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PRESSURE PIPE OBSERVATIONS AT PANJNAD WEIR.

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Introduction

This paper is a sequel to the two papers Nos. 138 and 142, presented by the Author at the 1930 Session of this Congress, in which the necessity for further observations and Laboratory Research was stressed (p. 208) and the lines on which such work was to be conducted were suggested.

As a result of the recommendation of the Islam Inquiry Committee of September, 1929, it had been decided to extend the Panjnad Weir by adding 14 new bays to the existing 33. The section of the weir in the "Extension" was designed more or less in accordance with the conclusions of the said papers and was different from the existing in some material respects. As the construction of this work was in progress during the year 1930-31, it afforded an excellent opportunity for full scale experiments. Mr. A. M. R. Montagu, the then Under-Secretary, Construction, took advantage of this opportunity and in consultation with the Author put up proposals for installing a series of pressure pipes suitably located in the weir under construction, and some in the part of the weir (bays 1 to 33) already constructed. The Author went down to Panjnad in November, 1930, to confer with Mr. James L. Roy, the Executive Engineer in charge of the Headworks Construction, as to the extent to which effect could be given to the proposals mentioned above without interfering with the progress of concreting and masonry work on the weir. As a result of this conference (28th November, 1930), it was decided to put in a total of 90 pipes in bays 43, 44, 45, 46 and behind the two flank walls of the "Extension." As for the old weir, it was decided not to interfere with the floor by inserting new pressure pipes. This latter decision was extremely unfortunate as it has deprived the profession, at least for the time being, of the detailed comparative data on the two different designs of section resting on the same type of subsoil—data so essential to a proper and comprehensive study of the problem of subsoil flow. This deficiency, however, it is hoped, will soon be made up and a series of pipes inserted in one or two of the bays of the old weir.

The Panjnad Weir is the only one of its kind in the world which has a set of pressure pipes located and fitted with scientific comprehensiveness; and, it is hoped, that the data it has supplied will stimulate research

and will lead to the acceptance of a general policy whereby all large hydraulic works of the future must be fitted with a series of such pipes under the advice of the Central Board of Irrigation, to ensure safety of that particular work and to produce further data for the advancement of our knowledge of Subsoil Hydraulics.

The Panjnad Weir

Figure 1 shows an aerial view of the Panjnad Headworks. It consists of a weir of 47 bays of 60 ft. clear span of which 33 bays were constructed between December, 1927, and September, 1929, and the annexe or extension bays on the right were constructed in 1930-31. The two sets of bays are divided by an earth bank, with armoured noses upstream and downsteam, known as the Junction Groyne. The first 4 bays on the left along with the 800 ft. long divide wall form the silt (excluding) pocket which feeds more or less clear water into the two off taking canals, the Panjnad and the Abbasia, with full supply discharges of 7,779 and 901 cusecs, respectively. The water is guided over the weir by means of two upstream guide banks 3,612 ft. long and two downstream guide banks 600 ft. long, all being protected on the slope and apron by suitable thickness of stone pitching. The two right and left marginal bunds link on to these guide banks.

The section of the old weir (*see* Plate III) consisted originally of a length of 60·5 ft. of pervious floor on the upstream, 110 ft. of similar pervious floor on the downstream, with an impervious length of 204 ft. in between, ending in a 5 ft. deep toe wall upstream and a 6 ft. one downstream and with a line of interlocked 30 ft. long steel sheet piles under the crest. In accordance with the recommendations of the Islam Inquiry Committee, 26 ft. length of blocks just below the downstream end toe wall was converted into a semi-impervious floor and a line of steel sheet piles 25·5 ft. in length was inserted just downstream of these blocks. The 32 ft. length of pervious protection of trangars in slope below this line of sheet piles was grouted in to a depth of 12″ and then given a 3″ concrete covering.

The section of the new weir or extension (Plate III) consists of 60·5 ft. length of upstream pervious floor followed in order by, 163 ft. of impervious floor containing one line of 20 ft. steel sheet piles 2 ft. inside the upstream edge, a similar line 50 ft. inside the upstream edge (under the toe of the upstream glacis) and another line of the same size 2·5 ft. inside the downstream edge ; a length of 20·5 ft. of concrete blocks over a 2 ft. deep graded filter ending in a line of 20 ft. deep wells and a pervious trangar and stone protection for another 100 ft. This section has been designed in accordance with recommendations (*iii*) and (*iv*), page 210 of paper No. 142 of 1930.

Location of Pressure Pipes

As will be seen from Plate III, the pressure pipes at pier 44 have been located at different depths along the sheet piles on both sides and at suitable points along the horizontal floor and in the underlying

subsoil, with a view to study the influence on the lines of flow of subsoil water and on the variations in uplift pressure under that floor, of the depth and spacing of sheet pile lines, of toe walls and of the length of impervious floor. Pipes Nos. 31, 24, 21, 7, 9, 10 have been duplicated at piers 43, 45 and 46 and pipes 20, 18, 17 in the middle of bay 46, to see if the results at the various piers and in the centre of bays are consistent. A set of pipes have been installed behind the wing walls (Plate I) to study the pressure distribution in the back fill with a view to determine the exact length the sheet pile lines should project beyond the flank walls in order that the pressure transmitted to the underside of the weir adjoining the flanks should be no greater through the back fill and under the flank walls than what it would be due to the direct flow under the weir.

Erection of Pressure Pipes

In the old weir, 3 pipes A, B and C were fixed in each of the two bays Nos. 25 and 30 during construction of these bays. They consist of $1\frac{1}{2}$" diameter G. I. pipes ending at the bottom in a filter of graded ballast (not metal strainers). The pipes 'A' are led up into the piers and plugged at the top with screwed caps. They are capable of direct reading all through the year. The pipes B and C end flush with the top of the floor with arrangement for extension by screwing on additional length of piping for purposes of observations. These, as they stand, cannot be observed except in winter and except when the gates are shut and even then the screwing of the extension piece has to be very carefully done to prevent any leakage. This latter, however slight, would upset observations.

The new set of pipes in bays 43 to 46 and behind the wing walls were erected between December, 1930, and February, 1931. These pressure pipes are also $1\frac{1}{2}$" diameter of galvanised iron. The strainer at the bottom is the standard 3 ft. long metal one consisting of an inner $1\frac{1}{2}$" G.I. perforated pipe screened round by a 50 mesh brass wire gauze which is in turn protected by a perforated brass sheet. The blind pipe from the strainer is led under the floor, up the piers through suitably located shafts. These terminate about an inch and a half above the top of the piers and are provided with blind plugs screwed on top. On pier 44 where they are collected in clusters, the tops are further protected by a cover plate which can be locked. Each pipe and blind plug has its number stamped on it and the same numbers are carved in the concrete of the pier just where the pipes come out on the surface. The filter or strainer points of pipes Nos. 1, 4, 7, 10, 13, 16, 17, 18, 21, 24, 27, 30, 31 and 34, were put in by excavating to the right level below the bed and then filling up. The points of the remaining pipes with the exception of two in the junction groyne were driven down to the correct level by an ordinary "monkey" falling on a special cap, attached to the pipe by means of a collar, to save the pipe-head from damage. The two junction groyne pipes referred to above were lowered in position through a bore pipe which latter was subsequently withdrawn. If any pipe got damaged in the course of driving it was replaced by a new one. Special precautions

were taken to see that the ends of the pipes were always kept secured with blind plugs to prevent foreign matter like mortar, ballast, etc., falling in and choking them. The filter points which were placed in excavated pits were surrounded with bajri to prevent any possible contact with concrete subsequently placed above. Practically all bends were encased in the concrete of the weir or the concrete cap of the pier. The bends occurring outside the floor were encased in blocks of concrete to ensure against leakage. All pipes were fitted in advance of the masonry. Inside the shafts in the piers, the pipes are spaced one inch apart and these spaces have been carefully filled with concrete to make the shaft and the pier absolutely solid. As stated above, all through these operations the blind plugs were kept screwed on top.

For purposes of observation, the level at which the pressure is recorded in the pipes is the level of the centre of the strainer where the latter is horizontal and the top of the strainer where it is vertical. This is an important point to bear in mind as it is possible that some of the readers may not quite agree with this assumption.

Geological Section of Subsoil

The information on the subject is not complete as the bore sections (Plate V) relate to the old weir (bays 1 to 33) only, but as the general subsoil belongs to the same formation they may be taken as fairly representative. The subsoil under the weir thus consists of strata of fine sand on medium sand and coarse sand respectively from top down. The coarse sand is mixed at places with kankar or clay. The bores at the sites of pressure pipes not being available, it is futile to argue about the influence of variations in the grades of sand on the pressure levels. It will be enough to treat the whole medium as a homogeneous mass of sand.

Method of Observation

Three methods were employed for taking observations of water level in the stand pipes :—

1. By suspending a tape weighted at the end into the water in the pipe and measuring above the wetted length. This is a crude method and very approximate as the water level in the pipe will temporarily rise due to displacement caused by the length of tape immersed.

2. The electric method (Plate VI, Figure 1) in which a float, suspended in a metal cylinder is let down by a twin electric cable with its terminal connected one to the float and the other to the cylinder. As soon as the float touches the water the contact points of the float and the cylinder come together and complete the circuit. The current is passed by a dry battery and as soon as the circuit is complete a small electric lamp is lighted. The lamp will remain lighted so long as the float is in contact with the water or submerged. Directly the float leaves the water surface it gets suspended in the cylinder and the circuit breaks putting

out the light. This device was not perfected, but it was somewhat more complicated than the third device, although the results were as quick and of the same order of accuracy.

3. The bell-sounder method. This is the simplest, the most direct and the most accurate. This consists of a solid brass rod $\frac{7}{8}$" diameter and $3\frac{1}{2}$ inches long ending in an inverted cup or a hollow hemisphere of $1\frac{1}{4}$" in diameter (Plate VI, Figure 2). This is suspended by a steel tape by means of a swivel screwed in at its top end. Directly the cup touches the water surface a definite sound is produced which can not be mistaken or drowned, even in the roar of water shooting past the weir crest. After the first sound is heard the cup is moved up and down and the exact reading where the sound occurs is noted. The readings can be accurate up to $\frac{1}{16}$th. of an inch but $\frac{1}{8}$" may be taken as the normal degree of accuracy. Each observation takes from 1 to 2 minutes and the whole set of 90 pipes in the new weir and 6 in the old take from 2 to 3 hours. The top level of each pipe is accurately known, and by deducting the length from top to water level the R. L. of the water in the stand pipe is obtained.

Recording Observations

The observations are first recorded in the field note-book or register and are then transcribed on the same day into another register kept in the Divisional Office. The record presented as Appendix I is an exact copy of that register. Ordinarily the observations were taken once a week, and sometimes twice or more if some special phenomenon was to be studied. The Panjnad Canal was run for the first time on the 1st April, 1932, and that was also the first ponding up. The first set of observations were taken on 18-4-32 and after 25-5-32 all observations were done by the bell sounder. The canal was closed and the pond fully released on 16th October 1932. This marked the end of the first series of observations.

The pond was again raised partly to feed supplies again into the canal for 2 days and partly to get observations for the range between 5 ft. and 10 ft. for which no observations had been taken in the first series. These were obtained by lowering the pond by about one foot after every 2 or 3 days. The observations were made each day and sometimes twice in the day to ensure getting at least one reliable set of results which could be independent of the time lag due to the change in pond level every 2 or 3 days. Observations were also taken both in the first and the second series at every 15 to 30 minute intervals, while the pond was being rapidly raised or lowered in order to determine the extent of time lag in the various pipes. It has not been possible to scrutinise these data in detail so far.

There was a distinct change in the relative drop of pressure at various points for the same heads between the 1st and 2nd series of

observations. There was, and so far is, no definite and satisfactory explanation of this phenomenon but it may be due to :

1. the pond in the second series being changed too rapidly for the full effect to be transmitted to the various pipes in the 2 or 3 days allowed ;
2. the temperature of water having definitely dropped after the first complete emptying of the pond on the 16th October, 1932.
3. a change in depth or class of silt on the upstream pervious floor.

The pond up to the date of release remained full ever since the 1st of April. On the 16th October, the discharge of river was only 7000 cusecs at a water temperature much below what it was a fortnight back. The temperature of the pond would naturally take some time to drop with this continuous inflow and outflow of 7000 cusecs of water. The presumption is that up to that date (16th October) the water temperature in the pond which influenced the flow under the weir was not far different from that in the previous month or months. But after the pond was completely emptied, the water that was collected to fill in the pond afresh had a distinctly lower temperature, so that if temperature has some influence on the water-levels in the stand pipes, these levels should obviously be different for the same heads in the 1st and 2nd series of results.

Unfortunately, no water temperatures were recorded during the hot weather and not till a new set of thermometers were obtained in November of that year. But, luckily, the maximum and minimum shade temperatures have been observed all along—thanks to Mr. James L. Roy's initiative in this respect. The data available at present are too meagre to give any useful results, but it is hoped to produce some definite results by the time the Congress meets. The phenomenon is being closely studied in the meantime.

In a critical analysis of the data it must be kept in mind that all through the observations there has been on the upstream pervious floor (and continued on the pacca) a silt deposit of 1½ to 2 ft. in bay 25 ; 2½ to 3 ft. in bay 30 ; 5 to 6 ft. in bays 43 and 46 and 6 to 7 ft. in bays 44 and 45. On the downstream there has been no silt on the pervious floor just below the impervious (pacca) portion. The influence of the depth of overlying silt in modifying transmitted pressures is problematic. There has been no opportunity of testing this so far, but later on as the silt is washed down it will be possible to determine this factor.

Analysis of Data

The observations are in progress at the time of writing and will continue to be taken for at least another year. These will certainly add still further to our knowledge and may also modify more or less some items of the present analysis. The entire set of observations have been reproduced in Appendix I, except those which were unreliable for obvious reasons.

The sources of errors may best be discussed before proceeding further with the discussion. They are—

(*a*) mistakes in readings—generally a whole foot;

(*b*) omission to add the length of sounder 4″ to 4½″;

(*c*) temporary fluctuations in the pressure levels in the pipes. In some cases fluctuations have been noted up to 2″ for no apparent reason;

(*d*) fluctuations in the river for regulating supplies in canals. Here the time lag comes in, for although the upstream and downstream gauges may be perfectly steady at the time of observations the full effect of the fluctuations in upstream and downstream levels may not have been transmitted to the various pipes.

It is suspected that the temperature of water in the pond also affects the pressures in the subsoil. This, if true, is not a source of error, but an important determining factor. This is under investigation and it will be premature to theorize on this till the investigation is complete.

Plate VII shows some plots of pressures above the downstream water-level as datum. In Figure I the pressures are plotted along direct (shortest) distances down the upstream sheet pile-line, straight across its bottom to the bottom of the downstream pile-line and up the latter. In Figure II, they are plotted against the "creep" line as commonly understood. The dotted lines in both figures represent plots of data of the second series obtained after the pond was partly released (16th October) and the temperature or silt effect is suspected. Figure III, shows for various heads the normal gradients below the region of influence of the sheet piles. The close fit of points on the various lines indicates that pipes 36, 26 and 12 are representative of the normal subsoil flow under the weir.

Plates IXA to XII show plots of pressure (p) at any pipe against total head (H) for piers 44, 43, 45, 46, bays 46, 30 and 25, the right flank and the junction groyne. It will be noticed that the relation (p/H) for any pipe is always constant. The variations will be discussed later, but they may be due, to either the sources of error referred to above, or the temperature or depth of silt changes.

Plate XIII shows the curves of pressure distribution in vertical flow in the downward direction on the upstream side and in the upward direction on the downstream side.

General Conclusions

From the above and a general perusal of the diagrams the following are obvious :—

1. The ratio of pressure at any point to the total head (p/H) is constant if the temperature and silt deposits on the pervious floors do not vary.

2. The variation of pressure in subsoil flow with regard to distance in the vertical direction downwards at the upper end of the impervious floor and upwards at the downstream end is either logarithmic or hyperbolic.

3. The variation of pressure under the impervious floor is linear in the horizontal direction and very nearly so in the vertical direction along intermediate sheet piles or other cut-offs.

These general observations coupled with other factors, the discussion of which will follow, point to the fact that the flow of water in the subsoil is in stream lines and follows the laws of hydrodynamics. The establishing of this fact, however, is just the first step. The aim of the present investigation is, as it was of the two previous papers, to determine from hydrodynamic considerations the pressure distribution in the subsoil and under the impervious floors under any given set of combinations of sheet piles, toe walls and horizontal impervious floor ; in other words, to reduce our problem of the design of weirs to comprehensive but simple rule of thumb formulæ which must not be at variance with the hydrodynamic conceptions. Not until this is achieved will it be possible to work out really economical yet absolutely safe designs.

Figures 2 and 3 which have been printed with the kind permission of Dr. Rehbock of Karlsruhe—their author—are an excellent visual demonstration of what exactly is meant by stream line flow in subsoil and how these stream lines are deflected and their even distribution disturbed by vertical obstructions at the end or in the general length of the impervious floor. The stream lines have been traced by means of aniline dyes let in at various points at the entrance. Photographs of the advance of these lines were taken every five minutes and these five minutes "wave fronts" were transcribed on the last photograph wherein the subsoil water started coming out at the downstream end. Plate IV shows Stream and Pressure lines.

A confirmation of the above is possible from the observations of pipes 28 and 29 (pier 44) which happen to be very nearly symmetrically placed with reference to the two upstream pile lines. From hydrodynamic conceptions, the ϕ line (hydrodynamic pressure line) at 29 should be vertical—the stream lines being horizontal—so that the pressure difference as indicated in the pipes, should be constant and due solely to the difference in elevation, *i.e.*, due to gravity and independent of the total head (H). From Appendix II it will be noticed that this difference is constant for almost the entire range of observations irrespective of head or temperature changes. This discovery along with the previous mathematics of it is due to Dr. Bose.

From Paper 140 by Dr. N. K. Bose (1930).

$$\Phi = p - g\rho Z.$$

and $\triangle^2\Phi = 0$ is the general equation of subsoil flow. These and other

mathematical equations are being presented in a more comprehensive manner in the new memoir by Dr. Bose on Subsoil Hydraulics (under print).

If p_{28} and Φ_{28} denote the indicated and hydrodynamic pressures respectively, at pipe 28.

and p_{29} and Φ_{29} at pipe 29.

Then $\Phi_{29} = p_{29} - g\ \rho\ Z_{29}$

$\Phi_{28} = p_{28} - g\ \rho\ Z_{28}$

If 28 and 29 are on a line of equal pressure,

then, $\Phi_{29} = \Phi_{28}$

Hence $p_{29} - g\ \rho\ Z_{29} = p_{28} - g\ \rho\ Z_{28}$

or $p_{29} - p_{28} = g\ \rho\ (Z_{28} - Z_{29})$

Now $g\ \rho\ (Z_{28} - Z_{29})$ is constant and if the above assumption is correct $p_{29} - p_{28}$ must also be constant.

From Appendix II $p_{29} - p_{28} = 0{\cdot}05 =$ constant so that, the assumption that pipes 28 and 29 are on the same equipotential line is correct and therefore the assumption of stream line motion is correct.

Now $$g\rho = \frac{p_{29} - p_{28}}{Z_{28} - Z_{29}} = \frac{0{\cdot}05}{(308 - 298)} = {\cdot}005$$

Correcting the indicated pressure 'p' for elevation, the 'Φ' at any point can be worked out from

$$\Phi = p - g\ \rho\ Z = p - 0{\cdot}005Z.$$

where Z is the elevation above a certain datum. The Φ contours can be plotted from these. But the difference in Φ and p is so small that for all practical purposes it will be just as well to trace the "p" contours. The Φ correction (as worked by Dr. Bose) for various pipes is, however, given in Appendix II.

Plate IV shows the 'p' contours for a 16·4 ft. head observed on 16th October.

Laws of Variation of Pressure in Subsoil Flow

It has been stated above (*see* Plates IX to XII) that for a given temperature the ratio $\frac{P}{H}$ is constant where (p) is the pressure at any pipe and (H) is the total head or difference between upstream and down-stream water-levels. This holds for all pipes whether along the horizontal floor or along the vertical piles. Therefore,

If p = pressure at any point

Y = distance of that point from the origin,

the following equations will satisfy the relation,

$P/H = K$ for that point.

1. Hyperbolic $py^n = H$.

when $y = 1$, $p = H$.

$y = \infty$, $p = \frac{H}{\infty} = 0$.

when (y) is fixed, *i.e.*, for any given point,

$$\frac{p}{H} = \frac{1}{y} = \text{constant for that point.}$$

2. Logarithmic.

$$\log\left(\frac{p}{H}\right) = -ky.$$

$$p = He^{-ky}$$

when $y = 0$; $p = H$.

when (y) is fixed for any point, $\left(\frac{p}{H}\right)$ is also fixed.

3. Linear.

$$\frac{p}{H} = cy.$$

Here also when y is fixed, $\left(\frac{p}{H}\right)$ is fixed.

The above three are all solutions of the hydrodynamic equation $\triangle^2 \Phi = 0$.

From Plate XIII, it will be seen that the curves of pressure drop can be either Hyperbolic or Logarithmic. At the upstream end the hyperbolic curves give a perfect fit (Fig. I) but they would be acceptable if there were no silt deposit at the upstream floor, for according to these curves the full pressure (H) is exerted at R. L. 319·5. Although frequent silt soundings have not been taken but it appears from the few observations that there has been 5 to 7 ft. silt there, all along. From a consideration of this the logarithmic curve (Fig. III) may perhaps be more applicable. In this case, however, the points fall on two lines meeting at pipe 35 at R. L. 308. A possible explanation may be that the soil changes, approximately at this level, from fine to coarse sand (*see* Plate V). It may be argued that any one pipe is recording wrong but the uniform trend of curves proves that it is not observational error. The logarithmic lines are, however, nearly parallel though not as good as the hyperbolic, showing that (p/H) is constant for any point whether the hyperbolic or logarithmic curves are taken. At the downstream end, both the logarithmic and the hyperbolic curves appear equally satisfactory. The departure from the straight lines is small and the maximum error in 16·4′ head is 0·18′ which may reasonably be taken as the observational error.

It will thus be seen that the pressure drop does not follow a linear law except along the horizontal floor in the region bounded by sheet piles. Or, in other words, for the flow in a horizontal direction where gravity does not come in, the pressure drop is approximately linear. At the upstream ends where the flow is vertical or in the region where the vertical is changing into the horizontal or *vice versa*, the pressure drop is either Hyperbolic or Logarithmic.

Figure 2, Plate VII, shows the Bligh pressure line for a head of 16·4 ft. It does not require much explanation to show how far it is out. According to Bligh the pressure drop should be at the rate of 1 ft. in every 17 ft. for 16·4 ft. head. Actually the rate is 1 in 80 ft. in the impervious horizontal floor inside the sheet piles and an extremely varying one (hyperbolically or logarithmically) along the outer faces of the upstream and downstream sheet piles, in *both cases decreasing in the direction of flow.*

The Creep theory of Bligh has, no doubt, given a workable basis for our designs, but from the foregoing it goes without saying that it must be altered beyond recognition, if our assumptions have to be in keeping with facts. It is wrong as to direction of flow and as to uplift pressures.

In the discussion on Paper No. 138 (p. 152 b) 1930 Proceedings, Dr. Bose proved mathematically that the drop in pressure from upstream to downstream side of intermediate sheet pile lines was given by

$$\delta p = 2\,c.\,v.$$
$$= 2\,c.\,k.\,i.$$

where c = length of sheet piles.
v = velocity of subsoil flow along horizontal normal stream lines.
i = normal gradient in horizontal flow.
k = constant.

In his memoir on Subsoil Hydraulics (under print) he has also shown that the drop on either side is equal and is given by $\frac{\delta p}{2} = c.\,k.\,i.$

This is approximately correct (Plate IXA). Of the 6·8 % drop at the intermediate 20 ft. piles nearly 3·8 % is lost on, the upstream side and 3·0% on the downstream. As will be shown later on, the 30 ft. piles of the original weir are responsible for a loss of 10·6 % of the total head only or just over one and a half times that for the 20 ft. piles. These compared with (about) 50% of the total head lost at the upstream end of the upstream 20 ft. sheet piles and 20% (about) at the downstream end of the downstream piles, is a very poor achievement, which shows that except as an additional factor of safety as a second line of defence, should a major disaster occur, the intermediate sheet piles are of little use.

Some Peculiarities in Observed Data

The following peculiarities are noteworthy. (Plate VII, Figure 2.)

Reading in pipe 30 is greater than that in pipe 31

,,	,,	24	,,	,,	27
,,	,,	25	,,	,,	24
,,	,,	11	,,	,,	10

These results are just the reverse of what one would normally expect. These eccentricities may be observational errors but as they persist through almost the entire range of readings, they may more likely indicate some definite hydrodynamic facts. It is surmised that in case of 31,30 and 27,24, there may be a heading up of pressure at pipes 30 and 24 due to velocity of approach as in pitot pipes facing the direction of flow, these two pipes being in a region of " dead flow." Pipes No. 31 and 27 may similarly be in " back water". Pressures in Pipes 25 and 11 being greater than 24 and 10 respectively indicate existence of " reverse flow " or " swirls." The two sets are similarly placed on the upstream of the sheet piles, both in the region of approximately linear flow and the agreement as to drop in both cases cannot be merely accidental.

Residual Head

From the plot of downstream pressures (Plate VII) it will be noticed that at the downstream floor at R. L. 315·0, the pressure is not Zero. It is 1·12 for 16·4 and so on and is just 7 % of total head or $\left(\frac{7H}{100}\right)$. This is as it should be, but the point is whether this high head at exit is permissible. For 19·5 ft. head—the maximum the weir is designed for—the residual head at exit will be 1·37 ft. It will be worth while to carry out experiments to determine the safe residual heads with and without inverted filters.

In the case of Panjnad Weir, it will be noticed that at the downstream end the rate of drop of pressure and, therefore, the velocity decreases in the direction of flow, *i.e.*, upwards. In the case of Dugri Siphon (Paper 138—1930 Session) the reverse was the case. The drop in pressure rate and therefrom the velocity increased in the direction of flow. While the former tends towards safety and is reassuring, the latter is the reverse and has given trouble. The conditions at the Dugri Siphon are complicated by side flow and general rise of spring level.

Analysis of Data for the Original Weir (Bays 25 and 30).

As there are only 3 pipes in each of the bays 25 and 30, no detailed examination is possible. The following statement, however, affords a comparison of the behaviour of the two sections of the weir.

See Plates III, VIII and XII.

Original Weir.		Extension equivalent.			
Pipe No.	Loss % of (H)	Pipe No.	Loss % of (H).	Difference %	Remarks
A	54·4	..	..	..	
Bottom of pile	*59·7	23	58·4	1·3	*half of (A+B) *vide* equation $\frac{\delta p}{2}$=k.c.i.
B	65·0	18	62·2	2·8	
C	74·4	13	70·3	4·1	
End of pacca floor	78·0	10	75·0	3·0	
Residue	78·0	7	90·6	12·6	

The results on the Original Weir downstream of gates are better by 1·3 to 4·1 % so far as uplift is concerned but they are on the danger side with regard to the residual head by 12·6% which for 19·5 (maximum head) is 2·46 ft. The impervious floor in this case is followed by a semi-impervious floor 26 ft. long consisting of concrete blocks over stone and spawls (not a graded inverted filter) the spaces between the blocks being sealed with cement concrete except for $1\frac{3}{4}$" diameter relief pipes spaced 3 to 6 ft. intervals. The water through these relief pipes will emerge with a head of 2·46 ft. in excess of the head at pipe 7 of the " Extension." To that extent, therefore, there is danger of piping through these relief pipes if they remain operative and as the toe wall at the end of impervious floor is only 5 ft. deep, there is danger of cavitation—slow though it will be. The Author is definitely of opinion that these pipes should be plugged. This later course will naturally raise the pressures by just a very small amount but the floor section is safe enough. These relief pipes are doing no good but might easily do considerable harm at least locally.

The relevant points of difference in the Original Weir and the Extension, where pressures are observed are :—Upstream bed is silted up to about 6 ft., downstream bed clear, upstream sheet pile line 20 ft. deep in the latter against upstream bed silt of 3 ft. downstream bed clear, middle sheet pile line 30 ft. The depth of deflection of stream lines from vertical to horizontal in the first case is (R. L. 326-298=) 26 ft. and in the second case (323—292=) 31 ft.

If in the original weir the entire floor upstream of the toe wall at the bottom of the upstream glacis were removed, the loss of pressure would still be the same within, say 5%. The results would be still better if the pile line were to be moved just under this toe wall with a Top R. L. of 318 and bottom R. L. of 288 or 4 ft. lower. Thus the whole or major part of the 91 ft. long horizontal floor upstream of the toe of the glacis is superfluous and could have been safely omitted. The saving would have been considerable. A knowledge of the true laws of subsoil flow is therefore, a real necessity both for economy and safety and any expenditure on this branch of research will be money well spent.

The investigation also shows that in the " Extension " the 40 ft. of the upstream horizontal floor and one line of sheet piles could have been omitted without increasing the uplift pressures downstream of the gate by more than for 2 to 5%. If the sheet piles had been 30 ft. instead of 20 ft. and only one line the results would have been better than even the existing ones. The second upstream pile line in this case may be said to be a second line of defence and as such the expenditure may, perhaps, be justified. It may be mentioned here that one pile line 30 ft. deep is about 30 % (final figures under investigation) better than two pile lines of 20 ft. The intermediate piles do very little work so far as reducing of pressures goes. Thus the drop between pipes 24 and 21 is only 6·6 % so that the intermediate sheet piles 20 ft. deep are responsible for about 6·6 % drop which for 19·6′ head is 1·3 ft. only. The drop at the 30 ft. piles under the crest of the Original Weir is only 10·6% or just over 50% more than for the 20 ft. piles. The 30 ft. piles have, however, helped in deflecting the stream lines down to R. L. 292 giving a vertical travel of 31 ft. as stated above which is responsible for the total drop from upstream water-level down to the sheet piles.

The conclusion from the above is that the upstream floor can be cut down considerably in length but the downstream length is essential from consideration of standing wave and retrogression of levels, in addition to uplift.

In the above discussion, observations after the 19th October have not been considered as the disturbing factor whether temperature or time lag, has not been fully investigated but it is surprising that while the pressures in the " Extension " and the two flanks have gone up, those in bays 25 to 30 have either remained stationary or gone down. Pipes B and C in the latter may have been slightly leaking but pipe A is coming up in the pier and must be reliable at least as much as any pipe in the Extension. This can possibly be explained by the unequal removal of silt in the two cases.

It has not been possible to analyse the data for the Right Flank and the Junction Groyne pressure pipes. This will be presented at the time of the Congress.

Summary and Conclusions

1. The flow of water in the subsoil under the weir is in stream lines and follows closely the laws of hydrodynamics for the flow of a a very viscous fluid.

2. The ratio $\left(\frac{p}{H}\right)$ for any point in the subsoil is constant except if modified by temperature and silt deposits on the pervious floor—factors which are under investigation.

3. The rate of change of pressure with respect to distance in vertical flow outside the upstream and downstream ends is either hyperbolic or logarithmic, the rate of drop and, therefore, the velocities being maximum where the water enters the subsoil at the upstream end, getting smaller and smaller in the direction of flow and finally reaching a minimum value at the exit at the downstream floor.

4. The rate of change of pressure and, therefore, the velocity is very nearly constant in the portion of the subsoil between the upstream and downstream end sheet pile lines. In the direct horizontal flow it is absolutely constant, and here the relation between rate of pressure drop and distance is nearly linear. The Bligh creep theory applies only in this region, with the creep ratios entirely different to the usual 1 : 15, etc.

5. The %age loss of head is a maximum at the upstream sheet piles (about 51% for 20 ft. sheet piles).

6. The horizontal floor (163 ft. length) of the " Extension " is responsible for a loss of 28%.

7. The downstream sheet pile line takes care of the balance of 21%.

8. The intermediate sheet piles line causes a local drop of 6·6 % out of the 28% for horizontal floor.

The 30 ft. intermediate sheet piles of the Original Weir cause a drop of 10·6 % only.

9. At the exit there is always a residual head just where the water emerges at the downstream floor level. In the case of bays 33 to 47, this residual head is 7 % of the total head and for a maximum head of 19·5 ft. will be 1·36 ft. which but for the graded inverted filter might perhaps be considered excessive.

10. With a correct knowledge of the laws of subsoil flow (subsoil hydraulics) considerable economies, consistent with absolute safety, are possible. In the old weir, the entire, or a major part of the upstream,

91 ft. length of *pucca* floor could be safely left out by moving the 30 ft. sheet pile at the toe of the upstream glacis. Similar economies were possible on the "Extension" where the 40 ft. upstream horizontal floor and line of sheet piles can only be justified on the score of additional factor of safety or a second line of defence.

11. A 30 ft. sheet pile line will any time be 50 to 75°/₀ better than two pile lines of 15 ft. each as in the latter case only the upper line will be operative. The exact relation between depth of sheet piles and percentage loss is under investigation and it will be premature to lay down any hard and fast rules here.

12. As a rough rule, on any large hydraulic structure, the depth of upstream and downstream sheet piles should not be less than the total head for which the structure is designed and not less than 20 ft. for river works. The upstream piles should preferably be longer so as to kill more of the head at the start, the downstream piles should not be much longer than required for obtaining a residual head at the exit, below the critical or maximum safe value. Within working limits the loss of head at the upstream pile line may be taken, on the average, as a percentage equal to twice the depth of such piles (for 25 ft. piles, 50°/₀) and half that value for the downstream piles (*i.e.*, 20°/₀ for 20 ft. deep piles). The actual relationship will be governed also by the ratio of the horizontal floor to the vertical depth of piles and the percentage proportion will decrease with increasing depth. The upstream piles appear to be, length for length, twice as effective as the downstream piles. The length of horizontal floor should be designed for the balance head, assuming (for sands) a gradient of 1/20 to 1/30 along the direct line of flow (not along the creep line) to suit requirements of standing wave and of retrogression of levels of the river bed downstream.

13. The aim of this investigation is to determine from hydrodynamic considerations the pressure distribution in the subsoil and under the impervious floor, under any given set of combinations of sheet piles, toe walls and horizontal impervious floor, or in other words to reduce the problem of the design of weirs to a comprehensive but simple formula which must not be at variance with the hydrodynamic conceptions.

14. This investigation seems to call for the acceptance of a general policy regarding provision for inserting a series of pressure pipes in all hydraulic structures of any magnitude which might be undertaken in the future, the location and extent of such pipes being determined under advice of the Central Board of Irrigation or some similar body specially interested in these problems.

Acknowledgments are due to Mr. Duncan, I.S.E., for his valuable help in general but more specially in the collection of accurate data.

DISCUSSION.

Mr. KHOSLA in introducing his paper demonstrated to the meeting his "Bell Sounder" and the "Electric Method" adopted by him for determining his observations in the Pressure Pipes at Panjnad. On the whole he preferred the former method, the sound in the pipe being quite clear above the noise of the water passing below the gates, the accuracy of the readings being within 1/16".

After thanking Mr. James L. Roy for the accuracy and thoroughness with which the pipes were fixed at Panjnad and which enabled such useful data being collected, he proceeded to give supplementary information on the Temperature effect and on the relative drop of pressures at the Right Flank and to examine some aspects of the subject-matter of the paper.

With regard to the temperature effect he remarked that the results of the third series of observations in December and January had been plotted on the Plates IX to XII. The data was not sufficient or conclusive to enable any definite conclusion to be derived but it appeared that other factors remaining unchanged, the pressure (P) increased and decreased with temperature and consequently the loss of head to any point increased as temperature decreased. The probable average temperature of water in the 1st series was about 90° to 75°F and that for the 3rd series about 60°/₀ 55°F. As would be seen the pressures (P) in the latter were everywhere lower by about 5°/₀. This difference might as well have been due to a deposit of more clay material on the upstream floor, cutting off head at the start. This, however, was highly unlikely in this case; as, during December and January, the water was absolutely clear and no more clay or silt could have had a chance to deposit. On the other hand, there appears every indication that such silt might have been there during the first series. The difference should, therefore, be due to temperature effect. The observations would have to be continued for another year or more to get at any definite results.

The Author visited Islam Headworks about 22nd December, 1932, when there was just a few inches of water over the downstream floor and the relief pipes in the semi-pervious floor were discharging seepage water from beneath the weir. The surface water temperature was about 60°F. but the water issuing out of the relief pipes was distinctly warm to the touch and was possibly about 75°F. This, coupled with the more or less constant temperature of 78° to 80° in the pressure pipes at the flank wall (Panjnad), leads to the belief that the temperature effect was considerably damped down in its passage through the subsoil.

For a high dam the temperature effect might be very different to that in a low dam or Weir. In the former the seasonal variations which might be apparent on the surface might even out at some depth below the surface. The rate and depth of penetrations of temperatures was being investigated by the U. S. A. Reclamation Bureau. In the case of

Weirs, the difference between the temperatures of surface water and that near the bed was no more than a few degrees. Temperature observations were taken in the pipes and these proved to be a few degrees higher than outside water in winter and a few degrees lower in summer. In the pressure pipes at the Flank Wall and the Junction Groyne, the water temperatures in winter and summer kept practically the same and in the neighbourhood of 78° to 80°F. because of the depth of overlying soil.

The difficulty in temperature measurements had been the getting of the right class of thermometer to go into the pipes. The author believed that the Six's pattern maximum and minimum thermometers manufactured by Messrs. Negretti and Zambra, would suit.

A set of data on temperature observations would be submitted later as a supplementary note.

Analysis of Pressures and Pressure Gradients at Right Flank Wall and Junction Groyne.

(*a*) *The Junction Groyne* (Plate II). Pipes 55 and 56 were both upstream of the 25·25′ sheet piles line and were at R. Ls. 318 and 298 respectively. The pressures in them were 50% and 50·8% of the total head. This was as might have been expected; as, all flow occurred under the piles or outside so that the stream lines would be, more or less, horizontal. Pipe 56 was comparable with pipe 33 of bay 44 and was 1·6°/₀ higher inspite of the deeper sheet piles. This was due to the direct flow past the end of the pile line, the pressure drop in which was partly controlled by the horizontal distance from the nearest water edge on the slope pitching and partly by the second line of piles which went right across. Pipes 57 and 58 showed about equal pressures, *i. e.*, 44·4 and 44·7°/₀. Pipe 58 corresponded to pipe 24 of pier 44 where the pressure was 45·6°/₀ and to Pipes A in bays 25 and 30 where it was 46·1 and 44·4°/₀ respectively. These, all more or less, agreed. Pipe 59 showed a pressure of 38·1°/₀ against 43·2°/₀ in pipe 26 and this was apparently due to the greater depth of piles by 5½ ft. at pipe 59. Pipe 60 showed a pressure of 36·6°/₀ against 38·8°/₀ in pipe 21 and 34·7 and 31·1°/₀ in Pipes B in bays 25 and 30. The result was thus better than at pier 44 but poorer than in the old Weir. The former was due to the additional depth of sheet piles across the Junction Groyne and the latter presumably to the silt upstream of these two bays. It might be noted that the nearest upstream water edge away from the wings from these pipes was about 230′ and this length was no more effective than the 111′ length of impervious floor upstream of the original weir. The only possible conclusion thus was that the horizontal length in either case had little effect in reducing pressures and appeared to confirm the belief expressed in the text that the weir would have been just as safe if the 91′ horizontal upstream floor had been omitted.

The Junction Groyne, therefore, was as effective in reducing pressures as the Weir sections and was of equal order of stability.

The Right Flank (Plate I). The three pile lines projected outside the wing wall by about three times the depth of piles, *i.e.*, 55 to 60′. The pile line extensions went 5·5′ deeper than the piles under the floor. The silt conditions in front of Pier 44 and the upstream right wing were about the same. The pressures in pipes 53, 54 were about the same as those in pipes 51 and, 52, respectively. Similarly, the pressures in pipe 47, 48 and 41, 42 were about the same as those in 46 and 39, 40, respectively (pipe 45 being out of order). This showed that the pressure lost outside of the sheet piles was almost equl to that lost through flow underneath them ; or, in other words, the length of extension equal to three times the depth of piles was suitable.

The corresponding pipes on the Right Flank and Pier 44 that mattered for safety compared as per below :—

Right Flank.		*Pier 44.*		Difference.
Pipe No.	P/H%	Pipe No.	P/H °/₀	
49	50·6	33	49·8	1·2
43	47·9	23) 26)	41·8) 43·2)	6·1 4·7
16	41·8	16	35·3	6·5
37	34·1	12	26·1	6·0
10	22·8	9	20·9	1·9

This showed that the uplifts round the flank were about 6°/₀ in excess of those along the direct flow under the Weir. The excess pressure, inspite of the 5½′ extra depth of sheet piles, showed that the pressure was transmitted from the soil beyond the sheet piles where apparently high pressures prevailed. However, considering the foundation wells of the downstream wing, the safety of the work was about the same at the wings as in the internal bays.

Old and New theory compared. According to the current creep theory, for a given class of soil, there was a creep or percolation coefficient which multiplied by the total head would give the necessary creep length for safe design. This creep length might be entirely along the horizontal or partly horizontal and partly vertical, each vertical contact face being considered just as effective, length for length, as the horizontal, provided

that the vertical cut-offs were not placed nearer than twice their depth. This assumed a constant hydraulic gradient throughout the floor length and, therefore, a constant velocity of flow.

From a study of the pressure-pipe observations at Panjnad and at other weirs in India and abroad, it might be accepted as proved that :—

(*i*) Most of the head was lost at the upstream face of the upstream line of sheet piles or wells.

(*ii*) Comparatively a much smaller loss per foot occurred along the horizontal floor.

(*iii*) A relatively large loss occurred along the outer face of the downstream end piling, this loss being much greater per foot than that along the horizontal but less than (perhaps half) that along the upstream piles.

(*iv*) A comparatively small loss—approximately the same per foot length as that along the horizontal—along both faces of intermediate sheet piles or cut-offs, provided that the latter did not go deeper than the upstream end piles. If they did, there would be a relatively large drop for this additional depth.

The above conclusions differred from the current-creep theory in that :—

(*i*) The loss along the horizontal contacts was not equal to that along the vertical ones, except when the latter were inner faces of the vertical cut-offs at the ends or both faces of intermediate ones.

(*ii*) The loss along all vertical faces was not the same. The outer faces were many times more effective than the inner and intermediate ones. The latter might be only as effective as the horizontal.

(*iii*) The pressure gradients and, therefore, the velocities of flow through the subsoil were not constant. They were high at the upstream and downstream ends, being a maximum at the entrance and decreasing in the direction of flow. They were, however, fairly constant along the horizontal (*see* Figures 2 and 3 and note the length traversed for each successive 10-minute period).

The determining factor, so far as the uplift pressures on the floor downstream of the gates was concerned, was the maximum depth of any of the vertical obstructions upstream, irrespective of the actual location of such obstructions. If there was a shallow line at the upstream end and a deep one just upstream of the gates, the result would be the same as if there was only the one deep line anywhere upstream of the gates.

From these remarks it would be clear that it would be neither rational nor safe to apply to the design of a flat floor a value of ' C ', the percola-

tion coefficient, derived from the analysis of an existing floor with sheet piles under it or *vice versa*.

The author referred to Mr. E. W. Lane of the United States Bureau of Reclamation, (technical memorandum No. 303), who had analysed over 200 dams in the U. S. A., India and elsewhere in the world and had found exactly what many others found before, that the vertical cut-offs were far more effective than the horizontal length. He had evolved a method of weighted creeps, in which he gave vertical contacts three times the value or weight of horizontal contacts. While the pressure gradient line obtained by the current-creep theory, where horizontal and vertical contacts were given equal weight, had absolutely no agreement, with the one actually observed (*see* Plate VII, Figs. I and II) the result obtained by the weghted-creep method were not far from actuals. For all practical purposes, this method was a decided advance ; but, as it was wrong in certain fundamentals, it might, under certain circumstances, lead to wasteful or unsafe designs. The error lay in the fact that *all* vertical faces or contacts were given equal weight—an assumption which was definitely wrong. The fact that intermediate cut-offs were of little use in killing head and that their effect was only local—a slight rise of pressures just upstream and a corresponding drop just downstream—had been clearly brought out in the Punjab Engineering Congress, Paper No. 138, and mathematically proved by Dr. Bose in the discussion on that paper. That had now been conclusively proved in the present paper (No. 162). In addition, the observations at Panjnad had clearly established :—

(*i*) That the inner vertical faces of end piles were no more effective than the horizontal and

(*ii*) That the flow was in stream lines or along what Mr. Lane called the " Short Path."

It had been assumed so far that the major flow occurred along the creep line or, in other words, up and down along the sheet piles and along the line of contact between the underside of the floor and the subsoil. This creep line had been considered as the line of least resistance because of the horizontal contacts—if not the vertical—being far from intimate due to subsidence of the subsoil under the floor. That, to the author's mind, was a misconception. The major flow occurred straight across from bottom of one sheet pile line to that of the next and so on, with only a very small fraction of flow occurring in the region bounded by the floor and the sheet piles. If the sheet piles were too close, this may be a region of dead flow; where they were widely spaced, a certain amount of flow would occur and some of the steam lines will branch into that region (*see* Plate IV). The line of creep could be ignored so far as piping was concerned. It had, however, to be considered in the determination of uplift pressures under the floor downstream of the gates and for that region it should be continued to be used as heretofore, although with percolation coefficients entirely different from those proposed by Bligh.

The Electric-flow Method.—Mr. Lane investigated the stream-line theory by the Electric-flow analogy. In this he had substituted for the weir a non-conductor with suitable allowance for vertical cut-offs. The subsoil was replaced by a brine solution—the electrolyte—and the flow of water by that of electricity under an electric head or potential. The lines of equal pressure were carefully obtained by means of a Wheatstone Bridge and plotted. The stream lines were then plotted orthogonal to the pressure lines. This chart he compared with the model tests by Colman and the two sets of lines were almost entirely different. In the Electric Stream line chart, the pressure distribution was uniform along the entire length, horizontal or vertical. In Colman's chart, the pressure drop varied, more or less, in keeping with the Panjnad results.

If then, the flow through the subsoil followed the laws of Hydro-dynamics, why such variations from the electric analogy, when the equations of flow in both cases were the same. The reason was not far to seek, the Electric analogy would hold if :

(*a*) gravity were absent ;

(*b*) if the transmission constant for flow in the vertical direction were exactly the same as in the horizontal. This apparently was never the case either due to stratification or some other causes which have yet to be determined.

If it were possible to duplicate the effect of gravity by superimposing an electric field and the stratification, etc., by putting in brine solutions of varying densities, one below the other, the results obtained by the electric analogy would be comparable with those with flow of water through the subsoil.

The experiments of Mr. Colman were described in Paper No. 1356 of 1915 of the American Society of Civil Engineers. In his model he settled the sand under water which, though not reproducing exactly the natural alluvial deposits in a river-bed, might be considered as somewhat representative. Although no definite stratification existed, the loss of head at the upstream piles was many times greater, length for length, than along the horizontal.

Force of subsoil water.—It could easily be proved mathematically and tested experimentally that the force of water on the porous medium through which it passed was equal to the hydraulic gradient multiplied by the specific weight of water and that this force acted in the direction of flow. This held both for constant velocities as under the flat floor and for variable velocities in vertical flow at the upstream and downstream piles. At the upstream end this force acted downwards in the direction of gravity. At the downstream, it acted upwards against gravity. If at this end the upward force was greater than that of gravity the soil particles would be lifted and damage would ensue. With upward vertical flow, it should be noticed that the velocity of water at which the sand was lifted was much less than that which was required to keep the sand particles in suspension.

As velocity was proportional to the hydraulic gradient at any place, the force 'F' of moving water can be represented by $F=\rho S=KV$ where K=constant, ρ= specific weight of water, S=gradient= $\frac{dP}{dY}$, P=pressure at any point, Y=the distance from origin and V=velocity.

If the pressure drop at the upstream and downstream ends be assumed to follow a logarithmic law (Plate XIII, Figs. III and IV) then

$$P=K' \frac{dP}{dY} = K'S.$$

or pressure at any point was proportional to the gradient. But velocity was also proportional to the gradient, hence,

$$P=K'S = K_1V$$

$$\text{But } F. = K V$$

$$\text{Hence } F=K_2P$$

or the indicated pressure (P) at any point was a measure of (F) the force of water acting at that point in the direction of flow.

At the downstream floor (P), the residual head was, therefore, a measure of the force which had not been counterbalanced. Since gravity was the containing force, it was obvious that the depth of stone protection over an inverted filter at this end was a major factor in determining the safety of the work. The proper maintenance of this protection was just as important as the maintenance of the loose protection at the upstream end.

Concluding, the author said that the observations at Panjnad would be continued for a period of at least one more year to study the effect, on indicated pressures, of temperature, silt including clay deposits and any permanent changes in subsoil with lapse of time. Other aspects of the problem would continue to be studied and the results, it was hoped, would be published in a supplementary paper.

Dr. BOSE congratulated Mr. Khosla on the admirable set of data that he obtained on the movement of water in the subsoil below the Panjnad Weir, and stated that as far as he knew these were the first of their kind in the engineering world and Mr. Khosla might well be proud of them. They threw a flood of light on the obscure science of "Subsoil Hydraulics" on which he had occasion to speak about three years back to the Congress. He had then proposed a general law of subsoil flow which stated that the pressure at any point in the medium could be calculated from a solution of the equation :—

$$\nabla^2\phi = o$$

(where $\phi=p+g \rho z$), that would satisfy the boundaries of a definite problem. Since then he had come across various suggestions about the laws of subsoil hydraulics from Schlichter, Forchheiner, Terzaghi and others. Some of them retained the gravity term in the above while others neglected it. Since these experiments started and Mr. Khosla was kind enough to keep the speaker supplied with his observational data, he

had been busy analysing some of the deductions from the above law for the Panjnad Weir and comparing them with the experimental results. A justification for retaining the gravity term was furnished conclusively by the pipes No. 28 and 29 a reference to which had been made by Mr. Khosla on pages 8 and 9.

FIG I

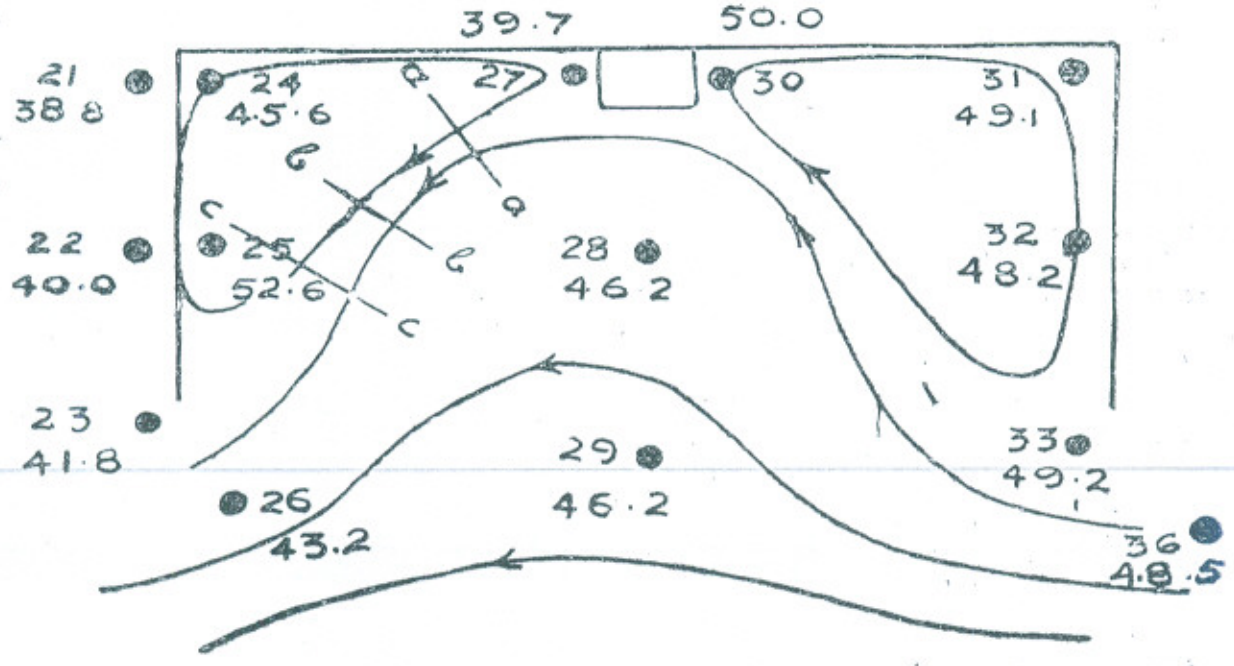
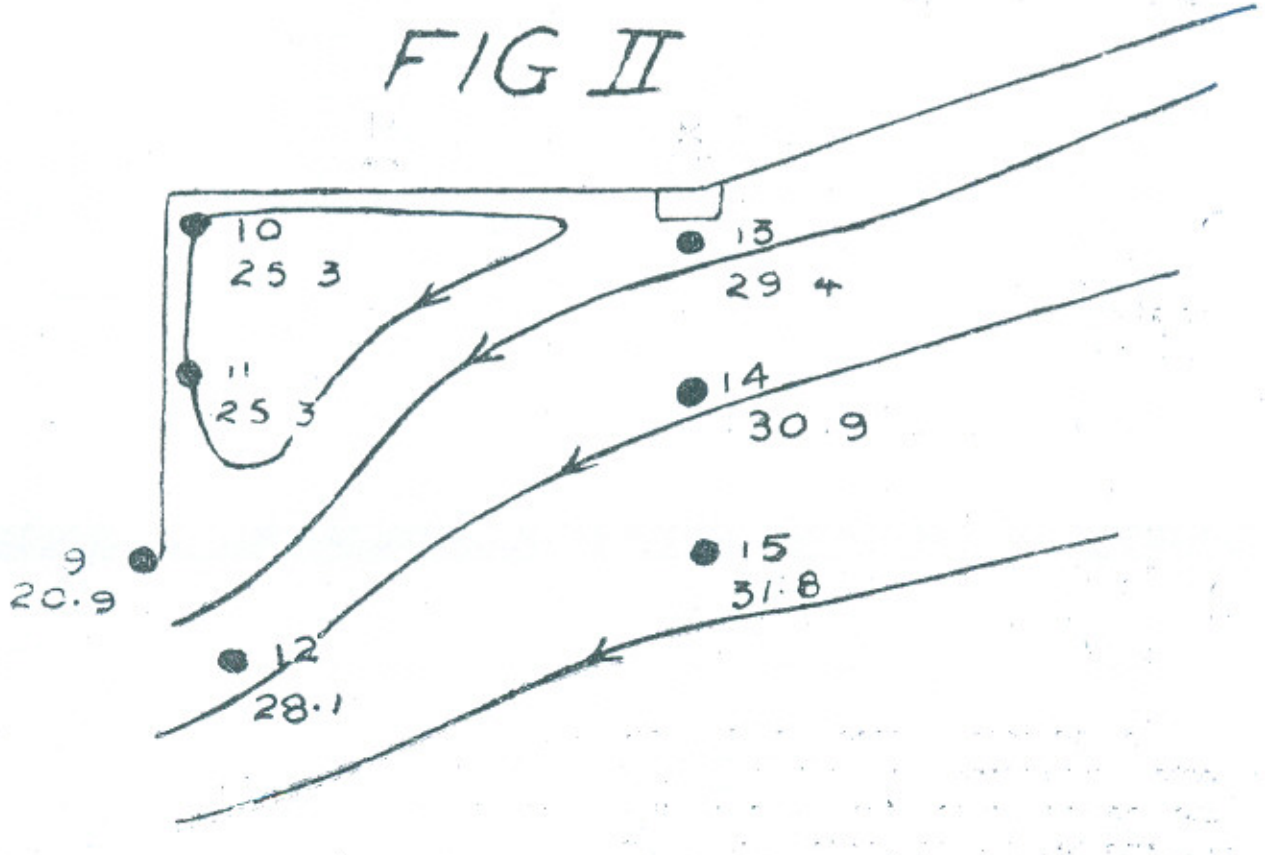


FIG II



The speaker said that there were many points of agreement between what Mr. Khosla had said in his paper and his own ideas. He would not refer to them but would refer only to those few points on which he did not agree and on which he hoped the engineers present would help him with their experience.

The main disagreement, according to him, was on the correctness of Plate IV and on the observations made by Mr. Khosla on p. 61. He stated he would take only the two relevant portions of the Plate IV and draw them.

In the sketch the pressure points were also put in position as well as the percentage of excess pressure at these points. These percentage pressures, it would be seen, were constant for each point irrespective of whatever head was put on the Weir. From Fig. I it would be seen that the pipe No. 25 had got the highest pressure in this region and then came pipe No. 30. Mr. Khosla, to get over these apparently irreconcilable data, had cut off the pipes 24, 25 and 27, 30, 31 and 32 from the general flow and put two vortices in these regions. With these simple strokes he made everything look reasonable and, if it had been ordinary free water flow in open channel, nobody would have objected to it. But in the subsoil where the flow was extremely slow, of the order ·00005 ft./sc. such vortex formation was extremely improbable. The drop of pressure from 36 to 26 was 5·3 °/₀ and the maximum head being 16 ft. the absolute drop was 0·84 ft. over a distance of 50 ft. so that the gradient was ·017 and velocity

$$v = Ki.$$
$$= 0{\cdot}003 \times 0{\cdot}017 \text{ ft./sc.}$$
$$= 0{\cdot}00005 \text{ ft./sc.}$$

This would be the maximum velocity in this region and between the sheet-piles it would be less. Now, in the corners if stream-line motion did not exist there were two possibilities :—

(*a*) This region might be dead, *i. e.*, there would be no motion.

(*b*) This region might have a vortex.

In the first case there should be no variation of pressure in this region. Pipes 10 and 11 showed it, but neither did the corners probed by 24, 25 and 27, nor by 30, 31 and 32 show any indication of it.

In the second case we can expect a variation of pressure inside the vortex but across the boundary regions between the vortex and the stream-line motion there should be a continuity of pressure, so that along the sections aa, bb, cc there should be no change of pressure. Now the

section cc entered the vortex region in the neighbourhood of 25, so that on the other end of the section the pressure should be the same at 25, *i.e.*, 52·6 °/₀. Now this was impossible as the pressure in this region could only lie between the pressures recorded by 29 and 23, *i. e.*, between 46·2 and 41·2. On the face of such deductions the speaker suggested that the pipe 25 was not behaving properly. Similar consideration would show that 30 also cannot be relied upon. It was a misfortune that these two points should go wrong—they were placed in a region of which very little was known and their correct records would have given a lot of very valuable information for which some more favourable opportunities must be awaited.

The speaker said that he had analysed the variability of the percentage pressures of all the pipes between the two lines of sheet piles and found the coefficient of variability for 25 was as high as 12 °/₀ whereas, for the others it was between 5 and 6 °/₀. The probable error of this coefficient for pipe 25 was also significant showing that pipe 25 was behaving erratically.

Now the percentage pressures of these pipes were more or less constant and this constancy we know was a characteristic of the stream line flow given by

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

and not of vortex motion. The flow in the two corners might be extremely slow but there cannot be any reverse motion.

There was another point on which the speaker did not agree with Mr. Khosla. On page 7 in his "General Conclusion" the author writes that the variation of pressure in subsoil flow with regard to distance in the vertical direction downwards at the upper end of the impervious floor and upwards at the downstream end was either logarithmic or hyperbolic.

If this was either of the two he would prefer the former as it was a solution of the general flow given by $\nabla^2 \phi = 0$ whereas the other was not.

This experiment by Mr. Khosla and a similar but small-scale experiment by Mr. Harbans Lall in the Punjab Research Institute throw a flood of light on some very obscure corners of the science of Subsoil Hydraulics. But still we know very little about it and he hoped many such experiments would be started. These should aim at clearing certain definite points that require elucidation. These controlled experiments he thought would be more useful in developing a rational theory of subsoil flow than the information that we could get by sticking pipes all over the weir. For this a small committee of experts should be formed to lay out every experiment and to analyse the data as they were obtained.

Before any small-scale experiments could be relied upon, one should become absolutely sure that the movement of the subsoil water under a foot head follows the same law as under say, 16 or 20 ft. head. It was well known in the flow of water through a pipe that as the head under which the flow took place went up the character of the flow suddenly changed from the stream line to the turbulent at a certain critical velocity. If the same thing happened in subsoil hydraulics, model experiments would not only be useless but misleading. This point, in the speaker's opinion, should be settled first before any model experiment was taken up. The Panjnad Pressure Pipe Observations were a complete set of reliable data, and if these observations were duplicated on a small-scale model of the Panjnad Weir, a comparison of these two parallel sets of data would decide this point.

The speaker said that in carrying out the small-scale experiment the following points should be borne in mind :—

(*i*) Accuracy of the pressure pipe observation should be very high—the smaller the model the higher the accuracy required. A small-scale model was at a disadvantage in this respect, where the capillary effect of the observation pipes might ruin the whole of the hydraulic effect. In such cases it was advisable to read all the pressures on the same pipe.

(*ii*) It was essential that there should be as little variation of temperature as possible during the course of the experiment. This was important for small-scale experiments where the capillary motion of the observation pipes play such an important part.

(*iii*) The U/S and D/S levels should be kept absolutely steady for at least a day before the pressure observations were taken. This was easier done in a small-scale model.

(*iv*) In a small model the boundary effects were very important and unless due precautions were taken after careful hydrodynamical considerations the results were liable to be misleading.

L. KANWAR SAIN observed that when Bligh gave to the world that glorious comfort—a neat formula—engineers jumped at it. It was treated by most of those in the profession as a kind of charter, giving absolution from further worry regarding the design of weirs on sand foundations. Some engineers occasionally half-heartedly revolted against Bligh's theory but for want of something better it was followed. It was left to Mr. Khosla to give a new lead in this direction, for which was due the gratitude of the entire body of hydraulic engineers. However, one should not fall into the trap of generalising on insufficient

data. Observations, similar to those made at Panjnad, should be made on as many existing weirs as possible. The cost incurred would be repaid many times over.

The author at Panjnad, found the loss of head on the horizontal floor to be 1 in 51. From observations made in October, 1898, on the Narora Weir in the U. P. this worked out to 1 in 29, while later in January, 1916, Mr. Woods found the loss of head on the Rasul Weir in the Punjab to be 1 in 38. L. Kanwar Sain wondered if the physical side of the sand had something to do with it? He suggested that a mechanical analysis of the sand together with a record of the temperature at the time of the observations should be added to the observation details. In small pipes and in flow through soils, the temperature of the water might be a very important factor. In the transactions of the American Society of Civil Engineers, 1903, Vol. 51, page 290, it was shown that for smooth pipes of 2" dia. and under, the loss of head has been found to increase about 4 per cent. for each 10° F. fall in temperature between 70/c and 40° F.

There was no doubt that the loss of head on entry into the soil would vary as some function of the head. The figure of 30 feet sheet piles being 75 per cent. better than two lines of 15 ft. piles, given on page 16 in the paper, might perhaps be too optimistic. Any colloidal matter from the river water depositing upstream of the upper pile line might possibly effect the initial loss of head. If this was true a new factor was introduced and the loss of head would also become a function of time.

He further enquired if the Author could give any reasons, other than that of entry and based on known hydrodynamic laws as to why the loss of head should be so much more at the upper line of piles than at the intermediate line.

Regarding the Author's definite opinion on page 13 about plugging the relief pipes; he enquired as to what objection the author had to filling these pipes with a suitably graded material and converting them into so many inverted filters.

MR. A. M. R. MONTAGU who had to do with the design of the extension to the Panjnad Weir (Bays Nos. 33 to 47) informed the meeting that evidence was even then available that sheet piling, properly placed, was cheaper than extending the upstream floor and more effective as a method of killing the pressure head on the Barrage. The primary reason for the middle line of sheet piles was to protect the structure in the event of cracks developing in the upstream floor and that the recommendations of the Islam Weir Committee precluded the placing of these piles directly below the crest, as was done in the original barrage (Bays Nos. 1 to 33). Regarding the residual head mentioned on page 13 it was recognised from the beginning that residual head must exist no matter what the length or type of interference might be inserted in the structure.

According to Mr. Lacey's theory no silt particles would be moved by a velocity of less than 0·84 feet/sec., but the officers advising on the design of the extension suggested that a properly designed and protected inverted filter could take care of residual head of about 1 foot. The barrage was, therefore, designed on this assumption. The author's observations indicated that the residual head was of the order of 1·37 feet.

Mr. GERALD LACEY said, that the author had presented the Congress with a very valuable paper as sequel to his previous communication to the Congress. If he had a small criticism to offer it was that the paper would be simplified with the addition of an appendix dealing with the notation.

Now that the work of Mr. Khosla and Dr. Bose had advanced the subject so greatly and removed it out of the plane of the early creep theory it was advisable to remember the early recommendations of Col. Clibborn who was the pioneer in subsoil flow under weirs in sand. Bligh had superimposed his own theories on empiric lines on Col. Clibborn's work, and the latter's chief recommendation was sometimes overlooked. The necessity for a floor in order to avoid severe action had led to its being regarded as an essential feature of subsoil flow design. Col. Clibborn particularly emphasised the necessity of deep upstream and downstream curtain walls, and the Author's paper had shown that they absorbed upwards of seventy-five per cent. of the difference in head. Effectively the lesson to be drawn from the paper was that engineers should look after the depth of the curtain walls and that, thereafter, works could be trusted to look after themselves.

There was also a great practical advantage of walls to which the late Mr. J. Power, Executive Engineer, U. P., had drawn his attention. By constructing deep curtain walls the water was constrained to follow a path through well compacted, water deposited sand, and lines of weakness, following the axis of the river were thus avoided. Many failures could be traced to one weakness passing right through the structure and near the surface. No amount of floor could compensate for neglect of these fundamentals.

The extent to which piling could be trusted to act as a curtain wall was very important. All piles did not function alike and some were anything but water-tight although they might restrict sand movement. This meant that they stopped surface piping but only partly relieved the pressure on the floor.

Mr. R.P. HADOW pointed out that in propounding his theory Bligh had misquoted or misinterpreted the results of Col. Clibborn's experiments in so far that he took the creep length round a number of vertical obstructions as being equal to twice the depth of those obstructions.

Col. Clibborn never proved this. Indeed, years ago in the Punjab, it was realized that it would be more correct to take the shortest line as the line of creep; but in view of lack of any more definite knowledge, Bligh's theory continued to be used in practice.

Valuable as the author's results undoubtedly were he thought that any conclusions drawn would only be applicable to the conditions examined and would not necessarily apply to a Barrage with different lengths of piling and floors. He suggested that it would be desirable to reproduce the Panjnad conditions in a model in the Irrigation Laboratory and record results. If they agreed reasonably well with the actual results at Panjnad, then it would be logical, as well as correct, to go on with the model experiments using different lengths of piles as well as changing their positions. From data gathered under such varying conditions some definitely valuable practical rules could be evolved for the design of works on sand foundations.

The AUTHOR thanked the various speakers for taking part in the discussion.

In reply to Dr. Bose, he stated that vortices were put in at pipes 25, 24 and 27 and at 30, 31 and 32 as no other explanation was forthcoming for the pressure drops in the reverse direction. The velocities in subsoil flow were certainly very low but perhaps some multiple of such velocities may cause turbulence in such flow and yet be insignificant compared to velocities causing similar turbulence in open water flow. Another explanation was that the corners where vortices are shown may be regions of dead estuaries, where there being no flow, the pressure will increase by the amount of velocity head, much the same as would occur in a pitot tube facing the direction of flow. Pipes 30 and 25 would thus naturally show higher pressures. Pipes 32 and 27 which were in the back water should show lower pressures by the amount of head due to velocity of recess. It was not possible to agree with Dr. Bose that these pipes were recording incorrect results. They might not be absolutely correct, but they indicated a definite phenomenon in subsoil flow which appeared to be controlled by the spacing of vertical obstructions. It will not be enough to brush it aside because they did not fit in with current conceptions of hydrodynamics; for, it must be admitted, that the Hydraulics of flow in subsoil was in its infancy. The strict mathematical derivations had to be modified to suit peculiarities of soil structures and other observed facts.

Dr. Bose was perhaps correct in accepting logarithmic variation for pressures in vertical flow.

Regarding tests on small-scale models, if conducted properly and with due regard to field condition this branch of research offered a fruitful

field but the abstruse mathematician and the matter-of-fact engineer must work together to get useful and lasting results. By a more extensive set of pressure pipes inserted in the various weirs, existing and future, and by duplicating results on models, a real advance was possible which would lead to economy and safety. The pressure pipes, in all cases, must be designed as to location with great forethought and laid with equal care.

Regarding L. Kanwar Sain the author agreed that the grade of soil particles, their packing and many other factors determine the actual gradient under the horizontal floor. The gradient increases in direct proportion to the head. Then it depended on the relative location and depth of sheet piles. A knowledge of the grade of soil, geological formation and temperature was, no doubt, essential to a proper study of the problem.

The deposit of clayey matter at the upstream end would necessarily cut off a good deal of the head at the entrance but these deposits are additional factors of safety and must not be taken into account, as these could just as easily be scoured out. The design should provide for the worst conditions. What, however, might be of a permanent nature in its effect, was the infusion of clayey matter into the pores of the soil by the water in its downward flow. The time factor would come in as the choking off would be progressive. Little was known of this at present. The evidence available to date showed that this introduction of clay into the pores of the soil did not penetrate far and was restricted to the surface which was likely to be disturbed in normal flow or in floods. The analysis of data at the various weirs like Narora, Khanki, etc., where observations had been taken for a series of years, would lead to useful indications regarding the time factor.

The question of loss of head at entry had been raised time and again but the author was of opinion that there was no such thing in subsoil flow, as loss of head at entry, unless, of course, there was a blanketting of tight clay at the surface. The loss was progressive from the surface down, the rate of drop decreasing in the direction of flow. This could be ascribed to gravity, stratification or some other similar phenomenon. In horizontal flow the stream lines apparently followed the planes of bedding (unless minor geological interferences intervened) where the soil pressure and gravity effect were uniform. This led to a constant drop of pressure per foot and, therefore, constant velocities. In vertical flow, the soil was subject to greater compression as it went deeper so that even though there be no marked stratification, the percolation water had to pass through pores under increasing compression in downward flow and *vice versa* in upward flow. While horizontal flow might be likened to that in cylinderical pipes, vertical flow would correspond to that in slightly conical pipes. Again, it was a proved fact that where layers of unequal porosity existed the percolation was governed by the layer of least porosity. It must, however, be remembered that pressure drop might not necessarily follow exactly the rate of drop in discharge but it would follow that trend.

Regarding relief pipes filled with Bajri, they would be safe if the bajri remained in them, but it was very frequently washed out under pressure of escaping water, with the result that the head would become great and might wash out fine sand from underneath. If the bajri remained in the pipe, the relief effected was little and the pressures under the floor remained undiminished. From either point of view, the utility of these relief pipes was questionable.

The author further remarked that Mr. Montagu's contribution to the design of the Extension to the Panjnad Weir and the insertion of pressure pipes had been already acknowledged in the paper. Mr. Montagu's remarks explained the reason for putting in the second line of piles upstream although it was clearly understood at the time that, so far as drop in pressures was concerned, this second line of piles would be almost ineffective. The residual head was taken note of at the time of design and provision was made for one foot of such head. The observed results were not far from what was anticipated.

Mr. Lacey had done well in reminding the profession, of the original recommendations of Col. Clibborn—the pioneer in the field of Subsoil Hydraulics—wherein stress was laid on deep upstream and downstream curtain walls and the horizontal floor was shown as essential to take care of the severe action of flowing water. Mr. Lacey had summed up the lessons of the paper in a pregnant sentence that "if the Engineer takes care of the depth, the work will take care of itself." These remarks about the importance of curtain walls in forcing flow through well compacted layers of soil, were worthy of note as also his warning against the sheet piles if not water-tight.

The author thanked Mr. Hadow for taking part in the discussion. He had drawn timely attention to the fact that Col. Clibborn's work was misquoted or misinterpreted and that his results were in keeping with those of modern research. That the flow occurred along the shortest line across the work and not along the so-called line of creeps had now been proved by the Panjnad observations.

The author, however, differed from Mr. Hadow in his remarks that the conclusions drawn from Panjnad would not necessarily be applicable to a Barrage with different lengths of piling and floors. The Panjnad Weir Extension was designed on certain assumptions which had proved to be correct. Those assumptions, the outcome of the Punjab Engineering Congress Papers No. 138 and 142 (1930 session) were :—

(*i*) that the major part of head was killed at the upstream sheet piles.

(*ii*) that the intermediate piles were ineffective in killing head and that their effect was local, but they were useful as second line of defence, should cracks occur in the upstream floor.

(*iii*) that the second major loss of head was at the downstream end sheet piles.

(*iv*) that the loss of head along the horizontal floor would be small.

(*v*) and that there would be a residual head at the downstream end irrespective of what depth of piles or length of horizontal floor was given. An inverted filter was given to take care of this residual head.

All the above assumptions had proved to be correct. Hence, to the extent stated in the paper, the conclusions were of general application. A fairly lengthy comment had already been offered in the introductory remarks and it would add greatly to the bulk of discussions if any attempts were made to show by analysis of the various dams for which data was available, as to what extent the general conclusions were applicable in each individual case. That the major loss occurred at the upstream end and that horizontal length was much less effective had passed the stage of controversy but the question might be asked if a deepening of the upstream vertical obstruction would correspondingly lower the pressures downstream of the obstruction if the other parts of the structure were left unaltered. Such evidence was impossible to be obtained from actual works where neither depth nor length could be altered once the work was complete. But luckily, Mr. Colman's paper afforded the necessary data which tended to prove that such was the case. The box in which the model was placed was 15′—2″ x 3′—11¼″ x 5′—6″ deep. The sand was filled to a depth of 5′ and a floor 8′—3″ long was centrally placed over the sand four feet from either end of the box. Then 1′, 2′ and 3′ length of piles were inserted at the upstream end. The results are tabulated below :—

Experiment No.	Depth of sheet piles.	Upstream water-level.	Water-level in pipe at bottom of sheet piles.	D/S water-level.	Total Head H.	Loss of head to Pipe-L.	Percentage loss L/H.
30/2	1 ft.	4·84	3·54	0·0	4·84	1·3	27°/₀
26/3	2 ft.	4·83	2·78	0·0	4·83	2·05	42·4°/₀
20/3	3 ft.	4·87	2·28	0·0	4·87	2·59	53·1°/₀

In the above experiments the temperature was about 50° F. in all cases, the length and location of horizontal floor and the depth of soil was unaltered. The only item that was altered was the depth of sheet piling. The test could not be called absolutely representative as the depth of soil left below the 3 ft. pile was only 2 ft. but there appeared little doubt that the indication was correct.

The Narora Weir with 17·5′ depth of piles below upstream floor level gave a pressure drop of about 45%, the Panjnad with 22½ ft. depth of piles gave about 51% the Pinkook dam (U. S. A.) with 31 ft. depth of piles gave about 60%. The Panjnad original Weir with sheet piles just upstream of the gates going 28′ below the upstream floor level gave a loss of head ranging from 65 to 69%. Rasul with 24′ depth of second line of wells gives about 70%. The subsoil, the depth and location of vertical obstructions and the length of horizontal floor varied in all cases although not much in the case of the first three. But the results bore out the general conclusions.

The author was in complete agreement with Mr. Hadow's suggestions regarding duplicating Panjnad results in the Research laboratory on a scale model and extending the scope of experiments later on. It was hoped that the suggestion would mature.

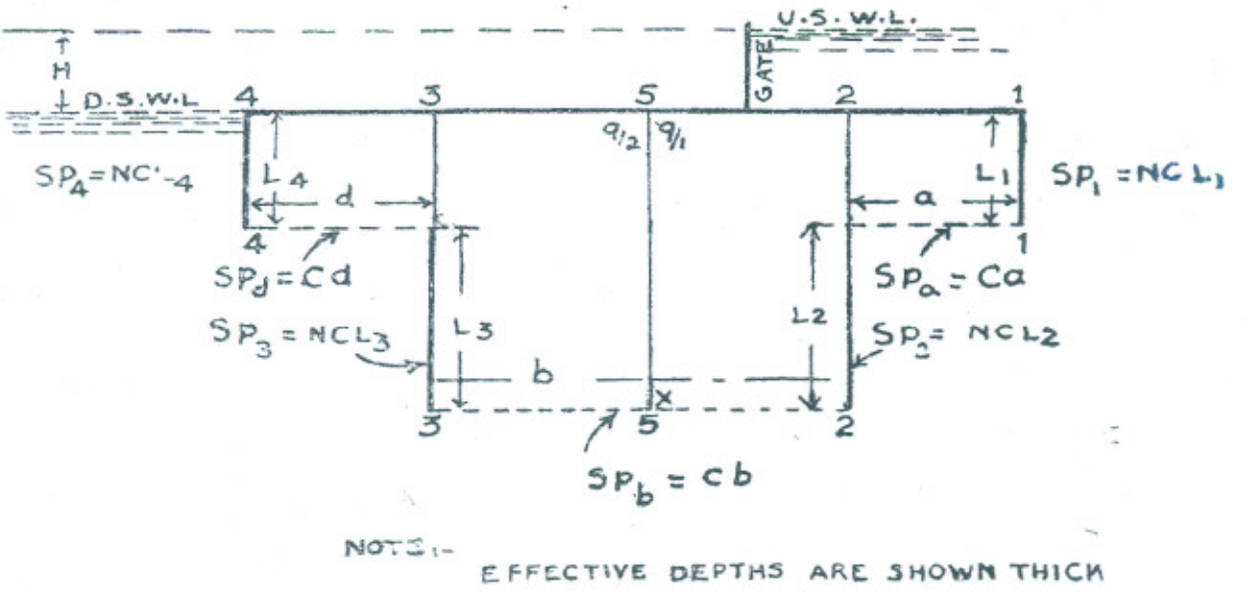
It would be premature, without further experiments and without an exhaustive study of existing data to lay down hard and fast rules. But the following provisional suggestions were offered by the author :—

(*a*) New values of the percolation coefficient 'C' depending on the angle of repose and other soil characteristics, would have to be introduced. For fine sands like those at Panjnad C may be accepted as 1/30.

(*b*) The length of horizontal floor downstream of the gates would depend primarily on two essentials (*a*) that the energy of flowing water must be dissipated to a safe degree before it reached the pervious floor and (*b*) that the standing wave must form well above that floor.

(*c*) The safe and economical way to deal with head was by means of a combination of vertical obstructions and horizontal lengths, the suitable proportions for which were yet in the realm of empiricism. It was extremely unsafe to depend entirely on a horizontal floor without vertical obstructions and it might be just as risky to go to the other extreme and to have practically no horizontal floor and to lay entire dependence on deep cut-offs.

(*d*) The relative importance of vertical cut-offs with reference to their depth and location was illustrated diagramatically in the sketch below. 'C' in this stands for the new percolation constant, 'N'—the number of times the vertical contact was effective compared to the horizontal, per foot length. 'N' would naturally vary with depth ; but, an average value would give results fairly close to actuals not only at the underside of the floor in more or less horizontal contact with the soil but all through the soil. 'δp_a'= drop in pressure along length '(a)', and so on.



Total drop along horizontal $= \delta p_a + \delta p_b + \delta p_d.$

$= C(a+b+d)$

Total drop along verticals $= \delta p_1 + \delta p_2 + \delta p_3 + \delta p_4.$

$= NC \left\{ L_1 + L_2 + (L_3 + L_4)/2 \right\}$

$$\text{Total Head} = H = C \left\{ a+b+d+N(L_1+L_2)+\frac{N}{2}(L_3+L_4) \right\}$$

(N) will probably have a value of the order of 10.

The intermediate pile line would be ineffective except that pressure at q_1 would be higher than that at x by about CL_5 and at q_2 less than that at x by about the same amount. Pile line I had masked the 2nd pile line from top by a length equal to L_1 and similarly line 4 had masked the upper length of the pile line 3 by length $= L_4$. The residual head was assumed to be included in L_4. It would be seen that the extreme depth of piles at either end was the controlling factor and that the actual point above the gate line where the deepest piles were located, had very little effect on the uplift pressures under the downstream floor.

CORRESPONDENCE.

Mr. E. W. LANE, Research Engineer, U. S. Bureau of Reclamation communicated :—

"The pressure pipe installation and observations described in Mr. Khosla's paper are undoubtedly the most thorough study of upward pressure ever made on any structure, and the account of them is an exceedingly valuable addition to knowledge on this subject. The graphical method used to represent the results is novel and very effective. In the main, it is believed that the conclusions drawn are sound, but there is one point which does not seem to be justified by the evidence presented.

"Mr. Khosla assumes a uniform porous foundation material with its surface approximately a continuation of the surface of the upstream and downstream aprons. Since the drop in pressure above the upstream and below the downstream piling had roughly a hyperbolic or logarithmic relation, he concludes that the ordinary law of linear flow did not hold. If the material was uniformly porous and of the extent assumed, the conclusion is justified, but there is strong evidence that these conditions did not exist. Since under Geological Section of Subsoil it is stated that the sand found in the borings increased in size from fine to coarse in a downward direction, and that this might be taken as fairly representative, it seems that the assumption of increasing porosity with depth would be more logical than that of uniform density. This variation in porosity would in itself account for the nonlinear pressure drop distribution, and seems to be the most probable explanation. This is further confirmed by the relation of the lines of the flow net given on Plate IV-A. If the foundation material was homogeneous, the curvilinear rectangles formed by the flow and pressure lines would all be of similar shape, *i.e.*, the ratio of the lengths of the flow line sides to those of the pressure line sides would be constant throughout. As plotted from the observed data, however, the curvilinear rectangles near the river-bed surface have their longer sides formed by the pressure lines and deeper in the deeper foundation material the longer sides are formed by the flow lines. This would only be true in a material of porosity increasing with depth. The steep slopes of the grade lines on Plates IX and X at the two ends of the diagrams are, therefore, due to a more impervious material near the surface, rather than a nonlinear flow relation. The greater fall at the upstream end as compared with that at the downstream end is probably due, in part at least to the thick layer of silt on the upstream floor. Another factor probably is that the flow of water from the river downward and under the dam, causes a filtering out of the sediment in the water as it passes through the bed terrain on the upstream face of the dam. This fine material tends to clog the void space in the bed material upstream from the weir and to decrease its permeability. This action was noted on the grand Valley Dam on the Colorado River in the United States.

" From a study of all of the available data on masonry dams on earth foundation which the present writer was able to collect, which included over two hundred of these structures, he was led to a belief that an intermediate line of sheet piling, as a second line of defence, is a valuable asset. In this study, no record of failure was found where piping certainly developed beneath a deep cut-off, and only one case, for which the information is meagre, where it may have. Three cases were found, however, where defects in the sheet pile cut-offs were causes of failure. In these three cases a second row of piling would probably have saved the structure. It is believed, therefore, that there are cases where two rows of fifteen foot piling would be preferable to one row of thirty foot piling, even though the latter might give lower upward pressures on the structure."

The AUTHOR in reply thanked Mr. E.W. Lane, Research Engineer of the United States Reclamation Bureau for his very interesting written communication and felt gratified at the world-wide interest which the subject of the paper had evoked in recent years. An interchange of views on the subjects of common interest to the Punjab and the United States of America and an exchange of research and other literature between the Punjab Irrigation Department and the Bureau of Reclamation, U. S. A., would undoubtedly be to their mutual benefit.

Mr. Lane had objected to the foundation material under the Panjnad Weir being assumed as a uniformly porous one. He would rather accept it as one of increasing porosity with depth. The geological sections at the sites of location of pressure pipes not being available, it was impossible to make a definite statement but there was no doubt that the law of flow along the upstream and downstream outer vertical faces was different from that along the horizontals. The reason may be gravity, the non-uniform porosity, stratification, infusion of clayey material with inflow at the upstream end or some other factors but the phenomenon appeared to be general and seemed to persist even in model tests. This had already been dealt with in replies to earlier discussions.

The rate of pressure drop at any point being proportional to the indicated pressure at that point the upstream sheet piles with higher indicated pressures would naturally be more effective than the downstream piles with much lower indicated pressure. Again, if porosity were the determining factor, the drop from pipe 34 to 35 being greater than that from 35 to 33 was understandable, the upper layers being of finer sand. But then the drop from pipe 9 to 8 should have been less than from 8 to 7 for the same reason, which was not the case. This shows that the rate of drop was governed not necessarily by porosity but by the indicated pressures. That was why the rate of drop was steadily decreasing in the direction of flow.

Mr. Lane's remarks about the curvilinear rectangles of flow net in Plate IV were very interesting but it must be clearly understood that while the spacing of the pressure lines and the curves of the pressure

and stream lines might be taken as more or less accurate, the spacing of the stream lines was pure guess-work and not based on any mathematical calculations.

Mr. Lane's remarks about the reduction of pressure at the upstream end due to a clay blanket or due to the pores of the surface layer getting clogged with the clayey material infiltering with the seepage water had already been dealt with in the introductory remarks and replies to discussions. In this connection, a reference was invited to Plate XVI, which showed for the entire period of observations the variations in the percentage pressures in pipes 33, 31, 23, 16, 12 and 9 of Pier 44 and also showed the observed river water temperatures, depths of silt on upstream pervious floor (the downstream floor being clear of silt) working heads and downstream water levels. From this diagram it was perfectly clear that *no reliance could be placed on silt deposits on the pervious floor*, for while such deposits might reduce the uplift pressures almost to a negligible amount so long as they existed, they could easily get washed away and again give rise to the full uplift pressures. On the 28th May 1933, the indicated pressure on pipe 33 (at the bottom of the 1st line of sheet piles) went down from its normal value of about 49 % to 12 % meaning a loss of head up to that point of 88 % leaving only 12 % of the total head to be dealt by the weir below the upstream pile line. Due, however, to the steady increase in temperature and a reduction in the depth of silt or perhaps to the removal of the clayey film on top, the pressures started rising after that date till about the middle of July they went up again to 60% and more. The observations of September 1933 were of special interest as the depth of silt deposit was in the neighbourhood of one foot only. In this period the rise and fall in pressures seemed to be governed by the temperature of river water.

The author was every time in favour of a second line of water-tight sheet piles as a second line of defence. His remarks in the text were not meant to discourage the provision of this additional safeguard but were designed to remove the blind adherence to the creep theory and to dispel the current belief that intermediate pile lines were just as effective in killing heads as the outer ones. As water tightness of a pile line was the essence of its effectiveness and as some Engineers had serious doubts about the water tightness of steel sheet piles arising out of faulty driving and other causes—it would certainly be an advantage to have the additional line of defence in a second row of sheet piles. The additional expense would be amply justified in case of important structures like the Headworks of Canals whose failure, even partial, might spell ruin to vast tracts of country served by them.

Dr. E. Mackenzie Taylor, Director of Irrigation Research (Lahore) had since carried out a set of very useful and instructive experiments to determine stream lines in subsoil flow with