account when calculating the length of the path of the subsoil water. The implication in this theory was that the loss of head as between piles and sand, or between masonry and sand, was only half that between sand and sand. For this there was little experimental justification on a large scale. The theory was that water would always creep meticulously round the outline of a masonry work where the path was assumed to be of less resistance, rather than strike boldly across a gap of sand pure and simple, for this reason it was sometimes known as the " musk rat theory " as that animal always skirted the boundaries of a room, however complicated, rather than strike across it.

Since Bligh assumed half the resistance, when the sand was in contact with masonry, it was quite clear that if a sandy embankment filled with water had a real hydraulic gradient of 1 in 7.5 , the same material if underlying a weir must be given a value of ' e ' of 1 in 15. This point was seldom brought out sufficiently.

Mr. Colyer remarked that the statement made on page 197 that 'Full uplift does not have effect, unless a cavity exists' (*i. e.*, until 'piping' occurs) was probably correct, and agreed with certain simple experiments made by the speaker, and added that quantitative investigation of the extent to which upward hydrostatic pressure is actually active against a slab laid on saturated sand below the level of the 'free-water-surface,' was desirable.

He asked the author to explain the best way to ascertain the 'True free-water-surface' mentioned on page 211; and to say, with regard to the operations described on page 207:—

(*i*) What proportions of soda (presumably ordinary 'washing soda') to cement should be used.

(*ii*) To what extent the addition of soda reduces tbe strength of the cement.

(*iii*) Where he fixes the relief-pipes, and

(*iv*) What height of relief pipe is suitable for a weir glacis.

The speaker then proposed that time had probably come to abandon the principle of designing weirs founded on sand, with glacis rendered partly porous by a layer of 'dry' ballast or stone, or by providing relief pipes either of the 'inverted filter' type or of the Tej type, and instead to design completely impervious structures capable of resisting the maximum uplift which experiment found likely to occur, and to protect it against restrogression by a line of sheet-piles or wells, and a sufficient 'dry' apron, or talus, preferably of heavy blocks.

Mr. Montagu said that two papers of the weight and importance of Mr. Khosla's could not be discussed adequately in the time allotted.

It would be essential to confine his (speaker's) remarks to certain essentials and to refrain from stepping down any of the by-paths, Mr. Khosla so invitingly held open.

1. The first point made by Mr. Khosla had no direct bearing on the theory set forward in paper No. 138, but on page 105 Mr. Khosla inferred that the *depth* of the protection upstream of a Weir was not always adequately examined. The point was underlined for consideration of the Congress. This depth would be simply found by means of the "free fall" formula.

2. On page 107 Mr. Khosla had indicated briefly the reasons for some of the Punjab Weirs still standing while considerations of accepted theory indicated that they were *unsafe*.

The apeaker had always felt that the full pressure available from the heading up above the Weir was never applied to the downstream floor and had held that this was due to "dissipation" into the subsoil water table. This view was found to be consistent with Mr. Khosla's more exact theory.

Mr. Khosla proved that the height of the L. S. W. L. was the determining factor when the actual F. S. L. was lower than that of the L. S. W. L. and the speaker submitted as a logical conclusion from the same

chain of reasoning that the result held though the relation of the levels was reversed.

3. The speaker failed to understand Mr. Khosla's use of the word "bound" on page 107, and also had grave doubts as to the accuracy of his suggestion that the full hydrostatic pressure was not brought to bear on the floor, for the reason given.

The only way in which Mr. Khosla's explanation was tenable lay n imagining the floor perched on a large number of earth pillars of relatively small diameter but large total area. Moreover Mr. Montagu suggested that the American theory to which Mr. Khosla had referred, was applicable to the case of pressures behind vertical retaining walls and not horizontal floors.

4. Mr. Khosla made a statement on bottom of page 197 and top of page 198 which appears to be in the nature of a general statement of a law, whereas it was in reality a statement of an unusual effect at one place only. If Mr. Khosla's general theory were true (and the speaker was convinced of this) then the response of the L. S. W. L. to canal F. S. L. was purely local and of no importance in relation to design. Moreover if the drains to which Mr. Khosla refers, were maintained in proper order, the Water Level in the drain would be but little above bed, and the response to the variation in Canal F. S. L. would be negligible.

Clearly it was in such cases that the maximum danger was to be feared.

5. A reference was invited to the curve on Plate III and the similar curve of Paper 138. The latter was for a fixed difference of levels and the former for a series of varying differences. The statement that the velocity varies with the difference in levels, if not actually wrong, was only a rough approximation. Consequently the statement that the variation in the second case is directly with the differences, was subject to the same criticism. The speaker emphasized that the deduction was wrong and not the curve.

6. The author of the paper had been at some pains to show that Bligh's theory was unsound and on page 201 had compared the effect of a pile line according to Bligh with actuals. The speaker suggested that a better comparison was as follows:—

Bligh 18 feet pile=18 × 2=36 feet length $36\times\frac{1}{15}=2{\cdot}4'$ loss of head.

This loss is constant.

In practice it was found that with high heads upstream, (9·8 feet) as much as 4·65′ head was lost over the pile line whereas with low heads upstream (4·1 feet) the effect of the pile line was only 1·9 feet.

It followed that *deep pile lines* and wells were generally *more effective than Bligh thought at high heads and less so at low.*

7. The speaker desired information as to the method of obtaining the profile of the standing wave on plate for Marala Weir. It was notable hat the method used gives a wave far too high up the glacis.

The outstanding lesson of Mr. Khosla's analysis of this failure was the necessity of impervious bottom layers of masonry on the downstream glacis.

8. In connection with Mr. Khosla's remarks on Suleimanki Weir, the speaker agreed as to the advantages of a sheet steel pile line at the downstream edge of the glacis but this line of piles must on no account be driven into the clay layer.

If the pressure applied by the upstream water level, and found to exist under the glacis was communicated through the interstices of the existing pile line under the crest, then to drive the new pile line down to clay was courting sure disaster.

9. In connection with hydraulic laboratory experiments, the speaker's recent visits to Poona and Karachi had reminded him in no uncertain manner of the difficulties involved in model experiments.

i. The scale factor affects velocity and rugosity in a proportion different from that of the scale.

ii. There is no silt movement below 0·88 f. p.s.

iii. Any natural swirl assumes its full size: in a model whose vertical and horizontal scales are not the same, this destroys the relation at once.

The speaker therefore supported a plea for research on a full or natural scale, into the problems set forth by the author.

10. The speaker held that the author's conclusions were warranted on the whole but urged caution in regard to—

i. wholesale condemnation of Bligh whose *results* if not his methods were sufficiently accurate for working rules over a long period of years;

ii. relief strainers: the speaker repeated and emphasized his reasons for distrust of this device.

11. Mr. Montagu concluded with a plea for a reasoned nomenclature and notation.

Mr. A. A. Musto, referring to page 197 (1), questioned whether the fact that sand was in contact with part of the underside of the masonry affected the upward pressure. Such grains of sand were themselves under equal upward pressure and transmitted this to the masonry, so that full upward pressure was borne over the whole area of the floor.

With reference to para. (2) on that page he said that Bligh himself gave a margin of 33 per cent. for safety. It was hardly fair to blame Bligh for errors in his theory, only discovered as the result of applying this theory to designs. Previously there had been no theory and only rule of thumb design. Bligh as a pioneer had done a great service to design however many mistakes there might be in his original tentative theories. The unbalanced head (non-apparent) referred to by Mr. Khosla ought to be very small if piping had not started and if the correct coefficient of creep had been taken for design. It seemed to him that the inconsistencies and dangers observed in existing old works were nearly

always much more probably due to *faulty work*, e.g., non-watertight lines of wells, than to mistakes in design or theory.

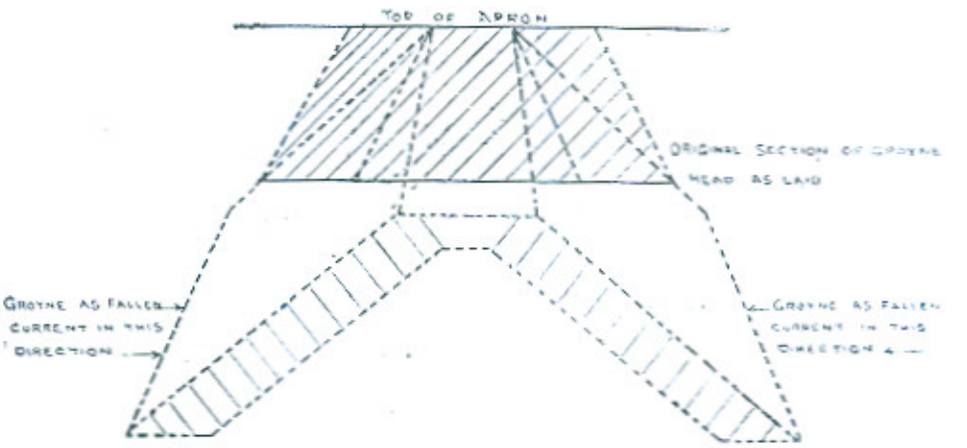
He doubted if all these works were protected from creep of subsoil water at their ends *i.e.*, *along the river banks* as well as in the direction of river flow; if not, they would fail from this direction. When designing, the *whole plans* of the work must be studied for protection and not merely a cross section in the direction of the river.

No complete *plans* were shown in the papers. Surely if a scour hole *beyond* the end of the pacca floor (toe wall) caused piping it could only be because the co-efficient of creep had been taken too low in the design. Of course if scour were allowed to come right back to the toe wall so that there was an easier passage for water than straight up to the top of the toe wall, then this reduced the length of the creep and obviously piping would start. The remedy was to prevent scour approaching *near* the toe wall by providing an ample falling apron which would keep scour held up *at a distance*.

As *cross* currents *i.e.*, currents parallel with the edge of the falling apron caused much more scour and disruption of the pitching (especially in practice on the upstream side and along the guide bank aprons both upstream and downstream) he suggested, throwing out groynes of pitching stone beyond the edges of such aprons thus:—



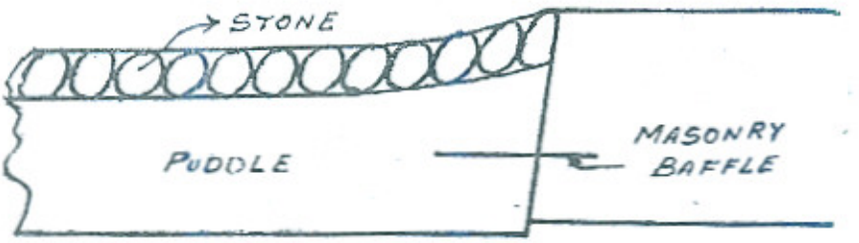
These groynes would fall sideways (with cross currents passing over them) and they should be much thicker at upstream or outer ends than at the edge of apron. They would then fall in this form:—



The spacing, contents and form of groyne would need care consideration. The above was only a rough suggestion.

Referring to the *Narora Weir* the speaker said that an upstream puddle apron was in his opinion unreliable, as it was practically impossible to make a watertight junction between puddle and a *vertical* surface of

either masonry or steel piling. The slightest settlement or contraction of the puddle would break what slight seal had been obtained by puddling against the wall. The only way to obtain a seal would be on to horizontal baffles built into the wall and surrounded by puddle. Even with this, uneven settlement might cause shear through the puddle and give a leak.



It was also doubtful if either of the lines of wells under the floor are really watertight. This was probably the main source of weakness in the floor.

Referring to *Rasul Weir* the speaker asked what was a Kharanja. When local terms were used they should be explained.

It seemed bad policy to lay masonry on dry stone filling which was almost certain to settle or compact itself and thus give irregular support to the masonry and even if the masonry did not crack and settle would leave pipes for flow of water below it. Would it not be better to lay the masonry direct on to the compacted sand?

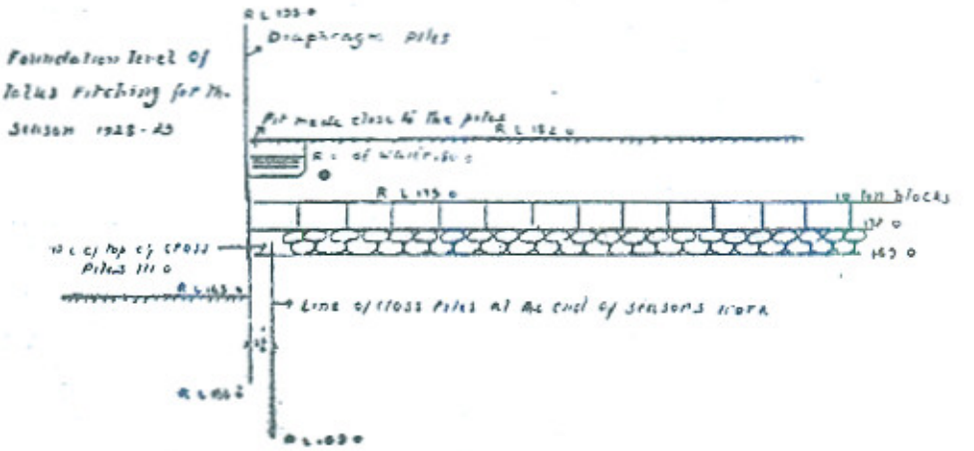
Referring to the *Marala Weir* he asked how the "hollow underneath the concrete blocks laid to replace the lifted Kharanja layer" was observed. Were not these raised cills a source of danger? Would not a lower cill (plus rising gates) get over much of the standing wave difficulty?

To prevent *uplift* of blocks it was obviously necessary to have their depth such a function of their surface area (and hence of uplift pressure that they could not be lifted.) With 9 feet of uplift head, the depth block should be not less than $9 \times \frac{62{\cdot}4}{{\cdot}125} = 4\frac{1}{2}$ ft.

Referring to *Suleimanke Weir* he said that the greater effect of vertical lines of obstruction *i.e.*, piling as compared with horizontal obstruction was due to the fact that any sand displaced against sides of piling was immediately replaced by further sand falling from above whereas sand removed from under the horizontal masonry would only be replaced by further sand carried by the stream of water and left deposited there. If piping once started it was unlikely that sand would ever be re-deposited in these pipes.

He attached 3 sketches with particulars of 4 tests of actual creep factor at Sukkur which might be of interest to members. These were recorded by Mr. M. P. Mathrani, Executive Engineer, Left Work Division, and verified by the speaker.

EXPERIMENTS OBSERVED BY MR. M. P. MATHRANI, EXECUTIVE ENGINEER, LEFT WORKS DIVISION, AND VERIFIED BY MR. A. A. MUSTO, SUPERINTENDING ENGINEER, LLOYD BARRAGE CIRCLE, TO FIND OUT THE CO-EFFICIENT OF CREEP BETWEEN SAND (NATURAL RIVER SAND) AND STEEL SHEET PILES AT SUKKUR.



Sketch showing the difference of water level on the two sides of diaphragm piles at the end of season's work (1927-1928).

$$\text{Co-efficient of creep} \quad \frac{(\text{R. L. } 180-149)+(165-149}{15}$$

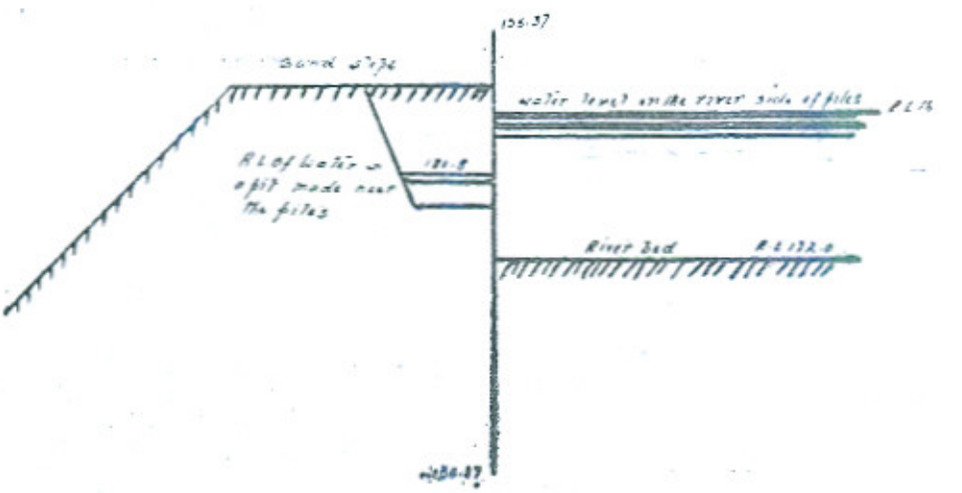
$$= \frac{31+16}{15}$$

$$= \frac{47}{15}$$

$$= 3{\cdot}2$$

Possibly these piles passed through a layer of clay.

(2) Sketch showing the water-level on two sides of Cofferdam piling (season 1928-29), 27th May, 1929.

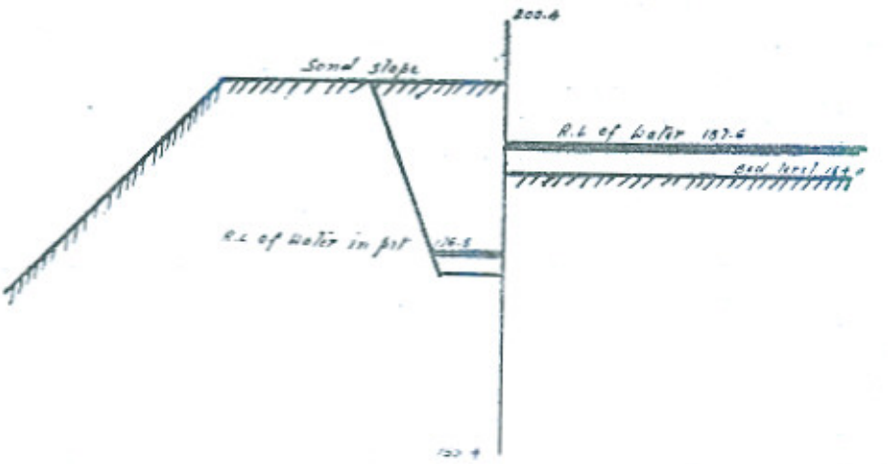


Length of creep on river side of piles = 172·0—150·37
= 21·6

Length of creep on cofferdam side of piles = 180·8—150·37
= 30·4

Co-efficient of creep.. $\frac{52·0}{6·5} = 8.$

(3). Sketch showing water levels on two sides of cofferdam piles near flume No. 5 (season 1928-1929) 30th May, 1929.



$$\text{Co-efficient of creep} = \frac{(184·0-155·4)+(176·8-155·4)}{10·8}$$

$$= \frac{28·6+21·4}{10·8}$$

$$= \frac{50}{10·8}$$

= About 5.

(4). It is found that a sand bund can stand a hydraulic gradient or 1 in 3 (with a total head of 18′ of water) without breaching. But under these conditions sand is always on the move.

If the section is wide enough to give a hydraulic gradient of 1 in 4, there is practically no movement of sand, but water simply oozes gently at the toe. This has been tested on various occasions (up to 18′ head).

Mr. Khosla, in reply to Mr. Kanwar Sen, appreciated Mr. Colyer's remarks suggesting extensive use of stand pipes for finding out the true laws of hydraulic flow as affecting the weirs. The question of upward water pressure acting on the whole or a part of the area was taken up by practically all the speakers.

Little experimental data existed on the subject but it was apparent that water in the capillary state could not exert the full indicated pressure; some tests were being carried out at Ferozepore. The results of these tests would be valuable when made available, but as far they went they helped to support the theory put forward by the author. Mr. Burkitt ascribes this phenomenon to the surface tension in the

microscopically small filaments of water between the grains of sand, which tension must be enormous and must act exactly like friction. A confirmation of this view could be had from the floor of the Passia siphon of the Upper Chenab Canal where the weight of the floor slab plus that of the middle and side walls was only 65 to 75 per cent. of the total indicated pressure and yet that light floor was standing intact. Careful experimental work was required to determine the exact percentage of indicated pressure which should be allowed for in designs. In the text, the phrase " water in bound state " had been used. It was the same as water in the capillary state. Directly, water under pressure was introduced into a boxed subsoil, there would be tendency for the soil to swell. This tendency would be counterbalanced by the super-imposed load of the structure which would keep the soil " bound." An increase of pressure beyond the limit of the super-imposed load would result in partial uplift of the structure followed by swelling up of the soil with an increase in the pore space. The water would then be passing the capillary or bound state, and would start exerting increasing upward pressure on the underside of the structure till in the limit this would get in contact over the whole area with a continuous sheet of water and the full indicated pressure would be exerted. Other points in this respect had already been explained in the text.

Mr. Gordon's remarks applied to definite area of pipes, a condition which did not occur in actual practice, where the depth of subsoil was considerable. The steep gradients, therefore, indicated large velocities.

Mr. Kanwar Sen had rightly summed up the conclusions which the author had in view in writing the paper. He, the writer, agreed that a monolithic form of concrete construction as was being done at Panjnad was the best possible safeguard against uplifts of the type that occurred at Marala and Rasul.

The bottom reinforcement of the floor slab was calculated on the assumption that the maximum cavity would not exceed half the effective span of the slab.

Regarding the difference between a reinforced concrete and a gravity section, there were both advantages and disadvantages. The gravity section was more elastic and therefore local damage could be spotted and set right much easier, but the reinforced concrete floor gave no indication of trouble until large cavities had occurred, which resulted in the first and final collapse.

Mr. Lacey introduced an important point that according to Mr. Bligh the resistance along sheet piles or masonry face was half of that which existed in sand. His remarks in the last paragraph were instructive.

Mr. Colyer suggested tests in the laboratory with which the author was in whole-hearted agreement.

Regarding soda cement, the proportions generally used were 1 in 1, to 1 in 4: the more the soda the less the strength.

The relief pipes were generally fixed 10 to 15 feet apart, or as the local conditions suggested.

The true free water surface is determined by inserting a filter point in the open bed of drain or river till such depth that the indicated water level in the pressure pipe does not rise any further. This driving should be done gradually to get good results.

The author was in general agreement with Mr. Colyer regarding the element of risk involved in having the tail end of the pacca floor resting over a pervious layer of stone and shingle. Mr. Musto had drawn attention to this very point. The best practice would apparently be to rest the pacca floor on compacted sand and give a length of thick inverted filter covered over with suitable thickness of blocks, just below this pacca floor.

Mr. Montagu's suggestion regarding logarithmic plotting was accepted with thanks, as it would help matters a great deal, and probably bring forth fresh ideas.

Mr. Montagu had rightly drawn attention to the fact that a line of sheet piling or wells became more effective as the head increased—a fact which Bligh had not allowed for. The loss of head—as mathematically established by Dr. Bose for intermediate sheet piling—varied with the velocity of under flow, *viz.*, $\delta p = v \times 2c$. Bligh had considered the length 2c but had not suspected the variation with v—the velocity. This explained why sheet piling became more effective at high heads.

The calculation of standing wave was based on Mr. Burkitt's method. He fully supported the suggestion of carrying out model experiments and experiments on full sized structures.

He also agreed with Mr. Montagu regarding the doubtful utility if not the utter uselessness, of relief strainers and regarding the change in nomenclature of some of the terms used in the paper.

In reply to Mr. Musto, he said that the question of uplift pressure had already been dealt with in reply to other speakers' criticisms. Mr. Musto's contention that the sand particles in contact with the underside of the floor slab were themselves under equal upward pressure, was correct, but these particles had their own weight acting downwards and as such could not transmit the full upward pressure to the floor slab. The author associated himself in appreciating the basis laid for the engineering profession by Mr. Bligh. This paper was in no way intended to belittle Mr. Bligh's efforts. Knowledge was a process of evolution and as such the new advance on old ideas was quite natural.

Regarding pressure from the sides, this question had already been considered in some of the recent designs and sheet piling was consequently provided upstream, downstream and on the flanks.

He also agreed with Mr. Musto that puddle never made a satisfactory joint with a pacca vertical face.

Kharanja meant stone on edge. The settlement of the floor below the blocks (or slabs) laid in 1929 occurred above winter water level and could therefore be distinctly seen in the dry.

The author finished by thanking the various speakers for the interest they had shown in the paper as evidenced by the discussions.

The President, in winding up discussions on Mr. Khosla's paper, thanked him on behalf of himself and all the Congress Members for the very interesting paper, which will greatly stimulate further investigation into the subject. He also thanked Mr. Musto for his candid criticism and assured him that the papers in future will see all the improvements so kindly suggested by him.