CANAL SEEPAGE STUDIES BY NUCLEAR TECHNIQUE

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C E R T I F I C A T E

Certified that the dissertation entitled' CANAL SEEPAGE STUDIES BY NUCLEAR TECHNIQUE' which is being submitted by Shri N.SESHADRI in partial fulfilment for the award of the degree of Master of Engineering in Hydrology of the University of Roorkee, is a record of the candidate's own work carried out by him under our supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for a period of more than six months from Ist Oct. 1977 for preparing this dissertation for Master of Engineering of this University.

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C O N T E N T S

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S Y N O P S I S

A brief survey of the many existing computational and experimental methods in estimating seepage losses from unlined canals is given. A method is also described in which the filtration velocity of seepage is measured using radio-isotopes.

In this dissertation an attempt has been made to evaluate quantitatively seepage losses from the Ganga Canal and also from Deoband Branch with the help of experimental measurements by single well dilution technique using radio-active isotopes as tracer.

The experiments were carried out on the left bank of Ganga Canal near Solani Aqueduct and the on the left bank of Deoband Branch near Tansipur village Inflow-outflow method was also employed to assess directly the seepage losses in order to have comparison of the results.

CHAPTER-I

INTRODUCTION

Much of water that is drawn from an irrigation source is lost during transit from the source to its field of utilisation. These losses are called transmission losses. These consist of mainly (i) evaporation losses (ii) transpiration through weeds and plants and (iii) seepage losses through canals etc. The ' Seepage ' is by far the greatest of all losses.

Seepage from canal not only results in heavy loss of irrigation water but also creates water logging conditions in adjoining fields. The reduction in seepage losses and prevention of waterlogging is becoming more and more important because of limited supply and increased demand of water for irrigation crops. It has been reported that only about 56 percent of the total water diverted through the Ganga Canal reaches the fields for effective artificial irrigation. The remaining 44 percent is accounted for by seepage and other losses. Out of this 15 percent has been estimated to be due to seepage from the main canal, 7 percent due to seepage from the distributaries and about 22 percent due to losses from the water courses etc.

The seepage loss is influenced by the nature and porosity of the soil, the depth, and temperature of water, the age and shape of the canal section, and the position of the ground water table. Two distinct conditions may exist under an earthen canal depending primarily on the first and last factors recounted above.

Quantitative knowledge of seepage rates is desirable in determining seepage losses from canals, in evaluating surface-subsurface water relationships, etc. Such knowledge can be obtained by prediction or by direct measurement. Prediction of seepage is based on knowledge of the relevant hydraulic properties of the soil and of the boundary conditions, subjecting the flow system in question to a hydrodynamic analysis. The methods available for direct measurement of seepage losses are the inflow-outflow, ponding, seepage meter and tracer techniques. The inflow-outflow and ponding methods are applicable regardless of canal or soil conditions. The seepage meter cannot be used where the channel has rocky bottom or heavy weed growth.

The tracer technique is relatively simple to use and can be applied in deep canals with turbid or fast flowing water, for canals with heavy weed growth and adverse bottom conditions. The earlier methods employed in the determination of ground water flow using salts, dichromates and dyes are being replaced by the use of radiotracers because of their high sensitivity to detection. The radioactive tracer is a mixture of isotopes of an element which may be incorporated into a sample to make possible observations of course of that isotope, alone or in combination, through a chemical, biological, or physical process.

Kaufman and Todd have reported a study on the usefulness of tritium and chloride as tracers in canal seepage measurements. They used single well dilution technique to find out the direction and velocity of ground water. Though similar problems of seepage in dams, etc. have been tackled in using radioisotopes, very little work has been done in this country in the measurement of canal seepage using radioisotopes. It was proposed to try out the utility and limitations of single well dilution technique using tritiated water, in a field near Roorkee, on the left bank of Ganga Canal (lined canal) and Deoband Branch (unlined canal) for estimating the sub-surface flow rate and velocity and other parameters concerning the problem of canal seepage.

CHAPTER-II

REVIEW OF LITERATURE

2.1. THEORY OF SEEPAGE AND GROUNDWATER FLOW(34)

The term seepage may be defined as the process of water movement into and through the soil from a body of surface water such as a canal, ditch, stream or a reservoir. The fundamental concept in the theory of ground water movement starts from the fact that the soil body consists of many interconnected openings which serve as fluid carrier.

The hydrodynamics of seepage flow are different from the usual hydrodynamics of fluid flow since the former is characterised by channels of irregular cross-sections, non uniformity of soil in horizontal and vertical extent, changing elevations of the water surface in the channel and of the water table and other complications.

Systematic study of the seepage phenomenon dates back to 1856, when the famous Scientist Henery Darcy proposed a law governing the flow of water through porous media.

2.2. DARCY'S LAW (36, 1959)

Because of the complicated nature of the soil body consisting of different sizes and shapes of grains and pore channels, etc. the ground water flow was not amenable to any scientific treatment until the advent of Darcy's law.

In 1856, Henry Darcy, a French Engineer first formulated an analytical approach to predict the flow of water through a porous medium in connection with the design of water supply system for the city of Dijon. The experiments of Darcy led to the conclusion called the Darcy's law which may be stated as :

' For a sand of given type it is possible to assume that the filtrating discharge is proportional to the pressure and inversely proportional to the length of the sand layer '.

This law may be expressed as

$$Q = KA\frac{H}{L} \qquad \ldots(1)$$

Where

Q = discharge

A = Cross-sectional area

L = Length of soil column

H = Head loss between the two exists of the column

and K = a coefficient, which according to Darcy is a function only of the type of soil. This he names ' coefficient of permeability '.

Later investigations have however proved that ' K ' depends on the fluid as well as the medium.

Equation (1) can be expressed

Since $Q = V A$ and $i = V/L$ = hydraulic gradient

Therefore $V = Ki$

Where V = velocity

2.3. SEEPAGE PROCESS

The process of seepage from an earthen channel starts from first in filling in the pores of the soil just around the canal bed and sides. Depending on the situation of ground water table and the types of soil underlying the bed of canal etc. two distinct process may bè considered (i) ' Percolation ' and (ii) ' Absorption '.

In the case of percolation there is zone of continuous saturation from the canal to the water table and there is a'direct flow'. In the case of absorption there is a small zone of complete saturation round the canal section surrounded by zones of decreasing saturation ratio (i.e. percentage of void space occupied by water).

2.4. FACTORS AFFECTING CANAL SEEPAGE

The seepage through canals has been influenced by many factors (3, 1972), chief among them are listed below.

1. Soil characteristics of the canal bed and slopes.
2. Depth of water in the canal
3. Velocity of flow in the canal
4. Wetted area or shape of the canal
5. Position of water table (ground water), relative to the canal
6. Position of impermeable layer relative to the canal.
7. Drainage level of a nearby drain if any and its location relative to canal.

8. Temperature of water and soil
9. Entrained air in the soil
10. Soil moisture tension
11. Age of the canal
12. Amount of sediment contained in water and its grade.
13. Salt concentration in the canal water and the soil
14. Surrounding vegetation of the canal
15. Ground slopes at right angles to the canal
16. Velocity of flow of underground water
17. Other biological factors
18. Frequencies of the canal usage.

All the factors mentioned above may or may not act simultaneously and the effects of individual factors is difficult to determine. Because of so many variables involved and the complexity of their relation no satisfactory formula for computing seepage has ever been evolved.

2.5. METHODS OF SEEPAGE ESTIMATION

The methods used in estimating seepage losses from canals can be classified under three categories.

1. Emperical formulae
2. Theoretical methods, and
3. Experimental measurements.

2.6. EMPERICAL FORMULAE

The emperical methods (26) have been in vogue in India and other countries from a very old time. These formulae are simple to use with a minimum of field investigation. Most of these formulae are based on large number of field observations by various authors. Their scope is therefore limited to particular field conditions. Their application to other regions may give unreliable results.

2.6.1. Highham's formula

Sir Thomas Highham suggested the following formula for Punjab canals.

$$S = 0.55\ C.\ \frac{BL}{10^6} \sqrt{d}$$

Where

S = Seepage lost in cusecs for a length L ft. of canal

C = Coefficient, average value being 0.45

B = Water surface width of canal in ft.

L = Length of canal in ft.

D = Depth of water in ft.

2.6.2. Dyas Formula

In about 1905 he suggested this formula

$$P = C\sqrt{d}$$

where

P = Seepage loss in cusecs/ MS ft

C = a constant

d = depth of canal in ft.

He later realised that seepage did not vary as $\sqrt{d}$ but as ' d '.

$\therefore$ he corrected the formula as

$$P = C.\ ad$$

Where a = wetted area in sq.ft.

The value of C ranges from 1.1 to 1.8 as per the observations made on some canals of Punjab.

2.6.3. Punjab Formula

In Punjab latest practice is to use the following formula.

$$P = 5 \times Q^{0.0625}$$

Where P = seepage loss in cusecs/ MS ft. of wetted area

Q = Discharge of canal in cusecs.

This formula is now being adopted in U.S. also.

From the above formula the total seepage loss in a length of L of canal works out to

$$S = 5 \times Q^{0.0625} \times \frac{WL}{10^6}$$

W = Wetted perimeter in ft.

Substituting the value of $W = 2.66/Q$ as per Lacey's formula

$$S = 0.0133\ L\ Q^{0.5625}$$

This formula has been used for calculating seepage from unlined canals of Bhakra Canal system.

For lined canal $S' = 1.25\ Q^{0.0625}$

Where S' is seepage per M.S. ft. of wetted area.

2.6.4. Davis and Wilson Formula(7)

They tried to correlate velocity of flow with seepage loss and obtained the following formula -

$$S = C \sqrt[3]{d} \quad \frac{P\,L}{4000000+2000\sqrt{V}}$$

Where

S = Seepage loss in cusecs in length L ft. of canal

C = Coefficient varying from 1 to 30

d = mean depth of water in canal

P = Wetted perimeter in ft.

L = Length of canal in ft.

V = mean velocity of water in canal

2.6.5. Merits formula

In U.S.A. Merits formula is used for computing seepage loss from canals.

$$S = 0.2\ C \sqrt{\frac{Q}{V}}$$

Where

S = Seepage loss in cusecs per mile of canal

Q = Discharge of the canal in cusecs

V = Mean velocity of flow in ft/sec.

V = Cubic foot of water lost in 24 hours through each square foot of the wetted area of canal prism.

The following values have been suggested for C

C = 0.34 for gravel and hard pan with sandy loam

= 0.66 for sandy loam

= 1.68 for sandy soil with rock

= 2.20 for sandy and gravelly soil

2.6.6. Muskat's formula

This formula (30) is applicable to deep water table conditions only

$$S = K L (B + 2d)$$

Where

S = Seepage loss in cft

K = Coefficient of permeability

L = Length of canal in ft.

B = Width of water surface in canal in feet

d = depth of water in ft. of canal

2.6.7. In India

As stated above already the commonly used formula is

$$S = C\ ad$$

or $S = 0.0133\ L\ Q^{(0.5625)}$

(vide details of symbols given above)

The other practice for approximation of absorption loss in earthen channels is to assume a loss varying from 6 cusecs per million square feet of wetted perimeter for a channel with a discharge upto 120 cusecs, to a loss of 8 cusecs per million square feet for a channel upto 2000 cusecs discharge.

2.7. THEORETICAL METHODS

The theoretical evaluation of canal seepage(5,1968) depends upon the hydraulic conductivity of the soil material and other boundary conditions which include the channel dimensions, drainage distance and depth to water table. The problem of finding analytical solutions for seepage from open channels has been attracting the attention of many Research workers from time to time and several authors have evolved several formulae for different soil and water table and drainage conditions. A few of the analytic solutions given by some authors have been furnished here.

2.7.1. Dachler's Method

Dachler (1969, 18) has obtained a solution from a trapezoidal channel underlain by a impermeable layer at shallow depth D_i from the bottom of the channel. He assumed a vertical drainage at a distance of L from the centre of bottom width of canal. The drainage has been assumed to be at a depth D_w from the water surface as shown in Fig.2.Φ.

He has divided the flow system on the basis of model studies into a region with curvilinear flow (region I) and one with Dupuit-Forchheither flow (region II), with the dividing line between the two systems at a distance

$$L_1 = \frac{W_s + H_w + D_i}{2}$$

from the center of the channel.

The flow in the region-I was analysed with an approximate equation for the potential and streamline distribution under a plane source of finite width. The pattern of streamlines and equipotentials calculated with the equation was used to develop factors F that enabled calculation of the flow in region I as

$$\frac{I_s}{K} = \frac{2\,F\,\Delta H}{W_s} \qquad \ldots(1)$$

Where ΔH is the vertical distance between the water surface in the canal and the ground water table at the dividing line between the two flow regions.

I_s = seepage rate per unit length of channel and per unit width of water surface (length/time).

K = Hydraulic conductivity of soil.

Values of F were presented in relation to $W_s/\ (D_i + H_w)$ and $(H_w + D_i - \Delta H)\ /\ (H_w + D_i)$ for relatively deep and for relatively shallow channels, using W_s/ Wetted perimeter as criterion (Fig.2.2)

Because the effect of channel shape on seepage is minor for the condition of flow assumed, this procedure of ignoring the actual shape and depth of channel is not objectionable.

The flow in region-II can be expressed with the Dupuit- Forchheimer theory as

$$\frac{I_s}{K} = \frac{2(D_w - \Delta H)}{W_s\ L_2}\left[D_i + H_w - 1/2\ (\Delta H + D_w)\right] \qquad \ldots(2)$$

In general the problem will be to calculate the seepage for a given value of D_w at a distance $(L_1 + L_2)$ from the channel center, and ΔH will not be known initially. A trial and error procedure has to be employed, assuming different values of ΔH and calculating I_s/K with equations (1) and (2). The correct magnitude of ΔH is then found as the value yielding equal values of I_s/K.

Bauwer has compared the values of Dachler taking a trapezoidal channel with side slopes 1:1 and $H_w/W_b = 0.75$ and taking $L_1 + L_2 = 10\ W_b$ and using different values of D_w and D_i. The results showed excellent agreement.

2.7.2. Theories based on Zhukovskii function

During the first half of this century, extensive development of ground water theories took place in Soviet Russia, primarily from the point of view of studying seepage problems in hydraulic structures. (Aravin and Numerov).

Symmetric seepage out of canals of curvilinear section is considered.

The theoretical solution for seepages from the above type of canals with a curvilinear perimeter is obtained by the conformal transformation of Z plane of the canal cross-section to the region of Zhukovskii's function W_j given as,

$$W_j = Z - \frac{iW}{K} \qquad \ldots(1)$$

Where $Z = x + iy$, W is a complex potential and K is the coefficient of permeability.

The actual procedure is semi inverse in practice where a shape for the region of Zhukovskii function is first assumed and the corresponding parametric equations for the perimeter of the canal are obtained.

S.N. Numerov has given the solution considering that there is no high-permeability underlying layer. He considers the shape of the region of Zhukovskii's function to be a semi-ellipse with semi axis B-q/k/2 and H and with the centre at the origin (Figure 2.2).

Thus, the region of the Zhukovskii function will be a half-plane with a semi-elliptical notch, as shown in Fig.2.2.

Let the region W_r be conformally mapped on to region W_j. We then obtain

$$Z = iW_r + i\,\frac{B-q/k}{2}\,\text{Sinh}\,\frac{\pi W_r k}{q} + i\,H\,\text{Cosh}\,\frac{\pi\,W_j k}{q} \qquad \ldots(2)$$

If we separate in the above the real and imaginary parts, we obtain the equations of the seepage flow grid, i.e.

$$x = \psi_r - \frac{B-q/k}{2}\,\text{Cosh}\,\frac{\pi\,\psi k}{q}\,\text{Sin}\,\frac{\pi\psi_r k}{q} - H\,\text{Sinh}\,\frac{\pi\phi_r k}{q}\,\text{Sin}\,\frac{\pi\,\psi_r k}{q} \qquad \ldots(3)$$

$$y = \phi_r + \frac{B-q/k}{2}\,\text{Sinh}\,\frac{\pi\phi_r k}{q}\,\text{Cos}\,\frac{\pi\,\psi_r\,k}{q} + H\,\cosh\,\frac{\pi\phi_r k}{q}\,\text{Cos}\,\frac{\pi\,\psi_r\,k}{q} \qquad \ldots(4)$$

$(0 \leq q_r < \infty$ and $0 \leq \psi_r \leq \frac{q_r}{2})$

Setting in $\phi_r = 0$, the parametric equation of the channel cross-section,

i.e.

$$x = -\psi_r - \frac{B-q/k}{2} \text{ Sin } \frac{\pi \psi k}{q}$$

$$\text{and} \quad y = H \text{ Cos } \frac{\pi \psi k}{q} \qquad \ldots(5)$$

$$(0 \leq |\psi_r| \leq q/2k)$$

If we set in (3) and (4), $\phi_r = y$ and $\psi_r = -\frac{q}{2k}$, we get the equation of the right hand branch of the phreatic curve (symmetric to the left hand branch),

i.e.

$$x = \frac{q}{2k} + \frac{B-q/k}{2} \text{ Cosh } \frac{\pi ky}{q} + H \text{ Sinh } \frac{\pi ky}{q} \qquad \ldots.(6)$$

The seepage discharge 'q' through the canal can be calculated knowing the coefficient of permeability 'k' and x and y co-ordinates at any point on the phreatic line.

2.7.3. Vedernikov Approach

Case I

Seepage from canals into sands with deep lying highly permeable beds.

Vedernikov and Numerov S.M. (1962) have obtained a solution for this problem for the case of trapezoidal and triangular shapes of canals. They assumed that the surface of seepage is not existing and that there is no loss due to evaporation or absorption due to infiltration from surface.

The highly permeable layer was assumed to occur at infinite depth below the canal. The velocity distribution tends to become uniform at deep layers and the percolation velocity at infinity equals to the coefficient of permeability as the potential gradient approaches unity with the infinite depth of water table or permeable stratum i .e. V = K.

The section to be investigated is shown in Fig.2.4(a) The hodograph is shown in Fig.2.4(b) and the inversion of hodograph is given in Fig.2.4(c).

Taking an auxiliary 't' plane as shown in Fig.2.4(d), we obtain for the mapping of the d_z/d_w plane onto the lower half plane of t

$$dz/dw = M \int_0^t \frac{t\,dt}{(1-t^2)^{1/2+\sigma}\,(\beta^2-t^2)^{1-\sigma}} + N$$

$$= M\,\phi\,(t) + N$$

Where $\sigma = \alpha/\pi$, and $\phi\,(t)$ is the indicated integral.

The quantity of seepage has been expressed in the form

$$q = k\,(B + A\,H)$$

Where A is given by $2/\tan\sigma\pi \quad \dfrac{f_2(\sigma,\beta)-f_1(\sigma\,\beta)/\cos\sigma\pi}{J_2\,\pi/2 - f_2\,(\sigma,\,\beta)}$

Taking a series of values for α and β, Vedernikov obtained the correspondence between A and B/H as given in Fig.2.4.

2.7.4. Vedernikov, Muskat and Harr

Case II

Seepage from canals of trapezoidal section into permeable layers at shallow depth (12, 1962).

The section to be investigated is shown in Fig.

The solution was first given by Vedernikov in 1934.

Taking the Zhukovsky function as

$\theta = Z + iw/k$, we have, as the corresponding θ plane, the solid rectangle. Selecting the t plane for mapping of the θ plane, we find,

$$\theta = M \int_{0}^{t/m} \frac{dt}{\sqrt{(1-t^2)(1-m^2t^2)}} + iH$$

From the correspondence at points D where $t = 1$ and $\theta = B/2 - q/2k$ and solving for q, we get

$$q = K \left[B + \frac{2H\,k}{k''} \right]$$

where q = seepage loss per foot length of the canal.

k and k' = are the elliptic integrals of the first order

K = coefficient of permeability

H = depth of water in the canal

B = Breadth of water surface in the canal

The mapping of the w plane onto the t plane is given by

$$w = M_1 \int_{0}^{t} \frac{dt}{\sqrt{(1-t^2)(1-\beta^2t^2)}}$$

Where the constant M_1 is found to be

$$M_1 = iq/2k\ (\beta)$$

On solving this, we obtain

$$q = 2k\ Tk(\beta)/\ k'(\beta)$$

and we can also find that

$$q/k = 2Hn/(1-F\ (\beta-\phi)\ /\ k(\beta)\ )$$

where $n = \cot\alpha = (\ B-B_1\ )/\ 2H$

Harr has given a plot of q/kh as a function of B/H for various ratios of T/H with $n = \cot\alpha = 1.5$ (vide Fig.).

2.7.5. S.P. Garg and Chawla, A.S.

Seepage from trapezoidal channels in homogeneous media to drains located at a finite distance from the canal considering vertical and horizontal drainages (1970).

Numerical method has been adopted in deriving the equations using the technique of conformal mapping.

2.7.5.1. Horizontal Drainage

The drain may be shallow and wide and the streamlines of seepage flow from the canal may join the bottom of the drain at various points along its width. In this case the bed of the drain represents an equipotential line.

2.7.5.2. Vertical Drainage

In this case the drain may be narrow. The stream line of seepage flow from the canal would reach the drain in a horizontal direction and a vertical plane along the line of flow of the drain may be considered to represent on equipotential surface. Such a situation may also arise when seepage occurs from a canal to a shallow water table. Far away from the canal in the zone of uniform flow of ground water, the streamlines would be almost horizontal and the equipotential lines nearly vertical. Such as equipotential line may be considered to represent a vertical drain.

2.7.5.3. Equation for free surface

The equation for free surface has been obtained as

$$\frac{Y}{H} = \frac{1}{K} \sin^{-1} \left[\sqrt{\frac{t(\acute{t}+\beta)}{r\,(t+\beta)}}\,,\ m \right] \qquad (1)$$

where $m = \sqrt{r/(\beta+\acute{r})}$ (2)

values of β and r can be obtained from Figure 2.6.2

The values of y/h can be obtained for different values of 't' with the help of equations (1) and (2) and tables for Jacobian functions. Corresponding to this value of 't', the value of x can be determined ~~by referring to Figure~~ by replacing $\acute{r}$ by t and L by x. Thus free surface is defined by obtaining y/h and X/H for various values of t between 0 and $\acute{r}$.

2.7.5.4. Seepage discharge

$$q = 2k'/k = kh$$

where q = volume rate of discharge per unit length of canal

k = coefficient of permeability

k and k' are complete elliptic integrals of first kind with modulus m and m', The value of m is given by eq.(2) and $m' = \sqrt{1-m^2}$.

For various values of β and r non-dimensional q/kh has been plotted for known channel dimensions, and drainage distance for horizontal and vertical drainages respectively.

2.7.5.5. Use of Curves

Horizontal Drainage

After knowing the channel dimensions and drainage distance i.e. B, α, L, H etc. Figure 2.6.2 is referred to and initial values of β and γ are obtained. Though in these curves the values of b/H and α' are to be used, the values of B/H and α are not different from b/H and α' and to the former can be used as the first approximation. The values of α' and b/H are read off from the curves in Fig. 2.6.3. With these values better approximations to β and γ are determined from Fig. 2.6.2 and the process is repeated till the variation in the values of β and from one trial to another is negligible. This condition will normally be achieved in two or three trials. Knowing β and γ the seepage discharge in terms of kh is obtained.

2.7.5.6. Vertical Drainage

The procedure for this case is similar with the difference that Fig. 2.6.2(b) is used instead of 2.6.2.(a) for determining β and γ .

2.7.5.7. Observations

It is observed that the phreatic surface is higher and the discharge is more in case of vertical drainage compared to that of horizontal drainage. The phreatic surface rises with increase in drainage distance or increase in water depth or bed width of the channel. Seepage discharge increases with increase in bed width or depth of water in the canal but decreases with increase in drainage distance.

2.8. EXPERIMENTAL MEASUREMENTS

2.8.1. Direct Methods

There are four direct experimental methods (27, 1972) for measuring seepage rates from open channels, namely the inflow-outflow, ponding, seepage meter, and salt penetration techniques.

2.8.1.1. Inflow-Outflow Method

The canal discharge is measured at several sections and any observed decrease in the canal discharge downstream is used to estimate seepage loss. Allowances are to be made for any additions to or diversions from the canal. The disadvantages of the method is that the seepage losses have to be considerably high for any meaningful measurement of the fall in discharge between any two sections.

2.8.1.2. Ponding Technique

A section of the canal is isolated to maintain zero flow and the rate of disapperance of the impounded water is measured. After correcting for the loss due to evaporation seepage loss is evaluated. The obvious disadvantage of this method is the required isolation of the canal section which is neither practicable nor desirable in many cases.

2.8.1.3. Seepage Meter Technique

Local seepage measurements can be obtained with the seepage meter, which is essentially a covered cylinderical infiltrameter or seepage ' cup' about 1 ft. in diameter. The cylindrical part is pushed a small distance into the channel bottom and seepage is measured as the outflow from the cylinder. The seepage meter technique is difficult to apply in deep canals, particularly if the water is turbid or flowing at high velocity, and in canals with heavy weed growth or rocky or gravel bottoms.

2.8.1.4. Salt Penetration technique

This is essentially a tracer method, whereby seepage is calculated from the rate of advance of salt in the bottom material (19, 1968). For this purpose, a layer of salt is maintained on the bottom of the channel for about 10 to 20 minutes. The salt will start to dissolve and part of the salt will enter the bottom material with the seepage flow. When the salt is completely dissolved and has disappeared from the canal bottom, normal channel water will enter the bottom again. Thus, the salt concentration distribution in the bottom material will have the form of a wave. If the salt was present on the channel bottom for a relatively short time (about 20 minutes) the wave will have a definite peak. If the salt was present for longer periods, the wave will have flat peak or plateau. Column studies have shown that the peak of the wave or the lower point of the plateau advances essentially as piston flow for atleast the first

5 inches or so. Thus the rate of advance of the peak or lower point of plateau is measured. The seepage rates can be calculated by multiplying the rate of advance of the salt by the porosity of the soil. The depth of the peak or the lower point of the plateau of the salt concentration wave is measured with an electrical conductivity probe. This probe is slowly pushed into the bottom material, until the conductivity just starts to decrease again after having reached a peak value. The depth of the probe at this point is recorded. Dividing this depth by the time since salt was applied then yields the average rate of advance of the salt wave. The porosity of the bottom soil can be measured on core samples. This technique yields a point measurement of the seepage. A number of such measurements should be made for evaluation of seepage rate from a channel section.

2.8.2. Indirect Methods

The radioisotopes are employed in the measurement of subsurface flow (22, 1966). The most important techniques are -

i) Double well technique

ii) Single well pumping method,

iii) Single well dilution technique

2.8.2.1. Double Well Technique

In this method an isotope is injected in one well and the observation for the tracer is made in another well located downgradient at a known distance. From the travel time of the peak of the tracer concentration and the distance between the injection well and the observation well, the velocity of flow can be calculated.

2.8.2.2. Single well pumping technique

This technique which is called as pulse technique is adopted where volume of water in the well is large, high permeability is encountered, and when more than one well cannot be economically drilled. The procedure is to add a small quantity of tracer into the aquifer through a well and the tracer is left to move with natural flow for a prefixed interval. Then the tracer is recovered by pumping water from the same injection well. The pumped water is continuously monitored for radioactivity. Knowing the time lapsed between injection and pumping rate, and the pattern of activity flowing out during pumping, the sub-surface flow rate can be calculated. This method requires a number of injections to be carried out to allow for incomplete recovery of tracer during pumping. One advantages is that only a small quantity of tracer is required for this type of work.

2.8.2.3. Single well dilution technique

Single well dilution technique has been tried out by many authors in determining localised velocity and direction of ground water flow. In this method, the radioisotope is injected in a single well and its dilution is noted with time after injection. The general principle of single well dilution method is that when a tracer is injected into the entire volume of water in a well uniformly, its concentration decreases with time. Since the well is encountered in sub soil water flow, fresh water enters the well and water concentrated with isotope leaves the well uniformly in the direction of subsoil flow. Thus the water in the well gets diluted. However, the total volume of water in the well remains the same. Thus, the decrease in concentration follows a simple exponential law as the fresh water flows into the well, at a constant rate throughout due to subsoil flow. Seepage rate can be calculated knowing the filtration velocity and the gradient of phreatic line.

2.9. REVIEW OF THE SOME OF THE EXPERIMENTS CONDUCTED IN INDIA

2.9.1. Punjab Irrigation Research Institute, Lahore

Attempts have been made to determine seepage losses in an indirect way by measuring the velocity of seepage flow in the neighbourhood of canal. Midha and others (1937) tried to develop a method using the measured velocities of seepage flow. The velocities were determined by measuring the transitive of a tracer. They also used the formula -

$$q = V_f d \left(\sec \theta + \frac{\sin \theta}{\theta} \right)$$

where V_f is the measured velocity of seepage flow,

d is the distance of observation point from the central line of the canal, and

θ is the angle the streamline makes with the vertical at the point of measurement.

There are many assumptions in the formula -

1) The canal was assumed to be a point source of seepage and all the streamlines were assumed to originate

originate from the centre of canal.

2) The angle θ the streamline makes with the vertical was assumed to be small and was equal to $\tan\theta$. Any streamline making an angle θ with the vertical was assumed to have a velocity $V_o \cos\theta$ where V_o is the velocity of the vertically downwards translocating streamline.

2.9.2. Irrigation and Power Research Institute, Punjab

Experiments have been conducted by Radioisotopic Laboratory, Irrigation and Power Research Institute, Punjab using I-131 as radioactive tracer for determining the seepage flux in Kasur Branch Lower, Lahore Branch and Upper Basi Doab canal. The double well technique is adopted and the seepage flux is calculated by the formula

$$F = V h\theta_1 \left(\frac{1}{\cos\theta_1} + \frac{\sin\theta_1}{\theta_1} \right),$$

with the same assumptions as stated in para 2.4.1. Here, h is the depth in the canal. The seepage flux per million sq.ft. of wetted surface worked out is 12.05 c/s, 8.96 c/s and 11.57 c/s for Kasur Branch lower, Lahore Branch and upper Basi Doab Canal, respectively.

2.9.3. Ganga Canal Study (1966 and 1967)

Radiotracer experiments were conducted by the Isotope Division, Bhaba Atomic Research Centre, Bombay, in collaboration with the Uttar Pradesh Irrigation Research Institute, Roorkee to estimate the seepage losses from the Ganga canal at a selected cross section near Asafnagar (Mile 23-4-0).

2.9.3.1. K. Krishnamurthy and S.M. Rao (27, 1968) evaluated the seepage per unit length on the basis of Zhukovsky's function and Numerov's approach. The formula adopted was -

$$X = q/2k + B-q/k/\ 2 \quad \text{Cos} \ \frac{\pi ky}{q} + H \sinh \frac{\pi ky}{q}$$

where q = seepage rate per unit length of canal

x,y = co-ordinates of any point on the phreatic surface.

B = top width of canal

They obtained the seepage rate as $2.2m^3$/day/metre ($0.03m^3/day/m^2$ or 11.4 c/s per million sq.ft.) for Ganga canal at Mile 23-4-0.

In this approach the cross section of the channel has been assumed to be elliptical and the effect of drainage distance has not been considered.

2.9.3.2. Uttar Pradesh Irrigation Research Institute, Roorkee (9, 1968) also evolved the seepage rate of Ganga canal (Mile 23-4-0), with the same data, but with a different approach.

The seepage discharge was calculated using the formula proposed by Dr. V.I. Vaidhianathan. The formula was slightly modified at the Institute.

$$F = 2\ V_f\ d\ \theta \cos \ \theta$$

Where

F = total seepage discharge per unit length of canal

θ = inclination of phreatic line with the vertical at the point of observation of V_f, in radians

V_f = filtration velocity, and

d = distance from the centre line of canal to the bore hole

The assumptions made in the above formula were -

1) The canal has been assumed to be a point source and all streamlines have been taken to originate from it.

2) The velocity along any streamline has been assumed to be constant and equal to V_o Cos θ where V_o is the vertically downward velocity along the y axis.

3) The above assumption is valid only in the case of very low velocities and for the condition that the sink is at infinity.

4) The streamlines have been assumed to be straight lines which practically may not be the case.

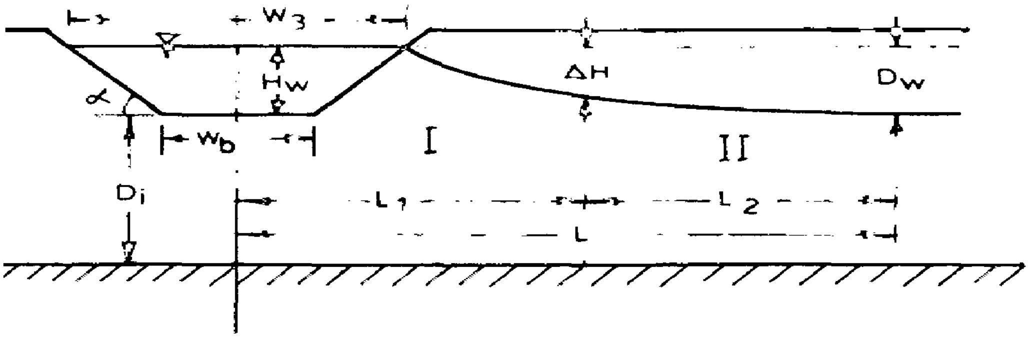
The seepage losses as calculated by the modified formula worked out to be of the order of 4.18 c/s per million sq.ft. of wetted perimeter for the Ganga Canal at Mileage 23-4-0.

The above experiment was carried out by single well dilution technique using I-131 as radioactive tracer.

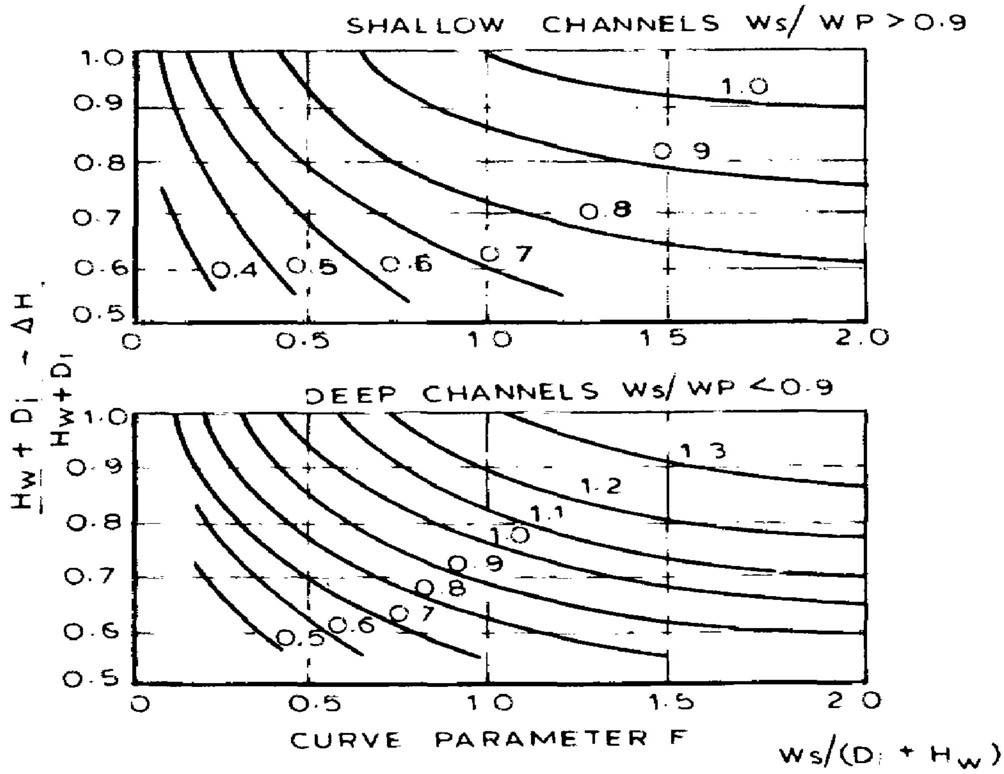
2.9.4. Present Study

The present study is based on the experimental measurement of filtration velocity by radiotracer point dilution technique. Tritiated water was used as radioactive tracer.

Experiments were conducted at two different sites. Conventional method was also tried to assess the seepage losses directly. These are described in the forthcoming chapters.



2.1 _ DIVISION OF FLOW SYSTEM IN REGIONS I AND II FOR DACHLER'S ANALYSIS



.2 _ DACHLER'S VALUES OF F FOR SHALLOW AND FOR DEEP CHANNELS

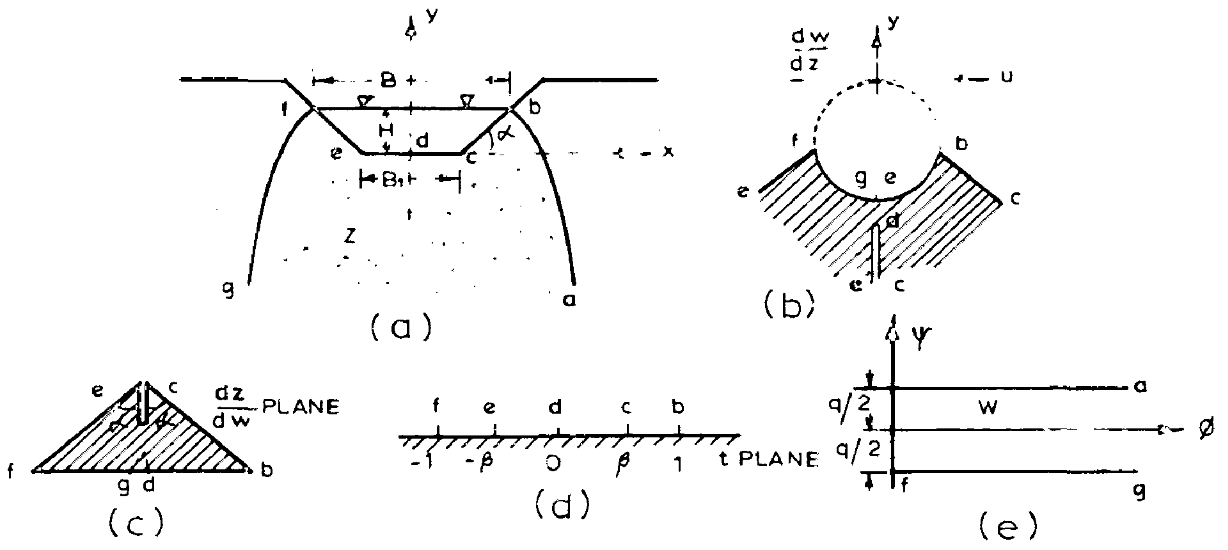


FIG. 2.4 – AFTER VEDFRNIKOV

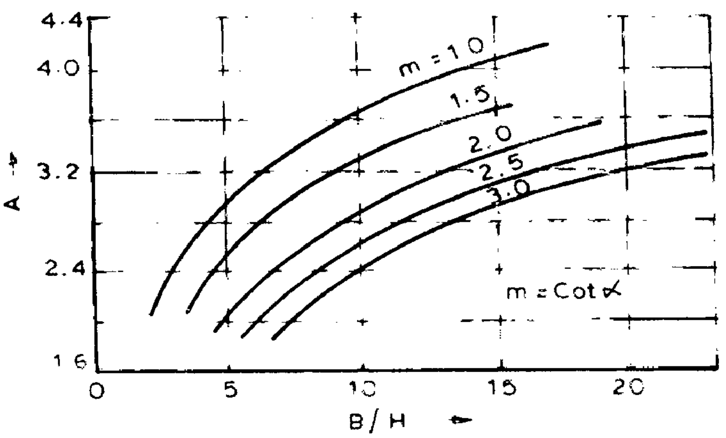


FIG. 2.4 – AFTER VEDERNIKOV

FIG. 2.3 – SHOWING THE CURVILINEAR SECTION CONSIDERED BY S.N. NUMEROV

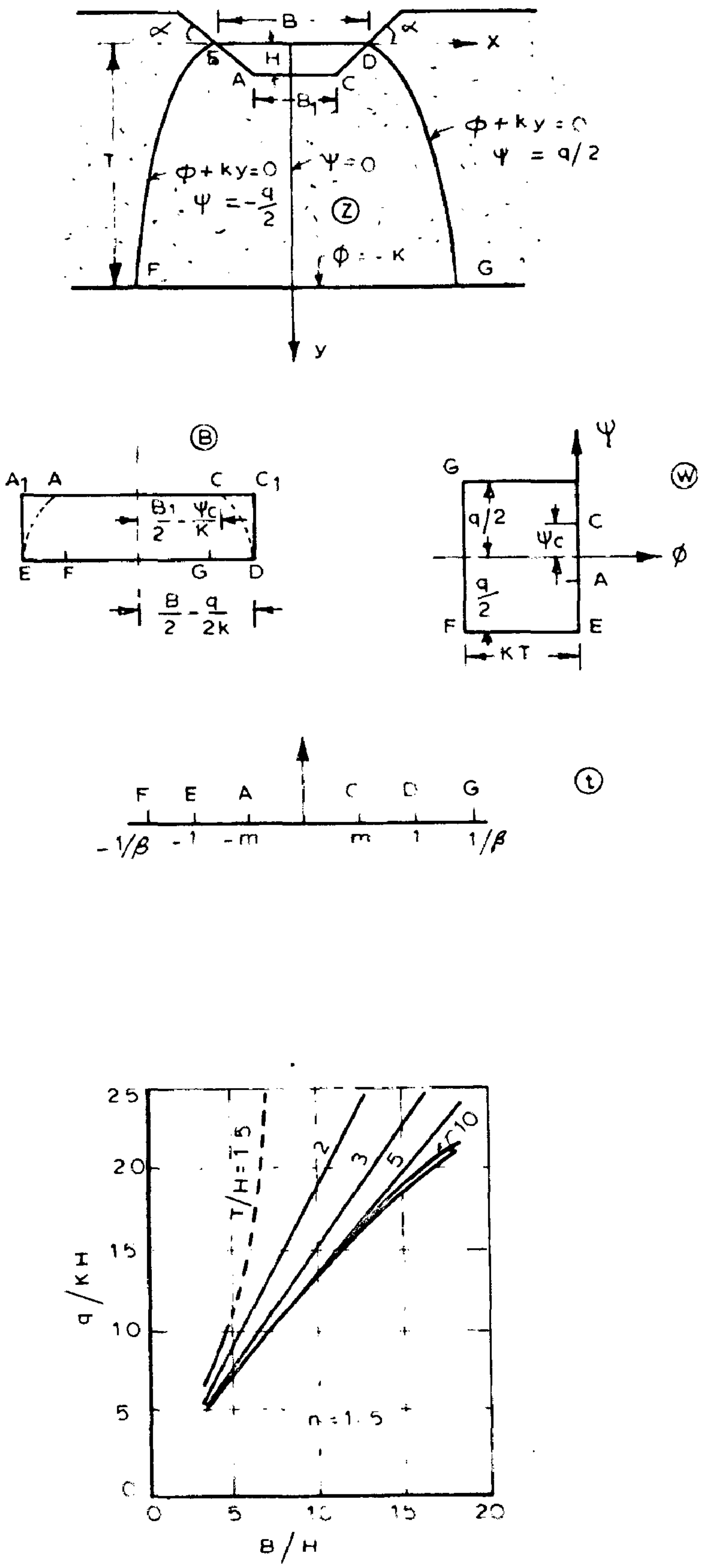
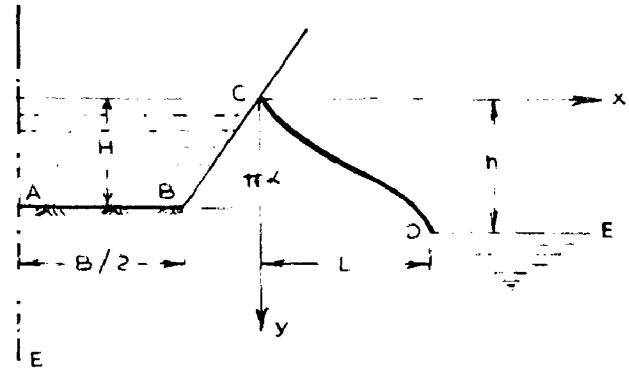
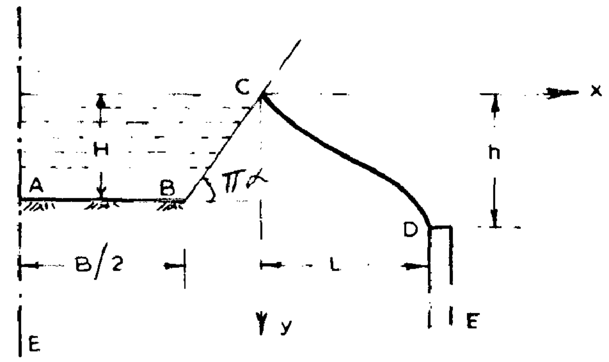


FIG.2.5 _ ADAPTED FROM MUSKAT AND VEDERNIKOV

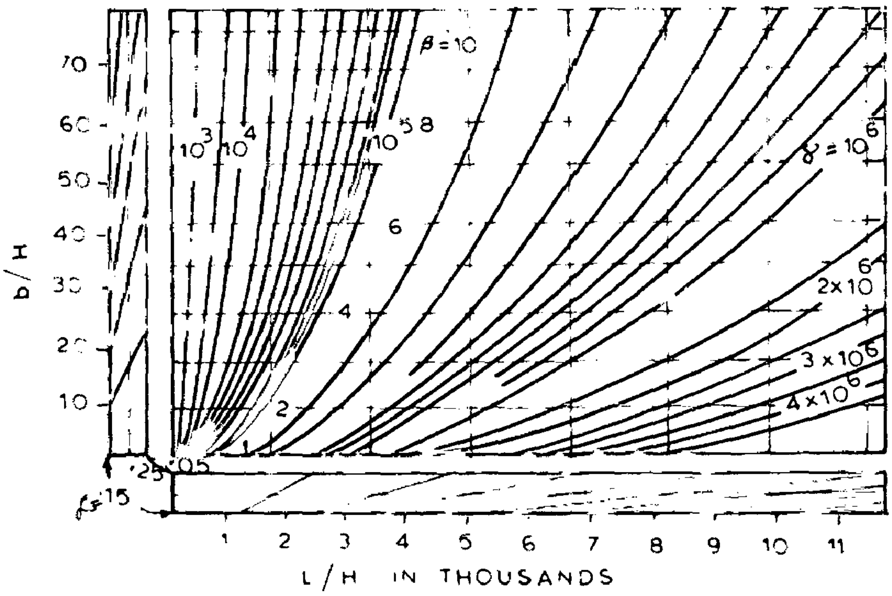


(a) Horizontal Drainage

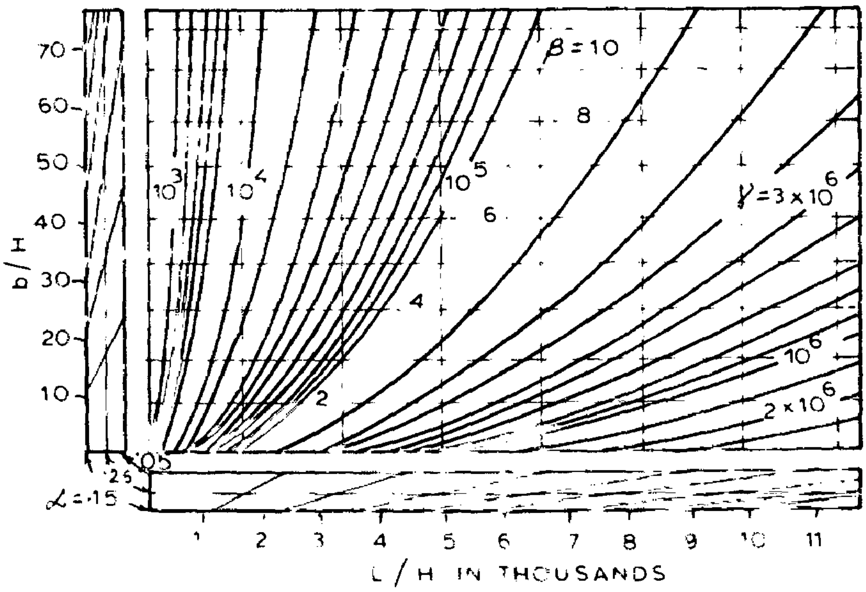


(b) Vertical Drainage

FIG.2.6.1 PHYSICAL PLANE

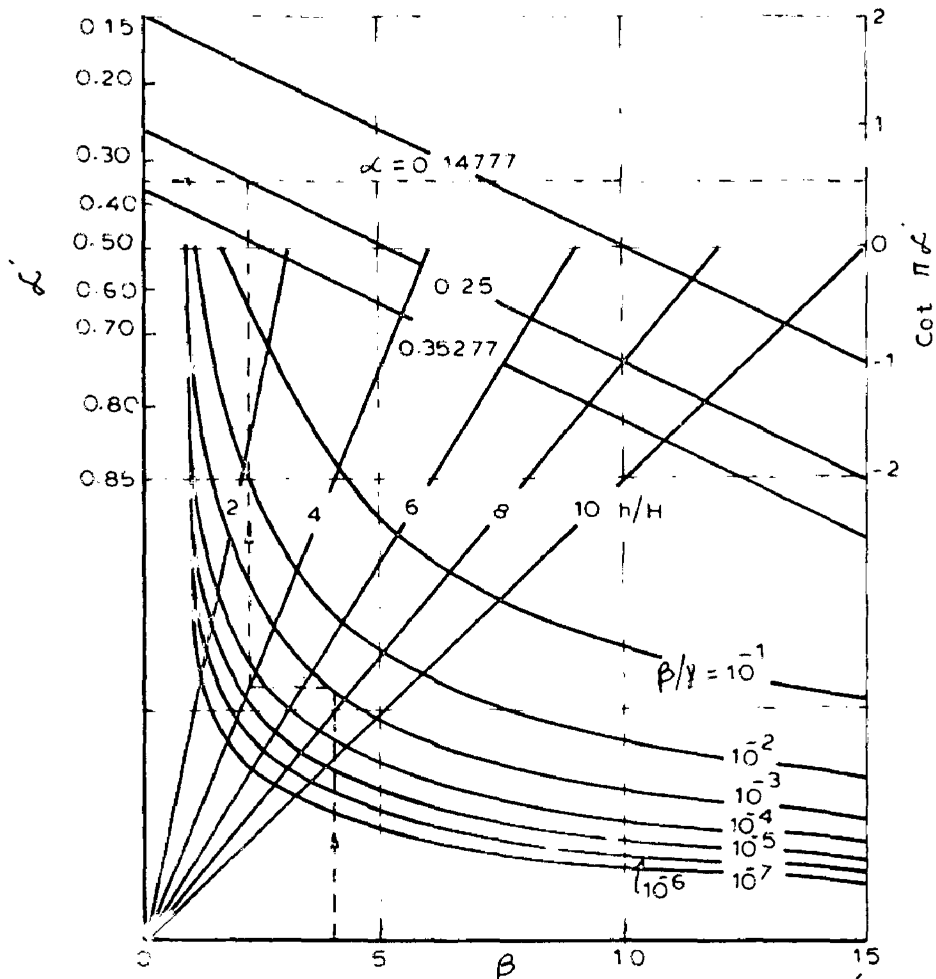


a_ VALUES OF β AND γ HORIZONTAL DRAINAGE

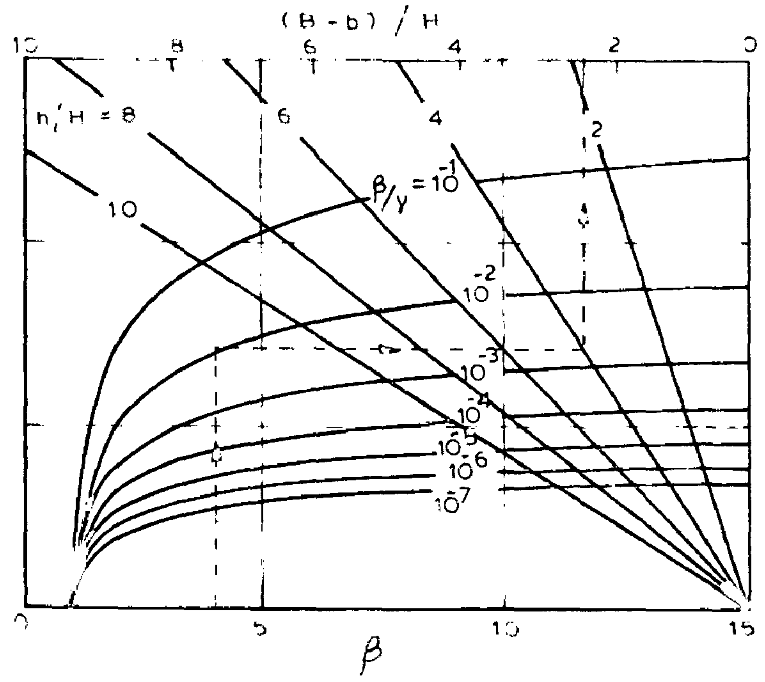


b_ VALUES OF β AND γ VERTICAL DRAINAGE

FIG 2



a _ RELATION BETWEEN α AND α'



b RELATION BETWEEN B AND b

FIG 2.6.3

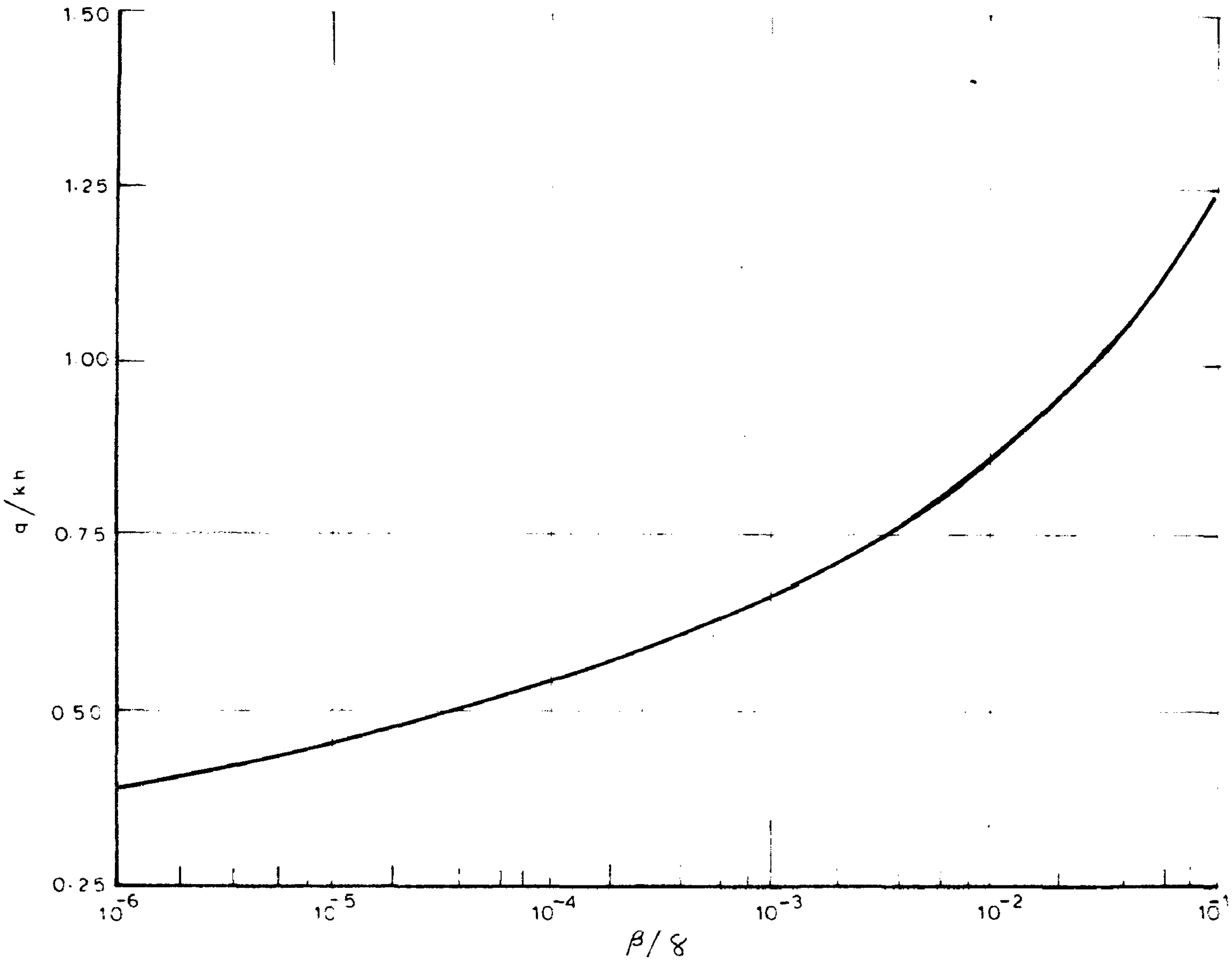


FIG.2.6.4_ SEEPAGE DISCHARGE q/kh

CHAPTER-III

EXPERIMENTAL SETUP AND METHODOLOGY

3.1. GENERAL

Radiotracer experiments were conducted at two different sites, one on the left bank of Ganga Canal and another on the left bank of Deoband Branch to estimate the seepage losses. The details are furnished in the succeeding paragraphs.

3.2. EXPERIMENTAL SETUP

3.2.1. Location of Sites

The first site taken up is on the left bank of upper Ganga Canal near Solani Aqueduct (Mileage 17-0-0 and 17-5-0) about 5 Kms from Roorkee town (Photo 1 and 2).

The Ganga Canal has a trapezoidal cross section and is cut through a gangetic alluvium. The normal discharge in the canal is about 180 m^3/sec. At the experimental station, the canal is about 54m wide at top and has a maximum depth of slightly over 3.5m. The sides are lined with brick masonry. About 75 meters on either side of the central line of the canal, there exists a small shallow drain running parallel to the canal.

The second site is on the left bank of Deoband Branch near Tansipur Village (Mileage 1-4-0 and 1-7-0) about 12 kms. from Roorkee town (Photo 9).

The Deoband Branch takes off from the right bank of Upper Ganga Canal at M22-1F and passes through Tansipur Village (Photo 8). The normal discharge in the distributary is about $19.8m^3$ At the experimental station, the distributary is about 20 ms. wide at top and has a maximum depth of slightly over 1.5 ms. The distributary is unlined.

The exact location of the sites is shown in the index map. (Figure 3.1).

3.2.2. Equipments used for the work

The equipment used for the experimental work is listed below -

1) Levelling instrument and its accessories for surveying work.
2) Hand pump and its accessories for boring work.
3) Filter of 2'' diameter for insertion in the bore holes with galvanished pipes extending upto ground level. Lengths varying, depending on the depth of bore holes.
4) Gauge tape with counterweight to note the water level fluctuations in the bore holes.
5) Sampling equipment for drawing water samples at regular intervals. (Separate equipment was used for each bore hole).
6) Plastic containers of 10cc capacity with screwing lids for sample collection. Other things such as measuring jar, bucket, pipettes, surgical hand

gloves, and polythene bags for soil sample collection were also used.

7) Parshall flume for measurement of flow in the seepage drain. This was used only for the I site experiment.

8) Current meter, velocity rods, sounding rods and other gauging accessories. Boat was used for gauging purpose.

9) Liquid scientillator spectro-meter type No.ECK (ISS 34) at I.R.I. for the estimation of tritium in samples.

10) Mechanical sieve shaker for analysis of soil samples.

3.2.3. Radio-tracer used for the work

Tritiated water (HTO) (i.e. Tritium having a half life of 12.26 years) has been used in both the experiments.

3.3. METHODOLOGY

3.3.1. Introduction

The point dilution method, sometimes called the single well dilution technique was employed for the experimental work at both the sites. The details of the method are furnished below.

3.3.2. The Principle of the point dilution method

The general principle of the point dilution method (also called as single well dilution technique) (8, 1972) is that when a tracer is injected into the entire volume of water in a well uniformly, its concentration decreases with time. Since the well is encountered in sub-soil water flow, fresh water enters the well and water concentrated with isotope leaves the well uniformly in the direction of sub-soil flow. Thus the water in the well gets diluted. However, total volume of water in the well remains the same. Thus, the decrease in concentration follows a simple exponential law as the fresh water flows into the well at a constant rate throughout due to sub-soil flow. The following assumptions are made for the derivation of the expression for the dilution of tracer in this method.

1. A permanent system of flow is established - This assumption is perfectly correct in the case of seepage flow from canals only when water level is in the canal remains constant at the cross-section during the period of experiments.

2. The gradient of vertical potential does not exist (Hypothesis of Dupuit).

3. The concentration of the tracer is homogeneous at any time during measurements. This is also true for tritiated water (HTO) due to similar properties as H_2O. Considering the decrease in concentration with respect to total concentration, the mass equation of the tracer can be written as

Loss of activity in volume Q_o = Activity gone in the direction of flow in time interval dt

or

$$d_c Q_o = - C_o \ d_q \qquad \ldots(1)$$

Where d_c is the change in concentration of the tracer produced by incoming water d_q in time dt.

Q_o is the quantity of water in the well $= \frac{\pi d^2 h}{4}$ where d is the diameter of the well and h isthe height of water column.

$C_o =$ initial concentration of the tracer at $t = 0$

Equation (1) can be written as follows -

$$d_c Q_o + C_o d_q = 0$$

$$\text{or} \quad d_c Q_o + C_o V_m A\, dt = 0 \qquad \text{.. (2)}$$

Where $d_q = V_m A\, dt$

V_m being tracer dilution velocity in the bore hole

A is area of cross-section of flow

Integrating the equation (2) we get

$$C = C_o\, e^{-(V_m A t / Q_o)}$$

Where C = concentration at time t. Substituting $Q_o = \frac{\pi d^2 h}{4}$ and A = dh for unit length of canal, we have,

$$C = C_o\, e^{-(V_m A t/Q_o)} \quad \text{or} \quad C/C_o = e^{-(4\, V_m t/\pi d)} \qquad \text{...(4)}$$

Taking log of either side,

$$\text{In} \left| \frac{C}{C_o} \right| = - \frac{4V_m t}{\pi d}$$

$$\text{or} \quad \text{In} \left| \frac{C_o}{C} \right| = \frac{4V_m t}{\pi d}$$

Therefore the observed velocity V_m of the horizontal water flow through the bore hole is given by the formula

$$V_m = \frac{\pi d \ \ln \left| \frac{C_o}{C} \right|}{4 \ t}$$

Where d = diameter of the bore hole

t = time required for the concentration to reduce by factor 'e' i.e. $\ln | C_o/C | = 1$ or $C_o/C = 2.713$. Where C_o and C are the initial and final concentrations.

Systematic studies on experimental flow patterns have led to the conclusion that the dilution velocity V_m inside the bore hole and the apparent or filtration velocity V_f, existing in the absence of bore hole, are not identical. Thus, the hydrodynamic regime of the ground water undergoes a local modification just at the measuring point, due to boring through the aquifer layer. The variation of the radioactive tracer concentration inside the measuring volume can also be due to vertical currents, radioactive disintegration, molecular diffusion, dispersion, thermal effects, adsorption, ion exchange, mixing phenomena. etc.

All these factors considered and the dilution velocity measured inside the borehole will be given by

$$V_m = \alpha V_f + V_c + V_v + V_t + V_d$$

where

V_f = filtration velocity

α = correction factor for hydrodynamic field distortion

V_c = velocity due to density convection

This may be neglected in the present study as tracer is HTO.

V_v = Vertical current velocity
Vertical current in the present study are also not considered.

V_t = Dilution velocity resulting from mechanical mixing up oftracer during homogenization process.
This effect is neglected as HTO is taken as mixed thoroughly.

V_d = apparent velocity due to molecular diffusion of tracer.
The correction for field distortion has been applied in the present experiment as it influences the filtration velocity considerably.

3.3.3. Factor influencing the measurement of the apparent Velocity

Several different parameters may influence the measured apparent velocity.

The most important parameter that should be considered is due to hydrodynamic field distortion. This correction has been applied in the present experiment.

3.3.4. Filter Tube

As to the degree of perforation of the filter tube, the ratio between void surface and total surface of the filter tube. should exceed 30 percent. Filter tubes with a perforated surface below this value can be used if approximate corrections are performed. Filter tubes should be perforated all along the height of the permeable layer. The filter tube used for the experiment had 47 percent perforations.

3.3.5. Isolation of the Measuring Volume

The relationship derived for the filtration velocity of the ground water can be applied in practice only if required conditions are met. If the measuring volume is not isolated, the filtration velocity is no longer correlated with the decrease of radioactive concentration. However, it was observed in practice that for small diameters (3''), the isolation of the measuring volume is not necessary (8,1972).

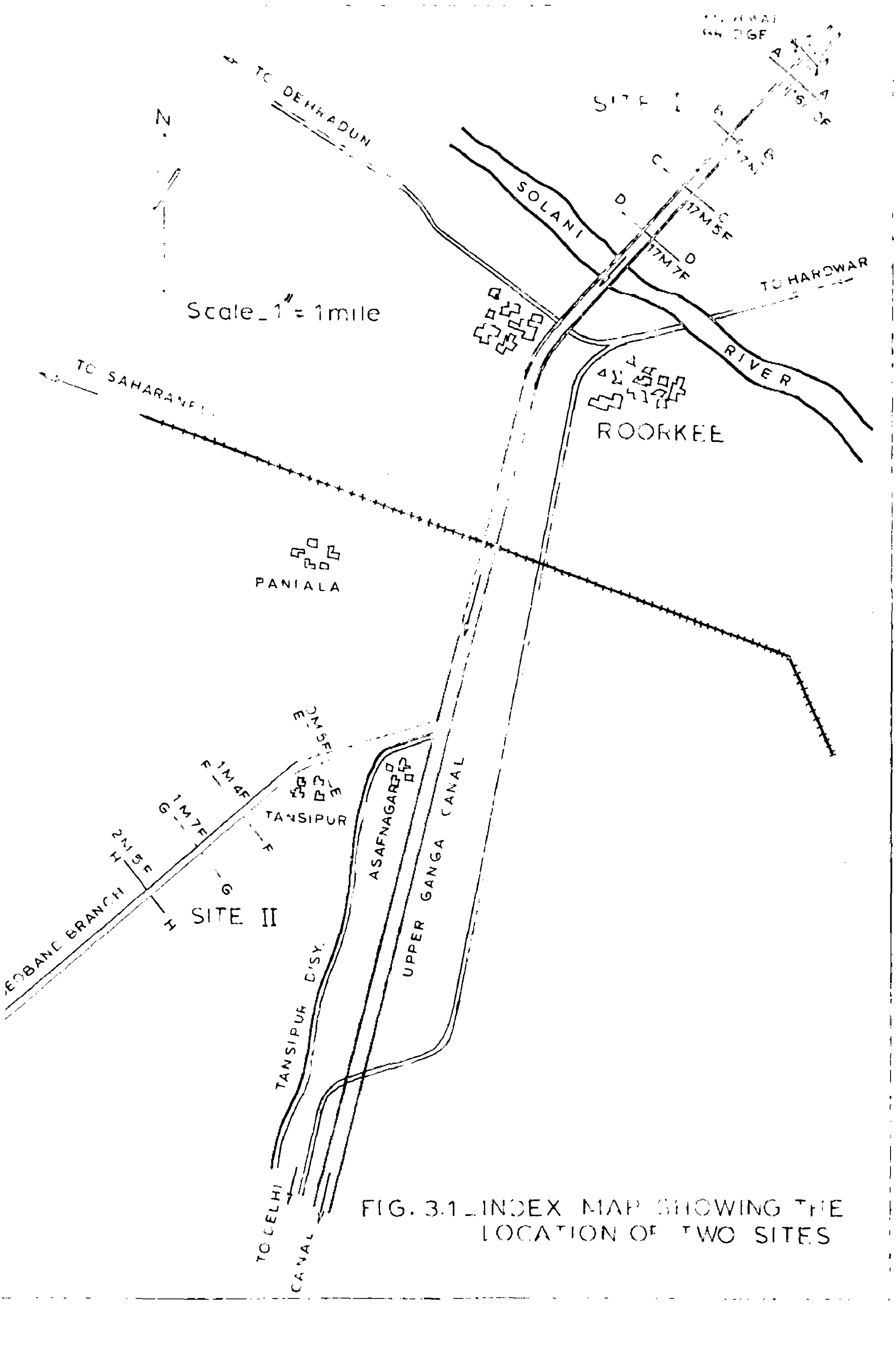


FIG. 3.1_INDEX MAP SHOWING THE LOCATION OF TWO SITES

CHAPTER-IV

WORK PLAN, OBSERVATIONS AND COMPUTATIONS

4.1. WORK PLAN

I- Site- GANGA CANAL

Four different cross section namely AA, BB, CC and DD are shown in the Fig.4.1. Discharge measurements were done at AA and DD by current meter gauging. Bore holes B_1, B_2 and B_3 were made along the section BB and C_1, C_2 and C_3 along CC as shown in the Figures 4.1.1. and 4.1.2.

II- Site DEOBAND BRANCH

In the same way, the cross sections at this site are named as EE, FF, GG and HH (Fig.4.2.). Gauging was done at the sections EE and HH. Bore holes, F_1,F_2,F_3 and F_4 were made along Sec.FF and G_1,G_2,G_3 and G_4 along GG as shown in the figures 4.2.1. and 4.2.2.

4.2. SCHEDULE OF EXPERIMENTAL WORK DONE

The field work for the experiment at I site was spread over for a period of twentyone days from 1.1.78 to 20.1.78. The work at Side II was completed in eighteen days from 27.1.78 to 13.2.78.

The countingwork at the Irrigation Research Institute, Roorkee, was carried out in ten days from 22 2.78 to 3.3.78.

4.3. DETAILS OF FIELD WORK INVOLVED

4.3.1. Surveying

Surveying was done to have the complete profile of the left bank cross sections at BB, CC (I Site) and FF, GG (II site), Levels were taken at every 5 meters interval (Photo 10). The profile of the bed of the canal was also obtained by taking soundings. Levels of water surface were also noted at all the cross sections.

4.3.2. Boring work

The position of boreholes were marked on the cross-sections BB, CC, FF and GG as shown in the Figures 4.1 and 4.2. It was decided to have six bore holes for the I site and eight bore holes for the II site. Boring was made by Sand pump manually (Photo 12 and 13). The diameter of the bore holes was 3.25'' and the depth varied with the depth to ground water. After the completion of boreholes, it was found that the walls inside some of the bore holes were sliding down frequently. Hence the necessity of filter pipes arose. These filter pipes of 2'' diameter were got done in a local workshop at Roorkee.

The filter pipes had a sealed cap at the bottom. The influence of vertical currents is prevented, thus eliminating the correction for the same. The filter pipes were connected by varying lengths of galvanised pipes depending on the depth of bore hole.

4.3.3. Soil sample collection

Soil samples at different depths were collected at the time of boring work (Photo 11). These were later tested for sieve analysis at the Ground Water Survey Division, Roorkee.

4.3.4. Experimental Part

1) The water levels in the canal at all the eight

cross sections (i.e. at AA,BB, CC, DD, EE, FF, GG and HH) were noted daily during the duration of experiment. The water levels in all the ten bore holes namely, B_1,B_2,B_3 C_1 C_2 C_3 F_1 F_2 F_3 F_4 G_1 G_2 G_3 and G_4 were also noted daily with the help of gauge tape and counterweight (Photo 4 and Photo 6).

2) Before injecting tritiated water, a background survey was made and water samples were collected from all the boreholes. Tritiated water was added to the bore holes B_2 B_3 C_2, C_3, F_1,F_2,F_3,G_1,G_2 and G_3 with the help of a funnel and a long plastic tube (Photo 15). Definite quantity of tritiated water was added at the timings and dates as given in the tables.

3) Mixing was ensured by stirring the water inside the bore hole slowly with a rod. In order to allow for good solution homogenisation the first sample was collected after a lapse of atleast 20 minutes from the time of injection (Photo 5). This has been taken as the initial activity. The samples so collected were sealed and marked properly. The sample drawing equipment consisted of a lengthy polythene tube with a suction pump at one end (Photo 17,18 and 19). The other end was connected to apipette. The upper portion of the tube is tied to a wooden stick to keep it straight while lowering the same into bore hole.

4) Every care was taken to keep the plastic containers containing samples free from any contamination. Separate sample drawing equipment was maintained for each bore hole

After drawing the samples, the whole equipment was cleaned with Teepol for 5 to 10 minutes and later in running water for about 30 minutes everytime (Photo 20). Surgical hand gloves were used for protection during sampling work.

5) Samples were drawn continuously at the desired interval for about ten days. At the I site, samples were drawn very frequently. It was later felt that the interval should be at a longer duration as the dilution rate was found to be slow. Moreover the possibility of contamination will get reduced if simples are drawn at a longer interval. Samples were drawn at a longer interval in the case of II site.

6) Gauging was got done by current meter and also by velocity rods at the cross-sections shown in the Fig.4.1 and 4.2. The gauging was done for four days at both the sites. Two point method (0.2 and 0.8 d) was adopted for current meter gauging of Ganga canal(Photos 3 and 4). One point method (0.6d) was adopted for Deoband Branch. (Photo 24,25 and 26).

7) The most important requirement of the experiment is the steady discharge in the canal. This was not possible in the case of Ganga Canal due to practical difficulties. In the case of Deoband Branch, steady discharge was maintained throughout the duration of experiment. Gauging was also done by velocity rods for Deoband Branch the details of which are furnished in Table 4.8.1. To have an idea of the quantity of seepage loss through the selected reach of Ganga Canal, 6'' Parshall flume was used to

measure the flow in the seepage drain at 16M-2F and 17MJF (Photo 6 and 7). This was carried out for four days. The readings are tabulated in Table. 4.4.2.

4.4. COUNTING WORK

4.4.1. THE DETECTION DEVICE

The radioactive tracer detection device, as any other device for nuclear radiation detection, consists of both a detector and auxiliary electronic equipment for radiation counting. The interaction process, in which the incident particle partially or totally releases its energy, occurs inside the detector. Otherwise, after interaction, the incident particle may leave the detector or remain inside it.

A certain amount of ionisation occurs inside the detector sensitive volume, as a result of interaction with the incident particle. The ionisation represents the 'information' about the incident particle transmitted by the detector to the electronic equipment for radiation countings.

The Liquid Scientillation System (LSS34) which is at present in the Isotope Laboratory, Irrigation Research Institute, Roorkee, transform the information from the detector into a more usuable form, number of pulses, counting rate. This quantity is plotted by a plotter. The details are given in the following paragraphs.

4.4.2. Liquid Counting Scintillation Systems

The liquid Scientillation System (LSS) finds applications in a variety of fields such as biophysics, biochemistry, pharmacy, medicine, geology, industrial agricultural and veternary research.

In hydrology and meteorology the LSS enables measuring of tritium concentration in rain and ground water. This helps in determining the age of reservoirs and holding times of water in atmosphere and on land.

The Liquid Scientillation system (LSS) provides precision counting of soft β-emitting radioisotopies such as carbon-14 (C-14) and Tritium (H-3). It consists of heavily shielded detector assembly in which selected and matched photo-multiplier tubes are housed with sophisticated and proven counting electronics to give accurate counting data. The LSS is shown in Fig.4.3.

The counting sample consists of(a) the radioactive material under study, (b) an organic solvent, and (c) an organic phosphor. Radiation emitted by the radioactive material is absorbed by the solvent with transfer of energy. The solvent, in turn, transfers this energy to the phospher which emits a burst of light photons. The photons are then collected on the photo-cathode of photomultiplier tube which converts them into an electronic pulse. The pulse which is proportional to the energy of radiation - after suitable amplification is recorded as a count signifying the emission of the particle or radiation.

4.4.2.1. Operational Details

The automatic Liquid Scintillation counting system (LSS 34) mainly comprises of a detector assembly, sample changer, fast electronics, timer dual counter, control unit and printer. The block diagram is shown in Fig.4.3.2. The detector assembly consists of two matched photo-multiplier tubes housed in a brass chamber with lead shielding. The use of two photomultiplier tubes in conjunction with a summing gated amplifer (which has a resolving time of 50 ns) minimises the background due to thermal noise. The inputs of the summing gated amplifer are separately amplified, summed up, gated through a liner gate which is closed or opened by the gate pulse from the gating circuit. The pulses from the summing gated amplifier are integrated and amplified in a logarithmic scale by means of a Video log amplifer. The output pulses of a log amplifer are fed to two single channel analysers which can be used as C-14 channel and H-3 channel respectively. The outputs of single channel analysers are fed to the inputs of a timer dual counter which consists of two scalars, timer and log rate meter. The control unit controls the sequential operation of the timber dual counter, the sample changer and the sample loading mechanism. The maximum attainable efficiencies for C-14 and H-3 are better than 80 percent and 30 percent respectively.

4.4.3. Preparation of Scientillator

The following scientillation chemicals are required to prepare the scientillator.

1) Dioxane -500 cc
$CH_2.CH_2.O.CH_2.CH_2.O$

2) PPO
Diphenyloxazole -3.5 grams
$C_{15}H_{11}$ NO

3) POPOP
1,4, - D_i 2 - (5-phenytoxazolyl) benzene
0.15 grams.

4) Napthalin 50 grams
$C_{10}H_s$

The chemicals at Sl.No.(2) and (3) were weighed in microbalance.

The scientillator so prepared will suffice for counting fifty samples.

4.4.4. Counting of Samples

Ten C.C. of scientillator was transformed into the special glass containers which were free from any contamination. These containers were placed in the counting chamber of the liquid scientillation equipment in order. The background counting was noted for all the containers. The containers were taken out after counting and kept on a clean table free from any contamination.

One C.C. of sample was drawn from the plastic containers with the help of standardised syringe. Separate syringes were used for each sample. Every care was taken to draw exactly 1 C.C. This was added to the glass containers containing scientillator which were later placed in the counting chamber. The counts were plotted by a plotter for every 100 sec.

4.5 OBSERVATION AND COMPUTATIONS

The details are given separately for the two sites.

4.6 I SITE - GANGA CANAL

4.6.1 Direct Method To Evolve The Seepage Discharge:

Inflow - outflow method was employed to estimate the losses. The discharge was measured at two different cross sections, one at 16 M -2F and another at 17 M-7F. Current meter was used to measure the velocity at 0.2 and 0.8 depths. The gauging was done by boat for three days continuously (Photo 3 & 4). The results are tabulated in Table 4.4.1. As the flow in the canal was not steady and as the reach was very short, this did not give any useful information. There used to be fluctuations in the water level from time to time.

Measurements were done in the seepage drain by means of six inches Parshall flume (Photo 6 and 7). The flow was noted daily for four days at 16 M-2F and 17 M - 7F. The data is tabulated in Table 4.4.2

4.6.2 Nuclear Technique To Evolve The Filtration Velocity:

Single well dilution technique was used to measure the filtration velocities at the two cross sections, namely 17 M 0 F and 17 M 5F.

Three bore holes of 3.25" diameter were made at each cross section on the left bank. The positions of bore holes can be seen in Photo 1 and 2.

Filter tubes of 2" diameter were used for bore holes B_2 and C_2. Tritiated water was injected into bore holes, B_2 , B_3, C_2 and C_3.

Samples were drawn for ten days at different intervals. Separate samplers were used for each bore hold.

Samples were counted at the Irrigation Research Institute, Roorkee.

The water levels in all the bore holes were noted daily. This is given in Table 4.2.

The rate of dilution of tracer with time is plotted on an ordinary graph (Fig. 4.4.1 to 4.4.4).

A straight line of In C Versus time was fitted by the method of least squares for each bore hold. The slope "m" of the straight line is directly related to the Filtration Velocity. This is determined for each bore hole. The details are tabulated in Tables 4.3.1 to 4.3.4.

Correction of hydrodynamic field distortion due to the presence of bore hold has also been applied. The correction "α" being 2, for the boreholes having a diameter of 3.25" (Gasper E and M, Oncescu,1972).

Premeability 'K' is evolved knowing the actual filtration velocity and hydraulic gradient of the free surface. The details calculated for each of the bore holes are tabulated in Table 4.4.

The results of soil tested for the two cross sections at different depths are shown in Table 4.5.

4.6.3 Computation Of Seepage Discharge By Different Methods

4.6.3.1 Emperical Method:

1. Punjab Formula For Lined Canals

$$S = 1.25 \quad Q^{0.0625}$$

where, S = seepage in cusecs per M.S.ft of wetted area

$$S = 1.25\ (6500)^{0.0625}$$

$$= 1.25 \times 1.7310396 = 2.1638 \text{ cusecs per M.S.ft.}$$

$$= 2.605\ m^3/\ \text{day} / \text{m length of Ganga Canal.}$$

4.6.3.2 Using I.R.I. Formula:

Once the filtration velocity V_f, at a distance 'd' from the centre line of the canal, has been determined, the seepage discharge may be calculated by the following formula, which is the modification of the formula proposed by Dr. V.I.Vaidhianathan.

$$F = 2\ V_f \ d\ \theta \ \ \text{Cosec}\ \ \theta$$

where, F = Total seepage discharge per unit length of canal.

θ = Inclination of phneatic line with the vertical at the point of observation of V_f, in radians.

Cross Section B

Bore hole B_2

V_f = 0.006825 m/day , d = 66.125

θ = 88^o 12' (1.5394 radians)

$$F = 2 \times 0.006825 \times 66.125 \times 1.5394 \times \operatorname{cosec} 88^\circ 12'$$

$$= \frac{1.3894719}{\operatorname{Sin} 88.2} = 1.39015 \ m^3/\ day\ /\ m$$

Bore hole B_3

$$F = \frac{2 \times 0.006825 \times 71.125 \times 1.555}{\operatorname{Sin} 89.083}$$

$$= 1.5098 \ m^3/day/m$$

Average seepage loss for cross section B $= 1.449975 \ m^3/day/m$

Cross Section C_1

Bore hole C_2

$V_f = 0.00686$ m/day, $d = 66.125$, $\theta = 89^\circ 5'$

$$F = \frac{2 \times 0.00686 \times 66.125 \times 1.555}{\operatorname{Sin} 89^\circ.083}$$

$$= 1.4109309 \ m^3/day/m$$

Bore hole C_3

$$F = \frac{2 \times 0.00686 \times 71.125 \times 1.5618}{\operatorname{Sin} 89.4833}$$

$$= 1.52412 \ m^3\ /\ day\ /\ m.$$

$\therefore$ Average seepage loss for cross section C_1

$= 1.4675254 \ m^3$ /day/m

Average loss between 16 M2 F and 17 M 7F is 1.459 m^3/day/meter length

Hence, seepage loss through the left bank of Ganga Canal considering the reach between 16 M2 F and 17M 7F is 0.729375 m^3/day/metre length of canal.

4.6.3.3 Direct Method:

Inflow - Outflow method was employed to estimate the loss through the left bank of Ganga Canal by measuring the flow in the seepage drain at Sections 16M-2F and 17M-7F, S (flow data is shown in Table 4.4.2) by 6" Parshall flume. The method has been described in

Seepage loss from the difference in the flow at two sections = 1.093 cusecs in a distance of 1M 5F.

seepage loss from left bank i.e.

$$= \frac{1.093 \times (0.3048)^2 \times 60 \times 60 \times 24}{8580/\ 3.28}$$

Seepage loss from both the banks:-

$$= 2 \times 1.02226 \ m^3 \ / \ day \ / \ \text{metre length of canal}$$

$$= 2.04452 \ m^3/day\ /m$$

4.7 II SITE - DEOBAND BRANCH

4.7.1 Direct Method To Evolve The Seepage Discharge:

The discharge was measured at section OM - 5F and 2M - 5F. The velocity was measured by velocity rods as well as by current meter.

The whole cross section was divided into six segments and velocities were measured at each of these segments.

Steady discharge was maintained throughout the duration of the experiment which is one of the main requirement for assessing the seepage loss accurately.

The gauging was conducted for three days at both the sections.

4.7.1.1 Velocity Rods:

In artificial canals where outcrops and weeds are not found, velocity can be best measured by simple equipment called ' Velocity Rods' (17). The velocity rods which were used for the present work consisted of special hallow telescopic tubular rods having diameter ranging from 2.5 cms to 5 cms and can be adjusted to the desired depth. Mean velocity is given by

$$0.99\ V_{0.8}\ \text{rod} - 0.09$$

The results are tabulated in Table 4.8.1

4.7.1.2 Current Meter

The velocity was measured at 0.6 depth and the observations were carried out by boat(Photo 22 and 23). The measurements were repeated for three days at both the cross sections. To have a clear check, soundings were taken daily at sections OM-5F and 2M-5F (Photo 21)

The results are tabulated in Table 4.8.1

There are four outlets between these two cross sections. The flow in these outlets is also noted. This is shown in Table 4.8.2

4.7.2 Nuclear Technique To Evolve The Filtration Velocity

Single well dilution method was employed using tritiated water as radio active tracer.

Four bore holes of 3.25 inches diameter were made at each cross sections. Filter tubes of 2 inch diameter were inserted into bore holes F_2, F_3, G_2 and G_3.

Tritiated water was injected into bore holes F_1, F_2, F_3, G_1, G_2 and G_3. The first sample was drawn after certain time to allow for proper mixing of tracer. Samples were drawn for ten days at different intervals from all the six bore holes (Table 4.7.1 to 4.7.6). Separate samplers were used for each bore hole. After each sampling, the samplers were cleaned with Tecpol chemical and later inrunning water to a remove contamination (Photo 20).

The water levels in all the bore holes were noted daily with gauge tape and counter weight.

Samples were counted at the Irrigation Research Institute, Roorkee.

The rate of dilution of tracer with time is plotted on an ordinary normal graph (Fig. 4.5.1 to 4.5.6). A straight line of In C V_s time was fitted by the method Least squares for each bore hole. The slope 'm' is calculated and filtration velocity ' V_f' evolved. Actual filtration velocity is obtained by applying the correction for hydro dynamic field distortion. This is got by dividing the observed filtration velocity by α. α is 2 in the present study.

Permeability 'k' is calculated knowing the actual filtration velocity and hydraulic gradient 'i' of the free surface. The details calculated from the experimental data is given in Table 4.9.1.

Table 4.5 gives the results of soil tested for the two cross sections at different depths.

4.7.3 <u>Computation Of Seepage Discharge By Different Methods:</u>

4.7.3.1 <u>Emperical Method</u>

1) <u>Davis and Wilson Formula</u>

$$Q = 8.64 \times 10^{-3}\ C\ P\ {}^{3}/\ H$$

where, q = the seepage rate in m^3/day per meter length of canal.

P is the wetted perimeter in meters

H is the water depth in the canal in meters, and

C is the soil coefficient ranging from 12 for loams to 70 for sandy gravels.

Taking $C = 20$, $P = 21$ m , $H = 1.622$ m

$$q = 8.64 \times 10^{-3} \times 20 \times 21\ {}^{3}/\ 1.622$$

$$= 4.2638\ m^3\ /\ \text{day}\ /\ \text{meter length of canal.}$$

4.7.3.2 <u>Using Experimental Data</u>

1. Numerov's Approach

The method has been explained in detail in chapter.

<u>Main Assumptions</u>:

A curvilinear perimeter of canal is assumed whereas the Deoband Branch has a trapezoidal cross section.

There is no high permeability under lying layer.

The equation for the phreatic curve as obtained

Numerov is :-

$$x = \frac{q}{2K} + \frac{B - q/K}{2} \text{Cosh} \frac{\pi ky}{q} + H \text{ Sinh} \frac{\pi k y}{q}$$

where x is the distance to the bore hole from the centre line of canal

y is the depth below water surface of canal in each bore hole.

B is the width of water surface in the canal

H is the depth of water in the canal

K is the Permeability of the medium

q is the seepage discharge per unit length

F Section

Bore hole F_2

$x = 22.50$ M's , $y = 1.723$ m's

$B = 19.25$ m's, $K = 1.0932$ meter/day

$H = 1.623$ m's $\quad x = \frac{q}{2K} + \frac{B-q/k}{2} \text{ Cosh} \frac{\pi ky}{q} + H \text{ Sinh} \frac{\pi ky}{q}$

$$x = \frac{q}{2K} + \frac{B - \frac{q}{k}}{2} \left\{ \frac{(e^{x'} + e^{-x'})}{2} \right\} + \frac{H}{2} (e^{x'} - e^{-x'})$$

where $\quad x' = \frac{\pi k y}{q}$

Substituting the values, we have

$$22.50 = \frac{q}{2 \text{ x}1.0932} + \frac{19.25 - \frac{q}{1.0932}}{2 \text{ x } 2} \left\{ e^{\frac{22\text{x } 1.0932 \text{ x } 1.723}{7 \text{ x } q}} + e^{\frac{-22\text{x}1.0930\text{x } 1.723}{7 \text{ x } q}} \right\}$$

$$+ \frac{1.623}{2} \left\{ e^{\frac{22\text{x}1.0932 \text{ x}1.723}{7\text{x } q}} - e^{\frac{-22\text{x}1.093\text{x}1.723}{7 \text{ x } q}} \right\}$$

This equation has been solved by trial and error procedure.

when $q = 4.09\ m^3$ /day/m

$$RHS = \frac{4.09}{2x1.0932} + \frac{19.25 - \frac{4.09}{1.0932}}{2 \times 2} \left(e^{1.4473922} + e^{-1.44739} \right.$$

$$\left. + \frac{1.623}{2} \left(e^{1.4473922} - e^{-1.4473922} \right) \right)$$

= 22.510 m

= L.H.S.

Hence $q = 4.09\ m^3$ / day / 'm' length of canal.

Similar calculations are also done for other bore holes namely F_3 , F_4 , G_2, G_3 and G_4 using the data from table. The final results are tabulated below

TABLE 4.9.2

SEEPAGE LOSS BY NUMEROV'S APPROACH

Bore hole	Seepage loss in m^3/ day per meter length of canal
F_2	4.090
F_3	3.6080
F_4	3.2860
G_2	4.1434
G_3	3.6900
G_4	3.3720
Average loss	3.6982

Using Numerov's equation, the values of $\pi y/B$ and x/B are calculated for various ratios of q/KB. This has been tabulated in Table 4.9.3. The curves are also plotted in Fig. 4.7. The seepage discharge can be directly interpolated from these curves using x, y, K and B values.

(2) Dachler's Approach

Dachler has obtained a solution for seepage from a trapezoidal channel underlain by an impermeable layer at shallow depth D_i from the bottom of channel. The drainage has been assumed to be at a depth of D_w from water surface.

This approach has been explained in Chapter 2. The flow system has been divided into two regions. $\triangle H$ is the vertical distance at the dividing line which is at a distance L_1 from centre line of canal.

$$L_1 = \frac{W_s + H_w + D_i}{2}$$

Where W_s = width of water surface

H_w = Depth of water in canal

D_i = Depth to the impermeable layer from bottom of channel.

F Section - 1M4F

$$L_1 = \frac{19.25 + 1.623 + 40}{2} = 30.43 \text{ m's}$$

ΔH from actual phreatic line

$$= 263.203 - 261.43 = 1.773 \text{ m}$$

To find out form Factor 'F'

$$\frac{H_w + D_s - \Delta H}{H_w + D_i} = \frac{1.623 + 40 - 1.773}{1.623 + 40}$$

$$= 0.9574033$$

$$\frac{W_s}{D_i + H_w} = \frac{19.25}{40+1.623} = 0.4624846$$

$$\frac{W_s}{W_1P} = \frac{19.25}{20.74} = 0.9281$$

W_1P = Wetted perimeter = WP

As $\frac{W_s}{W_1P} > 0.9000$, this comes under shallow channel.

Referring to Figure 2.1, we get

Form Factor = 0.78 corresponding to

0.4624846 and 0.9574033.

The flow is given by :-

$$\frac{I_s}{K} = \frac{2 F \Delta H}{W_s}$$

Where K = Hydraulic Conductivity

I_s = Seepage rate in unit time per unit width of water surface.

F = Form Factor,

ΔH = Vertical distance at the dividing line, and

W_s = Water surface width of canal.

Substituting the values, $K = 1.0932$

$\Delta H = 1.773$

$F = 0.78$

$W_s = 19.25$

$$I_s = \frac{2 \times 0.78 \times 1.773 \times 1.0932}{19.25}$$

$= 0.14368\ m^3$/day/unit width of water surface.

$I_s = 3.0236148\ m^3$/day/meter length of canal

Section G - 1 M 7 F

Similar procedure is carried out as under:

$$L_1 = \frac{19.09 + 1.60 + 40}{2} = 30.345\ m$$

ΔH from actual phreatic line

$= 263.10 - 261.375 = 1.725\ m$

$$\frac{H_w + D_i - \Delta H}{H_w + D_i} = \frac{1.60 + 40 - 1.725}{1.60 + 40}$$

$= 0.9585$

$$\frac{W_s}{D_i + H_w} = \frac{19.09}{40 + 1.60} = 0.45889$$

$$\frac{W_s}{\text{Wetted Perimeter}} = \frac{19.09}{20.50} = 0.9312$$

As this is $>$ 0.90, comes under shallow channel criteria.

From Figure 2.2, Form Factor, F = 0.80

$$\frac{I_s}{K} = \frac{2\ F \triangle H}{W_s}$$

$$= \frac{2 \times 0.8 \times 1.725}{19.09} = 0.144578$$

$$I_s = 0.144578 \times 1.1643$$

$= 0.1683321\ m^3$/day/meter width of water surface.

$$\therefore\ I_s = 0.1683321 \times 19.09$$

$= 3.2134\ m^3$/day/meter length of canal.

A_v $3.120\ m^3$/day/m

(3) S.P.Garg and Chawla A.S. Approach

Exact solutions of problem of seepage from canals in homogeneous media to drain located at a finite distance from the canal considering vertical and horizontal drainages are presented by S.P. Garg and A.S.Chawla (1970). This has been described in Chapter 2.

Section F - 1M - 4F

Bed Width of canal = B = 16.50 m's

L = Distance to the natural drainage (Thasipur Nala) = 970 meters.

Vertical Drainage has been considered.

h = Elevation difference between canal water surface and bed of natural drainage.

= 263.203 - 256.163 =7.04 meters.

H = Depth in the canal = 1.623 meters.

$$\frac{L}{H} = \frac{970}{1.623} = 597.65, \quad \frac{B}{H} = \frac{16.50}{1.623} = 10.166$$

From Figures 2.6.1 to 2.6.4, β and γ are

i.e. $\beta = 2.20$, $\gamma = 5500$

$\beta/ = 2.20/5500 = 0.0004$

$h/H = 7.04/1.623 = 4.3376$

α' and $\frac{B-b}{H}$ are read from Figures 2.6.2

$\alpha' = 0.26$, $B-b/H = 1.8$

$\therefore$ $b/H = 10.166 - 1.800 = 8.366$

Again for these, β and γ are read. This process repeated three times till there are no changes in the values, β and γ. Final values are $\beta = 2.40$, $\gamma = 6600$.

$\beta/ = 2.40/6600 = 0.0003636$

From Figure, 4 q/Kh is read,

$q/Kh = 0.570$

$\therefore$ $q = 0.570\ Kh$

$= 0.570 \times 1.0932 \times 7.04$

$= 4.3826\ m^2$/day/metre length of canal.

Section G - 1 M 7 F

$H = 1.60$ metres, $L = 1012$ meters

$B = 16.20$ meters, $h = 263.10 - 256.100 =$

$= 7.0$ m.

$L/H = 1012/1.60 = 632.5$, $B/H = 16.20/1.60 = 10.125$

From Figure 2.6.2, $\beta = 2.5$, $\gamma = 6800$.

$\beta/$ 0.0003676, $h/H = 7.00/1.60 = 4.375$

$\frac{B-b}{H} = 1.6$, $b/H = 10.125 - 1.6 = 8.525$

The final values of β and γ as read from the Figure 2.6.2 are 2.60 and 8800 respectively.

$\beta/ \quad = 2.60/8800 = 0.0002954$

From Figure 2.6.4, q/Kh corresponding to β/γ is 0.530

$\therefore$ $q/Kh = 0.530$

$q = 0.530 \times 1.1643 \times 7$

$= 4.31955\ m^3$/day/metre length of canal

Therefore seepage loss through Deoband Branch

is $\frac{4.3826 + 4.31955}{2} = 4.351075\ m^3$/day/metre length of canal.

4.7.3.3. Direct Method

Inflow-Outflow Method

The discharge measurements carried out by current meter has been considered.

There is a loss of (average value taken from three days readings) 5.8766 cusecs in a reach of 2 miles.

Converting it to metric units for one metre lmngth of canal, it gives

$$\frac{5.8766 \times (0.3048)^3 \times 60 \times 60 \times 24}{10560/3.28084}$$

$= 4.466894\ m^3$/day/metre length of canal.

Table 4.1.

Water Level (in ms) in Ganga Canal

Date	16 M-2F	17 Mile	17M-5F	17 M
10.1.78	265.680	265.509	265.338	265.202
11.1.78	265.451	265.274	265.108	264.97
12.1.78	265.680	265.509	265.338	265.202
13.1.78	265.680	265.509	265.338	265.202
14.1.78	265.451	265.274	265.108	265.99

Table 4.2.

Bore Hole Water Levels at 17M-0 F and at 17M 5F
(Section B)

Date	Levels in meters in bore holes					
	B_1	B_2	B_3	C_1	C_2	C_3
10.1.78	256.31	256.15	256.07	255.993	255.913	255.868
11.1.78	256.31	256.15	256.07	255.993	255.913	255.868
12.1.78	256.30	256.105	256.07	256.003	255.918	255.868
13.1.78	256.30	256.35	256.07	255.993	255.928	255.868
14.1.78	256.31	256.15	256.07	255.993	255.913	255.868
15.1.78	256.31	256.15	256.07	255.993	255.913	255.868
16.1.78	256.31	256.15	256.07	255.993	255.913	255.868
17.1.78	256.32	256.15	256.07	255.988	255.913	255.868
18.1.78	256.31	256.15	256.07	255.993	255.913	255.868

Finding Filtration velocity from the slope of straightline

The equation for the filtration velocity is,

$$V_{f_o} = \pi d/4t \ \ Ln \ C_o/C$$

$$V_{f_o} = \pi d/4t \ [Ln \ C_o - LnC]$$

$$\therefore \ LnC = Ln \ C_o - \frac{4t \ V_{f_o}}{\pi d}$$

$$y = C + (-m) \ x$$

Where d= dia. of bore hole

t = time

C_o = initial activity

C = concentration at time t.

Where $m = \text{slope} = - 4V_{f_o} / \pi d$

$\therefore$ Filtration velocity = $V_f = V_{f_o} / \alpha$

Where α = 2 is the correction applied for hydrodynamic field distortion

$$\text{Slope} = \frac{\Sigma xy - N \bar{x} \bar{y}}{\Sigma x^2 - N \bar{x}^2} = m$$

r = Correlation coefficient

$$= \frac{\Sigma xy - N \bar{x} \bar{y}}{\sqrt{\Sigma x^2 - N \bar{x}^2} \ \sqrt{\Sigma y^2 - N\bar{y}^2}}$$

The forthcoming tables gives the calculations for all the bore holes.

TABLE 4.3.1 : SAMPLE COUNT RATE DATA OF BORE HOLE B_2

(Ganga Canal)

Activity injected = 60 μci Dia. of Bore hole = 2"

Time of injection = 10.00 A.M. (10.1.78)

Time & Date of sampling	Mean counts per 100 secs. above back-ground "C"	Time 't' in hours = X	ln C = Y	From Fitted Straight line equation		
				Y'	Y = Y - Y'	Count per 100 sec.
1	2	3	4	5	6	7
10.1.78						
11.15 A.M.	26055	0	10.16797	10.142341	+0.02564	25396
12.25 P.M.	23301	1.16666	10.05625	10.131657	-0.07540	25126
2.22 P.M.	22975	3.11666	10.04216	10.113341	-0.07118	
4.25 P.M.	22869	5.16666	10.03754	10.095025	-0.05748	
8.05 P.M.	23101	8.83333	10.04763	10.061446	-0.013816	23422
11.1.78						
12.56 A.M.	21506	13.6833	9.97608	10.01703	-0.0409	
2.05 A.M.	22594	14.8333	10.02544	10.00649	+0.01894	
6.20 A.M.	22161	19.0833	10.00609	9.96757	+0.038513	21324
10.05 A.M.	22248	22.8333	10.01001	9.93373	+0.0767	
2.15 P.M.	20065	27.500	9.90673	9.89049	+0.01617	
4.45 P.M.	19322	29.500	9.86900	9.87218	-0.00318	19383
12.1.78						
10.40 A.M.	18034	47.4166	9.80001	9.70810	+0.09191	
3.40 P.M.	16149	52.4166	9.68334	9.66231	+0.02103	
6.19 P.M.	16803	55.0666	9.72931	9.63804	+0.09120	15337
13.1.78						
5.09 P.M.	11410	77.9000	9.34224	9.42893	-0.08668	

Table 4.3.1 contd.

1	2	3	4	5	6	7
<u>14.1.78</u>						
4.09 P.M.	9572	100.900	9.16659	9.218	-0.0517	
<u>15.1.78</u>						
7.14 A.M.	7981	115.9833	8.98481	9.080	-0.0953	8778
4.19 P.M.	7891	125.0666	8.97347	8.9969	-0.0235	
<u>16.1.78</u>						
7.09 A.M.	6976	139.9000	8.850231	8.8611	-0.01090	
<u>17.1.78</u>						
7.24 A.M.	6250	164.150	8.74033	8.639	+0.1012	5648
<u>18.1.78</u>						
7.14 A.M.	5043	187.9833	8.52575	8.420	+0.104	
<u>19.1.78</u>						
12.15 P.M.	3292	217.000	8.09925	8.155	-0.0558	3481

$\bar{X} = 64.9772$ $\bar{Y} = 9.54728$ $\Sigma XY = 12810.068$ $\Sigma X^2 = 184349.36$

$\Sigma Y^2 = 2012.9867$ $\Sigma \Delta Y^2 = 0.0859$

Correlation Coefficient $r = \dfrac{XY - N\bar{X}\bar{Y}}{\sqrt{X^2 - N\bar{X}^2}\sqrt{Y^2 - N\bar{Y}^2}} =$

$$= \frac{12810.068 - 22 \times 64.9772 \times 9.5472}{\sqrt{184349.36 - 22(64.9772)^2}\sqrt{2012.98 - 22(9.54)^2}}$$

$$= 0.9998$$

$$m = \frac{XY - N\,\bar{X}\bar{Y}}{X^2 - N\,\bar{X}^2}$$

$$= \frac{-837.639}{91464.56} = -\,0.009158$$

$$C = \bar{Y} - m\,\bar{X} = 9.54728 - 64.9772 \times (-0.009158)$$

$$= 10.142341$$

Equation for the straight line = $Y = -0.009158\,X + 10.142341$

$$\frac{4\,V_{fo}}{\pi\,d} = -\,m = 0.009158$$

$$\therefore\ V_{fo} = \frac{0.009158 \times 22 \times 2 \times 2.54}{7 \times 4}$$

$$= 0.00101 \text{ m/sec.}$$

$$V_f = \frac{0.00101}{2}$$

Where 2 is the correction applied for hydrodynamic field distortion.

$$= 0.000505 \text{ m/sec}$$

TABLE 4.3.2 : SAMPLE COUNT RATE DATA OF BORE HOLE B_3

Activity injected = 216 μ ci   Dia.of Bore Hole = 3.25"
Time of Injection = 10.10 A.M. (10.1.78)

1	2	3	4	5	6	7
<u>10.1.78</u>						
11.19 A.M.	42184	0	10.6498	10.6027	0.0471	40244
<u>11.1.78</u>						
6.19 P.M.	25937	31	10.16343	10.2343	-0.0709	27842
<u>13.1.78</u>						
7.19 A.M.	18294	68	9.81432	9.7947	+0.0195	17938
4.49 P.M.	16012	77.50	9.68109	9.6818	-0.0007	16023
<u>15.1.78</u>						
4.49 P.M.	9037	125.5	9.109083	9.1115	-0.0025	9059
<u>16.1.78</u>						
1.19 P.M.	7152	146.0	8.87514	8.868	+0.0069	7101
<u>18.1.78</u>						
6.29 P.M.	3780	199.166	8.23747	8.2363	+0.0011	3775

$\bar{X} = 92.4522$ $\bar{Y} = 9.5043$ $\Sigma XY = 5812.3167$ $\Sigma X^2 = 88324.83$

$\Sigma Y^2 = 636.3578$ $\Sigma \Delta Y^2 = 0.00768$

$r = -0.99833$ $m = -0.0118814$

$$Y = -0.0118814 X + 10.602761$$

$$V_{fo} = \frac{-m\pi d}{4} = \frac{22x3.25x2.54x0.0118814}{7 \times 4} = 0.00214 \text{ m/sec.}$$

$\therefore V_f = 0.00107$ m/sec

TABLE 4.3.3. : SAMPLE COUNT RATE DATA OF BORE HOLE C_2

Activity Injected = 60 μci   Dia.of Bore Hole = 2"

Time of Injection = 10.20 A.M. (10.1.78)

1	2	3	4	5	6	7
10.1.78						
11.[illegible] A.M.	26551	0	10.18682	10.1319	0.0549	25132
1.05 P.M.	22676	1.25	10.02906	10.120	-0.0910	24834
3.05 P.M.	23157	3.25	10.05048	10.1012	-0.0507	
5.00 P.M.	23426	5.166	10.0616	10.083	-0.0215	23933
8.25 P.M.	23169	8.5833	10.05007	10.050	+0.00095	
11.17 P.M.	21454	11.45	9.97366	10.0239	-0.05024	
11.1.78						
2.20 A.M.	23178	14.50	10.05048	9.995	+0.0553	21916
6.35 A.M.	21609	18.75	9.98086	9.955	+0.0257	
10.13 A.M.	21686	22.383	9.98442	9.9208	+0.0636	
2.35 P.M.	19907	26.75	9.89882	9.8796	+0.0192	19528
5.05 P.M.	19210	29.25	9.863186	9.856	0.0071	
7.05 P.M.	17825	31.25	9.788357	9.8371	-0.049	
12.1.78						
10.00 A.M.	16622	46.166	9.718483	9.6965	0.02198	16261
5.55 P.M.	16984	54.0833	9.740027	9.6218	0.1181	
13.1.78						
5.50 P.M.	11644	78.00	9.362546	9.396	-0.0342	
14.1.78						
3.55 P.M.	9926	100.083	9.202913	9.188	+0.01489	9779
15.1.78						
7.45 A.M.	8223	115.916	9.014691	9.0386	-0.0239	
11.50 A.M.	7917	120.00	8.97676	9.00018	-0.0234	8103
7.00 P.M.	7177	127.166	8.8763	8.9325	-0.0539	
16.1.78						
11.50 A.M.	6615	144.000	8.797095	8.7738	+0.0232	6463
17.1.78						
2 P.M.	5037	170.166	8.524566	8.527	-0.0024	
19.1.78						
12.50 P.M.	3242	217.00	8.083946	8.0853	+0.0014	3246

$\bar{X} = 61.144 \quad \bar{Y} = 9.5553 \quad \Sigma\ XY = 12062.77 \quad \Sigma X^2 = 165800.50$

$\Sigma Y^2 = 2016.22 \quad \Sigma\Delta\ Y^2 = 0.04567$

$r = 0.9978$

$m = 0.009431$

$C = \bar{Y} - m\,\bar{X} = 10.131949$

Equation for the straight line is

$$\underline{Y = -0.009431\ X + 10.131949}$$

$$\frac{4V_{fo}}{\pi\ d} = -m = 0.009431$$

$$\therefore V_{fo} = \frac{22 \times 0.009431 \times 2.00 \times 2.54}{7 \times 4}$$

$$= 0.00104\ m/sec.$$

$$\therefore V_f = 0.00104/2 = 0.000502\ m/sec.$$

TABLE 4.3.4: SAMPLE COUNT RATE DATA OF BORE HOLE C_3

Activity Injected = 216 μci   Dia.of Bore Hole = 3.25"
Time of Injection = 10.25 A.M.(10.1.78)

1	2	3	4	5	6	7
10.1.78						
11.53 A.M.	45039	0	10.71528	10.67081	+0.0444	43080
11.1.78						
7.03 P.M.	27711	31.16	10.22959	10.3022	-0.0726	29798
13.1.78						
7.53 A.M.	20944	68.00	9.949608	9.866	+0.0829	19264
5.58 P.M.	16530	78.083	9.712932	9.7473	-0.0344	17108
15.1.78						
5.08 P.M.	9445	125.25	9.153241	9.1865	-0.0334	9764
16.1.78						
1.53 P.M.	7866	146.00	8.970305	8.9441	+0.026	7662
18.1.78						
6.48 P.M.	4099	198.916	8.31849	8.3183	+0.0017	4098

$\bar{X} = 92.488$ $Y = 9.577$ $\Sigma XY = 5864.60$ $\Sigma X^2 = 88263.73$

$\Sigma Y^2 = 646.243$ $\Sigma\Delta Y^2 = 0.01708$

$r = 0.97104$, $m = -0.0118265$, $C = 10.670809$

The equation for the straight line is

$$\underline{Y = -0.0118265\,X + 10.670809}$$

$$\underline{V_{fo}} = \frac{-m\,\pi\,d}{4} = \frac{-22 \times 3.25 \times 2.54 \times 0.0118265}{7 \times 4} = 0.00213 \text{ m/sec.}$$

$V_f = 0.001065$ m/sec.

TABLE NO. 4.4

SHOWING THE DETAILS OF I SITE CALCULATED FROM THE EXPERIMENTAL DATA (GANGA CANAL)

PARTICULARS	B Section - 17 Mile			C Section - 17 M 5 F		
	Bore Holes			Bore Holes		
	B_1	B_2	B_3	C_1	C_2	C_3
Filtration Velocity 'V_f' in meters per day		0.0044	0.00925		0.00452	0.00920
Hydraulic gradient "i"		B_1 to B_2 0.032	B_2 to B_3 0.016		C_1 to C_2 0.016	C_2 to C_3 0.009
Angle of Phreaticline " θ "		$88^{\circ}12'$ (1.5394 radious)	$89^{\circ}5'$ (1.5550 radious)		$89^{\circ}5'$ (1.5550 radious)	$89^{\circ}29'$ (1.5618 radious)
Widh of water surface in canal 'B' in meter	51.46			51.65		
Depth of water in the canal 'H' in meter	2.489			2.485		
Distance to the bore hole from C/L of canal 'X' in meters	61.125	66.125	71.125	61.125	66.125	71.125
Depth below water surface of canal 'y' ordinate in meters	9.199	9.359	9.439	9.345	9.425	9.470
Premeability 'K' m/day	0.2844			0.5488		

Table 4.4.1. Inflow and Outflow Method
Current meter measurements
(Ganga Canal)

Date	16 M - 2 F		17 M - 7 F	
	Time	Discharge c/s	Time	Discharge c/s
11.1.78	2.30 PM	5912	4.30 PM	5974
12.1.78	11.15 AM	6452	2.00 PM	6365
13.1.78	11.15 AM	6782	12.46 PM	6493

TABLE 4.4.2

"SIX INCHES" PARSHALL FLUME MEASUREMENTS IN THE SEEPAGE DRAIN

Date	16 M 2 F	17 M 7 F	
17.1.78	Head 2.4" 0.16 C/S	Head 9" 1.307 C/S	1.147
18.1.78	Head 1.8" 0.102 C/S	Head 8.4" 1.17 C/S	1.068
19.1.78	Head 2.4" 0.16 C/S	Head 8.4" 1.17 C/S	1.01
20.1.78	Head 2.4" 0.16 C/S	Head 9" 1.307 C/S	1.147
			1.093 C/S

$q = 2.06\ ha^{1.58}$ in a reach of 1 M 5 F

where ha is the upstream head in ft. in the flume

TABLE 4.5 : Showing the results of soil samples

Depth in Cms.	GANGA CANAL											
	17M 0F						1 7M 5 F					
	Gravel %	Coarse Sand %	Medium sand %	Fine sand %	Silt %	Clay %	Gravel %	Coarse Sand %	Medium sand %	Fine sand %	Silt %	Clay %
0 to 30	0	2	3	37	50	8	0	1	4	31	58	6
30 to 80	0	1	10	39	41	9	0	1	7	32	57	3
80 to 140	0	0	6	74	16	4	0	0	5	70	22	3
140 to 250	0	0	4	94	2	0	0	0	1	91	8	0

	DEOBAND BRANCH											
	1 M 4 F						1 M 7 F					
0 to 3	0	1	4	11	82	2	0	1	0	2	97	0
30 to 80	0	1	0	11	84	4	0	0	1	21	78	0
80 to 140	0	0	1	69	30	0	0	3	3	14	80	0
140 to 250	0	0	5	89	6	0	0	0	5	86	9	0

Table 4.6. Water levels in Deoband Distributary

Date	Water levels in mts. at			
	OM 5F (E)	1M-4F(F)	1M-7F(G)	2M-5F(H)
3.2.78 to 11.2.78	263.499	263.203	263.100	262.798

Gauge height of 5'1'' (0-3F) was maintained throughout the above period.

Table 4.8.1 - Inflow and outflow method by velocity rods 'Deoband Branch)

Date	Discharge at OM-5F		Discharge at 2M-5F		Difference c/s
	Time	flow in c/s	Time	flow in c/s	
3.2.78	10AM	576.4	2PM	564.8	11.6
4.2.78	10AM	579.4	2PM	568.4	11
5.2.78	10AM	581.6	2PM	567.8	13.8

By Current meter

Date	OM-5F		2M -5 F		Difference in c/s
	Time	Discharge c/s	Time	Discharge c/s	
9.2.78	11AM	570.9	3PM	563.8	7.1
10.2.78	11AM	571.9	3PM	563.0	8.9
11.2.78	11AM	569.8	3PM	561.9	7.9

Table 4.8.2. - Number of Outlets between OM-5F and 2M-5F

Mileage	Bank	Discharge
1M-2F-98'	Right Bank 6'' pipe	0.67 c/s
1M-5F-490'	Left bank 4'' pipe	0.375 c/s
1M-5F-520'	Right bank 6'' pipe	0.67 c/s
2M- OF-400'	Left bank 4'' pipe	0.375 c/s
	Total	2.09 c/s

TABLE 4.7.1 : SAMPLE COUNT RATE DATA OF BORE HOLE F_1
(DEOBAND BRANCH)

Activity Injected = 230 μci   Dia.of Bore Hole = 3.25"
Time of injection = 7.25 A.M.(3.2.78)

1	2	3	4	5	6	7
3.2.78						
8.05 A.M.	33705	0	10.4254	10.4468	+0.0214	34434
6.10 P.M.	28253	10.0833	10.24896	10.2362	+0.0126	27895
4.2.78						
2.05 P.M.	18590	30.000	9.830379	9.8204	+0.00991	18405
6.2.78						
7.35 A.M.	8113	71.500	9.001223	8.95403	+0.0471	7739
8.2.78						
8.05 A.M.	2852	120.000	7.955776	7.94145	+0.0143	2811
9.2.78						
8.15 A.M.	1455	144.1666	7.282761	7.4369	-0.1542	1697
10.2.78						
8.05 A.M.	1081	168.0000	6.985642	6.9393	+0.0463	1032
11.2.78						
7.35 A.M.	1206	191.50	6.495266	6.4486	+0.0465	632

$\bar{X} = 91.906$ $\bar{Y} = 8.528$ $\Sigma XY = 5463.88$ $\Sigma X^2 = 106194.18$

$\Sigma Y^2 = 598.709$ $\Sigma \Delta Y^2 = 0.031221$

$r = -0.9982$, $m = -0.0208779$ $C = 10.446804$

The equation for the straight line is

$$\underline{Y = -0.0208779\ X + 10.446804}$$

$$V_{fo} = \frac{-m\ \pi\ d}{4} = \frac{22 \times 3.25 \times 2.54 \times 0.0208779}{7 \times 4} = 0.0000376 \text{cm/sec.}$$

$$= 0.00376 \text{ m/sec.}$$

$\therefore V_f = 0.00376/2 = 0.00188$ m/sec.

TABLE 4.7.2.: SAMPLE COUNT RATE DATA OF BORE HOLE F_2

Activity Injected = 18.56 ci Dia.of Bore Hole = 2"
Time of Injection = 7.30 A.M.(3.2.78)

1	2	3	4	5	6	7
<u>3.2.78</u>						
8.00 A.M.	8675	0	9.068201	9.0534	+0.0148	8148
6.00 P.M.	6968	10.0833	8.849084	8.8738	-0.0247	7142
10.35 P.M.	6575	14.5833	8.79103	8.79363	-0.00260	6592
<u>4.2.78</u>						
8.00 A.M.	5235	24.0000	8.563122	8.6258	-0.0627	5574
6.00 P.M.	4817	34.0000	8.479907	8.4476	+0.0322	4664
<u>6.2.78</u>						
7.50 A.M.	2625	71.8330	7.872836	7.7735	+0.09926	2377
1.00 P.M.	2021	77.0000	7.611348	7.6815	-0.0701	2168
<u>7.2.78</u>						
8.10 A.M.	1729	96.1666	7.455299	7.340	+0.115	1541
<u>9.2.78</u>						
8 A.M.	580	144.0000	6.363028	6.4877	-0.124	657
<u>11.2.78</u>						
8 A.M.	288	192.0000	5.662961	5.6324	+0.0304	279

$\bar{X} = 66.3666$ $\bar{Y} = 7.871$ $\Sigma XY = 4583.3792$ $\Sigma X^2 = 79983.32$

$\Sigma Y^2 = 631.109$ $\Sigma\Delta Y^2 = 0.05009$

$r = -0.9925$, $m = -0.0178177$ $C = 9.05348$

The equation for the straight line is

$$\underline{Y = -0.0178177\,X + 9.05348}$$

$$V_{fo} = \frac{-m\,\pi\,d}{4} = \frac{+22\text{x}2\text{x}2.54\text{x}0.0178177}{7 \text{ x } 4} = 0.00197 \text{ m/sec}$$

$V_f = 0.000985$ m/sec.

TABLE 4.7.3.: SAMPLE COUNT RATE DATA OF BORE HOLE F_3

Activity Injected = 18 μci   Dia.of Bore Hole = 2"

Time of Injection = 7.35 A.M.(3.2.78)

1	2	3	4	5	6	7
3.2.78						
8.10 A.M.	7990	0	8.985946	8.95995	+0.059	7785
4.10 P.M.	6900	8	8.839277	8.8445	-0.0052	6936
10.20 P.M.	6489	14.1666	8.777864	8.75559	+0.0222	6346
4.2.78						
8.10 A.M.	5338	24.0000	8.582606	8.61377	-0.03113	5507
5.2.78						
8.00 A.M.	3821	47.8333	8.248268	8.2699	-0.0216	3904
7.2.78						
7.10 A.M.	2096	95.0000	7.647786	7.5895	+0.0582	1977
12.20 P.M.	1718	100.1666	7.448916	7.5150	+0.0661	1835
8.2.78						
7.30 A.M.	1415	119.3333	7.254885	7.2385	+0.0163	1392
11.2.78						
7.50 A.M.	492	191.6666	6.198479	6.19510	+0.0033	490

$\bar{X} = 66.685$ $\bar{Y} = 7.998$ $\Sigma XY = 4322.04$ $\Sigma X^2 = 73163.585$

$\Sigma Y^2 = 582.652$ $\Sigma \Delta Y^2 = 0.01065$

$r = -0.9968$ $m = -0.0144253$ $C = 8.959911$

The equation of straight line is

$$\underline{Y = -0.0144253\ X + 8.9599511}$$

$$V_{fo} = \frac{-m\ \pi\ d}{4} = \frac{22x2x2.54x0.0144253}{4\ x\ 7} = 0.00159\ m/sec.$$

$V_f = 0.000795$ m/sec.

TABLE NO. 4.7.4 : SAMPLE COUNT RATE DATA OF BORE HOLE G_1

Activity Injected = 120 μ ci   Dia.of Bore Hole = 3.25"

Time of Injection = 7.55 A.M. (3.2.78)

1	2	3	4	5	6	7
3.2.78						
8:40 A.M.	21159	0	9.960293	9.9477	+0.01259	20904
4.2.78						
7.40 A.M.	12899	23	9.46490	9.4856	-0.0207	13169
6.25 P.M.	10533	33.75	9.262269	9.2696	0.00739	10610
5.2.78						
7.50 A.M.	8669	47.1666	9.067509	9.0001	0.06738	8103
12.50 P.M.	7614	52.166	8.937744	8.8996	0.03807	7329
5.55 P.M.	6444	57.0833	8.770905	8.800	-0.0299	6634
7.2.78						
7.55 A.M.	2646	95.25	7.880805	8.0341	-0.1533	3084
9.2.78						
7.40 A.M.	1380	143.000	7.229839	7.0748	+0.1549	1182
11.2.78						
10.10 A.M.	406	193.50	6.006353	6.0602	0.0539	428

$\bar{X} = 71.657$ $\bar{Y} = 8.508$ $\Sigma XY = 4871.637$ $\Sigma X^2 = 76836.35$

$\Sigma Y^2 = 664.06$ $\Sigma \Delta Y^2 = 0.05787$

$r = -0.9911$, $m = -0.02009$, $C = 9.9477037$

$$V_{fo} = \frac{-m \pi d}{4} = \frac{22 x 3.25 x 2.54 x .0.02009}{4 \times 7} = 0.00361 \text{ m/sec}$$

$V_f = 0.001805$ m/sec.

The equation for the straight line is

$Y = -0.02009 X + 9.9477037$

TABLE 4.7.5 : SAMPLE COUNT RATE DATA OF BORE HOLE G_2

Activity Injected = 17.8 μci   Dia.of Bore Hole = 2"

Time of Injection = 7.50 A.M.(3.2.78)

1	2	3	4	5	6	7
3.2.78						
8.30 A.M.	7797	0	8.96149	8.9275	+0.03395	7536
10.45 P.M.	59.55	14.25	8.691987	8.6813	+0.0106	5892
4.2.78						
8.30 A.M.	5516	24.00	8.6154	8.5129	+0.1024	4978
5.2.78						
8.50 A.M.	2981	48.333	8.000014	8.0925	-0.0924	3270
6.2.78						
8.50 A.M.	2291	72.333	7.736744	7.6779	+0.0587	2160
2.40 P.M.	1661	78.166	7.415175	7.5772	-0.1620	1953
7.2.78						
8.30 A.M.	1509	96.00	7.319203	7.2691	+0.0500	1435
9.2.78						
1.40 P.M.	536	149.166	6.284134	6.3507	-0.0666	573
11.2.78						
9.00 A.M.	294	192.50	5.68358	5.6021	+0.08138	271

$\bar{X} = 68.4915$ $\bar{Y} = 7.7444$ $\Sigma XY = 4679.4$ $\Sigma X^2 = 83083.41$

$\Sigma Y^2 = 610.538$ $\Sigma\Delta Y^2 = 0.0637$

$r = -0.998$ $m = -0.0172743$ $C = 8.92754$

The equation for the Straight Line is

$$\underline{Y = -0.0172743\,X + 8.92754}$$

$$V_{fo} = \frac{-m\,\pi\,d}{4} = \frac{22 \times 2 \times 2.54 \times 0.0172743}{7 \times 4} = 0.00191 \text{ m/sec}$$

$V_f = 0.0009505$ m/sec.

TABLE 4.7.6. SAMPLE COUNT RATE DATA OF BORE HOLE G_3

Activity injected = 17.8 μ ci   Dia.of Bore Hole = 2"

Time of Injection = 7.45 A.M.(3.2.78)

1	2	3	4	5	6	7
3.2.78						
8.35 A.M.	8831	0	9.0860	9.0373164	+0.04868	8411
4.45 P.M.	7231	8.166	8.8861	8.9184	-0.03232	7468
10.50 P.M.	6930	14.25	8.8436	8.8298	+0.0137	6835
4.2.78						
8.35 A.M.	6562	24.00	8.789	8.6879	+0.1010	5931
5.2.78						
1.40 P.M.	3050	53.0833	8.0228	8.2645	-0.24177	3883
6.2.78						
8.50 A.M.	2738	72.250	7.9149	7.9855	-0.0706	2938
7.2.78						
8.50 A.M.	2148	96.250	7.6676	7.6361	0.03142	2071
8.2.78						
7.50 A.M.	1898	119.250	7.5485	7.3013	0.24712	1482
10.2.75						
7.35 A.M.	748	167.00	6.61740	6.6062	0.01114	740
11.2.78						
8.35 A.M.	466	192.00	6.14418	6.2423	-0.09814	514

$\overline{X} = 74.624$ $\overline{Y} = 7.951$ $\Sigma XY = 5330.19$ $\Sigma X^2 = 97121.26$

$\Sigma Y^2 = 641.29$ $\Sigma \Delta Y^2 = .14811$

$r = -0.98196$

$m = -0.014572$

$C = 9.0373164$

The equation for the straight line is $Y = -0.0145572\ X + 9.0373164$

$$V_{f_o} = -\frac{m\pi d}{4} = \frac{22x\ 2x2.54x0.0145572}{7\ x\ 4} = 0.00161 \text{ m/sec.}$$

$$\therefore V_f = .000805 \text{ m/sec.}$$

Table No. 4.9.1.

Showing the details of II Site Calculated from the experimental data

(Deoband Branch)

Particulars	F Section - 1M 4F				G Section - 1M 7			
	Bore Holes				Bore Holes			
	F_1	F_2	F_3	F_4	G_1	G_2	G_3	G_4
Filtration Velocity "V_f" in metres per day	.01626	.008634	.0069		.0157	.00828	.00698	
Hydraulic gradient "i"	F_1 to F_2 .0181818	F_2 to F_3 .007272	F_3 to F_4 .003636		G_1 to G_2 .0175	G_2 to G_3 .005454	G_3 to G_4 .003636	
Distance to the bore hole from C/L of Canal 'X' in metres	11.50	22.50	28.00	33.50	11.00	23.00	28.50	34.00
Depth below water surface 'Y' of Canal 'Y' in Meters	1.523	1.723	1.763	1.783	1.480	1.690	1.720	1.740
Width of Water surface 'B' in Canal in metres	19.25	19.25	19.25	19.25	19.09	19.09	19.09	19.09
Depth in Canal H	1.623				1.600			
Permeability "K"=V_fi in metres per day	1.0932				1.1643			

Relationship between π y/b and x/b for various ratios of q/KB

x'	Cosh x'	Sinh x'	q/KB = 0.1, ∴ P=0.05 Q	R	X/B =(P+Q+R)	$\frac{\pi y}{B}$	q/KB = 0.13, P = 0.065 Q	x/B (P+Q+R)	$\frac{\pi y}{B}$	q/KB = 0.15, P=0.05 Q	x/B	$\pi y/B$
0.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.05						
0.5	1.12763	0.52110	0.5074335	0.043925	0.6014	0.05	0.490519	0.599	0.065	0.479	0.598	0.075
0.6	1.18547	0.63665	0.53346	0.053665	0.3671	0.06	0.51567	0.634	0.078	0.5038	0.632	0.09
0.7	1.25517	0.75858	0.564826	0.063943	0.6787	0.07	015459	0.6748	0.091	0.5334	0.673	0.105
0.9	1.43309	1.02652	0.64489	0.0865285	0.7814	0.09	0.62339	0.775	0.115	0.609	0.7705	0.135
1.1	1.66852	1.33565	0.750834	0.112586	0.9134	0.11	0.72580	0.9033	0.143	0.709	0.8965	0.165
1.3	1.97091	1.69838	0.886909	0.1431617	1.080	0.13	0.85734	1.065	0.169	0.8376	1.0557	0.195
1.5	2.35214	2.12928	1.05858	0.1794836	1.288	0.15	1.02329	1.267	0.195	0.999	1.2534	0.225
1.8	3.10747	2.94217	1.39836	0.2480046	1.696	0.18	1.3517	1.665	0.234	1.320	1.643	0.27
2.0	3.76220	3.62686	1.69299	0.3057192	2.248	0.20	1.6365	2.007	0.26	1.598	1.979	0.3
2.3	5.03722	4.93696	2.266749	0.4161516	2.732	0.23	2.188	2.599	0.299	2.1408	2.632	0.345
2.5	6.13229	6.05020	2.759305	0.5099964	3.319	0.25	2.6675	3.24	0.325	2.6062	3.1912	0.375

$$\frac{X}{B} = \underbrace{\frac{q}{2\,KB}}_{P} + \underbrace{\left(0.5 - \frac{q}{2\,KB}\right)\,\mathrm{Cosh}\left(\frac{KB}{q}\right)\frac{\pi y}{B}}_{Q} + \underbrace{\frac{H}{B}\,\mathrm{Sinh}\left(\frac{KB}{q}\right)\frac{\pi y}{B}}_{R}$$

$$x' = \frac{KB}{q}\,\frac{\pi y}{B}\ , \quad \frac{H}{B} = 0.0842931$$

q/KB = 0.2 , P = 0.1			q/KB = 0.22 , P = 0.11			q/KB = 0.24 , P = 0.12		
Q	x/B	πy/B	Q	x/B	πy/B	Q	x/B	πy/B
0.451	0.594	0.1	0.439	0.593	0.11	0.428	0.5919	0.12
0.474	0.6276	0.12	0.462	0.625	0.132	0.450	0.623	0.144
0.508	0.6659	0.14	0.[illegible]89	0.6629	0.154	0.476	0.6590	0.168
0.573	0.7595	0.18	0.5589	0.7554	0.198	0.5445	0.75102	0.216
0.66[illegible]	0.8795	0.22	0.650	0.8725	0.242	0.6340	0.8665	0.264
0.788	1.[illegible]3116	0.26	0.768	1.[illegible]2116	0.286	0.7489	1.0120	0.312
0.94[illegible]	1.2194	0.3	0.9174	1.2068	0.33	0.8939	1.19338	0.36
1.2428	1.59[illegible]8	0.36	1.2119	1.5699	0.396	1.18[illegible]8	1.5488	0.432
1.505	[illegible].911	0.4	1.467	1.8827	0.44	1.4296	1.8553	0.48
2.[illegible]148	2.531	0.46	1.9645	2.49[illegible]6	0.5[illegible]6	1.914	2.45[illegible]	0.552
2.4529	3.[illegible]628	0.5[illegible]	2.391	3.[illegible]109	0.55	2.330	2.[illegible]599	0.6

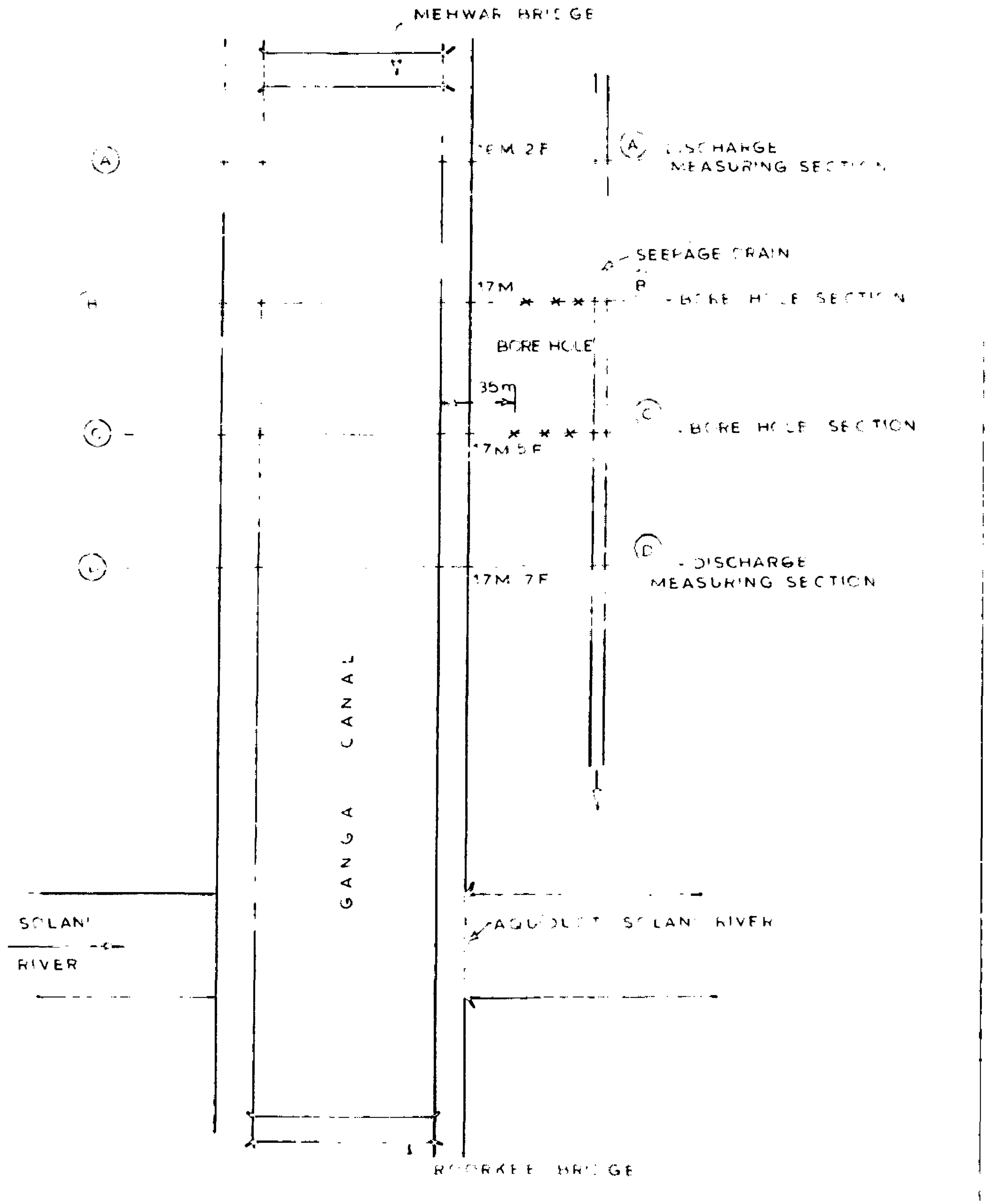
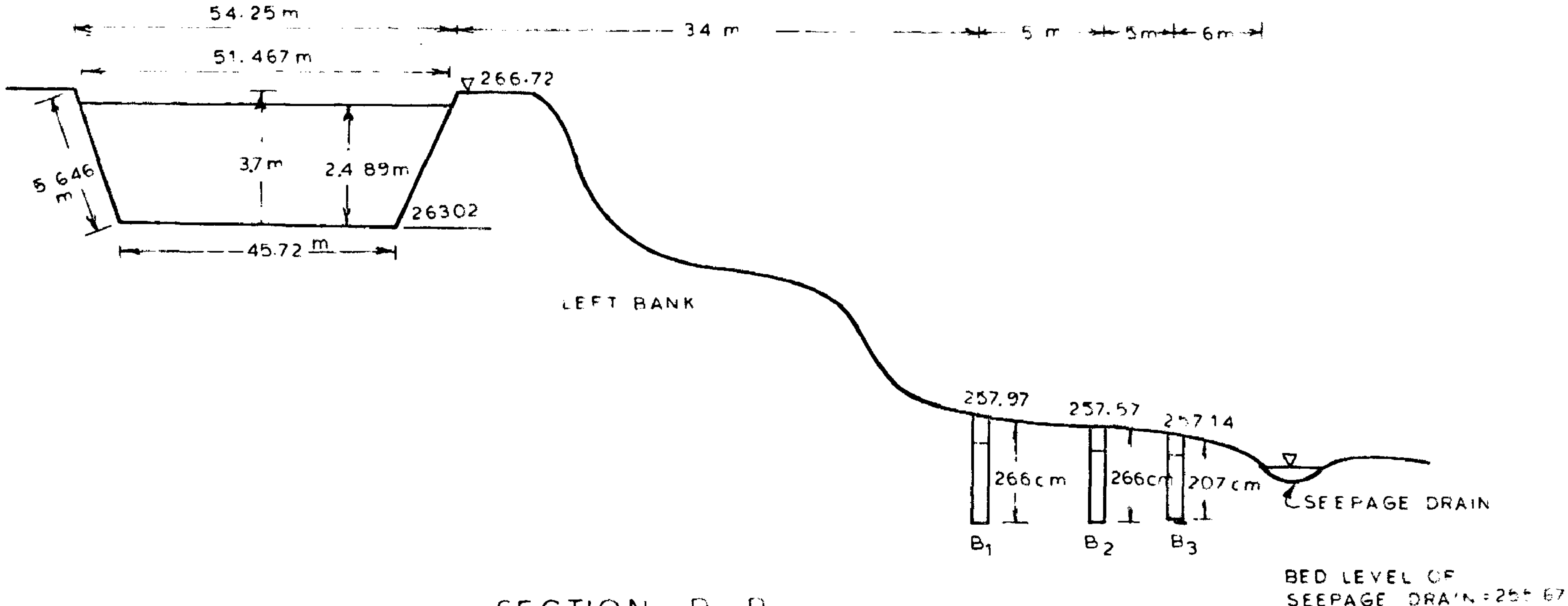
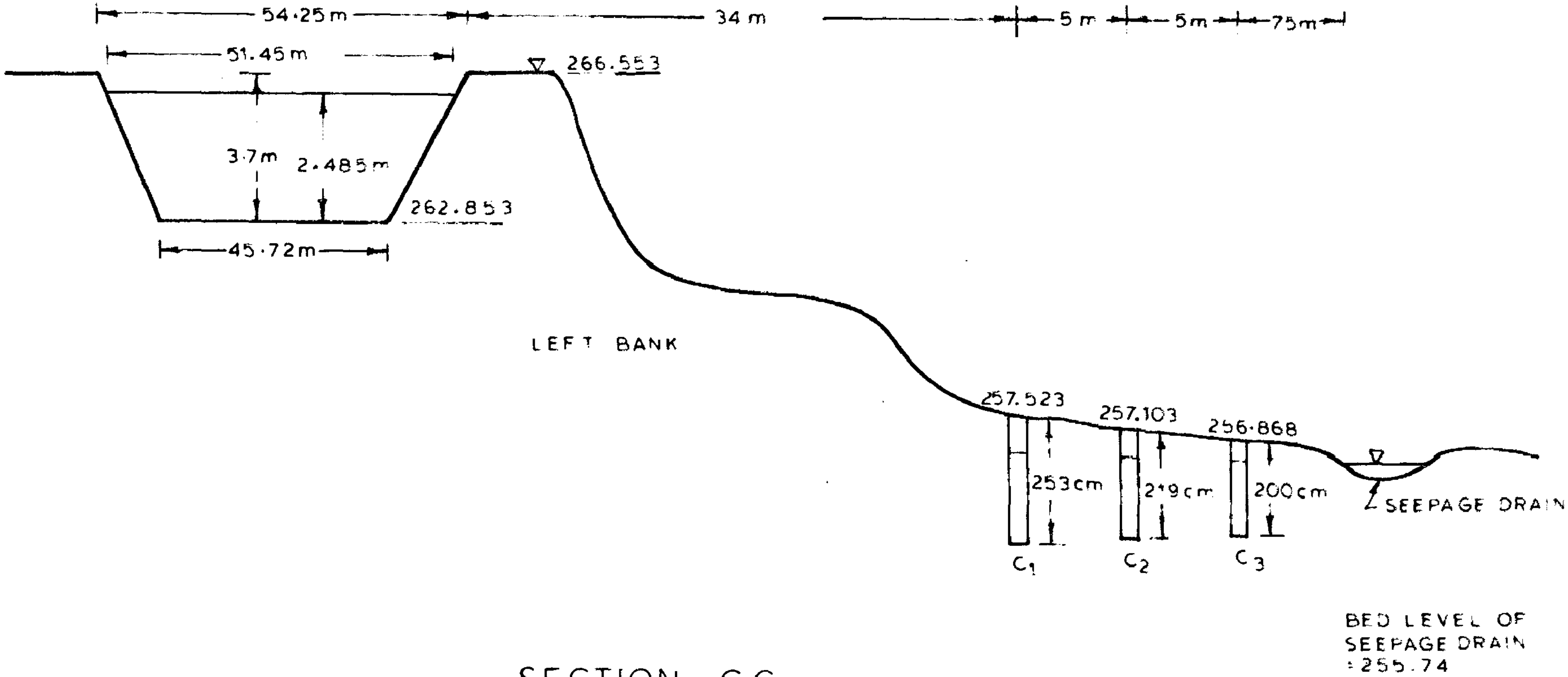


FIG 4.1. PLAN OF SITE I (GANGA CANAL)



SECTION B-B

FIG. 4.1.1 _ GANGA CANAL—CROSS SECTION (17M-CF)



SECTION CC

FIG.4.1.2 _ GANGA CANAL—CROSS SECTION (17 M-5 F)

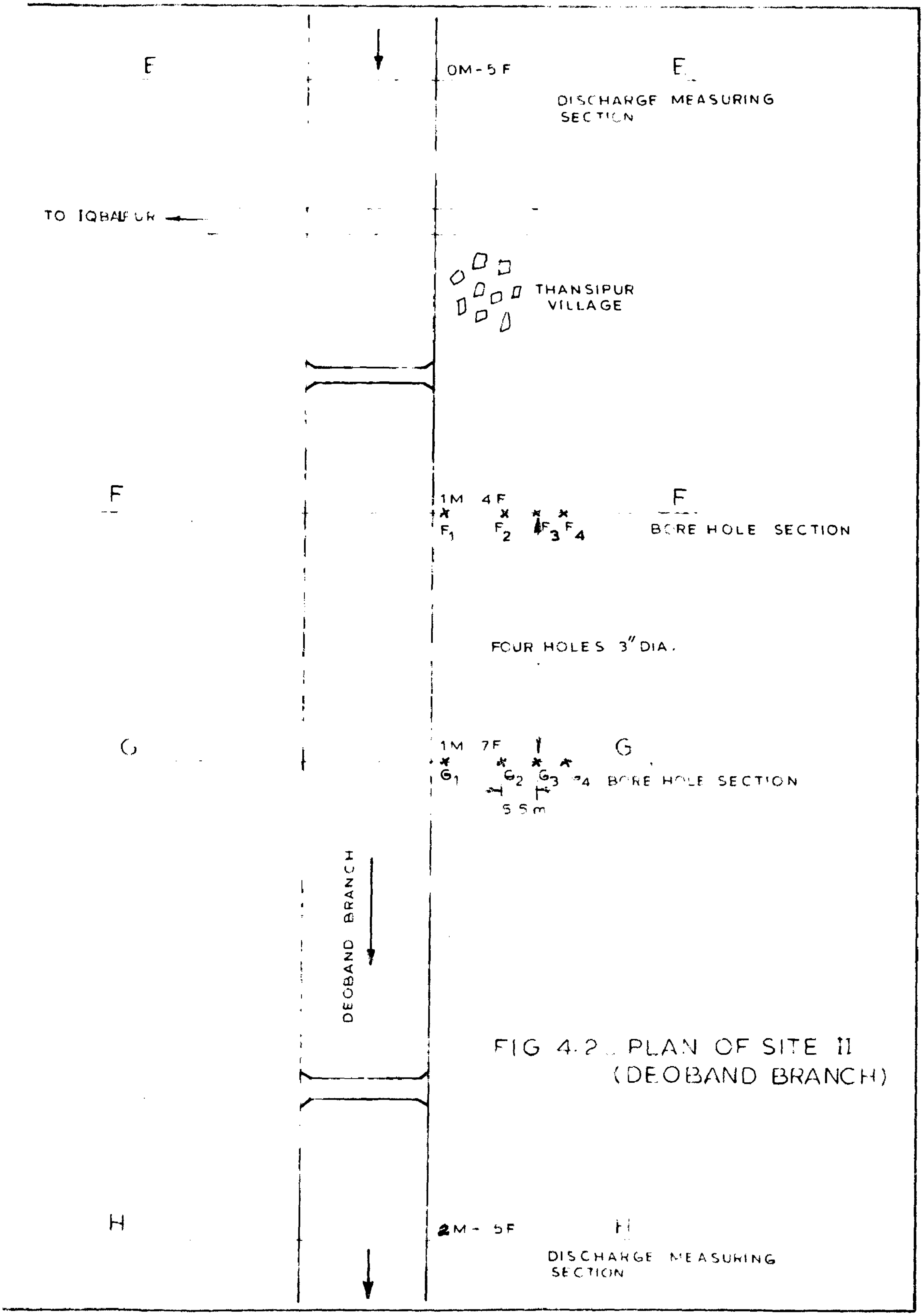
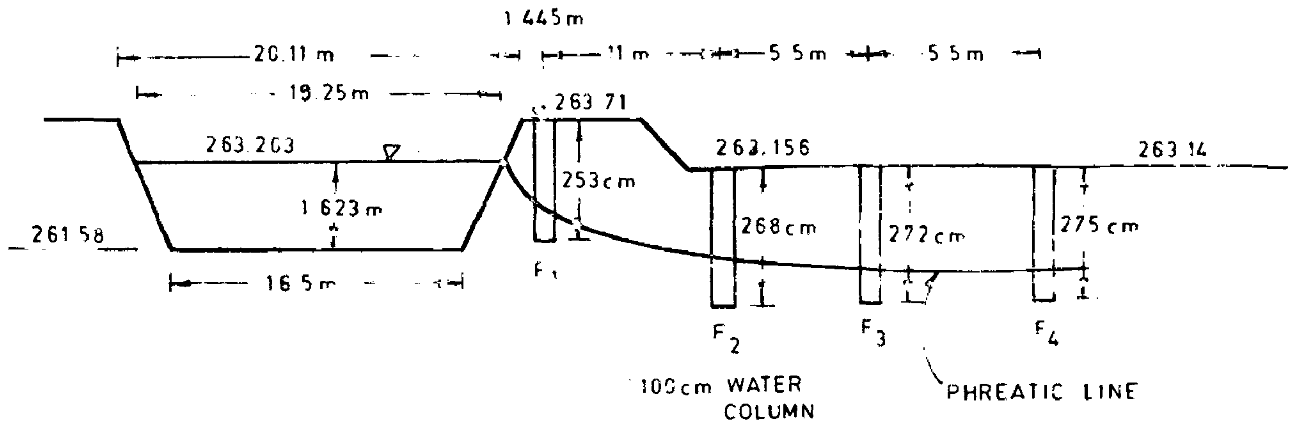
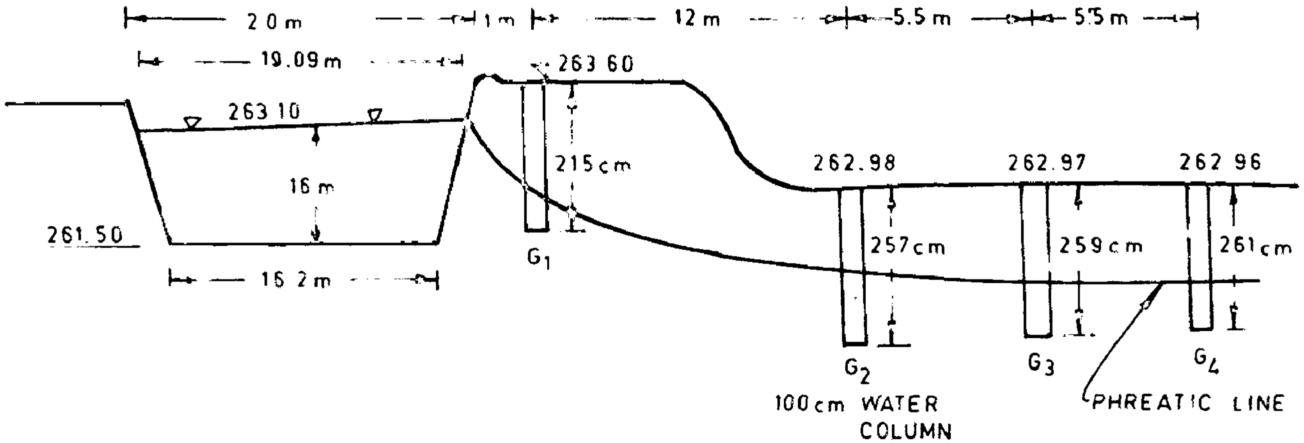


FIG 4.2 PLAN OF SITE II (DEOBAND BRANCH)



ACTIVITY WAS INJECTED INTO BORE HOLE F_1 F_2 AND F_3

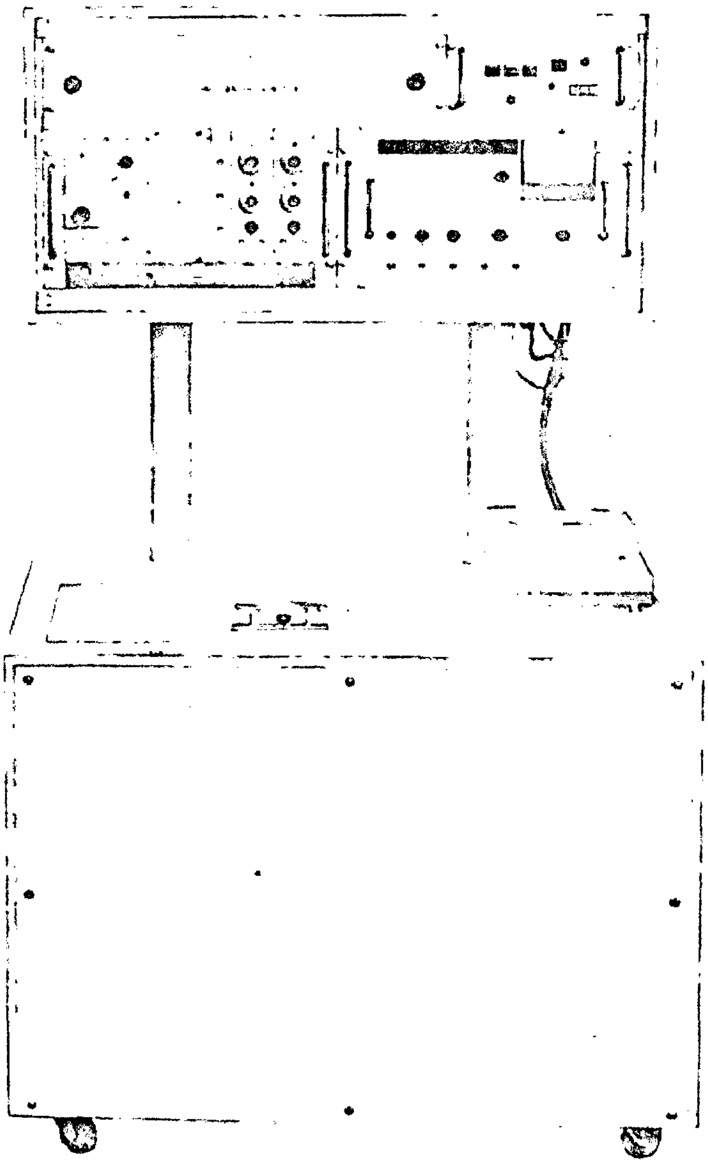
FIG. 4.2.1 _ DEOBAND BRANCH CROSS SECTION (1M-4F)



TRITIATED WATER WAS INJECTED INTO BORE HOLE G_1 G_2 AND G_3

FIG 4.2.2 _ DEOBAND BRANCH CROSS SECTION (1M-7F)

FIG 4·3·1



FEATURES

* Solid state modular construction
* Ambient temperature operation
* 100 sample capacity
* Manual, automatic and repeat counting modes
* Pulse summation and logarithmic amplification
* Two independent channels
* Simultaneous assay of two isotopes
* S ... ber disolay
* I ... mode of
* Digit...
* Single c... capability

APPLICATIONS

The Automatic Liqu... S... tion system, type LSS34 fin... de applications in advanc... search.

* An essential equipment for low energy and low active beta counting
* Used for radio tracer techniques in Biology, Bio-chemistry, Agriculture, C... ncer Research etc.
* Used in industrial research for Petroleum products etc.

AUTOMATIC LIQUID SCINTILLATION SYSTEM LSS 34

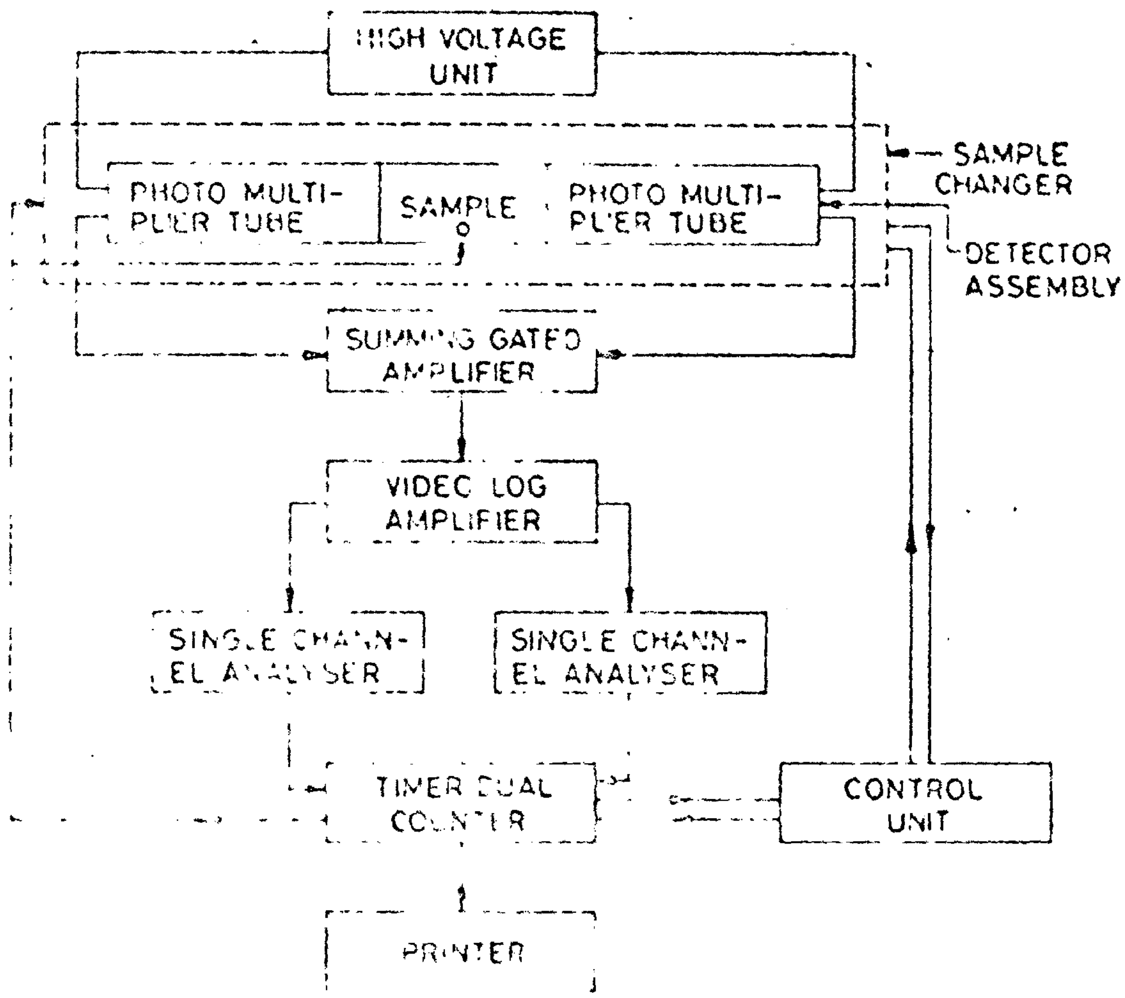


Fig 4.3.2 BLOCK DIAGRAM OF LSS34

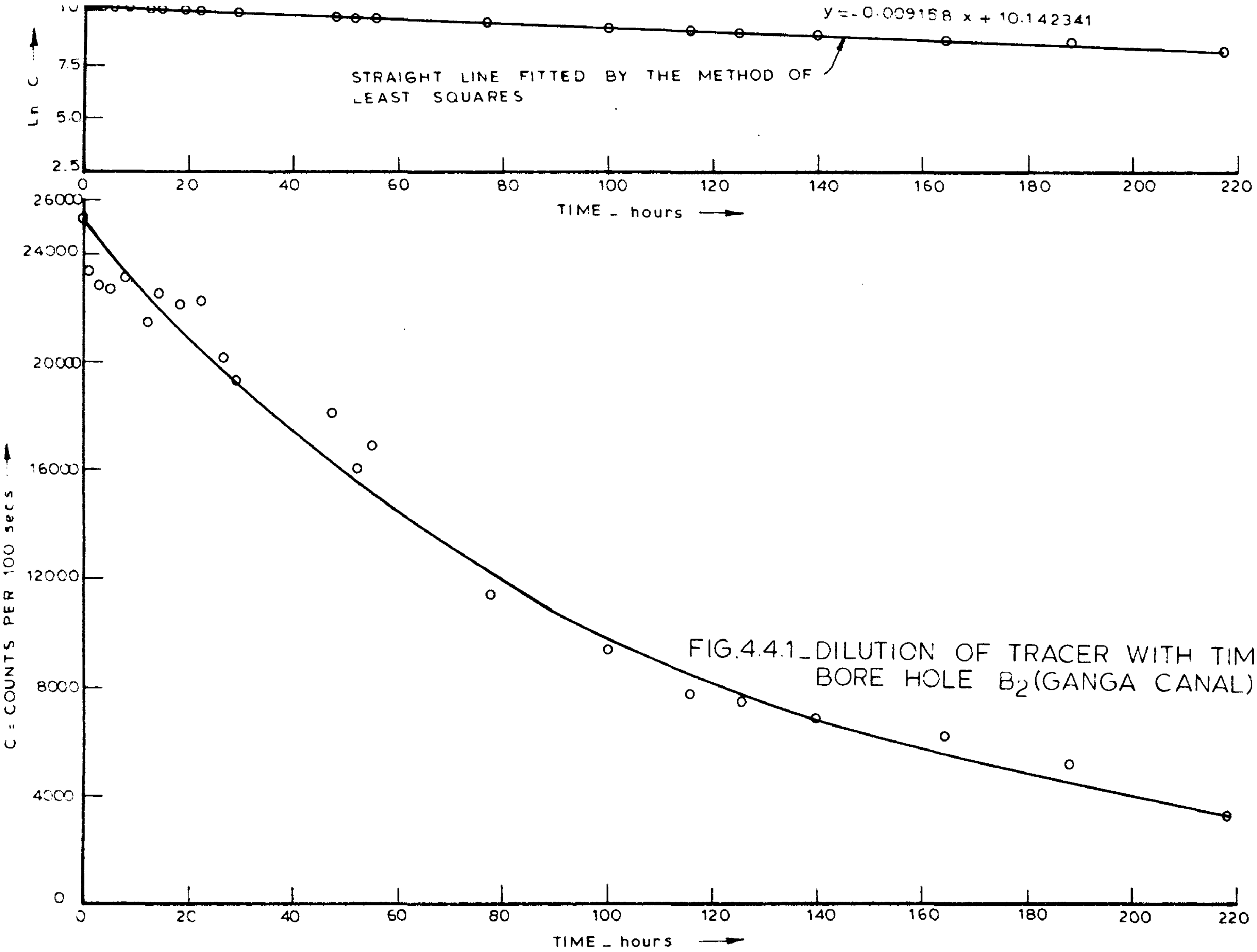


FIG.4.4.1_DILUTION OF TRACER WITH TIME IN BORE HOLE B_2 (GANGA CANAL)

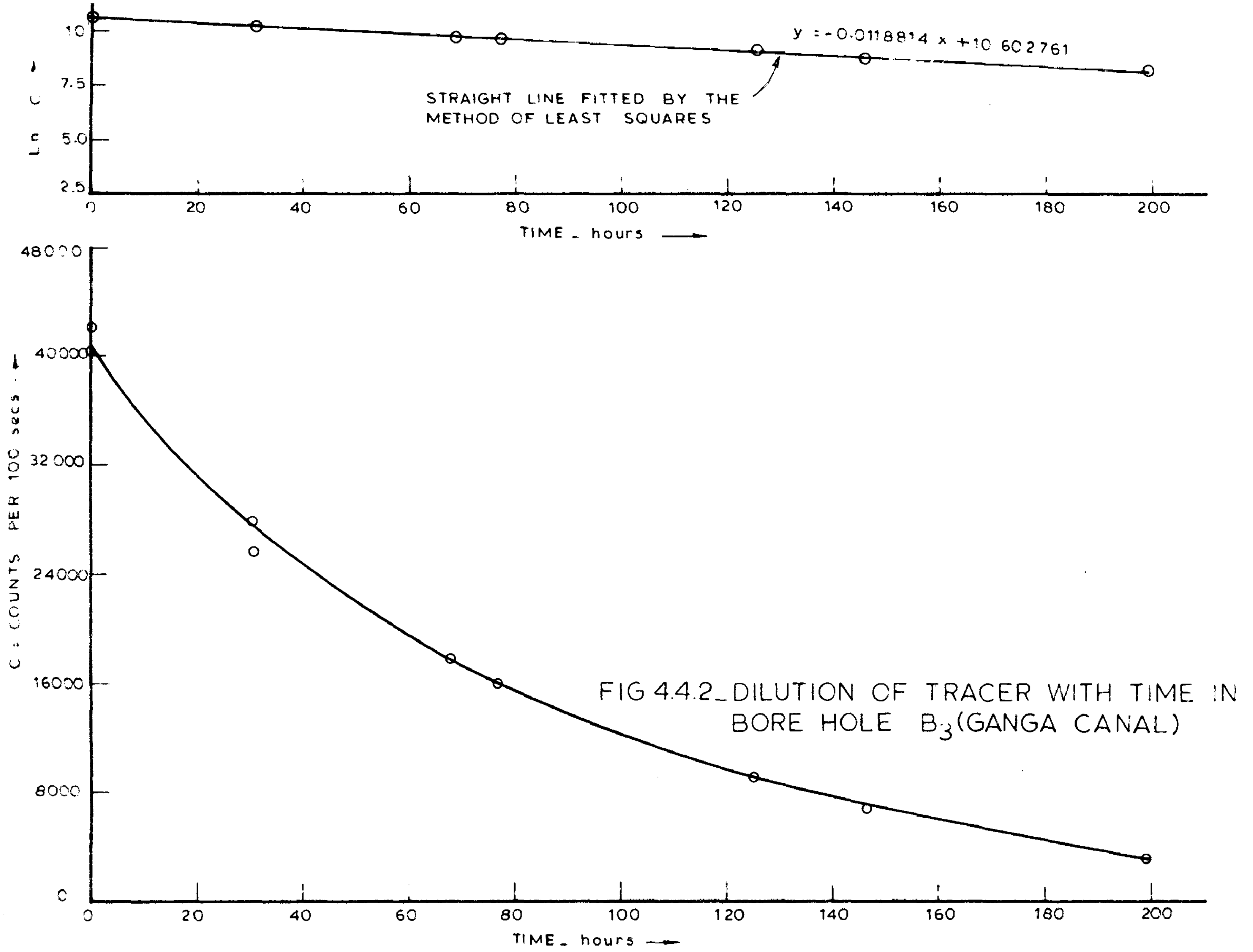


FIG 4.4.2 – DILUTION OF TRACER WITH TIME IN BORE HOLE B_3 (GANGA CANAL)

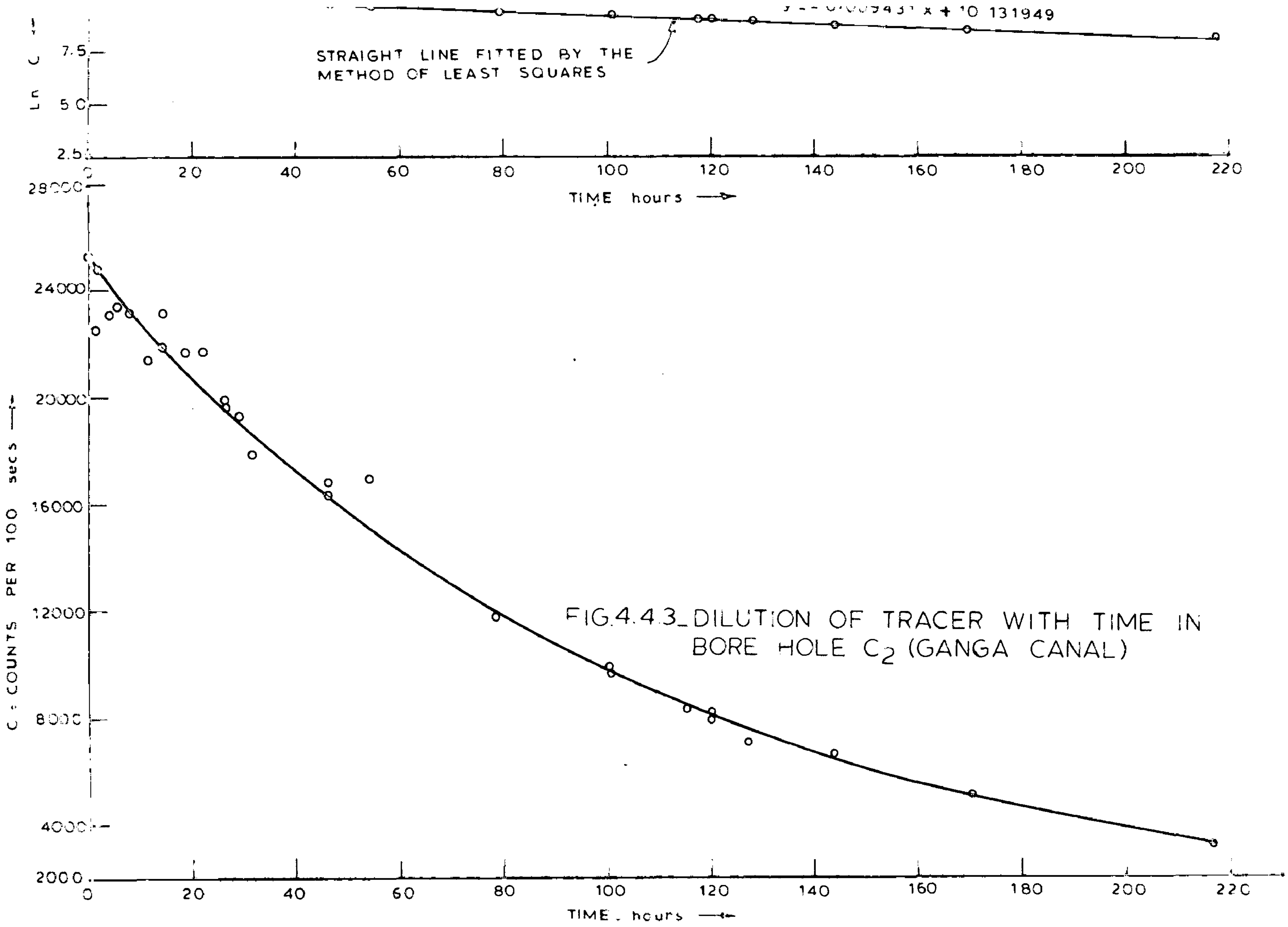


FIG.4.4.3_DILUTION OF TRACER WITH TIME IN BORE HOLE C_2 (GANGA CANAL)

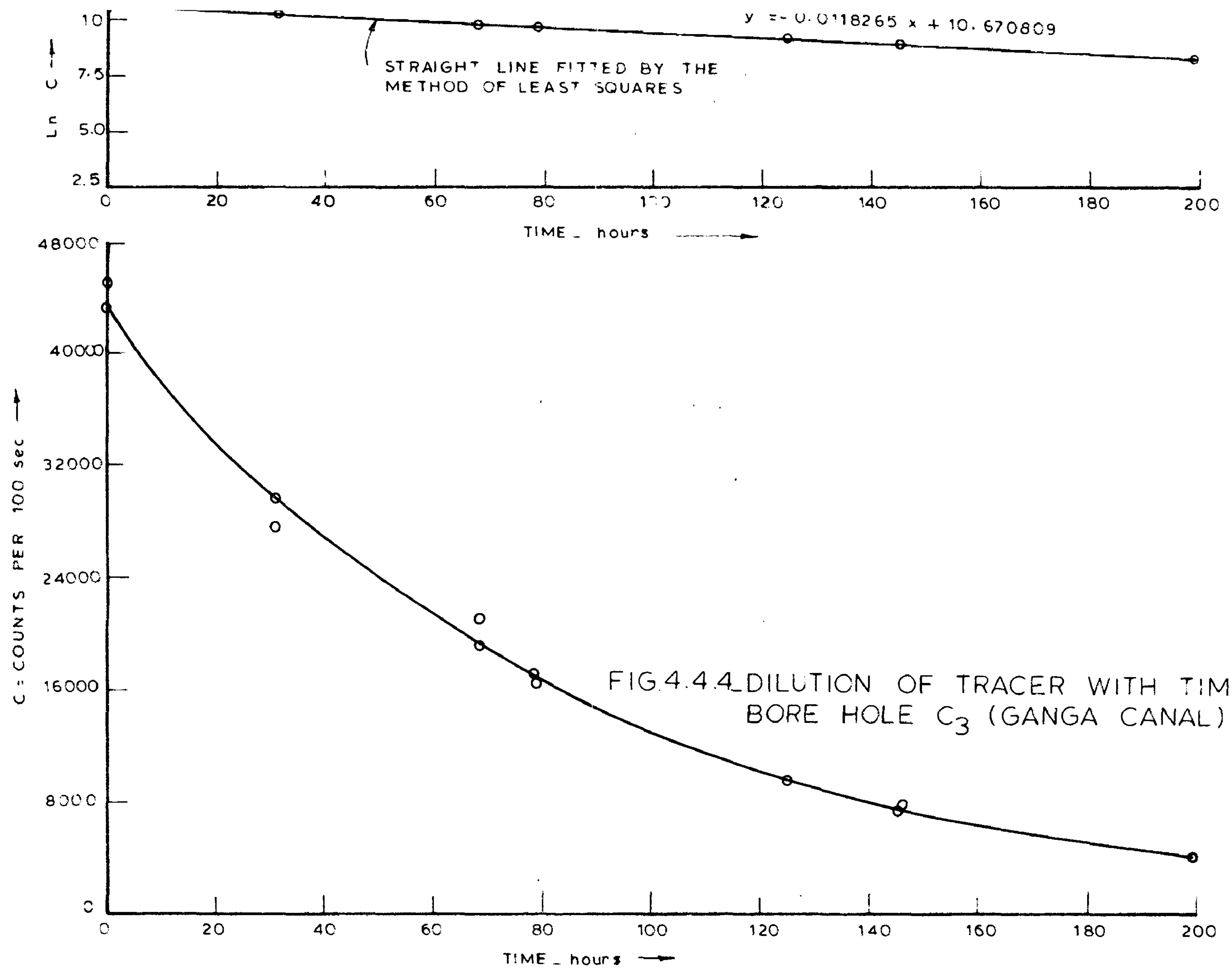


FIG.4.4.4 DILUTION OF TRACER WITH TIME IN BORE HOLE C_3 (GANGA CANAL)

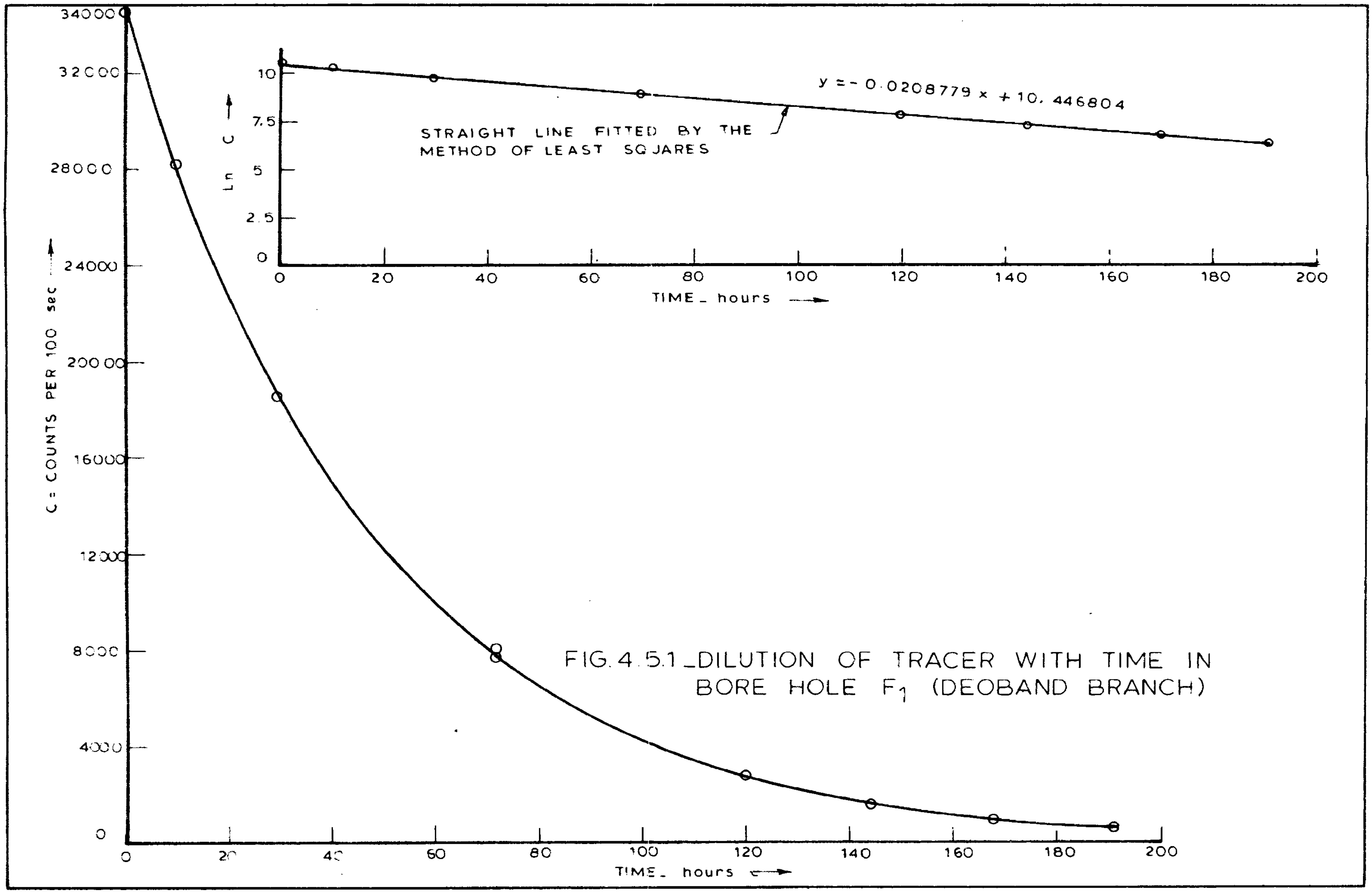


FIG. 4.5.1 _ DILUTION OF TRACER WITH TIME IN BORE HOLE F_1 (DEOBAND BRANCH)

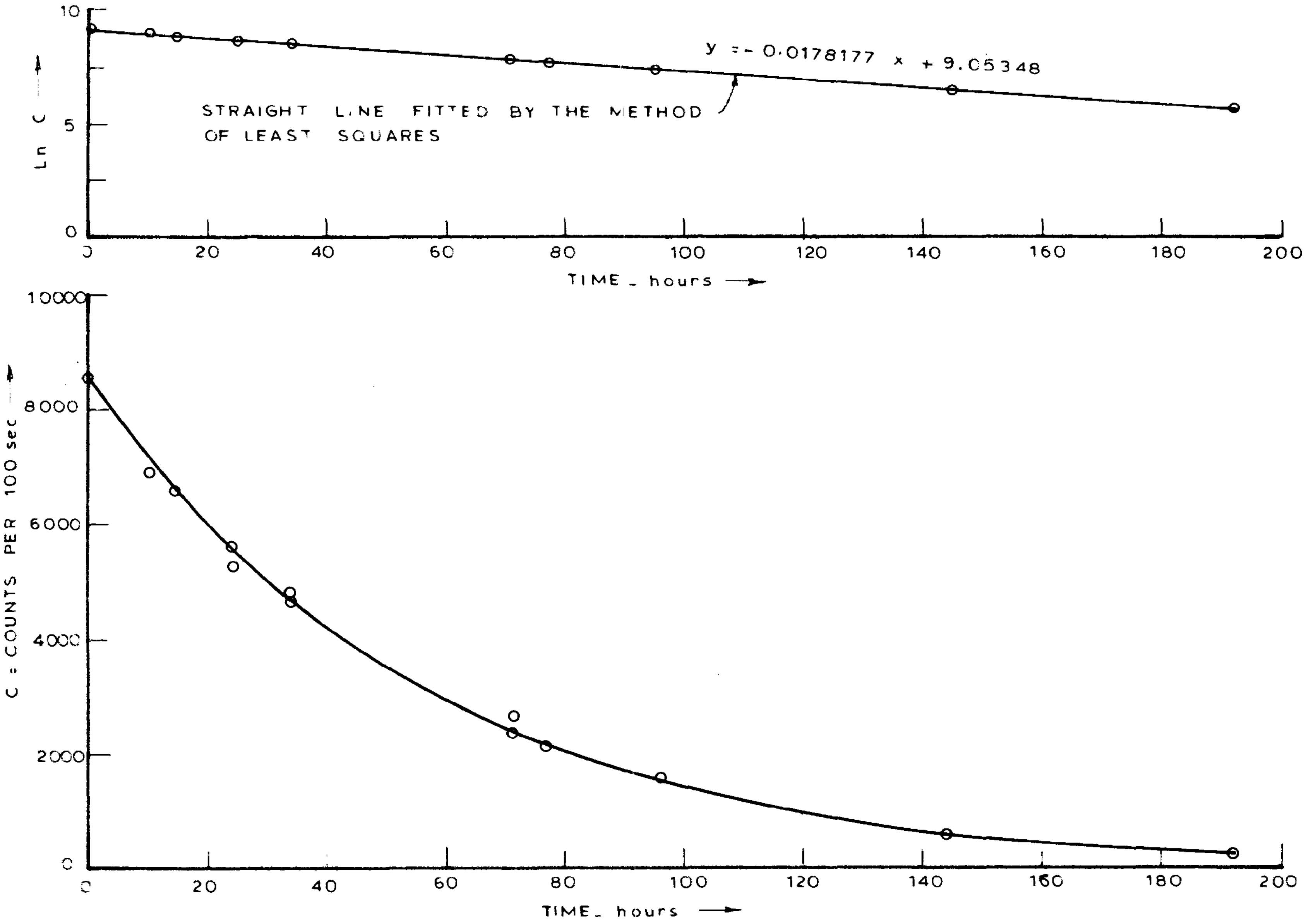


FIG.4 5 2_DILUTION OF TRACER WITH TIME IN BORE HOLE F_2
(DEOBAND BRANCH)

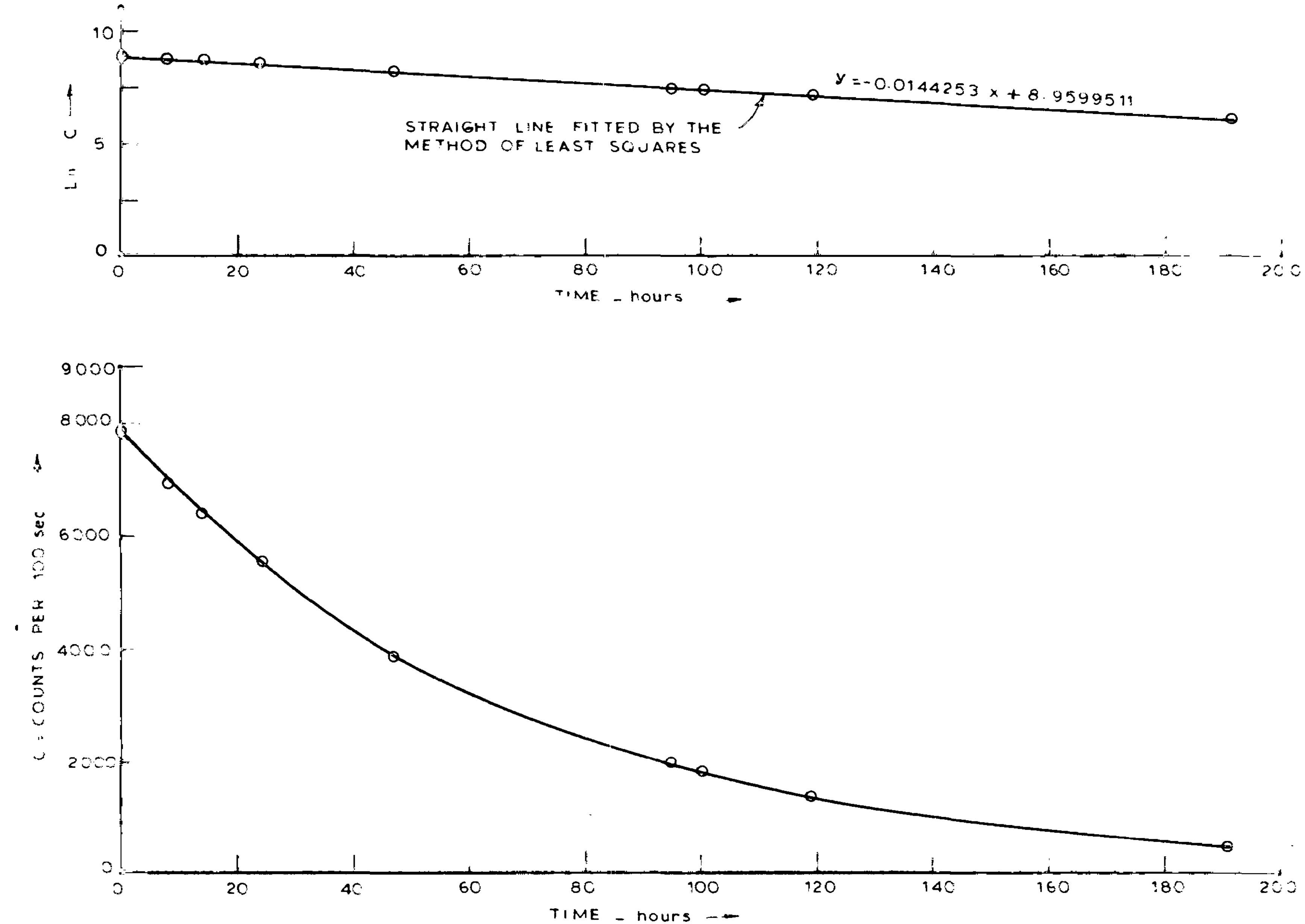


FIG.4.53_DILUTION OF TRACER WITH TIME IN BORE HOLE F_3
(DEOBAND BRANCH)

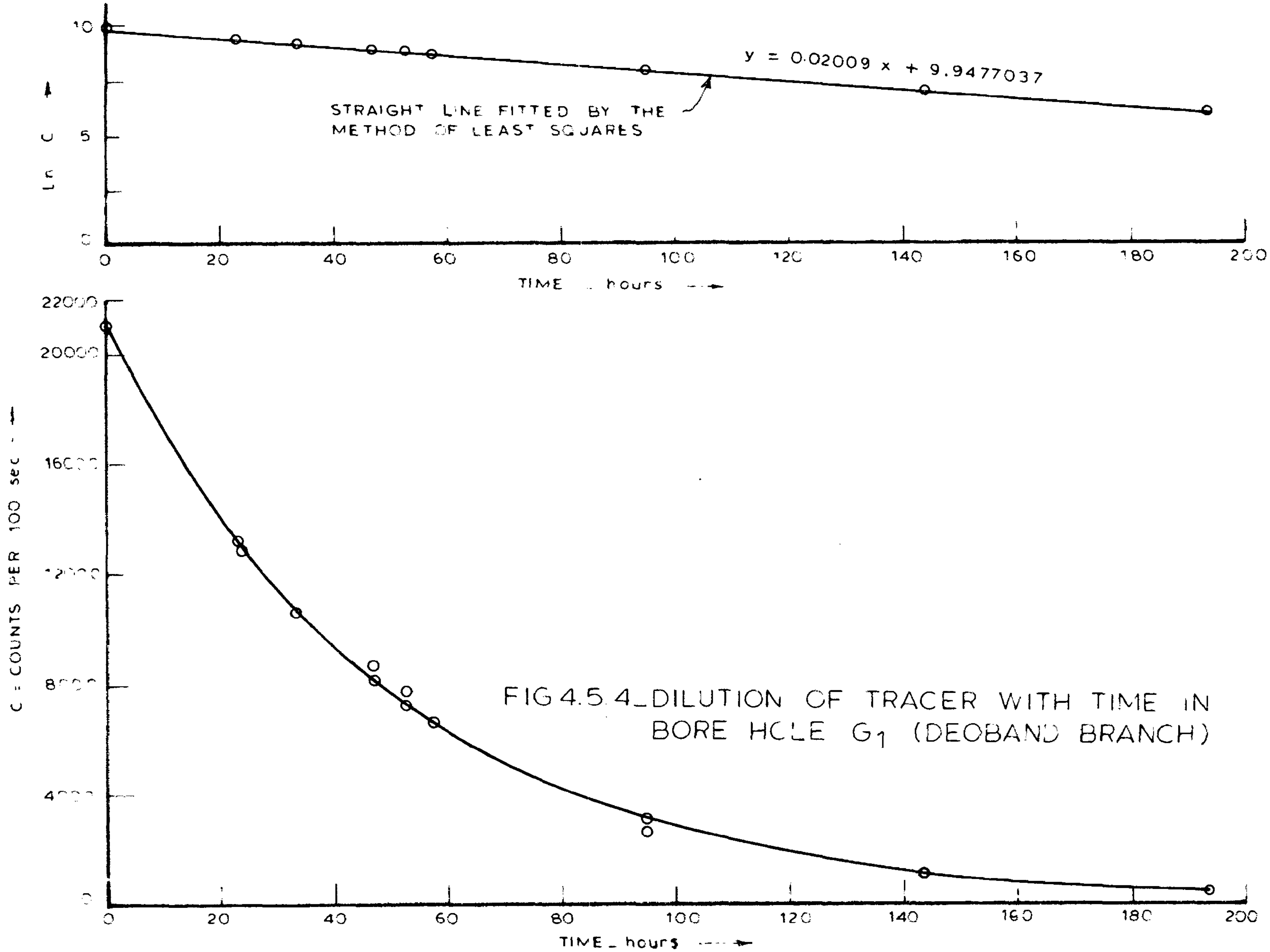


FIG 4.5.4_DILUTION OF TRACER WITH TIME IN BORE HOLE G_1 (DEOBAND BRANCH)

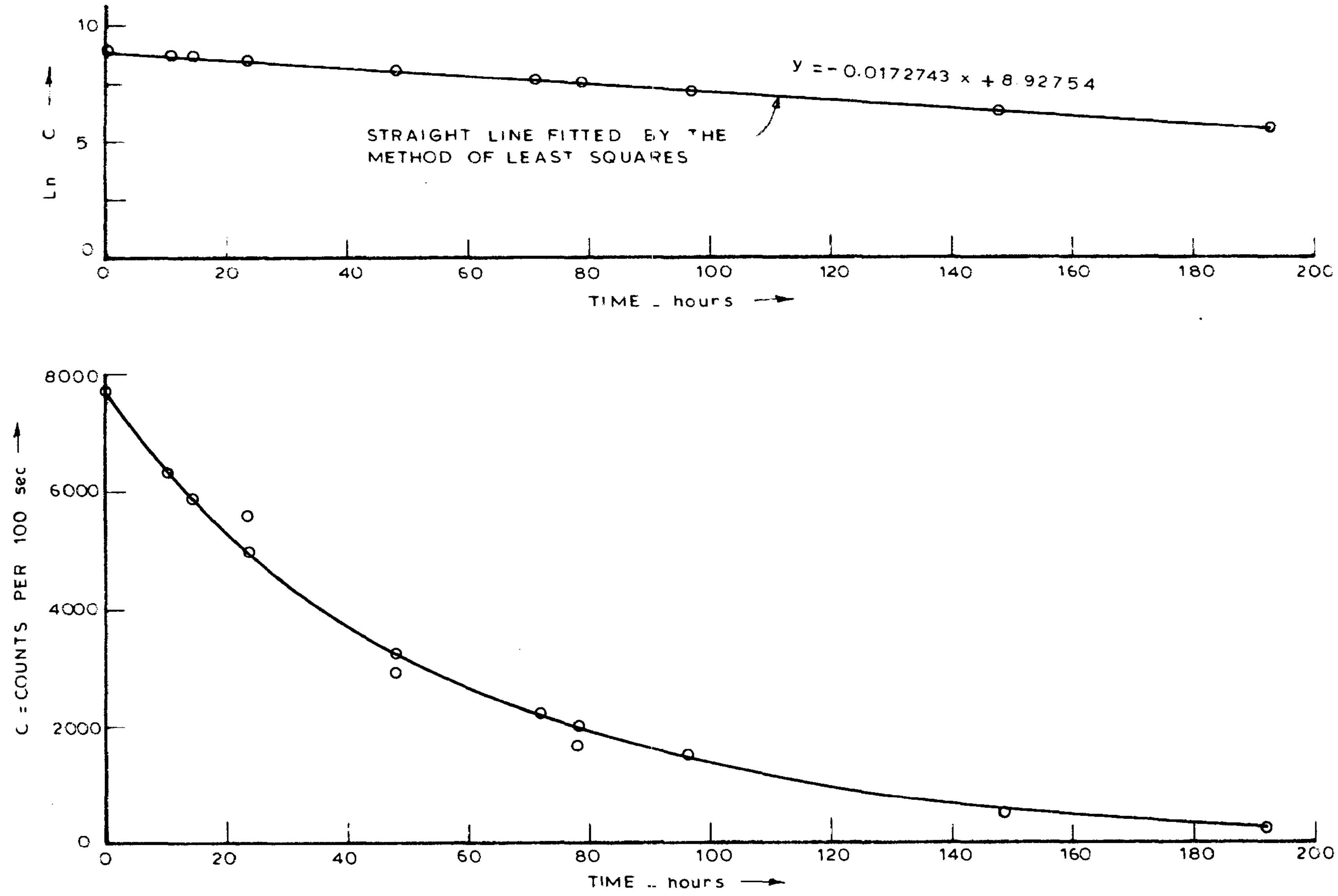


FIG.4.5 5_DILUTION OF TRACER WITH TIME IN BORE HOLE G_2
(DEOBAND BRANCH)

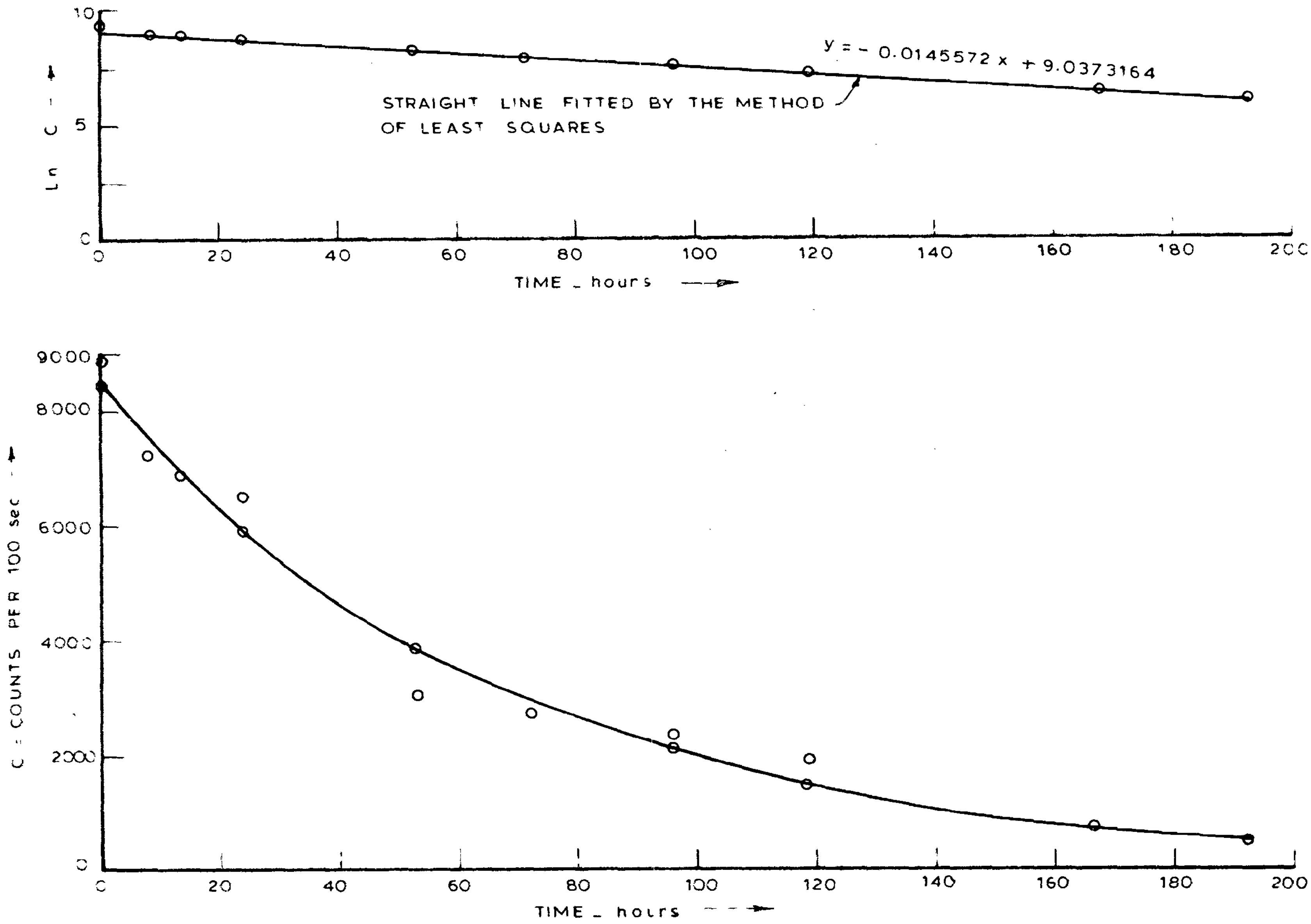


FIG 4.5.6_DILUTION OF TRACER WITH TIME IN BORE HOLE G_3
(DEOBAND BRANCH)

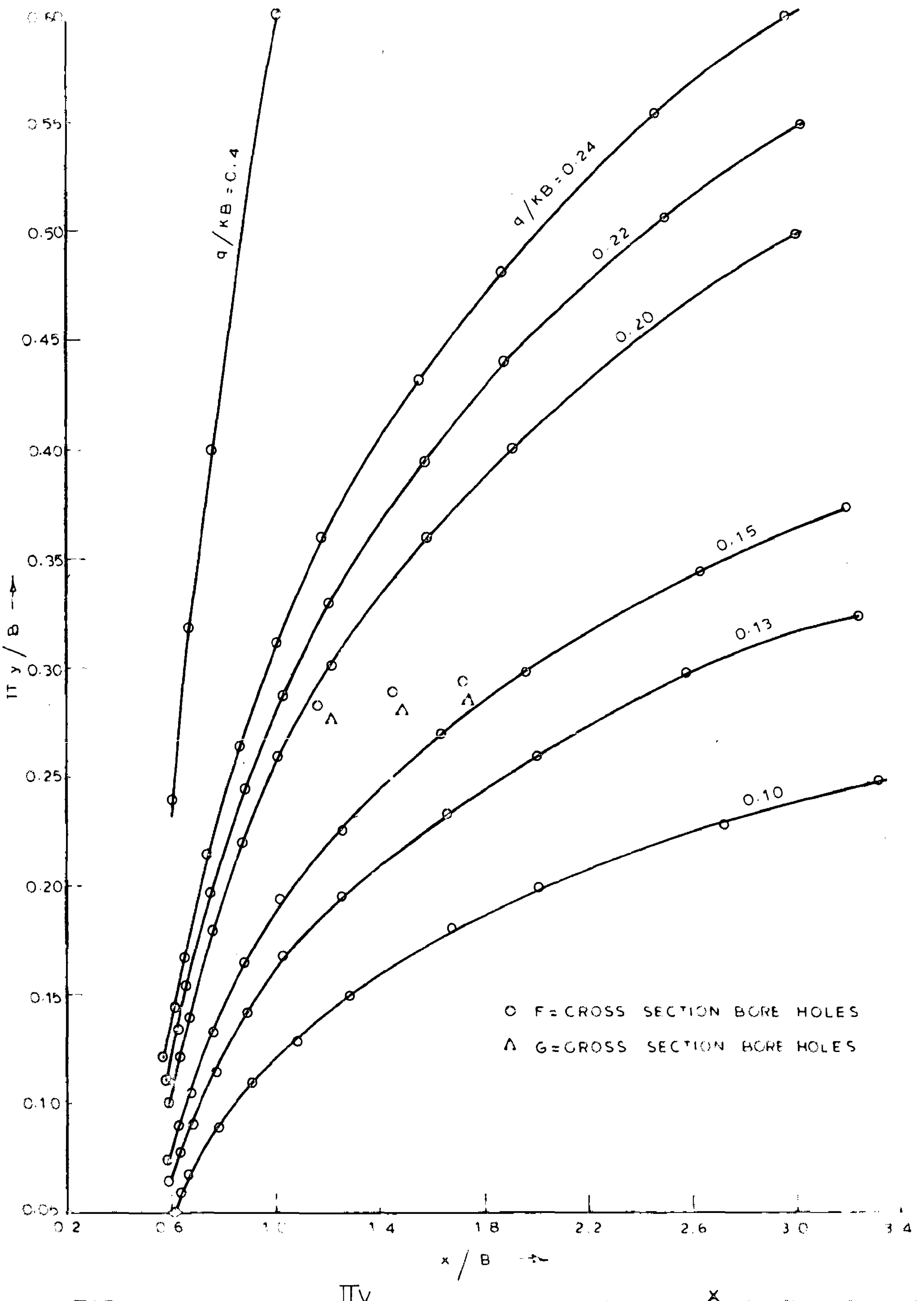


FIG 4.7 PLOT OF $\frac{\pi y}{B}$ AS A FUNCTION OF $\frac{x}{B}$ FOR VARIOUS RATIOS OF q/KB

CHAPTER-V

SUMMARY OF RESULTS AND DISCUSSIONS

5.1. SITE-I- GANGA CANAL

The numerical values of filtration velocity in respect of bore holes B_2, B_3, C_2 and C_3 works out to be 0.0044, 0.00925, 0.00452 and 0.0092049 metres/day respectively. This indicates that the velocity values are increasing from bore hole B_2 to B_3 or towards the seepage drain. The same is observed in respect of bore holes C_2 and C_3. The increase in filtration velocity may be due to the effect of vertical currents in the absence of filter tube.

The seepage losses as obtained by different approaches are given below -

1) Emperical method 2.605 m^3/day/m

2) IRI formula 1.459 m^3/day/m.

3) Direct r method -
Inflow-outflow method
(Parshall flume measurements in the seepage drain) 2.044 m^3/day/m.

The reach taken up for investigation is running in high banks. The sides are lined with brick masonry. The seepage loss is calculated considering that the section is unlined.

The seepage loss as obtained by inflow-outflow method using parshall flume measurements in the seepage drain closely agrees with that got from emperical formula.

for the reach between 16R 2F and 17R 7F

The seepage loss of Ganga canal may be taken as varying from 1.50 to 2.50 m^3/day/m. This appears to be a more reasonable figure for the type of soil strata available at site and considering the limitations of the formula employed.

5.2. SITE-II - DEOBAND BRANCH

The numerical values of filtration velocity in respect of bore holes F_1, F_2, F_3, G_1, G_2 and G_3 work out to be 0.01624, 0.008634, 0.0069, 0.0157, 0.00828 and 0.00698 metres/day respectively. There is apparent decrease of filtration velocity from bore hole F_1 to F_3 and from G_1 to G_3. This shows the effect of the natural drainage. The filtration velocity in respect of bore holes F_1 and G_1 are quite high due to steep hydraulic gradient. These bore holes were located very near to the bank.

The seepage losses as evaluated by different approaches are given below.

1)	Emperical method	4.2638 m^3/day/m
2)	Numerov's approach	3.698 ''
3)	Dachler's method	3.12[illegible] ''
4)	S.P. Garg and Chawla A.S. [illegible] Approach	4.351[illegible] ''
5)	Inflow-outflow method	4.467 ''

The rate of seepage loss obtained above appears to be a quite reasonable figure for the Deoband Branch which has unlined section. The percentage of fine sand and silt are predominant in the soil strata available at site. The soil type is almost same at both the sections. There is a little variation in the rate of seepage at the two sections.

The rate of seepage loss obtained by different approaches compare favourably and the results are encouraging.

Determination of the permeability from the experimental data coupled with the calculations based on Numerov's approach appears to provide a reasonable method for estimating seepage losses from unlined canals. In this analysis, a curvilinear perimeter of the canal is assumed whereas the Deoband Branch has a trapezoidal cross-section.

In Dachler's approach, an impermeable floor at a depth of 40 meters has been assumed. The figures obtained by Dachler's and Numerov, method have better agreement.

The result got from Inflow-outflow method has very good agreement with the result obtained by S.P.Garg and Chawla A.S. approach. In thiscase, the effect of natural drainage has been considered. This method seem to be reasonably more accurate than the other methods.

The seepage losses for the reach taken up for investigation through Deoband Branch may be taken as varying from 3.70 to 4.30 m^3/day/m. To have a correct assessment of the seepage loss, it is necessary to conduct the experiment at number of cross-sections on both the banks.

The Radiotracer point dilution technique can thus find a useful place in evaluating the filtration velocity and hydraulic conductivity concerning the problem of seepage.

BIBLIOGRAPHY

1. Aravin, V.I., and S.N. Numerov., 1965, Theory of Fluid flow in undesformable porous media.

2. Bharat Singh, 1975, Fundamentals of Irrigation Engineering.

3. Bhemalingaiah, H.M., 1972, Modern trends in evaluating and controlling seepage losses from canals, M.E. Thesis.

4. Bruce W. Hunt, 1972, Seepage from shallow open channels, Hydraulics Division.

5. Cedergren, H.R., 1968, Seepage, Drainage and Flow nets.

6. Daniel and Wood, Fitting equation to Data.

7. Davis and Wilson, 1946, Irrigation Engineering.

8. Gasper, E. and M. Oncescu, 1972, Radioactive tracers in Hydrology.

9. Garg, S.P., 1968, Canal Seepage Measurements along Ganga Canal at mile 23-40 using radiotracing.

10. Garg, S.P., and Chawla, A.S., June, 1970, "Seepage from trapezoidal channels", Journal of Hydraulics Division, ASCE, Volume 96.

11. Grmzenix, J. et.al. 1963, "Apparatus for measuring the relative velocity of groundwater by Point Dilution Method.

12. Harr, M.E., 1962, Groundwater and Seepage.

13. Halevy, E. et.al. "Borehole dilution technique - Acrificerl review," Isotopes in Hydrology, IAEA, Viema (1967).

14. Hazza, I.B. et. al., 1965, A method of determination of aquifer parameters using radioactive isotope, International Journal.

15. Hammad, H.Y., 1960, Seepage losses from parallel canal systems, ASCE Engg. Mechanics Division.

16. Halevy, E. et. al., 1960, "Use of Radioisotopes in studies of ground water flow" Tahal Water Planning for Israel, Tel Aviv.

17. Hiranandani and Chitale, Stream Gauging.

18. Herman Bawer, 1969, Theory of Seepage from open channels in Vol. 9 of Advances in Hydroscience, page 121 to 170.

19. Herman Bouwer and Rice, R.C. (1968) A salt Penetration technique for seepage measurement, Irrigation and Drainage Division Proceedings, ASCE, Prof. 94.

20. International Atomic Energy Agency Vienna, 1971, Isotope Hydrology.

21. International Atomic Energy Agency, 1967, Isotope and Radiation Techniques in Spil Physics and Irrigation Studies.

22. Iya, V.K., K. Krishnamurthy, K.S. Aggarwal and P.A. Gokulan, 1966, A preliminary radiotracer study on Canals Seepage.

23. Jeppson, Roland, W., 1968, Seepage from Ditches - "Journal of Hydraulics Division", ASCE, Vol.94.

24. John. C. Rodda, 1976, Facets of Hydrology.

25. Kauffman, W.J., Todd D.K., 1962, Application of tritium tracer to canal seepage. Tritium in the physical and Biological Sciences.

26. Khushlani. Irrigation Practice and design Vol. VI

27. Krishnamurthy, K, and S.M. Rao, 1968, Theory and Experiment in Canal Seepage.

28. Lisley, Kohler and Paulhus, Applied Hydrology

29. Mario G. Salvadori and Melvin, L. Baron, 1963, Numerical Methods in Engineering.

30. Muskat, M, Flow of Homogeneous fluids through porous media.

31. Midha, D.C., 1937, A new method of determining seepage from canals.

32. Nemec, 1973, Engineering Hydrology

33. Parker, F.L., 1958, "Radioactive tracers in Hydrological Studies", Trans. American Geophysical Union.

34. Polubarinova-Kochina P.Y., Theory of Groundwater movement.

35. Technical Report Series No.126, Nuclear Well logging in Hydrology, IAEC, Viema (1971)

36. Todd, D.K., 1959, Groundwater Hydrology

37. U.P., I.R.I., 1972 A note on the assessment of seepage losses from canals.