PAPER No. 294

OBSERVATION, RECORD AND ANALYSIS OF PRESSURE PIPE DATA OF WEIRS ON PERMEABLE FOUNDATIONS

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1. INTRODUCTION.

This paper is written to provide handy information to the Field Officer, about the methods of observation, record, and analysis of pressure pipe data of weirs on permeable foundations. Pressure pipes are installed on a number of weirs and other irrigation works such as level crossings and siphons etc., and pressure observations are recorded as a matter of routine to keep an eye on the safety of the work.

A comprehensive treatment of the theory of Design of weirs on permeable foundations is contained in C. B. I., publication No. 12[1] and the methods of observation and record of pressure pipe data have been described by Khosla in C.B.I., publication No. 8[2]. In none of the above publications have the methods of analysis of pressure pipe data been clearly and seperately treated as a subject in itself, though it is dealt with in the former by implication in the treatment of the theory of flow under weir structures and comparison of model and prototype data.

It it not the purpose of this paper to go into the detailed mathematical treatment of the subject, nor into the experimental details, but only into the basic physical principles connected with the problem of analysis of pressure pipe data, with a view to afford clearer understanding of the subject and to make the paper self sufficient in the matter of analysis. Developments which have occurred after the preparation of C B.I. Publication No 12, especially with reference to the analysis of pressure pipe date, are dealt with in greater detail here.

2. DATA REQUIRED FOR ANALYSIS.

The data required for keeping a check on the safety of a work on sand foundations, from the point of view of flow under the work, fall into two main categories.

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(1) Khosla, A. N. Bose, N. K & Taylor, E. M. Design of weirs on permeable foundation C.B.I. Publication No. 12

(2) Khosla, A. N Observation & Record of pressures below works on permeable foundation.

(A) Basic Design data of the work.

(B) Observational flow data.

3. (A) BASIC DESIGN DATA OF THE WORK.

This comprises the design of the weir, the geological formation of the substratum from place to place under the work, and the lay out of the pressure pipes.

The following basic record is an essential part of the folio to be maintained for pressure pipe data.

(1) A complete plan of the weir showing the position of the sheet piles and pressure pipes and a statement showing reduced levels defining the position of the strainers

(2) Cross Section of the weir showing the location and reduced levels of the strainers. In the case where the design changes across the weir, it is desirable that the pressure record folio should have the outlines of the bays shown in vertical series with the gate line forming a vertical axis.

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(3) Results of tests holes bored at different places to determine the condition of geological substratum should be suitably represented diagramatically.

(3) A record of all additions and alterations to the weir, which directly or indirectly affect the subsoil flow or the hydraulic stability of the work should be kept, with dates of completion. The alterations should also be shown on the plans and X—Sections.

(5) A record should be kept of cavities or springs detected from time to time, and also of all cement or sand grouting carried out. The amount of material grouted in, on each occasion should be recorded.

4. (B) OBSERVATIONAL FLOW DATA.

The following observations are required for a proper analysis of the pressure pipe data.

(1) Upstream water level.

(2) Downstream water level.

(3) Temperature of river water, upstream and downstream, at such a suitable depth below the surface at which it is approximately constant and unaffected by the atmospheric or surface temperature.

(4) Temperature of Subsoil water in the first, middle, and the last pipe in each line of pipes.

(5) Water levels in all pipes.

(6) The depth of silt on the upstream and downstream floor and on the permeable portion upstream and downstream of the impermeable portion.

(7) Depth of scour upstream and downstream, if any.

(8) Spring levels at the two flanks of the weir.

5. THE METHOD OF OBSERVATION.

Upstream and downstream levels are read from gauges suitably placed. The temperature of water well below the surface, should be taken by a maximum-minimum thermometer, and not by an ordinary thermometer. For observing the temperature of the subsoil water in the pipes, also, a maximum-minimum thermometer, should be employed which should be able to pass freely into the pipes. A suitable type is the Six's pattern maximum-minimum thermometer manufactured by Messrs Negretti and Zambara, London.

The water levels in pipes can be easily read by means of a bell sounder, consisting of a solid brass rod about $3\frac{1}{2}''$ long and $\frac{7}{8}''$ diameter, ending in an inverted cup of $1\frac{1}{4}''$ diameter. A swivel is screwed on to the upper end, to which a steel tape is attached. The steel tape with the cup at its lower end is lowered into the pipe, and as soon as the sounder hits the water surface, a definite sound is produced which can be heard distinctly even in the roar of the standing wave. By slightly moving the cup up and down, the position where the sounder touches the water surface in the pipe can be determined with an accuracy of 1/8 of an inch. Knowing the reduced level of the upper end of the pipe, the reading on the tape corresponding to the top of the pipe (at the position where the sound is heard) plus the length of the sounder below the zero of the tape, will give the depth of water in the pipe below its top. The reduced level of the water surface in the pipe is thus determined.

Yet an other method which could be used is to paste a thin layer of MeCabe water level indicator (a sort of a paste manufactured by Renick & Mahoney, Inc New York, which changes colour in water)

on a steel tape with a weight at its lower and to keep it stretched, the tape is lowered into the pipe, the dipped portion of the tape changes colour giving a sharp demarcation line. The R. L. of the water in the pipe can be measured with reference to the R. L. of the top of the pipe.

The depth of silt or scour upstream and downstream can be obtained by soundings.

6. SOURCES OF ERRORS.

The accuracy of an observation lies in the degree of skill displayed by an observer in noting the behaviour of the pipe and in avoiding different sources of errors. This is very important if the analysis is to yield correct results. A wrong reading due to abnormal functioning of a pipe, or a serious observational error passed as a genuine reading, will cause in accuracies in the final results, making it almost an impossible task for an analyser to differentiate between an observational error and a hydrodynamical effect on pressure on account of cavity or a loose contact etc. Carelessness in observations will only create confusion which can vitiate the very purpose of analysis.

The errors can be divided into the following main groups :

(*a*) error of installation.

(*b*) errors of observation,

and (*c*) errors due to faulty working of pipes.

(*a*) The errors of installation include the following :

(1) Sinking of strainers to depths different from those designed.

(2) Inadvertant stamping of wrong number on the blind plugs and cover plates in the cluster of pipes, or a careless change of plugs.

(3) Loose joints, leading to leakage.

(*b*) The errors of observation may comprise :

(1) Mistakes in reading generally by a whole foot.

(2) Failure to add the length of the sounder.

(3) Fluctuation of water level in the pipe, due to observation having been made when the upstream and downstream

levels are not steady, or when the process of silt deposition or erosion is still not complete.

(4) Incorrect record of temperature by maximum and minimum thermometer if raised by a jerk.

(*c*) Abnormal functioning of a pipe, in recording correct levels, can be due to any of the causes given below :—

(1) The strainer is choked, fully or partially, and either does not record correct levels, or takes a very long time to attain proper level after a change in the upstream or downstream level has taken place.

(2) Partial blockage of a pipe due to anything falling in when the cover plate is left open.

(3) Air looks forming in bends.

(4) Damage to the strainer at the time of installation or clearance, allowing sand to enter the pipe.

(5) Incorrect record due to leakage in the under water portion of a pipe subjected to differential head.

(6) Inability of the pipes to respond accurately to change in level at heads below 3 feet.

7. PRECAUTIONS AND METHODS TO CHECK AND CORRECT ABNORMALLY FUNCTIONING PIPES.

The erratic behaviour of the pipes can be checked only by patient testing. It is always helpful to test the pipe by filling it with water upto the top level, noting the rate of fall of level in the pipe, and then blowing into the same pipe to decrease its level and again noting the rate of rise. The fall and the rise of levels in the pipes when plotted against time will tend to approach asymptotically towards a common value, if the pipe is not fully or partially choked. If the pipes are partially choked the rate of fall or rise will be much slower than in the case of freely working pipes, as shown in fig. 1 (*a*). If some sand is doposited in the strainer due to damage or from any other cause the water level remains low when it rises, and remains too high when it drops. The slower the rate of rise or fall, the more would the pipe be choked. The air locks will also have a more or less similar effect. The above explanation shows how partially chocked or damaged pipes can cause errors, especially when the readings are taken after rising or falling of the upstream or downstream levels, notwithstanding the fact that the transmission of pressure is considered to

be almost instantaneous. It also indicates that pressure observations should be taken when the upstream and downstream levels have remained steady for at least 24 to 36 hours. Observations should be taken not less than 24 hours after clearing the pipes.

Airlocks or sand deposition in the pipe can be removed by suction from the pipe by means of an exhaust pump, or sometimes, by blowing water into it under pressure. Excessive suction or blowing should not be done when it is suspected that the strainer is even slightly damaged, for, in that case, a cavity is likely to be created.

It is advisable that each pipe should be tested in the presence of, or by, a responsible officer thrice a year, and that a record of the functioning of the pipe should be kept. With the history of each pipe so available, the analysis of pressure pipe data will be far more correct than it could otherwise possibly be.

8. THE FREQUENCY OF OBSERVATIONS.

Fortnightly, or at least, monthly, observations would, for all practical purposes, be sufficient, but from what has been said before it is clear that it is not the frequency of observation, but the accuracy, and the personal attention given to each pipe, that would yield useful and reliable results.

9. RECORD OF DATA.

The observational results may be recorded on standard forms in the folio for pressure record. As the observations are to be taken only twice a month, the entire data for one year, of one line of pipes in a particular bay, will be recorded on two facing pages. If the weir consists of different sets of bays differing in design from each other, as in the case of Islam weir, the results of all the lines of pipes in similar sets of bays can be grouped together. The folio should be provided with graph sheets for plotting of the data. This aspect of the record will be described more fully after dealing with methods of analysis. The folio should preferably be of the loose-leaf type.

10. PURPOSE OF PRESSURE PIPE OBSERVATIONS & ANALYSIS.

The main purpose of the observations and analysis is to keep strict watch over the safety of the work. The essential considerations for the safety of a weir on permeable foundations are :

(A) The uplift pressure at any point of the weir must not exceed the downward force comprising the weight of the masonry and of the water if any, over the floor.

(B) To safeguard against undermining, the gradient at the toe of the weir-termed the exit gradient — should not exceed a certain safe limit, depending on the nature of the soil and the form of the structure.

(C) No cavities, piping, or hollows, should develop under a weir.

11. A. UPLIFT PRESSURE AND THE SAFETY OF THE WORK.

The main purpose of the weir is to head up supplies in a river to a level sufficient to feed the offtaking canal systems. In winter, when rivers pass low supplies, it is necessary to completely or partially hinder the flow over the weir by the barrage gates; in summer when the gates are up, the weir causes an afflux of varying order depending on type and proportions of design. In either case there is a difference in the upstream and downstream water levels. This difference in water level, generally known as the 'head' across the weir, causes seepage to take place in definite stream lines; through the permeable medium below the weir. In flowing through a granular medium resistance is encountered by the flowing fluid, resulting in a pressure drop or loss of head along the length of percolation. Figure 1 (*b*) shows a stream tube bounded by the underface of the weir and a stream line ABCDEFG with strainers embedded to record pressures at points 1, 2, 3 — — — & 12. If the difference of upstream and downstream levels is denoted by a head H, across the weir, different pipes will record levels at different heights, h_1, h_2... ...h_{12} above the downstream level taken as the datum, giving a continuous line termed the Hydraulic gradient. The effect on the hydraulic gradient, of factors other than the form of the base of the structure and the head across the weir, will be discussed at a later stage. It may be pointed out here that for a given form of the base of the structure, a plot of hydraulic gradient in terms of head of water over the downstream water level would change with H — the head across the weir; the drop between adjacent pipes will increase with increase in head and vice-versa. For convenience of analysis, it is essential to express the result in such a manner as to render it independent of the upstream or downstream water level, and as a unique function of the form of the base of the structure considered as resting on homogeneous foundations. For this purpose, the results are usually expressed, in non-dimensional form, as a ratio of the head above the downstream water level to the total head across the weir, and are expressed by percentages, as :

$$\phi = \frac{h}{H} \times 100 \qquad \ldots\ldots(1)$$

It should be noted that *h* increases directly with H, and the ratio ϕ at a given point under a structure remains constant for a given type of structure

and type of sub-stratum. The hydraulic gradients are generally plotted in terms of ϕ. If ϕ is is known for any point of a structure, the upward thrust or the head in feet of water (h) can be calculated for a given value of H - the head across the weir, — by :

$$h = \phi . H/100 \quad \ldots\ldots (2)$$

The stability of the weir floor at any point, (fig. 1 (b), depends upon the forces acting there. These forces are :

(1) An uplift pressure equal to h in feet-head of water.

(2) The weight of masonry and that of water, if any over the floor.

If the former exceeds the latter, blowing up of the floor can result. In designing a new work, therefore, an optimum thickness of floor should be provided at different points depending on the magnitude of the uplift pressure which is determined by the form of the structure and the head. For an existing work there would be a particular value of safe uplift pressure (or safe value of ϕ) at any point of a given design and thickness of floor and a maximum permissible head which should not be exceeded.

12. SAFE VALUE OF ϕ FOR THE STABILITY OF THE WEIR FLOOR.

For any particular point of the floor two cases arise for consideration :

(1) When the downstream floor is dry.

(2) When it is under water.

Figure 1 (C & D) delineate the conditions governing the safety of the work at floor level, FL, when downstream water level is lower and higher than FL respectively.

From equation (1).

$$\phi = h/H \times 100$$

or, the head in feet of water above downstream water level is

$h = \phi \cdot H/100$ or $h = (\phi/100)$ (USL − DSL)

or $h = h_1 + L = (\phi/100)$ (USL − DSL)

where h_1 is head in feet of water above point FL and L is the elevation of point FL with respect to the downstream water level [(refer fig.1 *c*]

Thus $h_1 = (\phi/100)\ (USL - DSL) - L.$

But $L = (FL - (DSL)$

therefore $h_1 = (\phi/100)\ (USL - DSL) - (FL - DSL)$

$= (\phi/100)\ (USL - DSL) + DSL - FL$ ..... (3)

(1) If DSL is lower than FL, as in fig. 1 (*c*), the floor will be dry; its weight in terms of head of water will be $P_d \times$ t sec. θ Where P_d is the ratio of the weight of unit volume of the material to that of water *i.e.* $P_d = \frac{W_m}{W}$ or in foot lbs. system $P_d = \frac{W_m}{62{\cdot}4}$

The condition for safety will be :

$P_d \times t \text{ Sec } \theta > h_1 + t \text{ Sec } \theta$ ..... 3 (A)

Where θ is the angle made by the floor with the horizon. When θ is small, the thickness of the floor resisting the uplift pressure, *i.e.* t Sec. θ can be taken as equal to t. It has been shown [3] that for slopes flatter than 1 in 3, the difference is not appreciable, though it is considerable for slopes of 1 : 1 and 1 : 2. For slopes flatter than 1 in 3, the condition of stability is obtained from equation 3 (A) as :

$$h_1 < (P_d - 1)\ t \quad \ldots\ldots (4)$$

Substituting the value of h_1 from (3)

$(P_d - 1)\ t > (\phi/100)\ (USL - DSL) + DSL - FL$

$$\text{or } \phi < \frac{(P_d - 1)\ t + (FL - DSL)}{(USL - DSL)} \times 100 \quad \ldots\ldots (5)$$

$$\frac{(P_d - 1) \times \text{thickness of floor} + \text{elevation of point FL above DSL.}}{\text{Total head over the weir}} \times 100$$

For this case, ϕ and t being constant for a particular point, the worst condition will be, when

$\frac{FL - DSL}{USL - DSL}$ is smallest *i.e.*, when DSL approaches FL.

That is, the value for worst condition will be

$$\frac{(P_d - 1) \times t \times 100}{USL - DSL} \quad \ldots\ldots (6)$$

(2) When DSL is higher than FL — the point under consideration (fig. 1 (*d*), the floor is submerged and its effective weight in term of

(3) Annual Report, Punjab Irrigation Research Institute, December 1944.

head of water is given by :

$$\rho_s = \frac{W_m - W}{W}$$

i e., by the ratio of the loss of weight of unit volume of material in water to water to the weight of unit volume of water. In the foot — lb system $\rho_s = \frac{W_m - 62{\cdot}4}{62{\cdot}2}$

The downward pressure in feet head of water is :

$$\rho_s \times t + DSL - FL$$

and the upward pressure in feet head of water is :

$$(\phi/100)\ (USL - DSL) + DSL - FL.$$

so the condition of safety is :

$$\rho_s \times t + DSL - FL > (\phi/100)\ (USL - DSL) + DSL - FL, \text{ or}$$

$$\phi < \frac{\rho_s t \times 100}{USL - DSL} \qquad \ldots\ldots(7)$$

The worst case would occur when DSL is minimum or DSL = FL.

The above formulae provide a method of plotting the pressure pipe results with a view to keep a check on the safety of the weir from the point of view of uplift pressure and thickness of floor. The method of plotting would consist in selecting suitable pipes, say immediately downstream of the gates, near the toe of the glacis, and near the downstream end of floor, and calculating value of ϕ for each pipe for the worst condition of flow from the above formulæ. The observed values of ϕ from the weir are plotted for each of the above points seperately against time as abscissa. The maximum safe value of ϕ are also plotted as straight lines on the graph. If the observed ϕ does not exceed the limiting safe value of ϕ, the work can be assumed to be safe at the point under consideration.

13. MAXIMUM PERMISSIBLE HEAD FROM THE POINT OF VIEW OF STABILITY OF WEIR.

For dry downstream floor, the condition of safety is :

$$\phi = \frac{(\rho_d - 1)\ t + (FL. - DSL)}{H} \times 100.$$

For the worst case, when DSL approaches FL. it is :

$$\phi = \frac{(\rho_d - 1)\ t \times 100}{H}$$

Taking the theoretical value of ϕ for a normal weir, the maximum permissible head for the worst condition is given by :

$$H \text{ max.} = (100/\phi) \times (\rho_d - 1)\, t$$

The method of determining the theoretical value of ϕ will be given later.

14. B. EXIT GRADIENTS & THE SAFETY OF A WORK.

Figure 2 (a) shows the conditions determining the stability of a small element of the substratum of length *dl* and cross sectional area dA. The difference of pressure between the faces A & B, or the force acting on the cylinder, is $pdA - (p + dp)\, dA = - dpdA$, or force per unit volume of cylinder $= - \frac{dp\, dA}{dl\, dA} = - \frac{dp}{dl}$. This means that the force per unit volume is equal to the pressure gradient at the point. Elements at different positions will be acted on by this force in different directions. The easiest point for starting a displacement of the particles is near the tail end, and obviously this force at this point, is in the upward direction. This force will lift up the particles, if it is more than the submerged weight of the latter. Let us consider the stability of the element at the tail end. The force acting per unit volume in the upward direction is $F = - \frac{dp}{dl}$. The submerged weight acting downwards

$$= Ws = w\,(1 \epsilon)\,(\rho - 1)$$

Where w = weight of unit volume of water

ρ = specific gravity of sand particles

ϵ = pore space in unit volume.

Since both F and Ws, act vertically at the tail end, than, when F is equal to W, the particles of sand would be just stable; as soon as F exceeds W, the sand particles at the toe start lifting. This is the critical gradient; any further increase in F will cause blowing at the tail end. The critical, or the flotation, gradient for w = 1, $P = 2.65$ and $\epsilon = 0.4$ (pore space of 40%) is :

$$F = Ws = 1\,(1 - .4)\,(2.65 - 1) = .99 \text{ (say 1)}$$

It will be seen that the downward force is a function of pore space and specific gravity of the sand. The upward force, or the pressure gradient at exit, depends on the head across the weir, the depth of the downstream sheet pile, or the floor depression (D), and the length of the floor.

In ordinary structures on sand foundation, the critical gradient is almost impossible of occurrence, for, usually, the maximum head for flat

weirs on sand foundations varies between 10' to 25'. It is very unlikely to occur even in the case of depressed-floor weirs, which is the worst case and one of only academic interest. Even in the case of depressed floor weirs, 100 long, under a head of 20 feet, the exit gradients, for depressions of 2' and 1', will be 0.59 and 0.84 respectively. Evidently no undermining should occur from theoretical considerations. However, some factors inherent in the working system or the subsoil formation, acting separately or in conjunction with each other, may create conditions suitable for the realisation, of critical exit gradient. The factors are :

(*i*) A scour hole extending right upto the toe wall.

(*ii*) The presence of a local surge or a wave-working like an intermittent pump — which can increase the exit gradient many times.

(*iii*) The effect of sudden application or reduction of head, and high spring level.

(*iv*) Non-homogeneity of soil-zones of very porous material, or local intrusion of clay beds — which may induce concentration of flow.

(*v*) Faults and fissures in the subsoil formation.

Due to the above uncertain factors coming into the problem, Khosla [4] proposed that instead of taking 1 : 1 as the critical gradient for design and analysis of existing works, the following factors of safety, depending upon the type of the subsoil grade may be adopted.

Shingle	1/4 to 1/5
Coarse sand	1/5 to 1/6
Fine sand	1/6 to 1/7
Clay	1/3 to 1/5

It is to be noted that the safety factors are purely arbitrary without any physical or experimental basis, but these should be adhered to till further experiments place them on more rational grounds.

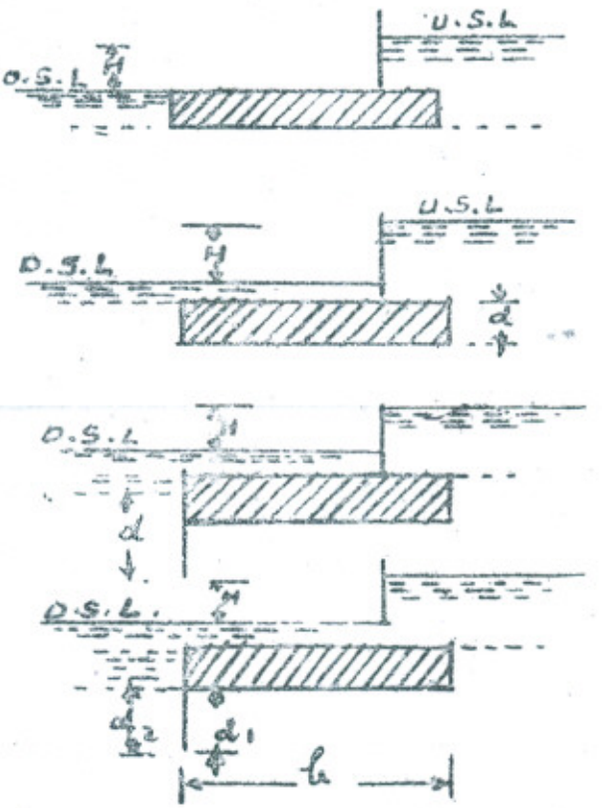
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(4) Khosla, A.N. 'Note on the Design of Hydraulic structures resting on clayey soils' C.B.I. Eleventh Annual Meeting of the Board 1940 P. 156.

15. DETERMINATION OF EXIT GRADIENTS FOR SOME STANDARD FORMS OF WEIRS.

The formulæ for determining the exit gradients for standard forms of weir are tabulated below from C.B.I. Publication No. 12.

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Only the standard forms 3 and 4 are of practical importance. To calculate the exit gradient, a curve between $\frac{1}{\pi\sqrt{\lambda}}$ and $\alpha = b/d$ is reproduced, from C.B.I. publication No. 12 in fig 2 (b). Two formulæ have been given for working out the exit gradient for standard form 4. The exit gradient can be more accurately obtained from the former, than the latter, but since the difference is small the latter formula, giving simple curves, will do for all practical purposes.

16. DETERMINATION OF SAFE HEAD FROM THE POINT OF VIEW OF EXIT GRADIENT.

For a given design, say with a given length of floor (b) and

down-stream sheet pile depth (d), there would be a safe limit of the head (H) which should not be exceeded. To calculate this, curves between H/d and b/d have been drawn for different exit gradients, after Kanwar Sain (5), in fig (3) The method of calculating maximum permissible head consists of the following steps :

(*i*) Select appropriate value of G_E for the subsoil grade .

(*ii*) Read the value of H/d against the value of $\propto$ = b/d for the structure, for selected G_E. If this is termed X, them H/d = X or H = d.X

17. C. SAFETY OF A WEIR WITH CAVITY FORMATION OR LOOSE CONTACT WITH SUBGRADE.

The effect of a cavity or of loose contact is to draw the flow towards such place and to reduce the effective length of the floor.

The effect of a cavity on the hydraulic gradient for a simple case of a pileless floor was studied in the Irrigation Research Institute laboratory and the results are shown in fig. 4 (*a*). The indications are :

(*i*) The gradient through the cavity is almost horizontal, as it should be, due to lack of resistance to flow.

(*ii*) Immediately above and below the cavity the gradient is steeper than the normal gradient.

(*iii*) If the cavity happens to be near the upstream or downstream end of a floor, the Hydraulic gradients at the entry or exit are steepened to a greater extent than for a cavity at any other place.

(*iv*) The effect of the cavity is local and cannot be felt at some distance from the cavity.

Due to the fact that the hydraulic gradient is steepened to a greater extent when the cavity or loose contact happens to be near the exit, the exit gradient in such a case is very likely to exceed the permissible limit. Under the circumstances the cavity will work upstream causing undermining and resulting in complete failure of the work.

One of the purposes of analysis of pressure pipe data is to get an indication of the presence and location of cavities or loose contacts underneath a weir. Since the cavity affects the hydraulic gradient, the method of detecting a cavity or a loose contact under a weir consists in comparing the observed gradient with the theoretical hydraulic gradient. If, after eliminating the effect of silt blanket, (to be described later), while keeping in view the functioning of the pipe, the pecularities

(5) Kanwar Sain and Madan Lal. Design curves for Irrigation structures on permeable foundation'.

enumerated above are noticeable in the comparison, a cavity or a loose contact can be inferred to be existing at the position of flat gradient. The importance of the proper functioning of a pipe is again stressed because a wrongly recorded, or recording, pipe is liable to show gradients flatter or steeper than normal due merely to error and without any ascribable physical reason.

18. METHOD OF DETERMINING HYDRAULIC GRADIENT OR THEORETICAL VALUE OF ϕ UNDER A WEIR SECTION ON HOMOGENEOUS PERMEABLE FOUNDATIONS.

It has been shown before that for checking the safety of a given weir floor against blowing up, or for obtaining indications of the presence of cavities underneath it, it is necessary to determine the Hydraulic gradient or the ϕ - line. The calculation of theoretical uplift pressure is, therefore, a pre-requisite of analysis of pressure pipe data for the stability of the weir. It has been possible to deduce exact mathematical solutions for determination of the uplift pressure, only for simple forms of weirs on homogeneous foundations. A complete mathematical solution for a weir section with three or four sheet pipe lines and having sloping or level aprons, is not yet available, but by "the method of Independent Variable", (6), (7), it is possible to determine uplift pressure with a fair degree of accuracy. The method of calculating the uplift pressure as evolved by Khosla, Bose and Taylor, is briefly given here to enable the reader to make the calculations himself when required.

In the case of a complex weir with a number of pile lines and sloping and level floors, the method of determining uplift pressures requires the splitting of the complex weir section into elementary forms i.e. the entire length of the floor is successively considered with each pile line, in turn, taken as unique, assuming the other pile lines to be non-existent. For each elementary form, the pressures are determined for key points E, C, and D in fig 4 (*b*), where E. is the external junction point of the floor and the pile line i.e. on the upstream corner of pile and floor, C is the internal corner at the junction of floor and the pile line on the downstream side, and D is the bottom point of the sheet pile.

The values of ϕ at the key points are read from fig. 5 (*a*) Full details for using figure 5 (*a*) are given thereon.

The value of ϕ for points E & C are then corrected for :—

(*a*) the mutual interference of piles.

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(6) Khosla, A.N., Bose N. K. & Taylor E.M. Loc. cit.

(7) Malhotra J.K., & Taylor E. M. Note on Khosla's principle of independent variables; Punjab Irrigation Research Publication Vol. II, No. 20.

(*b*) the thickness of the floor, and

(*c*) the slope of the floor.

(a) *The mutual interference of piles.*

The effect of mutual interference of pile lines can be worked out within about 2 per cent of the experimental values by a simple formula, (see fig. 5 (*b*),

$$C_1 = 19 \sqrt{\frac{D'}{b'}} \left(\frac{d + D'}{b} \right)$$

where C_1 = the correction to be applied as percentage of head,

b' = distance between two piles,

D' = the depth of pile, the influence of which has to be determined on the neighbouring pile of depth d,

d = depth of pile on which the effect of pile (D) is required to be determined, and

b = total floor length.

The limitations of the equation are :

(*i*) It does not apply to the effect of an outer pile on an intermediate pile if the latter is equal to or smaller than the former and is at a distance less than twice the length of the outer pile

(*ii*) The correction (C_1) is additive for points in the rear and substractive for points forward in the direction of flow.

(b) *Correction for the thickness of floor.*

The values of ϕ so obtained will refer to point E′C′ but it is required to find uplift pressure at E and C, (fig 5 c). For this purpose, it is necessary to determine pressure drop along each face of sheet pile line. Assuming the drop to be linear along each face of pile line, the drop in the value of ϕ along the upstream face of the pile line fixed at a distance b1 from the upstream edge of the floor, as determined empirically, is $(70 - 40 \times \frac{b1}{b})$ per cent, in all cases, of the total drop along both the faces of the sheet pile.

From the values of ϕE′, ϕD and ϕC′ determined from the curves in fig. 5*a* the values of ϕE and ϕC are calculated as below.

$$\phi_E = \phi_E' \text{ corrected for depth } = \frac{t}{d}(\phi_E - \phi_D)$$

$$\phi_C = \phi_C' \text{ corrected for depth } = \frac{t}{d}(\phi_D - \phi_C)$$

(*c*) *Corrcetion for slope.*

Where a sloping floor exits, a correction for slope is to be applied to the results. The correction is given, with reference to the fig 5 (*d*) by :

$$C_s = \frac{b_3}{b'} \times K,$$

where b_3 = the horizontal length of the slope.

b' = the spacing between the sheet piles, and

K is the slope correction factor read from fig. 5(*d*) for difference slopes.

The percentage correction found from this curve has to be multiplied with b_3/b' ; where b_3 is length of slope along horizontal and b' is spacing of piles.

Since the floors sloping up or down in the direction of flow, decrease or indecrease the values of ϕ under a horizontal floor with the same base ratio the percentage correction for the glacis sloping down and up in the direction of motion would be plus and minus respectively.

19. CONDITIONS UNDER WHICH THE ABOVE METHOD FOR CALCULATING ϕ OR HYDRAULIC GRADIENT IS ADMISSIBLE.

The above method of calculating values of ϕ for any point under a structure applies only under the following restrictions :—

(*a*) The flow is two dimensional *i.e.*, only in the XY plane and not in the direction of Z axis (*i.e.*, the horizontal axis perpendicular to the direction of flow)

(*b*) The substratum is homogeneous and deep ; this implies a reasonable homogeneity of particle size, its packing, and resulting pore space.

(*c*) There is no silt blanket, or semipervious layer upstream or downstream of the impervious floor.

(*d*) There is no temperature gradient in the direction of or against the flow.

A variation in each one of the above factors could cause departures of various magnitudes in the observed values of ϕ, from the theoretical values obtained for simplified conditions. In an analysis of pressure pipe data it is assumed that none of the above disturbing factors are present, or that the few factors which are present, can be eliminated. Conversely any variation in the observed values of ϕ from theoretical value, is due to any of the above causes working in conjunction or separately.

20. METHOD OF ELIMINATING, OR FINDING THE TREND OR THE MAGNITUDE OF THE EFFECT OF THE ABOVE FACTORS ON THE UPLIFT PRESSURES OR HYDRAULIC GRADIENTS.

Unfortunately it is not possible, in the present state of knowledge, to eliminate or determine the effect of all the factors causing changes in the values of ϕ. However, for some cases, the possible trend of the effect, and in others the probable magnitude also, are known, and for one case — which is very important and causes considerable variation in the values of ϕ — the methods of eliminating the effect are also known. A brief discussion of the disturbing factors, which is important from the point of view of analysis, is given below.

(*a*) *Three dimensional flow.*

The problem of three dimensional flow is yet to be solved, and the value of ϕ obtained by the theoretical method—which tacitly assumes two dimensional flow — will not strictly hold good. It is, therefore, necessary to know when a flow along the Z axis, is likely to occur. Such possibilities occur in actual practice for the following cases :

(*i*) Flow under the side bays of a weir, where, due to high level in the canal, water level on the flanks may be different from the upstream and downstream levels across the weir, thus inducing a cross flow at the flanks.

(*ii*) When a weir changes in design along its length, so that the pressures in any bay as given by the theory differ from those in an adjacent bay thus evidently causing cross flow and thereby changing the normal pressure distribution.

The observed pressures, at the flank bays, and in the bays where the design of the underface of the weir changes, will not stand strict comparison with the theoretical values of ϕ. Only lines of pipes in bays away from such structural changes can be usefully analysed in comparison with

the theoretical ϕ value. The effects for such complicated structures, can however be determined from hydraulic models in case of a serious doubt about the safety of a work.

(*b*) **The non-homogeneity of the substratum.**

It is not possible to deal with all the variations in the substratum which can possibly occur; a few simple and common cases of non-homogeneity are, however, dealt with qualitatively. The exact uplift pressures for such cases can only be determined from hydraulic models, but the general trend of changes of uplift pressures from the normal gradient for typical cases of non-homogeneity can help in the proper understanding of departures in observed hydraulic gradients from theoretical ones. The non-homogeneity can be broadly divided into three main types :

(*i*) Discontinuity of media across the lines of flow under the weir i.e. vartical or oblique stratification of geological faults etc.

(*ii*) Discontinuity occuring parallel to the direction of flow under the weir, or horizontal stratification etc.

(*iii*) Conglomerations of sand and clay, or sand and shingle.

I—Vertical or oblique stratification.

Material considerably coarser or finer than the average subgrade, occurring across the path of percolation below the weir, would cause a change in the flow pattern and pressure distribution. Evidently, a layer of coarser grade or of shingle in contact with the underface of a weir will offer any easy path to flow lines, and the lines of flow would be drawn on to it, as shown in fig 6 (*a*). If such a layer exists under part of the length of the weir, the effective length of the weir will be decreased. The observed hydraulic gradients are compared with those for homogeneous foundations in fig. 6 (*c*). The main indication is that the gradient through the medium of greater permeability is flatter than the normal gradient, and upstream of it the gradient becomes steeper.

If, however, the medium obstructing lines of flow is considerably finer than the general subgrade, or consists of an oblique on vertical clay layer, the pressure on the floor would increase or decrease according as the oblique or vertical clay layer is near the downstream or upstream end of the floor; an oblique or vertical clay layer near the downstream end will act analogously to a downstream sheet pile, and will pile up pressure on the floor; while a clay layer near the upstream end will tend to reduce pressure on the floor.

II—Horizontal stratification.

(*i*) Works Resting on sand underlaid by shingle or coarser material.

Evidence so far available shows that if the medium changes in depth from less to greater permeability (or is horizontally stratified), the difference in the hydraulic gradient from that of a work resting on a homogeneous bed, is not as great as in a case of vertical stratification. Experiments carried out in connection with Marala weir, which was known to be build on a 25 foot thick layer of sand foundation underlaid by shingle, gave the flow lines and Hydraulic gradient as shown in fig. 6 (*b*) & 6 (*d*). A comparison with the hydraulic gradient for a model on a homogeneous foundation of great depth, shown in the same figure, bears out the above conclusion.

(*ii*) Works Resting on sand, underlaid by an impermeable clay stratum of finite depth.

To this type belongs the Sulemanki weir and Bay 4 of Khanki weir. If a sheet pile is imbedded into the clay stratum or pierces it, cutting off the upstream portion from the downstream, the sheet pile will cut off all the head, and no pressure will be transmitted to the downstream floor. The pressure line downstream of the floor will follow the downstream water level and will be completely independent of the upstream level.

(*iii*) Woks Resting on sand underlaid by an impermeable clay stratum of infinite depth.

For a pileless floor resting on sand of finite depth, the uplift pressures have been found [8] to be less under the downstream half and more under the upstream half, than in the case where the sand is infinitely deep, though the difference is not appreciable unless the floor is at least four times as long as the depth of sand. When the sand depth is very small, the fall of pressure tends to be very nearly proportional to distance except for a certain rapid loss at entry and exit. This incidentally explains the linear relationship of clibborn which later appeared as the creep hypothesis.

For a floor with a pile driven into the clay stratum, the effect would be similar to case (*ii*) above.

III—Conglomerations of subgrade.

Works on a mixture of sand and clay or sand and shingle, more or less uniformly distributed.

(8) Malhotra, J. K. A horizontal flush floor on a finite substratum, chapter V of thesis for Ph. D. (Unpublished).

No experimental data exist for the case of works resting on sand and clay to show the trend of variation from the normal uplift pressures. A few experiments carried out on a model of Kalabagh weir on a foundation of sand and shingle mixed in different ratios, (fig 7), show that the uplift pressures are generally higher when a weir rests on a mixture of sand and shingle, than those for homogeneous sand foundations.

(21. C) THE EFFECT OF SILT BLANKET ON THE UPLIFT PRESSURES, AND METHODS OF ELIMINATING THE EFFECT.

The effect of an upstream silt blanket on the uplift pressure is explained with reference to an experiment carried out by the author under simple and controlled conditions A model of a pileless depressed floor 4′ long, (fig 7 (*b*), was constructed on a homogeneons bed, and pressure observations were made for no-silt deposits, and for silt blankets of 1/8″, 1/4″ 3/8″ and 1/2″ thickness respectively. The hydraulic gradients, plotted in fig. 7 (*c*), indicate that the values of ϕ for the point immediately below the silt blanket decrease with an increase in the thickness of the silt blanket. It has been shown earlier that a comparison of the observed ϕ line (for no-silt blanket) with the theoretical gradient, can give an indication of the pressure of cavities under a weir. A similarity of hydraulic gradients will indicate a sound contact and absence of a cavity, while an observed gradient which is flatter between any two adjacent pressure points than the theoretical gradient will indicate a cavity or loose contact, subject to the absence of other affecting factors. Since a silt blanket generally flattens the gradients, a complication crops up in the analysis and it becomes essential to eliminate the effect of a silt blacket before an analysis is made. This elimination can be effected in various ways.

The graphical method.

The graphical method for plotting the results is given in C.B.I. Publication No. 12 (See figure 7 (*d*) ϕ values are plotted along the Y axis as shown, and along the X-axis any arbitrary length OX is taken, and the point X is joined to the point Y where $\phi = 100$. Values of ϕ, without any silt blanket, for different pipes, are projected horizontally on to the line XY and from these points perpendiculars are drawn to the line OX. The intercepts on the line OX represent the relative positions of the pipes as regards the pressure cut-off. The results of the experiment described in thc subpara above are plotted by this method in fig. 7 (*d*). This method of plotting shows a linear relation between the value of ϕ and the pipe number.

TABLE I.

Thickness of silt blanket Pipe No.	1 1/8"		1/4"		1/8"		1/2"		Model experiment without silt.
	With silt	calculated	With silt	calculated	With silt	calculated	With silt	calculated	
1	57.1	96.5	37.9	96.6	20.5	96.4	13.1	96.4	96.9
2	53.9	91.1	31.1	88.6	19.2	90.2	12.2	89.8	91.2
3	47.2	79.8	27.5	75.6	16.5	77.6	10.6	78 8	79.3
4	41.6	70.3	24.9	71.9	14.5	68.1	9.2	67.7	74,4
5	38.8	65.6	22.8	65.0	14.0	65.8	9.0	66.2	65.3
6	—	—	—	—	—	—	—	—	—
7	31.0	52.4	18.5	53.7	11.4	53.6	7.0	51.5	53.3
8	28.3	47.8	16.2	46.2	10.0	47.0	6.2	45.6	48.0
9	25.3	42.8	14.7	41.9	9.4	44.2	6.1	44.8	43.2
10	22.2	37.5	13.2	37.6	8.5	39.9	5.2	38.3	38.2
11	18.9	31.9	11.1	31.6	7.1	33,4	4.3	31.6	31.9
12	15.1	25.5	8,9	25.4	5.6	26 3	3.4	26.2	25.0
13	10.0	16.9	6.1	17.3	4.0	18.8	2.4	17.7	18.5
14	4.5	7.6	2.8	7.9	1.9	8.9	1.0	7.3	7.2

If by this method of plotting, the values of ϕ, for all the points below a silt blanket, lie on one line, the hydraulic gradient would be normal. The draw back in the method is that it does not clearly indicate regions of flatter gradient as compared to normal gradient. Another method of eliminating the effect of silt blanket is therefore described.

Formula to eliminate the effect of silt blanket.

The formula follows obviously from the graphical method of eliminating the silt effect by plotting a graph between ϕ and the proportional relative position of pipe, as described above.

From fig. 8 (*a*) we have :

$$\frac{\phi_{ox}}{\phi_{sx}} = \frac{\phi_{ol}}{\phi_{sl}}$$

$$\text{or } \phi_{sx} = \frac{\phi_{sx}}{\phi_{sl}} \times \phi_{ol} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

ϕ_{sx} = the theoretically calculated value of ϕ at any point X under the weir on a homogeneous bed.

ϕ_{sx} = the observed value of ϕ at any point X under the weir with a given upstream silt blanket.

ϕ_{sl} = the theoretical value of ϕ at point No. 1 in a homogeneous bed.

The calculated values of ϕ, after eliminating the effect of silt blanket, are compared with the experimental values in table I.

It should, however be noted that the accuracy of the above method depends upon the accuracy of the reading given by the first pipe. If this pipe appears to be behaving abnormally or erratically, the calculations should be made with reference to the second or third pipe from the upstream end. Even if there are only two pipes under a weir, the above formula can still be applied to eliminate the effect of silt blanket, and a comparison of the hydraulic gradient so obtained with the normal gradient can yield information about a cavity or loose contact, if such happens to be situated in the region of observation.

Statistical method of analysis.

Accumulated data of 6 months to a year can be analysed statistically also. The method of analysis is outlined below.

Step 1. A statement of the values of ϕ is made from the folio, and all

the values which seem to be very different from the general run, are marked.

Step 2. All those values are to be rejected which are :

(1) more than twice the general run of values.

(2) less than half the general run of values.

(3) negative.

Unless the ϕ to be so rejected, as also the general run of ϕ, are both less than 10, no rejection under (2) is made. Such rejections are to be confined to issolated values and if more than 5 successive values of ϕ come under (1) or (2) they may not be rejected but should be treated as a seperate group of values. If a record of depths of silt upstream is also available it would help in forming the groups.

The above limits are no doubt arbitrary but it is believed that where an isolated value is more than twice or less than half the general run of values, some extraordinary factors, such as partial choking of a pipe, or serious error of observation or record, must be operating to render the value un-representative of stable conditions. The negative values of ϕ must be rejected because they have no significance.

Step 3. The groups of values of ϕ are averaged separately for each pipe and, as a measure of dispersion of individual values about the mean value, the root mean square deviation, or the standard deviation, is calculated also separately for each pipe for each group.

The standard deviation is given by :

$$\sigma = \sqrt{\frac{\Sigma (X - \overline{X})^2}{N}}$$

where X is any individual value

and $\overline{X}$ is the mean of all the values.

$X - \overline{X}$ is difference or deviation of any value from the mean value.

N is the number of observations of which X represents the mean.

This will give an idea of the scatter of the values about the mean.

Step 4. The values of ϕ are again studied and all those values which differ from the mean value by more than three times the corresponding

standard deviation are to be rejected. The values which remain are averaged and standard deviations calculated afresh.

The science of statistics shows that if a value differs from the mean by more than 3σ the odds in favour of the value being influenced by factors different from those which influence the rest, are more than 300 : 1

Step 5. The mean values are now divided by the theoretical values and the results expressed as percentages :

$$\text{i.e. } \frac{\phi \text{ mean}}{\phi \text{ Theoretical}} \times 100$$

The values of ϕ are, as stated earlier, liable to be influenced by depth of silt deposit. If the theoretical values for three different points under a weir are 60, 50 and 40 while the actual observed values are 30, 25 and 20, then it is obvious that the silt blanket at the upper end has cut off 50 % of the total head.

Step 7. The average percentage is applied to all the actual values and the results are compared with the theoretical, or vice versa.

A PROBLEM

Pressure observation data of a weir are recorded in table III.

To make an analysis by the statistical method given above to find out the percentage of head absorbed by the silt blanket, where it exists, and, by applying the average percentages to the theoretical values, to compare the results

None of the values are to be rejected under Step 2. A perusal of the data, and of the record of silt depth upstream, suggests three distinct groups of values

(*i*) October to December.	(6 values)
(*ii*) May to September.	(9 values)
(*iii*) January to May.	(9 values)

The groups of values are averaged separately for each pipe and the mean, and the standard deviations are calculated and recorded in column 3 and 4 of table III. In column 5 of the same table, $3 \times \sigma$ is calculated and again the mean value of ϕ for each pipe is compared with the undivided values of ϕ of the pipe in this group, to see if in any case the variation is greater than 3σ .. No such case is noticed and consequently there will be no rejections under this step.

TABLE II

DATA OF SUKHAR (JINNAH) BARRAGE

1941

DATE	15:12:41	1:12:41	17:11:41	3:11:41	20:10:41	6:10:41	22:9:41	9:9:41	25:8:41	11:8:41	29:7:41	14:7:41	30:6:41	16:6:41	19:5:41	5:5:41
U. S. L.	195.3	194.9	194.9	194.6	195.2	195.7	195 8	195.7	195.8	196	196.7	195.9	195 75	195.5	197.7	194.5
D. S. L.	181.0	182.1	183 0	184.7	186.2	187.0	190.8	193.4	194.4	193.5	190.8	194.3	191.0	192.4	188.8	184.0
Silt U/S of end pile	8.0	8 0	8.	7.0	6.0	5.0	2.5	3 0	2.5	2 5	2 5	2.5	2 5	4.0	6.0	6.0
Depth of silt D/S of weir.	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil
$\phi 1$	41	38	40	42	48	55	66	83	93	76	74	96	86	76	67	46
$\phi 2$	33	34	36	39	44	49	60	75	87	67	67	86	76	74	61	40
$\phi 3$	33	33	35	38	42	48	58	73	86	64	65	85	75	68	60	38
$\phi 4$	22	22	23	25	28	30	38	48	67	45	44	68	48	45	38	24
$\phi 5$	18	18	19	19	23	26	31	48	61	36	36	60	40	40	31	20
$\phi 6$	15	14	15	17	18	19	23	33	55	33	30	57	31	31	24	15
$\phi 7$	10	10	10	10	13	13	16	23	45	25	23	49	22	19	17	10
$\phi 8$	9	9	9	9	12	12	14	19	34	18	19	36	17	11	11	6

TABLE 11 (Contd.)

1941

	21-4-41	7-4-41	24-3-41	10-3-41	25-2-41	10-2-41	27-1-41	13-1-41
U. S. L. ...	194.5	195.2	192.75	192.75	192.8	193.4	193.4	194.5
D. S. L. ...	184.75	181.1	174 70	174.90	174.85	174.9	175.1	179.4
H ...	9.75	14.1	18.05	17.85	17.55	18.50	18.3	15.1
Temp. shed maximum	105.5	102.2	98	87.9	89.8	75	72.5	74
,, ,,	78	76	73.3	67.0	55.9	53	42.8	47
Temp. surface water		79	78	70	74	65	58	60
10 H ...	84	79	78	72	74	67	61	62
16 H	84	77	75	69	69	62	56	58
Temp. before 10 H surface ..	78	75	75	71	70	64	58	60
16 H ..	77							
Depth of silt ...	Nil	Nil	Nil	Nil	Nil	Nil	Nil	Nil
U/S at pier ...	5.5	5.0	8.0	8.0	5.0	5.0	5.0	5.0
60 away ...	Nil	8 0	5.0	5.0	8.0	8.0	8.0	8.0
ist pipe ...	5.0	9.5	9 5	9.5	9.5	9.5	9.5	9.5
1 ...	53	54	65	65	62	60	63	69
2 ...	45	47	52	54	57	50	53	60
3 ...	43	45	51	52	50	48	51	58
4 ...	24	29	35	35	34	33	36	38
5 ...	23	25	30	30	30	29	31	33
6 ...	18	20	25	25	25	24	26	27
7 ...	12	15	20	20	20	19	21	20
8 ...		11	17	17	17	17	19	18

TABLE III.

Pipe No.	Number of values	Mean & (observed)	Standard deviation	$3 \times \sigma$	$\theta_{th.}$	$\frac{100 \times \phi mean}{\theta theoratical} = K$	K Mean	ϕmean (calculated
	1	2	3	4	5	6	7	8
			Group I (period 15-12-41 to 6-10-41)					
1	6	44	5.8	17.4	99	48.89		41.7
2	6	39.17	5 68	17.04	77	50.87		35.796
3	6	38.17	5.37	16.11	73	52.29		33.959
4	6	25.0	3.05	9.15	60	41.07		27 01
5	6	20.50	2.99	8.97	46	44.57		21.362
6	6	16.33	1.82	5.46	40	40.83		18.576
7	6	11	1.41	4.23	24	45.83	46.44	11 146
8	6	10	1.41	4.23	9	111.11 *		4.1796
			Group II (period 22-9-41 to 19-5.41)					
1	9	79.6	10.5	31.5	90	88.44		80.919
2	9	72.56	9.17	27 51	77	94.23		69.231
3	9	70.44	9.59	28 77	73	96.49		65.634
4	9	49.00	10.47	31.41	60	81.67		53 946
5	9	41.67	10.58	31.74	46	90 59		41.359
6	9	35.22	11.62	34.86	40	88 05		35.964
7	9	26.56	11 30	33.90	24	110 67*	89.41	21.5784
8	9	19. 89	8.59	25.77	9	221.00*		8.0919
			Group III (period 5-5-41 to 13-1-41)					
1	9	59.67	6.83	20.49	90	66.30		55.449
2	9	50.89	5.81	17.43	77	66.09		47.4397
3	9	48.44	5.52	16.56	73	66.36		44.975
4	9	32.00	4.85	14.55	60	53.30		36.966
5	9	27.89	4.01	12.03	46	60 63		28.341
6	9	22.78	3.87	11 61	40	56.95	55 449	24.644
7	9	17.44	3.85	11.55	24	72.67*		13.0776
8	9	15.25	4.15	12.45	9	169.44*		.499041

The average value of $\frac{100 \times \phi \text{mean}}{\phi \text{Theoretical}} = K$ is calculated for each group, discarding the values, marked with an asterik, which differ considerably from the normal scatter. It will be interesting to note that for all the pipes in one group the values of K are scattered around an average value which is different for each group; this indicaters that different percentages of head are cut off by the various thickness of silt blanket. The average value of K for each group is recorded in column 7.

Applying the average value of K to the theoretical values, the ϕ values are calculated and tabulated in column 8. A comparison with the observed mean values in column 2 shows a fair agreement, indicating that, apart from the silt effect, the pressure distribution is normal, and the foundation of the work is sound.

23. (D). THE EFFECT OF TEMPERATURE GRADIENT ON THE UPLIFT PRESSURES.

In Western Pakistan, the temperature of river water in the plains changes, due to seasonal variation, from about 10°C in January to something like 35°C in June. There is however a lag in the subsoil temperature. While the minimum temperature of water occurs in January, the minimum temperature of the subsoil occurs in March. All the year round a temperature gradient of varying magnitude along or against the direction of flow, will be obtained in the subsoil below a work. Allen Hazen (9)* while experimenting on the effect of temperature on flow of water through sand, found the relation

$$v = 3.28 \ cd^2 \ \frac{(T+10)}{60} \ \frac{H}{L}$$

Which is the same as Darcy's law $v = K\frac{h}{L}$ if $K = 3.28 \ cd^2 \frac{(T+10)}{80}$, K being a " transmission constant ".

In the Hazen formula v is velocity of flow in feet per 34 hours.

d = effective size of sand,

c = a coefficient,

h/L = hydraulic gradient,

T = temperature in degrees Farenheit.

This shows that K, the permeability constant, is a function of size of material and temperature of flowing water. Evidently a change in

*(9) Irrigation Pocket Book, by R. B. Buckley, Third Edition 1920, Page 493.

temperature of flowing water will be equivalent to a change in the Permeability of the medium. The effect of variation of temperature at Panjnad was studied by Khosla[10] in the field, and by Uppal[11] in the Laboratory on a model. Khosla concluded from his observations on the prototype a that change of 35°F to 38°F in temperature caused a change of 20% in pressure, or approximately one-half of one percent for every degree Farenheit differential variation in temperature. If this is correct, the temperature variation will have a considerable effect on pressure variation and consequently it must play an important part in the analysis of pressure pipe data. To test the magnitude of the temperature effect the author carried out experiments under simplified conditions, to determine the magnitude of pressure variation for a temperature variation of about 15°C. A model of a depressed floor, 4 feet long, as described in the silt-effect experiments, was constructed. The schematic diagram showing the position of pipes and thermometers is given in fig. (7) Tap water, after passing through glass wool and a sand filter to catch any suspended fine clay particles, was led through a glass tube in which it was heated electrically, or cooled by a surrounding cooling chamber, to different temperatures, and than finally delivered to the upstream side of the model. Three conditions were tried.

(*i*) Upstream water temperature was kept higher than the subsoil temperature.

(*ii*) An approximately uniform temperature was kept in the medium.

(*iii*) The temperature of the upstream water was kept lower than the temperature of the medium.

By the side of each pressure, pipe a thermometer was embedded and both were simultaneously recorded by two observers. The temperature gradients maintained in the above three cases are shown in fig. 8 (b).

The corresponding values of ψ plotted in fig. 8 (c) show that for a temperature difference of 15.1°C between the upstream free water and the subsoil water, the change in the value of ψ from the normal or no-temperature gradient case. for pipe No. 7, is equal to 4. The same order of variation in θ was observed for temperature gradient against the flow It is to be noted that the above experiments do not coroborate Khosla's conclusions ftom his Panjnad observations. The former show that in the absence of silt blanket the temperature difference between the upstream and subsoil water of about 15°C (or+27°F) can cause a change in the value of ψ by only +5%. The effect of temperature gradient is

(10) Khosla, A. N. Observation and Record of Pressures below Works on Permeable Foundations. C. B. I Publication No. 8.

(11) Uppal H. L., Pressures under a Model of Panjnad Weir and under the Prottoype. Paper No. 185 of Punjab Engineering Congress.

therefore minor as compared to the silt blanket effect, and may be completely masked by the observational errors of prototype observation, wkich may be of the same order.

Since however temperature variation does effect the uplift pressure to some extent, the observation of temperature of the upstream and subsoil water has been included in the data required to be collected.

23. PLOTTING OF RESULTS

To ensure a ready check on the safety of the work, the plotting of the data should be made in such a way as to show up the following points.

(*a*) Region of the work, or the period of year, which is unsafe against uplift pressures.

(*b*) Existence of cavities or loose contacts.

The method of plotting the data for the two cases is given below.

(*a*) Region of work or the period which is unsafe against uplift pressures.

This aspect of the problem has already been discussed from theoretical considerations in para 12. For pipes downstream of the crest, the observed values of ϕ are plotted graphically against time as abscissa. The maximum safe value of ϕ is calculated as in para 12 and is then plotted as a line Parallel to the abscissa. On each graph sheet in the folio for record and analysis of pressure pipe data, results, of pipes in each line should be plotted to cover a period of 4 years. A suitable ordinate scale can be selected to include all the pipes in one line below crest on one page. On the bottom of the page the head across the weir and the thickness of silt blanket, and also to maintain a check on the maximum permissible head.

(*b*) Existence of cavities or losse contacts.

To check for the possible existence of cavities, proceed as follows:

(*i*) Calculate the theoretical ϕ line.

(*ii*) Eliminate the effect of silt blanket by the use of the equation 8 or by the statistical method.

(*iii*) Plot the hydraulic gradient and, by comparing with the theoretical gradient, look for the features described in para 17.

On another sheet of the graph in the folio, comparison of the theoretical and the observed hydraulic gradients should be made after eliminating the affect of silt blanket. Consistant sets of readings with nearly the same thickness of upstream as well as downstream silt blanket, may be grouped together and the mean Hydraulic gradients may be compared with the theoretical.

In conclusion, I have great pleasure in thanking Late Mr. C. King, Director, Irrigation Research Institute to whose keen interest in the subject, careful perusal of the manuscript and valuable suggestions, the paper owes much.

Since the paper was first written Mr. Muhammad Hussain, Superintending Engineer Mechanical Circle haa also very carefully gone through it, and discussed with the author the various suggestions and amendments he would like to make. This was a very valuable discussion and some of his suggestions have been incorporated with thanks.

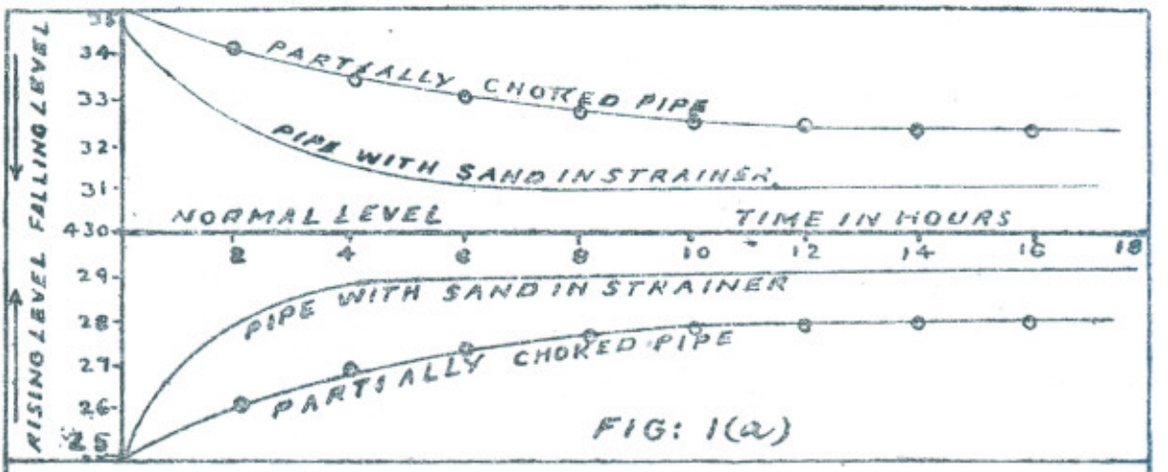


FIG: 1(a)

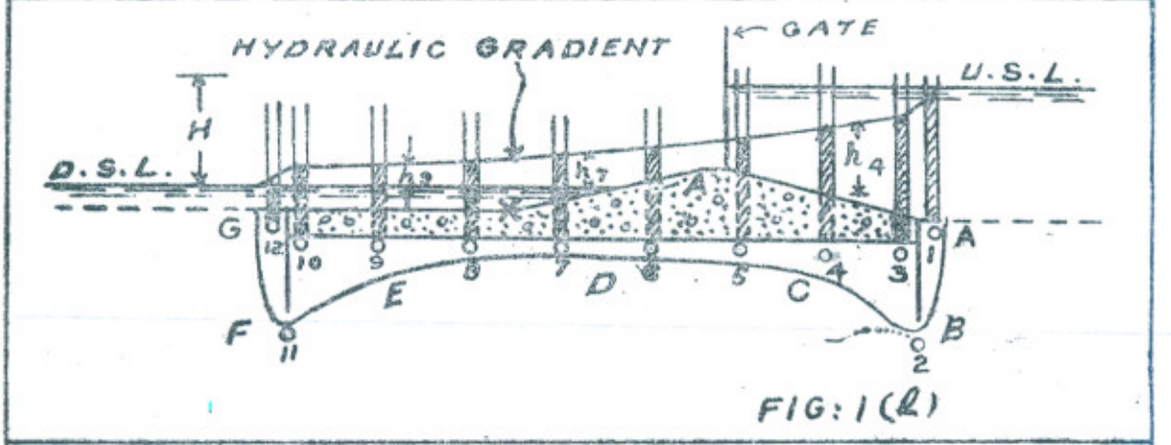


FIG: 1(b)

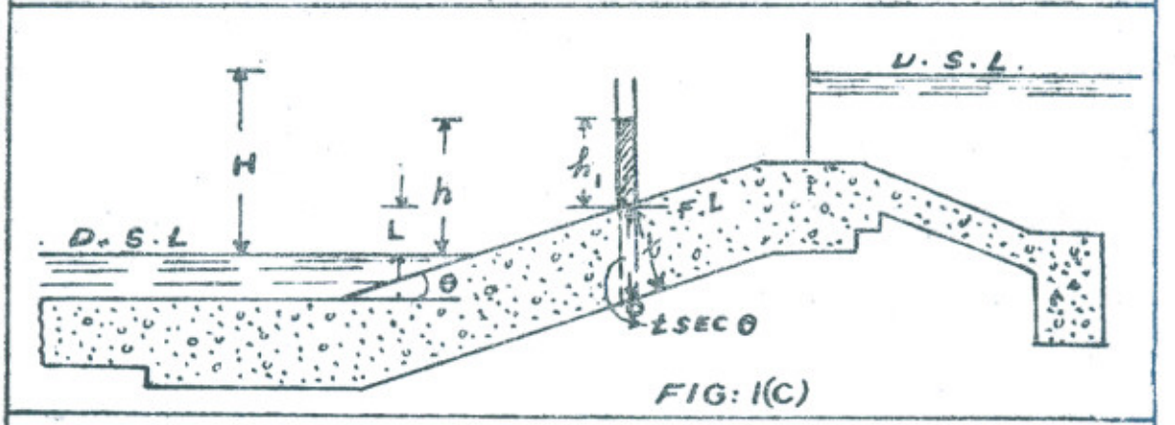


FIG: 1(C)

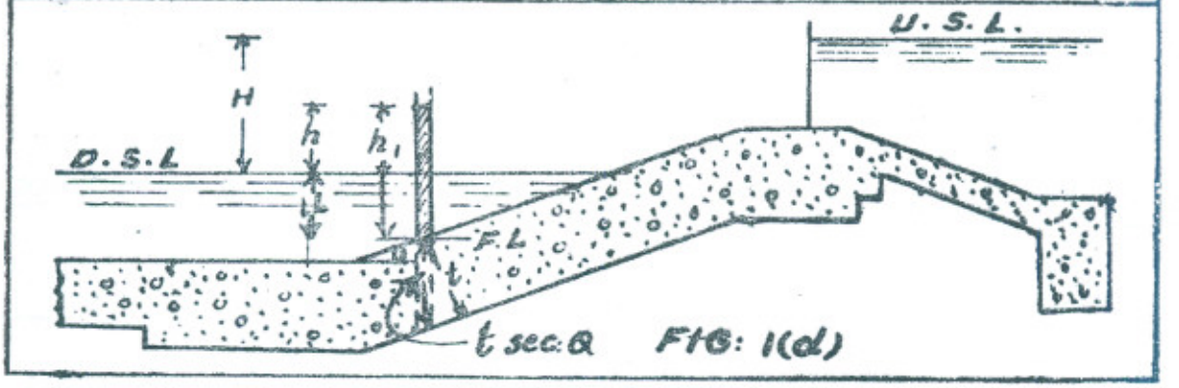


FIG: 1(d)

FIG. 2(a)

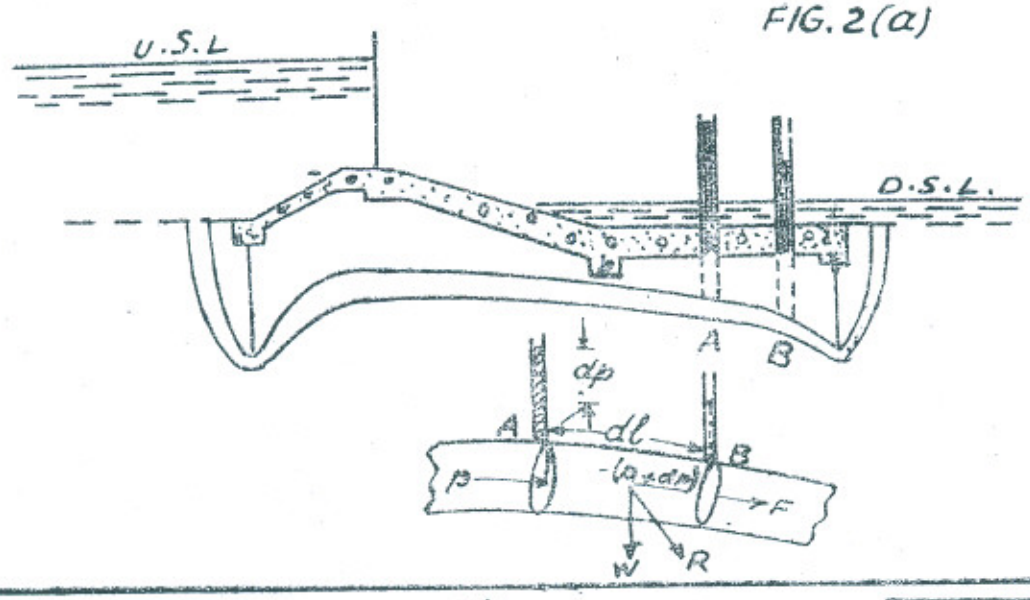
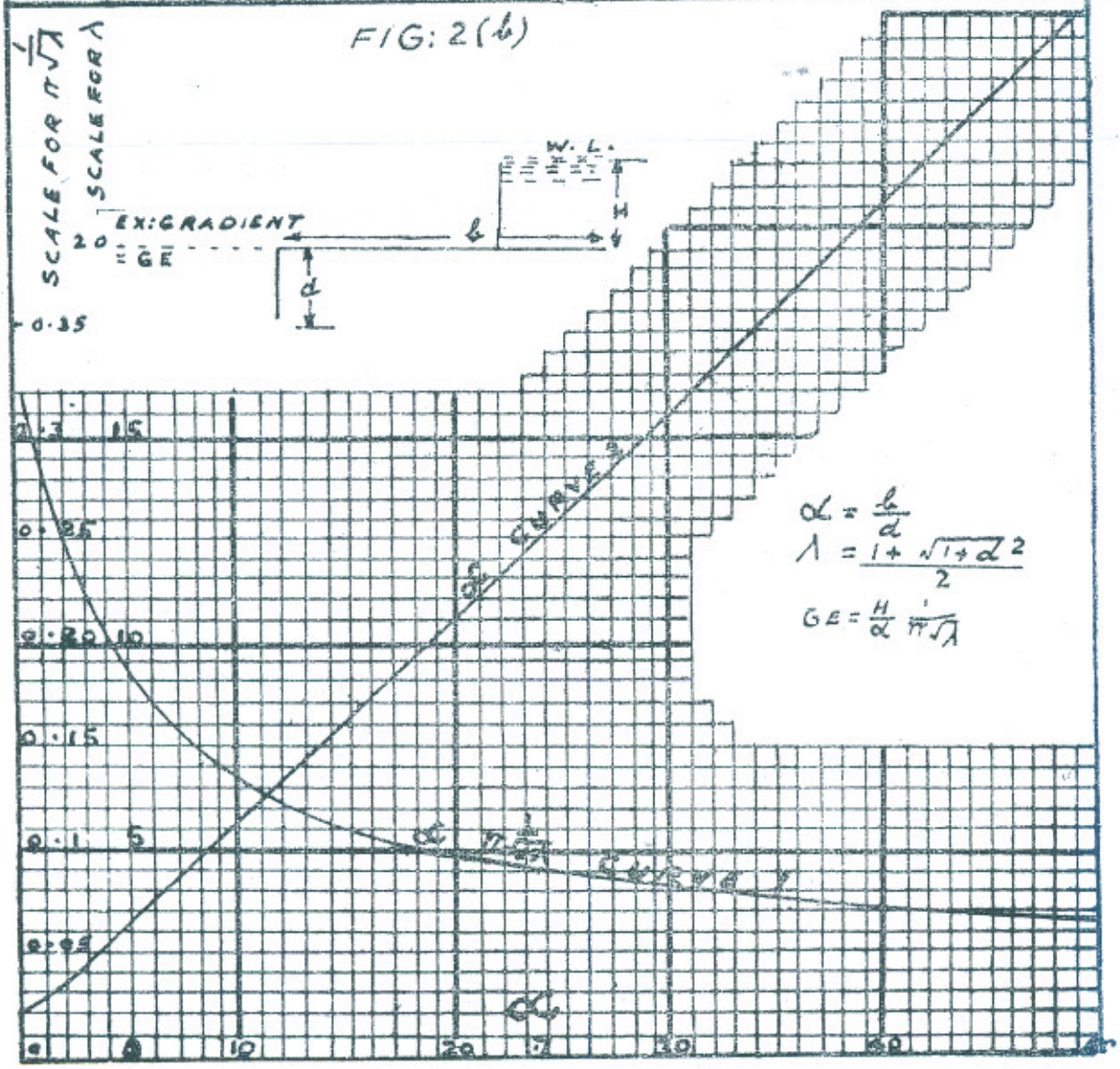

U.S.L
D.S.L.
A
B
dp
dl
A
B
F
W
R

FIG: 2(b)


SCALE FOR $\frac{1}{\pi\sqrt{\lambda}}$
SCALE FOR λ
EX: GRADIENT
GE
W.L.
H
b
d
0.35
20
15
10
5
0.25
0.20
0.15
0.1
0.05
0
10
20
30
40
$\alpha = \frac{b}{d}$
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$
$GE = \frac{H}{\alpha}\frac{1}{\pi\sqrt{\lambda}}$

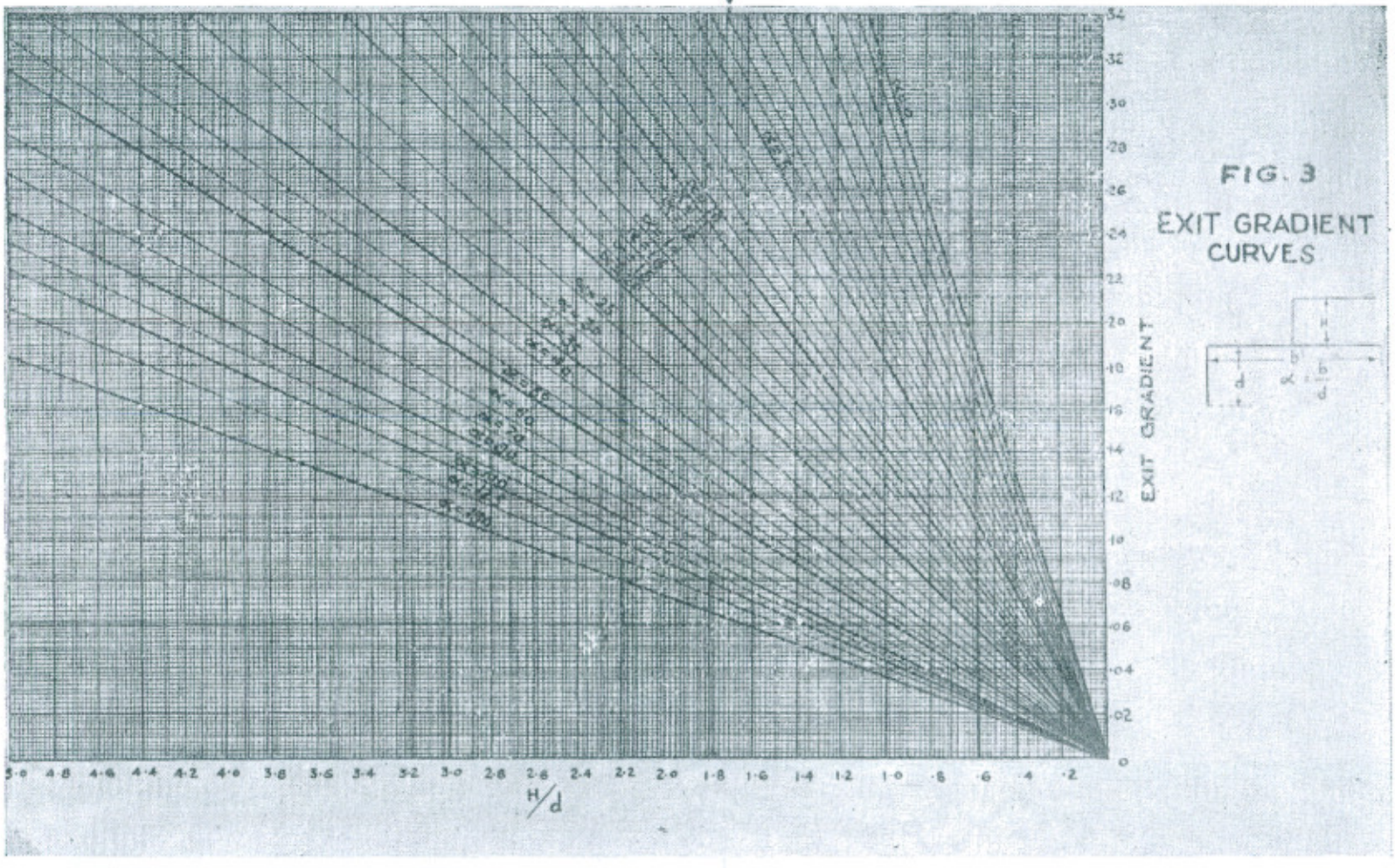


FIG. 3

EXIT GRADIENT CURVES

FIG: 4 (a)

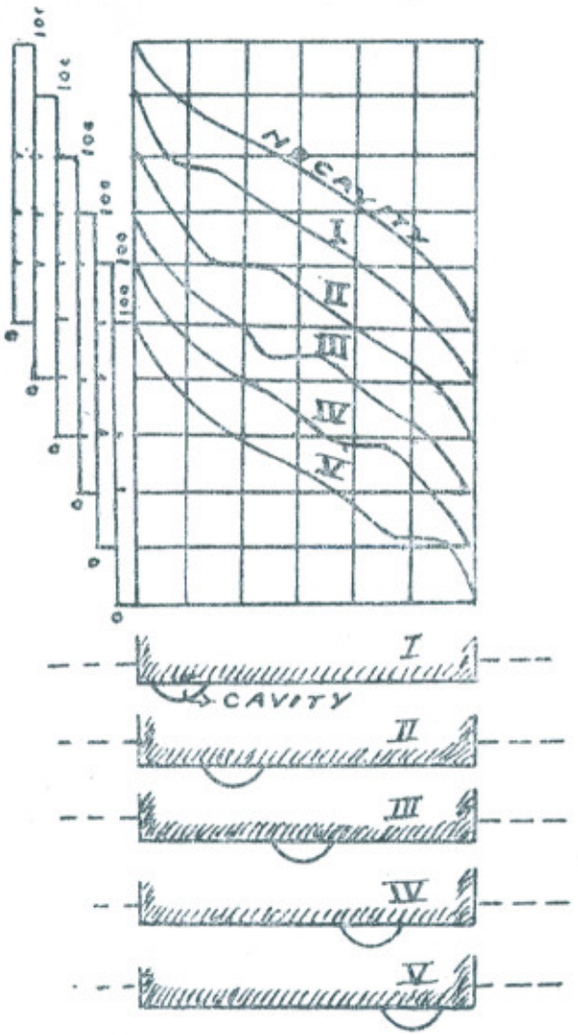


FIG: 4 (b)

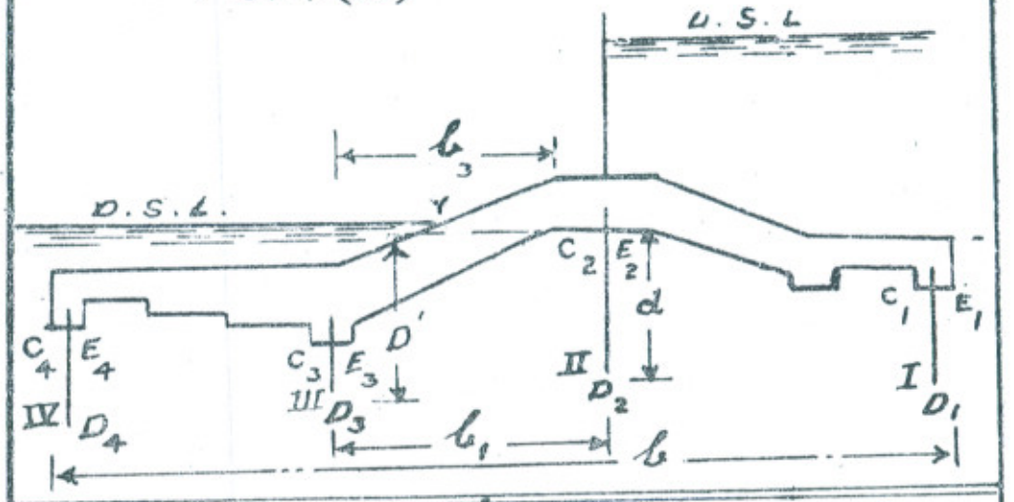


FIG: 5 (c)

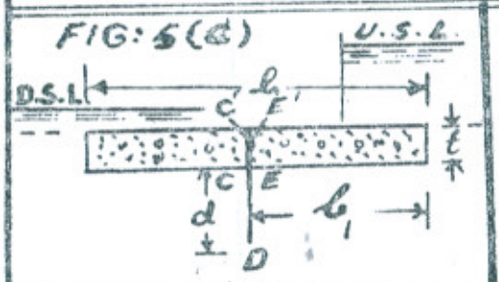


FIG: 5 (b)

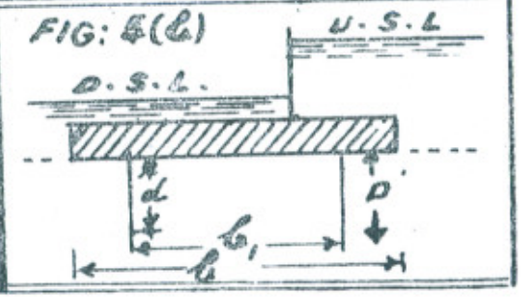


FIG: 6 (d)

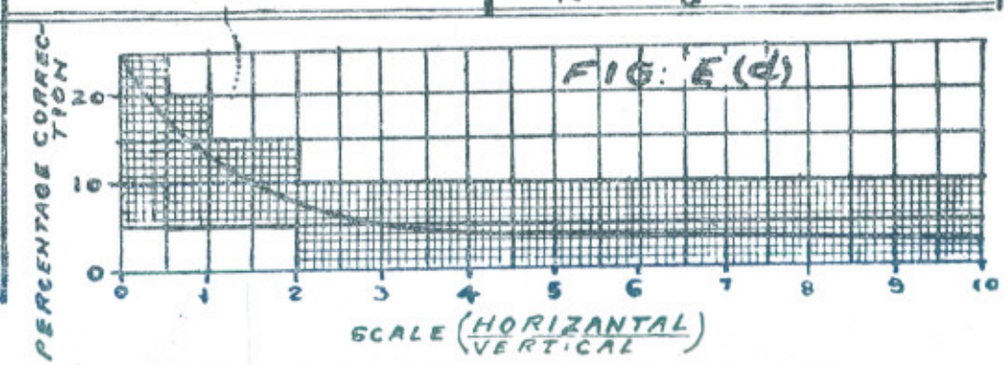
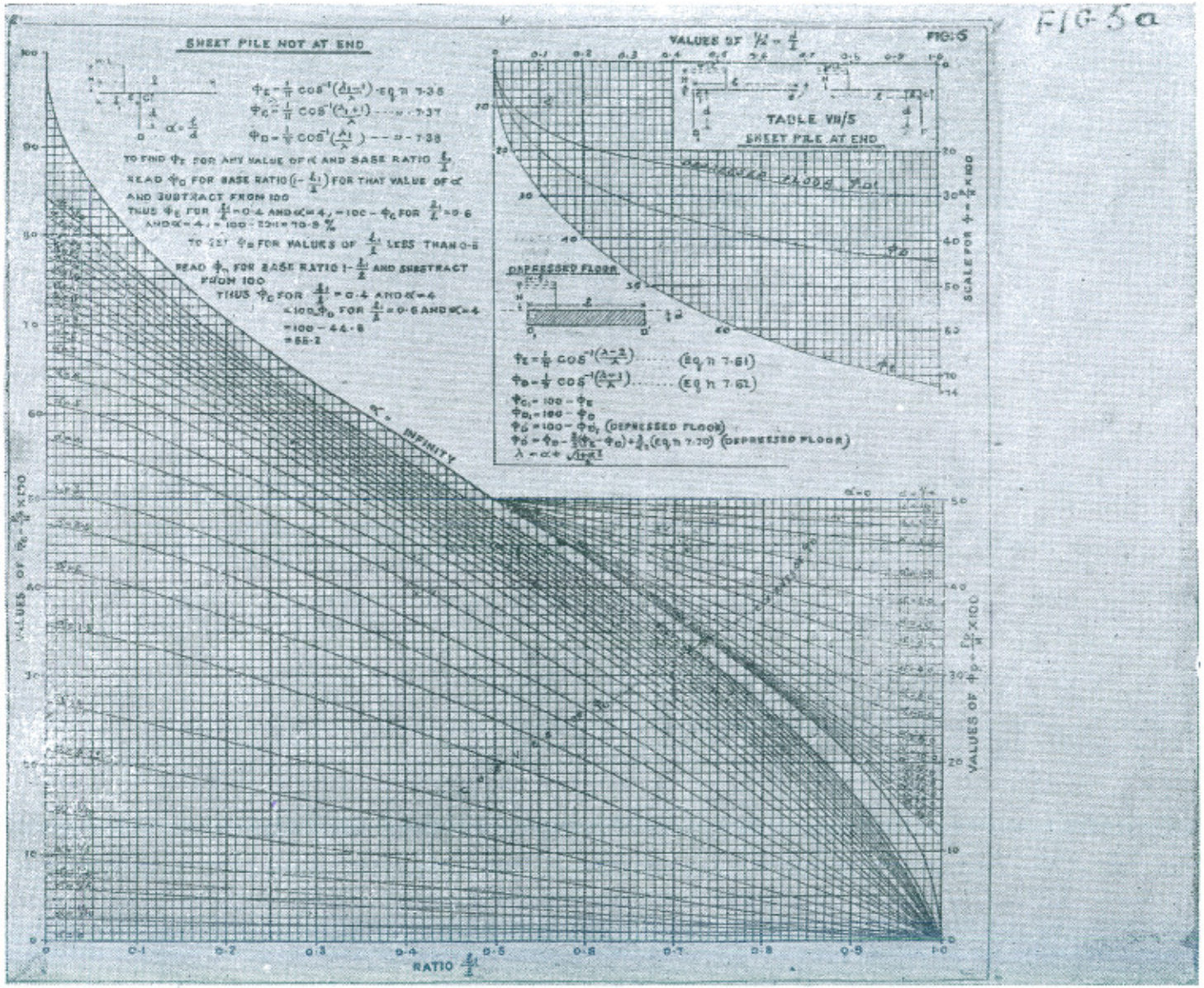
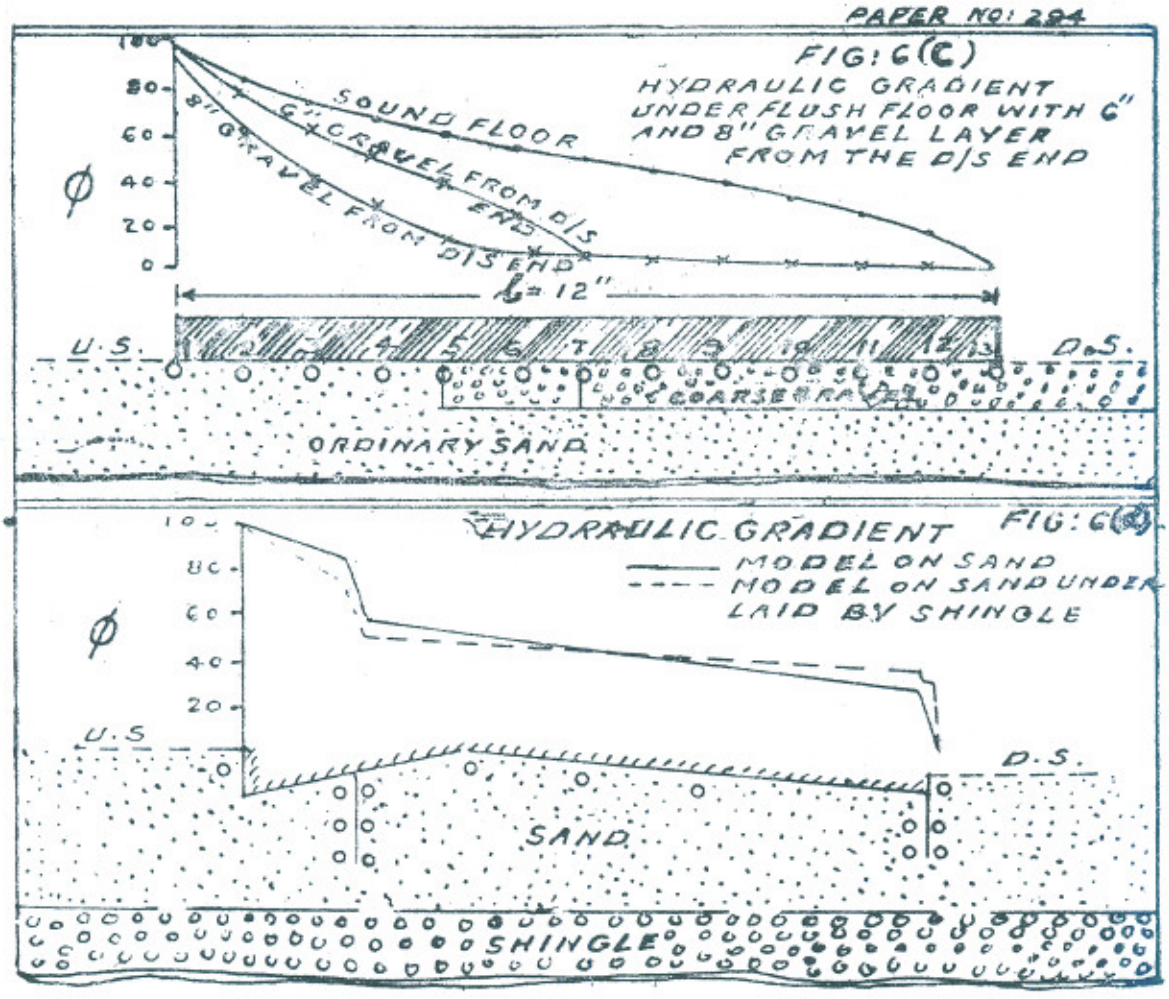


FIG 5a


SHEET PILE NOT AT END
FIG:5
VALUES OF $\frac{t_1}{t} - \frac{t_2}{t}$
$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - \alpha^2}{\lambda}\right)$ Eq n 7.35
$\Phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$ --- " 7.37
$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$ --- " 7.38
$\alpha = \frac{t}{d}$
TO FIND Φ_E FOR ANY VALUE OF α AND BASE RATIO $\frac{t_1}{t}$
READ Φ_D FOR BASE RATIO $\left(1 - \frac{t_1}{t}\right)$ FOR THAT VALUE OF α
AND SUBTRACT FROM 100
THUS Φ_E FOR $\frac{t_1}{t} = 0.4$ AND $\alpha = 4$, $= 100 - \Phi_C$ FOR $\frac{t_2}{t} = 0.6$
AND $\alpha = 4$, $= 100 - 29.1 = 70.9$ %
TO GET Φ_D FOR VALUES OF $\frac{t_1}{t}$ LESS THAN 0.5
READ Φ_C FOR BASE RATIO $1 - \frac{t_1}{t}$ AND SUBTRACT
FROM 100
THUS Φ_C FOR $\frac{t_1}{t} = 0.4$ AND $\alpha = 4$
$= 100 - \Phi_D$ FOR $\frac{t_1}{t} = 0.6$ AND $\alpha = 4$
$= 100 - 44.8$
$= 55.2$
TABLE VII/5
SHEET PILE AT END
DEPRESSED FLOOR
$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$ ..... (Eq n 7.61)
$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$ ..... (Eq n 7.62)
$\Phi_{C_1} = 100 - \Phi_E$
$\Phi_{D_1} = 100 - \Phi_D$
$\Phi_{\bar{D}} = 100 - \Phi_{D_1}$ (DEPRESSED FLOOR)
$\Phi_{\bar{C}} = \Phi_D - \frac{1}{2}(\Phi_E - \Phi_D) + \frac{1}{2\lambda}$ (Eq n 7.70) (DEPRESSED FLOOR)
$\lambda = \alpha + \sqrt{1 + \alpha^2}$
$\alpha = \infty$ INFINITY
VALUES OF $\Phi_E = \frac{P_E}{H} \times 100$
VALUES OF $\Phi_D = \frac{P_D}{H} \times 100$
SCALE FOR $\frac{t}{d} \times 100$
RATIO $\frac{t_1}{t}$


FIG: 6(C)
HYDRAULIC GRADIENT UNDER FLUSH FLOOR WITH 6" AND 8" GRAVEL LAYER FROM THE D/S END
SOUND FLOOR
6" GRAVEL FROM D/S END
8" GRAVEL FROM D/S END
100
80
60
40
20
0
Φ
L = 12"
U.S.
D.S.
1
2
3
4
5
6
7
8
9
10
11
12
13
COARSE GRAVEL
ORDINARY SAND
HYDRAULIC GRADIENT
FIG: 6(D)
MODEL ON SAND
MODEL ON SAND UNDER LAID BY SHINGLE
100
80
60
40
20
Φ
U.S
D.S.
SAND
SHINGLE

HYDRAULIC GRADIENT UNDER KALABAGH WEIR ON MIXTURE OF SAND AND SHINGLE

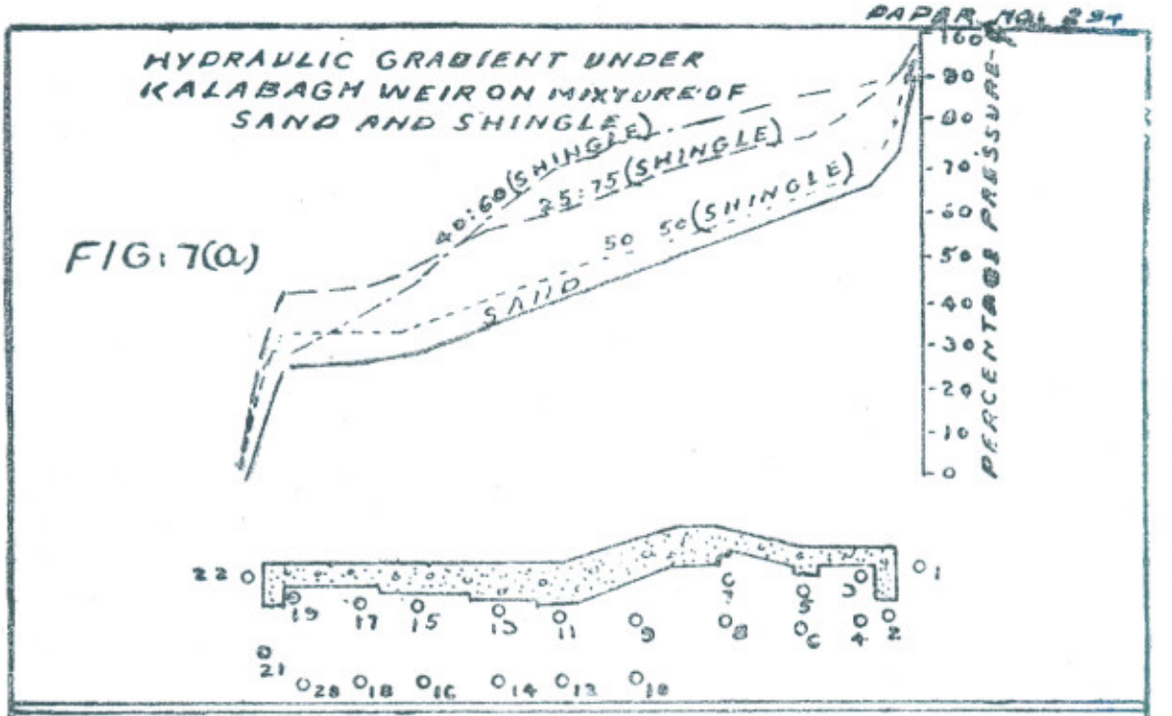


FIG: 7(a)

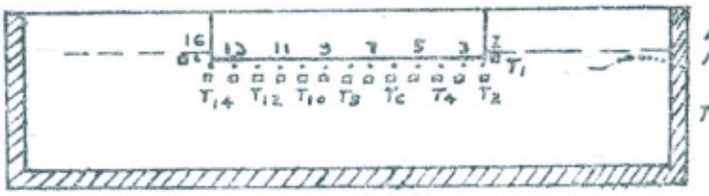


FIG: 7(b) SHOWING POSITION OF PRESSURE PIPES AND THERMO-METERS

HYDRAULIC GRADIENT FOR DIFFERENT THICKNESSES OF SILT BLANKET

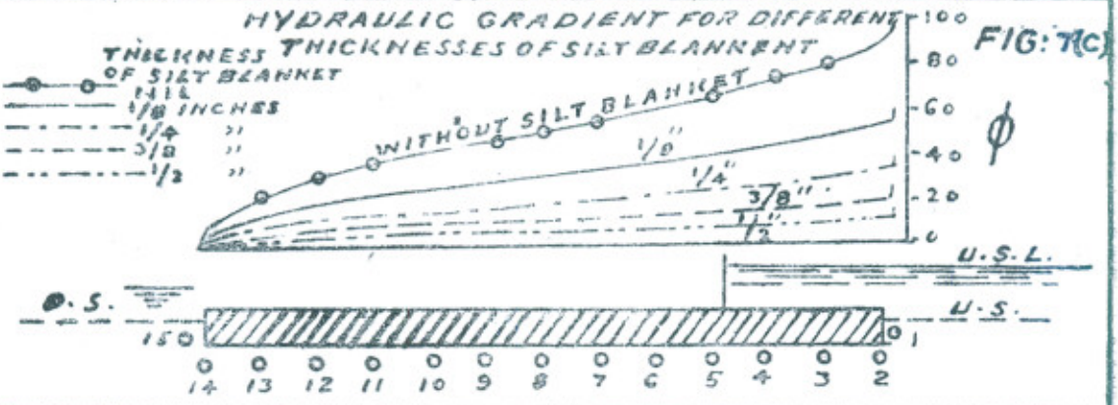


FIG: 7(c)

ϕ PLOTTED AGAINST PIPE NUMBER

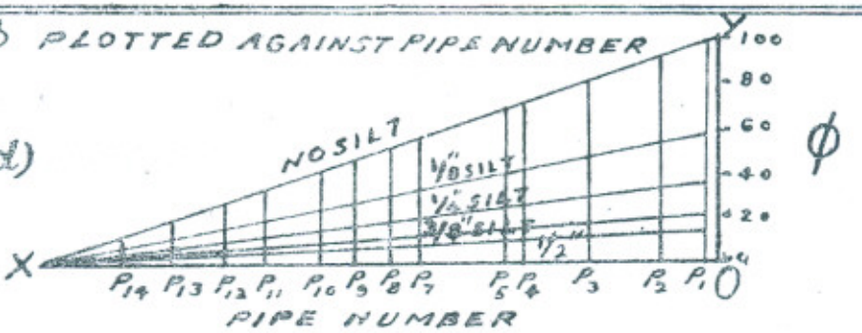
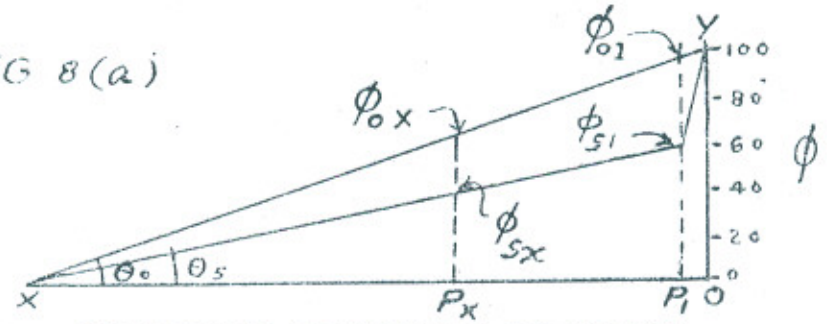


FIG: 7(d)

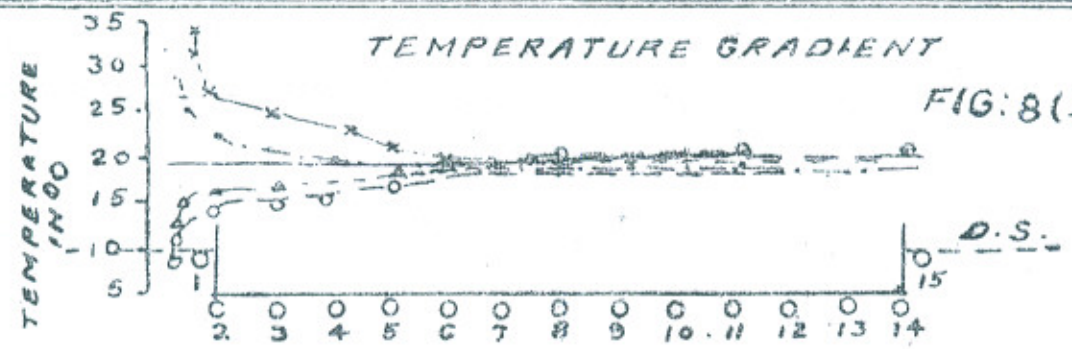
FIG 8(a)



RELATIVE POSITION OF PIPES

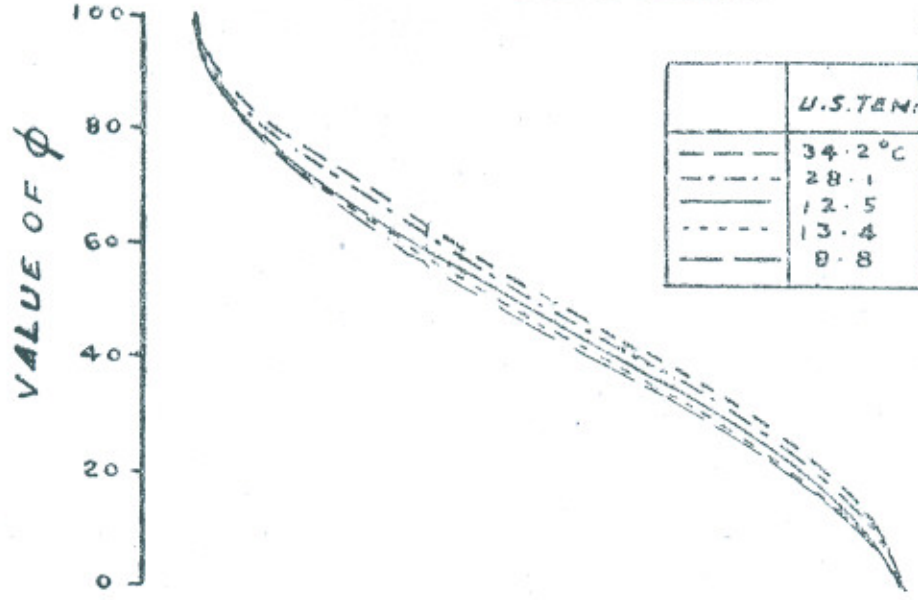
TEMPERATURE GRADIENT

FIG: 8(b)

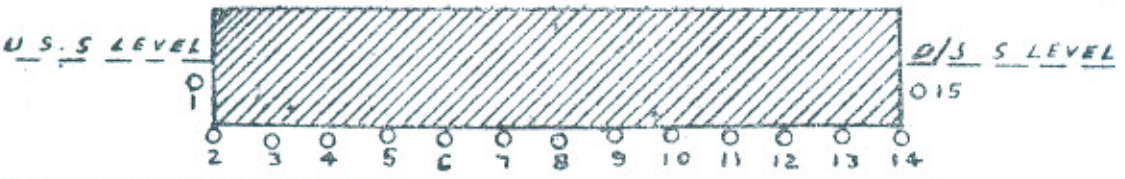


HYDRAULIC GRADIENTS FOR DIEFFERNT TEMPERATURE GRADIENTS

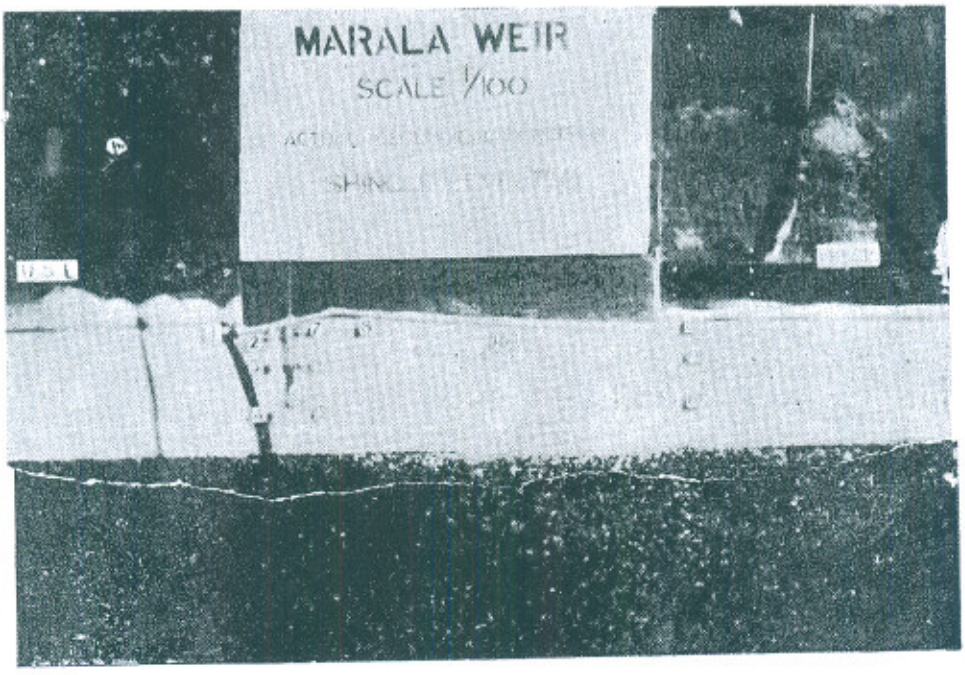
FIG: 8(c)



	U.S. TEM	D/S TEM	TEM DIFF
— — —	34·2 °C	19·1 °C	15·1 °C
— · — · —	28·1	18·1	9 9
———	12·5	19·4	-1·1
- - - - -	13·4	19·6	6·2
— — —	8·8	19·4	10 6




FLOOR
BAJRI DS
BAJRI BEYON
THE TOE


MARALA WEIR
SCALE 1/100