# STREAM AQUIFER INTERACTION - DETERMINATION OF RUSHTON'S PARAMETERS

A DISSERTATION

*Submitted in partial fulfillment of the requirements for the award of the degree*

*of*

**MASTER OF TECHNOLOGY**

*in*

**WATER RESOURCES DEVELOPMENT**

By


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## Candidate's Declaration

I hereby certify that the work which is presented in the Dissertation entitled ***''Stream aquifer interaction –Determination of Rushton's parameters''*** is being submitted in partial fulfillment of the requirements of the Degree of Master of Technology in Water Resources Development , Indian Institute of Technology , Roorkee, is an authentic record of my own work carried out from July 17, 2002 to December ,2002 under the supervision of Dr.G.C.Mishra , professor , WRDTDC, Indian Institute of Technology , Roorkee.

The Matter embodied in this Dissertation has not been submitted by me for the award of any other degree.

Dtaed: 2-12-02 GETACHEW BERHANU

This is to certify that the above statements made by the candidate are correct to the best of my knowledge and belief.

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(GETACHEW BERHANU)

## Synopsis

A rectangular channel in an isotropic and homogeneous porous medium is assumed. The seepage is computed for both effluent and influent cases using the method of fragments and Dupuit-Forchheimer assumptions.

The methods of Herbert and of Aravin and Numerov assume that there is a linear relationship between the flow to the aquifer and the potential difference between the aquifer and the river. However, field evidence suggests a non-linear relationship (K.R. Rushton, 1978). Typical non-linear relationships, which appear to give a fair representation, are as follows:

If $h_2 \geq h_1$

$$Q = C_1(h_2 - h_1) + C_2\{1 - \exp[-C_3(h_2 - h_1)]\}$$

If $h_2 \leq h_1$

$$Q = 0.3C_2\{\exp[C_3(h_2 - h_1)] - 1\}$$

Where $C_1$, $C_2$ and $C_3$ are constants depending on field conditions. Q is the seepage. $h_1$ and $h_2$ are the groundwater potentials at the river boundary and at half of the aquifer depth below the river bed respectively.

It is found that the non-linear relationship comes into picture for higher values of ratio of bed width of the river to depth of the aquifer below the riverbed. It is also found that there is a close agreement of seepage values computed by Dupuit-Forchheimer assumption and the method of fragments. There is an error of 10% between the two for B/D =1.

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# CHAPTER 1

## Introduction

### 1.1 General

Rivers passing through a region underlain by a phreatic aquifer (and in special cases even by a confined aquifer) may either contribute water to the aquifer or serve as its drain. Much of the low water flow in streams is derived from groundwater whose water table elevations in the vicinity of a stream are higher than the stream. Such streams are called effluent streams .On the other hand, when the water level in a stream is higher than the water level in the adjacent (or underlying) aquifer ,water will flow from the river to the aquifer .The river is then called an influent river. When a stream cuts through an impervious layer, establishing a direct contact with an underlying confined aquifer, the stream may be either an influent one or an effluent one depending on whether piezometric heads in the aquifer are above or below the water level in the stream. The same stream can be an influent one along one stretch and an effluent on another, or it can be both effluent and influent at the same point.

Obviously, the entire discussion presented above is based on the assumption that the riverbed is not completely clogged and that water can flow freely through the riverbed. Otherwise, there is no hydraulic contact between the water in the river and the aquifer and no relationship exist between the two. It is possible that the profile of a stream is such that its deeper part accommodating for low flows is completely clogged, while above a certain level, the riverbed is pervious.

The volume of water contributed to an aquifer by stream flow (or drained into a stream from an aquifer) is part of the regional water balance .In view of the different possible situations discussed above, the rivers may play several roles when solving a groundwater forecasting problem.

### 1.2 Two Dimensional Steady state flow of Groundwater

In a steady two-dimensional seepage flow through a homogeneous and isotropic medium, all quantities depend on two co-ordinates only. The fundamental equations of this flow are obtained by modifying the general equations of seepage flow. The fundamental equations of the two-dimensional seepage flow in a homogeneous isotropic medium are,

$$v_x = \frac{\partial \varphi}{\partial x} = -k\frac{\partial h}{\partial x} \quad , v_y = \frac{\partial \varphi}{\partial y} = -k\frac{\partial h}{\partial y} \qquad (1.2.1)$$

Where $v_x$ and $v_y$ are the components of Darcy velocity in the direction of the co-ordinate axes, and $\varphi(x,y)$ is the potential of seepage flow.

$$\varphi = -kh \qquad (1.2.2)$$

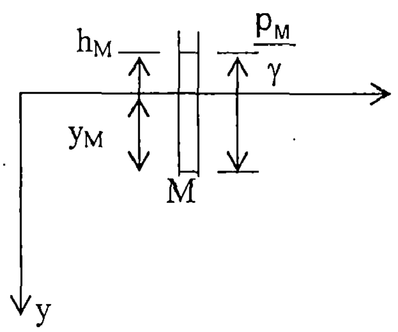


**Figure 1. 2.1 Hydraulic head at the point M**

h(x,y) is the hydraulic head at the point (x,y) above the chosen reference plane. For the direction of co-ordinate axis being considered (Halek and Svec, J, 1979)

$$,h = \frac{p}{\gamma} - y + c \qquad (1.2.3)$$

Where p(x,y) is the hydrostatic pressure at the point (x,y), C is a constant dependent on the choice of the reference plane used in the determination of the piezometric head, h . If we put this reference plane on the level of the axis x (Fig 1.2.1), C=0

For two dimensional steady state flows the continuity equation is,

$$\frac{\partial v_x}{\partial x}+\frac{\partial v_y}{\partial y}=0 \qquad (1.2.4)$$

By substituting in eqn.(1.2.4) according to eqn.(1.2.1) or (1.2.2) we obtain Laplace's equation of the potential $\varphi$ :

$$\Delta\varphi=\frac{\partial^2\varphi}{\partial x^2}+\frac{\partial^2\varphi}{\partial y^2}=0 \qquad (1.2.5)$$

or of the hydraulic head h:

$$\Delta h=\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=0 \qquad (1.2.6)$$

Hence both the potential $\varphi$ and the hydraulic head h are harmonic functions of the co-ordinates of points in the region of seepage. Solving equation (1.2.5) or equation (1.2.6) under prevailing boundary conditions gives the magnitude of the potential $\varphi=\varphi(x,y)$ or the hydraulic head $h=h(x,y)$ in the region of seepage (except for an arbitrary additive constant ); all the remaining quantities in which we are interested ,$v_x$ , $v_y$ and p can then be determined with the help of the above equations.

### 1.3 unsteady state Flow

The law of conservation of mass for transient (unsteady state) flow in a saturated porous medium requires that the rate of fluid mass flow into any elemental control volume be equal to the time rate of change of fluid mass storage within the element. With reference to Fig 1.3.1 the equation of continuity takes the form,

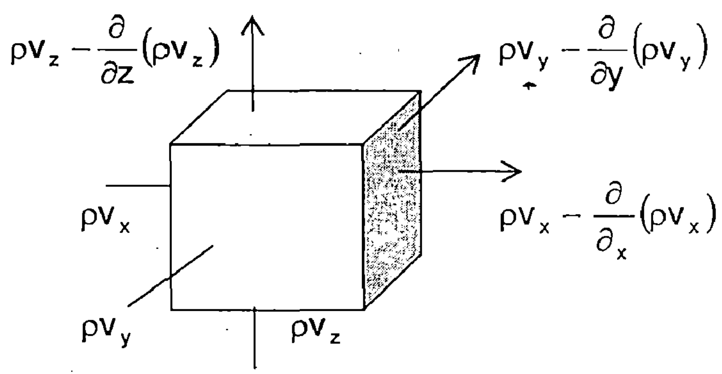


Fig 1.3.1 Elemental control volume through Porous media.

$$-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z}=\frac{\partial(\rho n)}{\partial t} \tag{1.3.1}$$

Or, expanding the right –hand side,

$$-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z}=\frac{\partial(\rho n)}{\partial t}=n\frac{\partial\rho}{\partial t}+\frac{\rho(\partial n)}{\partial t} \tag{1.3.2}$$

The first term on the right-hand side of Eq. (1.3.2) is the mass rate of water produced by the expansion of the water under a change in its density$\rho$. The second term is the mass rate of water produced by the compaction of the porous medium as reflected by the change in its porosity n, the first term is controlled by the compressibility of the fluid $\beta$ and the second term by the compressibility of the aquifer,$\alpha$.

After further simplification of Eq. (1.3.2) we obtain

$$\frac{\partial}{\partial x}\left(k_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(k_z\frac{\partial h}{\partial z}\right)=s_s\frac{\partial h}{\partial t} \tag{1.3.3}$$

Where $s_s$ is the specific storage for transient flow through a saturated anisotropic medium.

The solution h (x,y,z,t) describes the value of the hydraulic head at any point in a flow field at any time .A solution requires the knowledge of the three basic hydrogeological parameters, k, $\alpha$ and n , and the fluid parameters , $\rho$ and $\beta$(Freeze and Cherry, 1979).

## 1.4 conformal Mapping

### 1.4.1General

A transformation that possesses the property of preserving angles of intersection and approximate image of small shapes is said to be conformal. The usefulness of conformal mapping in two dimensional flow problems stems from the fact that solutions of Laplace equation remain solutions when subjected to conformal transformations.

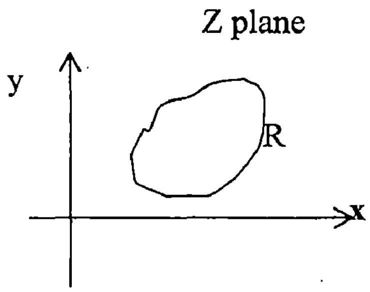


**Fig 1.4.1.1**

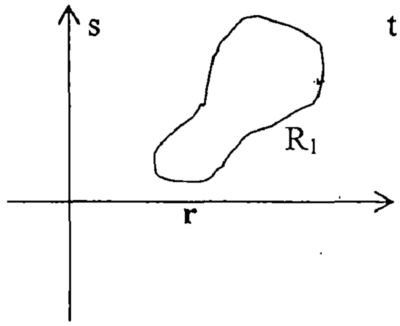


**Fig 1.4.1 .2**

Let $w = \phi + i\psi = f(z)$ be the complex potential and let its real and imaginary parts satisfy Laplace's equation in the region R of the Z plane (fig 1.4.1.1), so that

$$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}=0 \qquad \frac{\partial^2\psi}{\partial x^2}+\frac{\partial^2\psi}{\partial y^2}=0$$

Now suppose that there is a second analytic function z=F(t) , with t =r+is , which maps the region R into $R_1$. The function w =f[F(t)] is an analytic function of an analytic function, which in turn is also analytic, and hence

$$\frac{\partial^2\phi}{\partial r^2}+\frac{\partial^2\phi}{\partial s^2}=0, \qquad \frac{\partial^2\psi}{\partial r^2}+\frac{\partial^2\psi}{\partial s^2}=0$$

The solution of groundwater problem could be reduced to one of seeking the solution of Laplace's equation subject to certain boundary conditions within a region R in the Z-plane. From the standpoint of an analytical solution to Laplace's equation, unless the region R is of very simple shape a direct approach to the problem is generally difficult. However, by means of conformal mapping, it is often possible to transform the region R into a simple region $R_1$ wherein Laplace's equation can be solved subject to the transformed boundary conditions. Once the solution has been obtained in region $R_1$, it can be carried back by the inverse transformation to the region R, the original problem. Hence the crux of the problem is finding a transformation that will map a region R conformally into a region $R_1$ so that $R_1$ will be of simple shape (Harr, 1962).

### 1.4.2 The Schwarz-Christoffel Transformation

Theoretically the transformation exists which will map any pair of simply connected regions conformally onto each other. This is assured by Riemann mapping theorem; however the determination of a general solution for the mapping problem has thus far defied discovery. At first this may appear somewhat disturbing; however, appropriate auxiliary mapping techniques enable us to transform even complicated flow regions into regular geometric shapes. Generally these figures will be polygons having a finite number of vertices (one or more of which may be at infinity).

If a polygon is located in the Z-plane, then the transformation that maps it conformally onto the upper half of t-plane (t=r+is) is

$$z = M\int \frac{dt}{(t-a)^{1-\frac{A}{\pi}}(t-b)^{1-\frac{B}{\pi}}(t-c)^{1-\frac{C}{\pi}}\ldots} + N \qquad (1.4.2.1)$$

where A, B, C ... are the interior angles of the polygon in the Z-plane and a,b,c ...are the points on the real axis of t-plane corresponding to the vertices of the polygon in Z-plane.

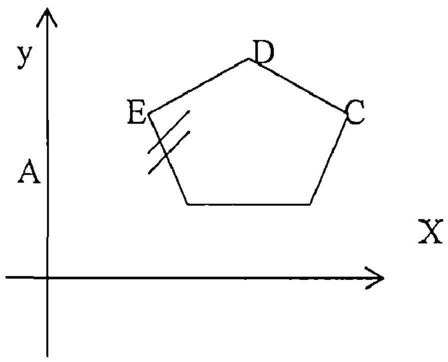


Fig 1.4.2.1 z plane.

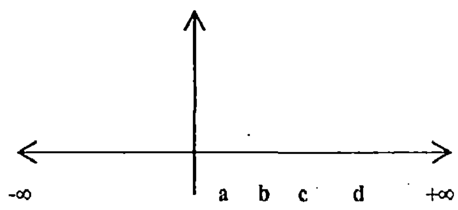


Fig 1.4.2.2 t-plane

### 1.4.3 Zhukovsky Function

A special mapping technique, of particular value when dealing with unconfined flow problems, makes use of an auxiliary transformation called Zhukovsky's function. Noting that the relationship between the velocity potential and the pressure

$\left[\phi = -k\left(\frac{p}{\gamma_w} + y\right)\right]$ can be written as $\frac{-kp}{\gamma_w} = \phi + ky$ if we define $\theta_1 = \frac{-kp}{\gamma_w}$, then

$$\theta_1 = \phi + ky \qquad (1.4.3.1)$$

$\theta_1$ is seen to be an harmonic function of x and y as $\nabla^2\theta_1 = \nabla^2\phi \equiv 0$. Hence its conjugate is the function $\theta_2 = \psi - kx$ (1.4.3.2)

Defining $\theta_1 + i\theta_2 = \theta$, we observe that

$$\theta = \theta_1 + i\theta_2 = w - ikz \qquad (1.4.3.3)$$

$w = \phi + i\psi$ $\quad$ $z = x + iy$

Definition (1.4.3.3) and any function with its real or imaginary part differing from it by a constant multiplier is called Zhukovsky function.

## 1.5 Objective of the Study.

In the light of the status of the studies on seepage from partially penetrating stream the objectives of the present study are:

1. Computing the flow to or from an aquifer to a rectangular stream for different ratios of bed width of the river to depth of the aquifer below bed of the river.
2. Determination of Rushton constants using the computed flows.

The following assumptions have been made in the study:

i) the flow is two dimensional.

ii) symmetrical conditions exist on either side of the stream .

iii) the soil is homogeneous and isotropic.

iv) A stream of finite width partially penetrates the aquifer.

v) The stream forms the boundary of a single layer of aquifer.

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# CHAPTER 2

## Literature Review

### 2.1 General

Stream aquifer interaction has been studied in greater details in recent years. There are two aspects of the process: i) the exchange of flow between the stream and the aquifer during the passage of a flood wave; and ii) the effluent discharge during lean flow period. The groundwater flow during the passage of a flood wave remains in an unsteady state where as the flow during the lean flow can be regarded as steady. Solution of Laplace equation, which satisfies the boundary conditions prevailing at the flow domain boundaries, enables quantification of steady state seepage from an aquifer. Effluent or influent seepage can be evaluated using analytical approach only for idealized stream aquifer system.

Partially penetrating rivers offer additional resistance against flow. Therefore, the effect of a partially penetrating river can be modeled as an applied potential with the flow from the river acting in a similar manner to leakage through an overlying stratum. An alternative graphical method of estimating the additional resistance was proposed by Numerov(Aravin and Numerov,1965). From detailed analyses of the sharply deformed seepage patterns, the additional resistance can be superimposed on the normal flow pattern. From graphical presentations contained in the above publications, values of the additional seepage resistance for different river cross-sections can be estimated.

### 2.2 Dupuit-Forchheimer Assumptions.

Dupuit based his assumptions on the observation that in most groundwater flows, the slope of the phreatic surface is very small. Slopes of 1/1000 and 10/1000 are commonly encountered (Bear, 1979).

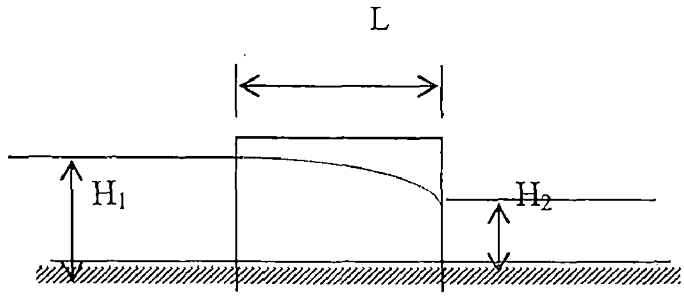



Fig 2.2.1 One-dimensional flow in an unconfined aquifer.

The total discharge through a section of length B (perpendicular to the plane drawn in fig) is $Q = kB\left(\frac{H_1^2 - H_2^2}{2L}\right)$

Although this formula has been derived by disregarding the variation of head with the vertical co-ordinate, and although the so-called seepage surface at the downstream boundary was not taken into consideration, the formula has been found to give excellent results, even when the length L is very small and the head difference ($H_1$-$H_2$) is very large (Veruijt, 1982).

Proof, that the Dupuit-Forchheimer can yield exact solutions for flow below mildly slopping water tables, was given by Charny (Polubarinova-Kochina, 1962) .The Dupuit-Forchheimer theory loses accuracy if the depth of the impermeable layer below the river bed increases, because of the increased importance of vertical flow. In comparing seepage rates based on the Dupuit-Forchheimer theory with solutions obtained with an electrical resistance network analog, which takes vertical flow components into account, Bouwer(1969) found that the Dupuit-Forchheimer theory gave reasonably accurate seepage values if the distance of the impermeable layer below the stream bottom was not more than twice the width of the water level in the stream.

The Dupuit assumptions are probably the most powerful tool for treating unconfined flows. (Bear 1979).

# CHAPTER 3

## Seepage from an Unconfined Aquifer to a Rectangular Channel.

### 3.1 General

Flow from an unconfined aquifer to a channel can be analyzed by different Methods. The direct method of attacking the problem is by using Zhukovsky function. There are also other approximate methods such as, graphical flow net, electrical analogue, Hele-shaw model, the method of fragments, etc. In the present study the method of fragments is used.

### 3.2 Statement of the Problem.

A rectangular channel in an isotropic and homogeneous porous medium is assumed to receive steady state seepage. Symmetrical conditions are assumed on both sides of the stream. It is required to compute the flow from an aquifer to the channel. Then making use of the computed values, the Rushton constants would be determined for different ratios of width of the river to depth of the aquifer below the riverbed.

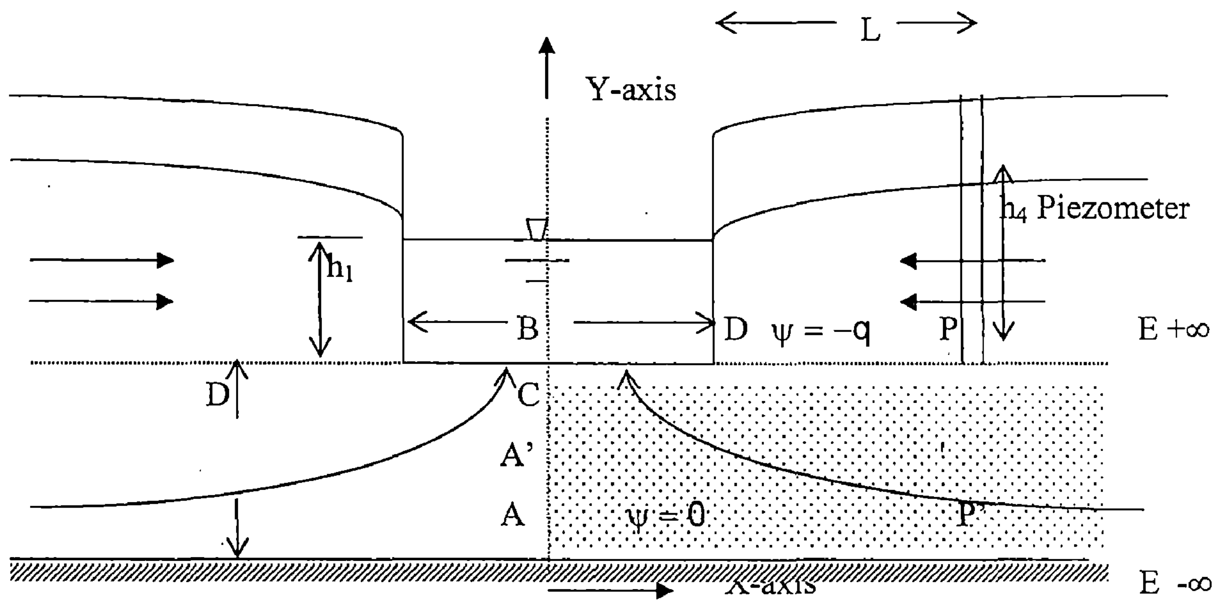



Fig 3.2.1 Physical flow domain (Z plane)

W-plane

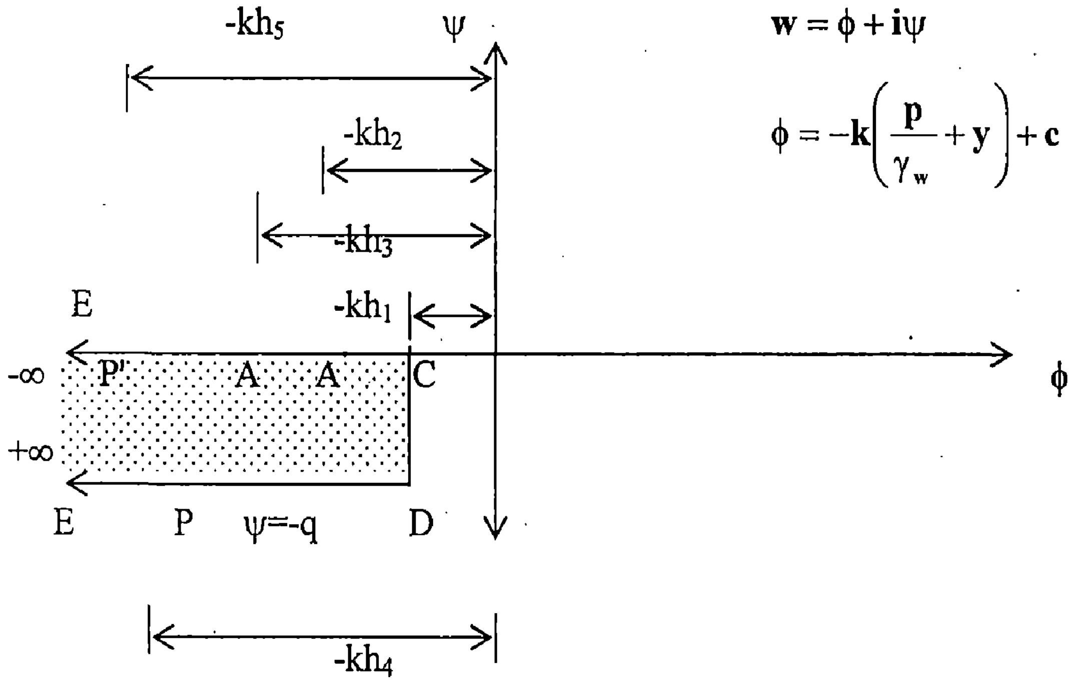



Fig 3.2.2 The complex potential plane.

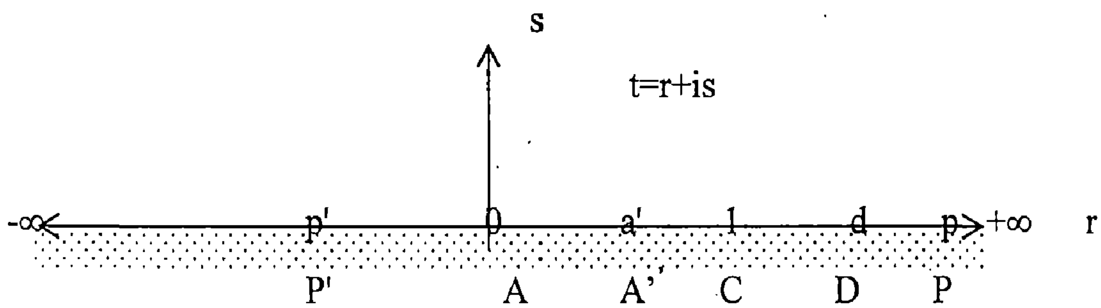



Fig 3.2.3 Auxiliary Plane.

### 3.3 Analysis

The problem is tackled by splitting the original problem into three segments. Dupuit-Forchheimer assumption is used for the portion of flow above the riverbed. That portion of flow, which is below the riverbed, is considered as confined flow and the method of conformal mapping is used for this portion. For some cases Dupuit-Forchheimer assumption is used for the whole flow system and the result is compared with the previous method.

Flows from aquifer to river and vice-versa have been calculated for different values of B/D ratio and different depth of water in stream and the result is tabulated below. Numerical values are assumed for hydraulic conductivity and bed width of the river.

According to Schwarz-Christoffel transformation, the mapping of complex potential to the lower half of the auxiliary plane is given by

$$\frac{dw}{dt} = \frac{M}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}}$$

Integrating, $w = M\int \frac{dt}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}} + N$

Replacing $t = Re^{i\theta}$ and $dt = Re^{i\theta}\, i d\theta$

$$W = M\int \frac{Re^{i\theta}\, i d\theta}{\left(Re^{i\theta}-1\right)^{\frac{1}{2}}\left(Re^{i\theta}-d\right)^{\frac{1}{2}}} + N \qquad (1)$$

As one traverses on t-plane along a radius of infinity from $\pi$ to $2\pi$, the corresponding change in w-plane is -iq

Hence, $-iq = M\int_{\pi}^{2\pi} \frac{Re^{i\theta}\, i d\theta}{\left(Re^{i\theta}-1\right)^{\frac{1}{2}}\left(Re^{i\theta}-d\right)^{\frac{1}{2}}}$

$$\lim_{R\to\infty} M\int_{\pi}^{2\pi}\frac{Re^{i\theta}\,i d\theta}{(Re^{i\theta}-1)^{\frac{1}{2}}(Re^{i\theta}-d)^{\frac{1}{2}}}$$

$$=M\int_{\pi}^{2\pi}\frac{Re^{i\theta}\,d\theta}{Re^{i\theta}\left(1-\frac{1}{Re^{i\theta}}\right)^{\frac{1}{2}}\left(1-\frac{d}{Re^{i\theta}}\right)^{\frac{1}{2}}}$$

$$=Mi\int_{\pi}^{2\pi} d\theta = Mi\theta \Big| = Mi\pi$$

Hence, $-iq = Mi\pi$

And, $M=\frac{-q}{\pi}$. Substituting M in (1), and noting that the constant N corresponds to the lower limit of integration,

$$W=\frac{-q}{\pi}\int_{d}^{t}\frac{dt}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}}-kh_1-iq$$

Applying the condition at P where t= p and w= $-kh_4 - iq$,

$$-kh_4-iq=\frac{-q}{\pi}\int_{d}^{p}\frac{dt}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}}-kh_1-iq$$

Equating the real parts,

$$k(h_1-h_4)=\frac{-q}{\pi}\int_{d}^{p}\frac{dt}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}}$$

Thus,

$$q=\frac{\pi k(h_4-h_1)}{\ln\left[\frac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]}$$

Therefore total confined flow equals $2q = 2\frac{\pi k(h_4-h_1)}{\ln\left[\frac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]}$.

The parameters p and d are unknown, they are to be found from the relation between Z and t plane.

The unconfined flow is calculated using Dupuit-Forchheimer assumption.Thus $q_u = K\left(\frac{h_4^2-h_1^2}{2L}\right)$ where L is the distance between the bank of the river and the piezometer.

In this case L=4B

Therefore total flow from aquifer to river equals $2\dfrac{\pi k(h_4 - h_1)}{\ln\left[\dfrac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]} + K\left(\dfrac{h_4^2 - h_1^2}{4B}\right)$

Similarly the mapping of Z-plane to the lower half of the auxiliary plane gives,
$$\frac{dz}{dt} = \frac{M}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}} = \frac{M}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}}$$

Substituting $t=\sin^2\theta$ and $dt = 2\sin\theta\cos\theta d\theta$,

$\sqrt{t} = \mathrm{Sin}\theta$ and $\theta = \sin^{-1}t$

For point C, Z=iD and t=1 Hence, $iD == 2M\frac{\pi}{2}$. Therefore, $M = \frac{iD}{\pi}$

Thus for $0 \le t \le 1$ , $Z = \dfrac{i2D\sin^{-1}\sqrt{t}}{\pi}$ and

For $t \ge 1$ $\quad Z = M\displaystyle\int_1^t \frac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}} + iD$

For Point D, $Z = \frac{B}{2} + iD$ and t= d, hence, $\dfrac{B}{2} + iD = \dfrac{iD}{\pi}\displaystyle\int_1^d \frac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}} + iD$

Thus, $\dfrac{B}{2} = \dfrac{D}{\pi}\displaystyle\int_1^d \frac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}}$

Therefore, $\dfrac{B}{2} = \dfrac{D}{\pi}\left\{2\ln\left(t^{\frac{1}{2}} + (t-1)^{\frac{1}{2}}\right)\right\}_1^d$

After further Simplification,

$d^{\frac{1}{2}} + (d-1)^{\frac{1}{2}} = e^{\frac{B\pi}{4D}}$

Finally the parameter d is found to be; $d = \dfrac{1 + 2e^{\frac{B\pi}{2D}} + e^{\frac{B\pi}{D}}}{4e^{\frac{B\pi}{2D}}}$

Similarly for point P, $Z = \frac{\mathbf{B}}{2} + \mathbf{L} + \mathbf{iD}$ and t =p,

Then, $\frac{\mathbf{B}}{2} + \mathbf{L} + \mathbf{iD} = \frac{D}{\pi}\int_1^p \frac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}}$ +iD. Equating the real parts,

$$\frac{\mathbf{B}}{2} + \mathbf{L} = \frac{D}{\pi}\int_1^p \frac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}} = \frac{D}{\pi}\left\{\ln\left(t - \frac{1}{2} + (t^2 - t)^{\frac{1}{2}}\right)\right\}_1^p = \frac{D}{\pi}\left\{\ln\left(p - \frac{1}{2} + (p^2 - p)^{\frac{1}{2}}\right) - \ln\left(\frac{1}{2}\right)\right\}$$

$$\text{Hence, } \frac{\pi}{\mathbf{D}}\left(\frac{\mathbf{B}}{2} + \mathbf{L}\right) = \mathbf{ln}\left(\frac{p - \frac{1}{2} + \sqrt{p^2 - p}}{\frac{1}{2}}\right) = \ln\left(2p - 1 + 2\sqrt{p^2 - p}\right)$$

$$\text{Then, } e^{\frac{\pi}{D}\left(\frac{B}{2}+L\right)} = 2p - 1 + 2\sqrt{p^2 - p}$$

$$\text{Therefore, } p = \frac{1 + 2e^{\frac{\pi}{D}\left(\frac{B}{2}+L\right)} + e^{\frac{2\pi}{D}\left(\frac{B}{2}+L\right)}}{4e^{\frac{\pi}{D}\left(\frac{B}{2}+l\right)}}$$

Numerical values are assumed for bed width of the river and the hydraulic conductivity of the medium. The seepage is calculated and tabulated below.

**Table 3.3.1 calculation of flow for B/D =5 and $h_1$ =5m**

| B/D =5 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) x10-5 | h2-h1 (m) $x\,10^{-5}$ |
|---|---|---|---|---|---|---|---|
| | 5 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 5.1 | 0.1 | 9.78 | 5.05 | 1.48 | 8.68 | 8.68 |
| | 5.2 | 0.2 | 19.57 | 10.2 | 2.98 | 17.4 | 17.4 |
| | 5.3 | 0.3 | 29.35 | 15.5 | 4.49 | 26 | 26 |
| | 5.4 | 0.4 | 39.13 | 20.8 | 5.99 | 34.7 | 34.7 |
| | 5.5 | 0.5 | 48.92 | 26.3 | 7.52 | 43.4 | 43.4 |
| | 5.6 | 0.6 | 58.7 | 31.8 | 9.05 | 52.1 | 52.1 |
| | 5.7 | 0.7 | 68.48 | 37.5 | 10.6 | 60.7 | 60.7 |
| | 5.8 | 0.8 | 78.27 | 43.2 | 12.1 | 69.4 | 69.4 |
| | 5.9 | 0.9 | 88.05 | 49.1 | 13.7 | 78.1 | 78.1 |
| | 6 | 1 | 97.84 | 55 | 15.3 | 86.8 | 86.8 |

**Table 3.3.2 Calculation of flow for B/D =5 and $h_1$ =10m**

| B/D =5 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) x10-5 | h2-h1 (m ) $x10^{-5}$ |
|---|---|---|---|---|---|---|---|
| | 10 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 10.1 | 0.1 | 9.78 | 10.05 | 1.98 | 8.68 | 8.68 |
| | 10.2 | 0.2 | 19.57 | 20.2 | 3.98 | 17.4 | 17.4 |
| | 10.3 | 0.3 | 29.35 | 30.45 | 5.98 | 26 | 26 |
| | 10.4 | 0.4 | 39.13 | 40.8 | 7.99 | 34.7 | 34.7 |
| | 10.5 | 0.5 | 48.92 | 51.25 | 10.0 | 43.4 | 43.4 |
| | 10.6 | 0.6 | 58.7 | 61.8 | 12.1 | 52.1 | 52.1 |
| | 10.7 | 0.7 | 68.48 | 72.45 | 14.1 | 60.7 | 60.7 |
| | 10.8 | 0.8 | 78.27 | 83.2 | 16.1 | 69.4 | 69.4 |
| | 10.9 | 0.9 | 88.05 | 94.05 | 18.2 | 78.1 | 78.1 |
| | 11.0 | 1 | 97.84 | 105 | 20.2 | 86.8 | 86.8 |

**Table 3.3.3 Calculation of flow for B/D =1 and h1 =5m.**

| B/D =1 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 5+ | h2-h1 (m) |
|---|---|---|---|---|---|---|---|
| | 5 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 5.1 | 0.1 | 43.75 | 5.05 | 4.88 | .01 | .01 |
| | 5.2 | 0.2 | 87.5 | 10.2 | 9.77 | .02 | .02 |
| | 5.3 | 0.3 | 131 | 15.5 | 14.6 | .03 | .03 |
| | 5.4 | 0.4 | 175 | 20.8 | 19.6 | .04 | .04 |
| | 5.5 | 0.5 | 218 | 26.3 | 24.4 | .05 | .05 |
| | 5.6 | 0.6 | 262 | 31.8 | 29.4 | .06 | .06 |
| | 5.7 | 0.7 | 306 | 37.5 | 34.4 | .07 | .07 |
| | 5.8 | 0.8 | 350 | 43.2 | 39.3 | .08 | .08 |
| | 5.9 | 0.9 | 394 | 49.1 | 44.3 | .09 | .09 |
| | 6 | 1 | 438 | 55 | 49.3 | .10 | .10 |

**Table 3.3.4 Calculation of flow for B/D =1 and h1 =10m**

| B/D =1 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 10+ | h2-h1 (m ) |
|---|---|---|---|---|---|---|---|
| | 10 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 10.1 | 0.1 | 43.75 | 10.05 | 5.38 | .01 | .01 |
| | 10.2 | 0.2 | 87.5 | 20.2 | 10.8 | .02 | .02 |
| | 10.3 | 0.3 | 131 | 30.45 | 16.1 | .03 | .03 |
| | 10.4 | 0.4 | 175 | 40.8 | 21.6 | .04 | .04 |
| | 10.5 | 0.5 | 218 | 51.25 | 26.9 | .05 | .05 |
| | 10.6 | 0.6 | 262 | 61.8 | 32.4 | .06 | .06 |
| | 10.7 | 0.7 | 306 | 72.45 | 37.8 | .07 | .07 |
| | 10.8 | 0.8 | 350 | 83.2 | 43.3 | .08 | .08 |
| | 10.9 | 0.9 | 394 | 94.05 | 49.2 | .09 | .09 |
| | 11.0 | 1 | 438 | 105 | 54.3 | .10 | .10 |

**Table 3.3.5 Calculation of flow for B/D 0.2 and h1 =5m**

| B/D =0.2 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | Confined flow ($m^3$/s/m) x $10^{-5}$ | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | $h_2$ (m) 5+ | $h_2$-$h_1$ (m) |
|---|---|---|---|---|---|---|---|
| | 5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| | 5.1 | 0.1 | 9.5 | 5.05 | 10.0 | .07 | .07 |
| | 5.2 | 0.2 | 19 | 10.2 | 20.0 | .14 | .14 |
| | 5.3 | 0.3 | 28.5 | 15.5 | 30.1 | .21 | .21 |
| | 5.4 | 0.4 | 38 | 20.8 | 40.1 | .28 | .28 |
| | 5.5 | 0.5 | 47.5 | 26.3 | 50.1 | .35 | .35 |
| | 5.6 | 0.6 | 57 | 31.8 | 60.2 | .42 | .42 |
| | 5.7 | 0.7 | 66.5 | 37.5 | 70.3 | .49 | .49 |
| | 5.8 | 0.8 | 76 | 43.2 | 80.3 | .56 | .56 |
| | 5.9 | 0.9 | 85.5 | 49.1 | 90.4 | .63 | .63 |
| | 6 | 1 | 95 | 55 | 101 | .70 | .70 |

**Table 3.3.6 Calculation of flow for B/D 0.2 and h1 =10 m.**

| B/D =0.2 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 10+ | h2-h1 (m ) |
|---|---|---|---|---|---|---|---|
| | 10 | 0 | 0.0 | 0 | 0.0 | 0 | 0.0 |
| | 10.1 | 0.1 | 9.5 | 10.05 | 10.5 | .07 | .07 |
| | 10.2 | 0.2 | 19.0 | 20.2 | 21.0 | .14 | .14 |
| | 10.3 | 0.3 | 28.5 | 30.45 | 31.5 | .21 | .21 |
| | 10.4 | 0.4 | 38.0 | 40.8 | 42.1 | .28 | .28 |
| | 10.5 | 0.5 | 47.5 | 51.25 | 52.6 | .35 | .35 |
| | 10.6 | 0.6 | 57.0 | 61.8 | 63.2 | .42 | .42 |
| | 10.7 | 0.7 | 66.5 | 72.45 | 73.7 | .49 | .49 |
| | 10.8 | 0.8 | 76.0 | 83.2 | 84.3 | .56 | .56 |
| | 10.9 | 0.9 | 85.5 | 94.05 | 94.9 | .63 | .63 |
| | 11.0 | 1 | 95.0 | 105 | 106 | .70 | .70 |

The graphical presentation of the seepage values is given below.

Fig 3.3.1 Flow from aquifer to river or vice-versa for B/D =5

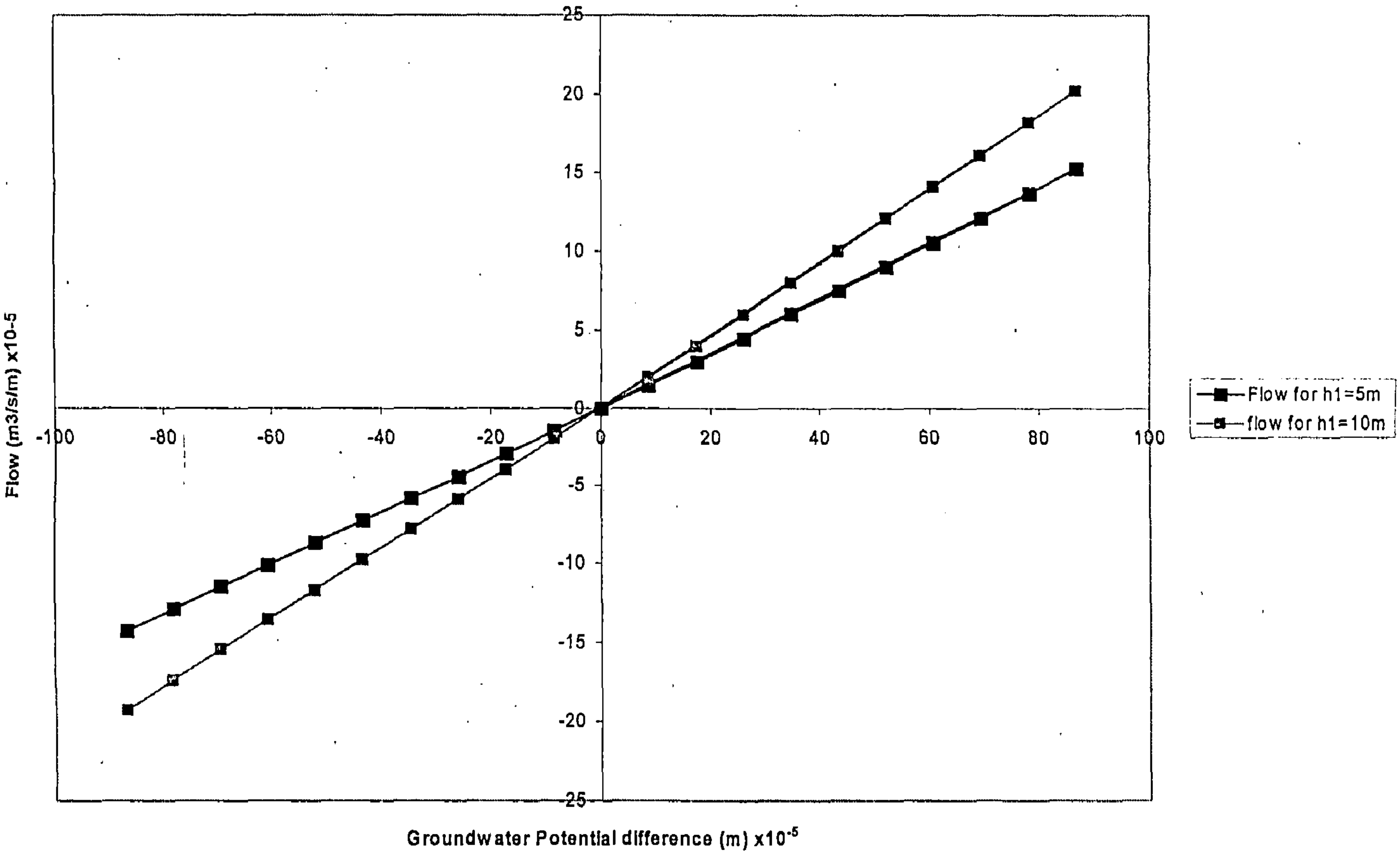


**Fig 3.3.2 Flow from aquifer to river or vice-versa for B/D =1**

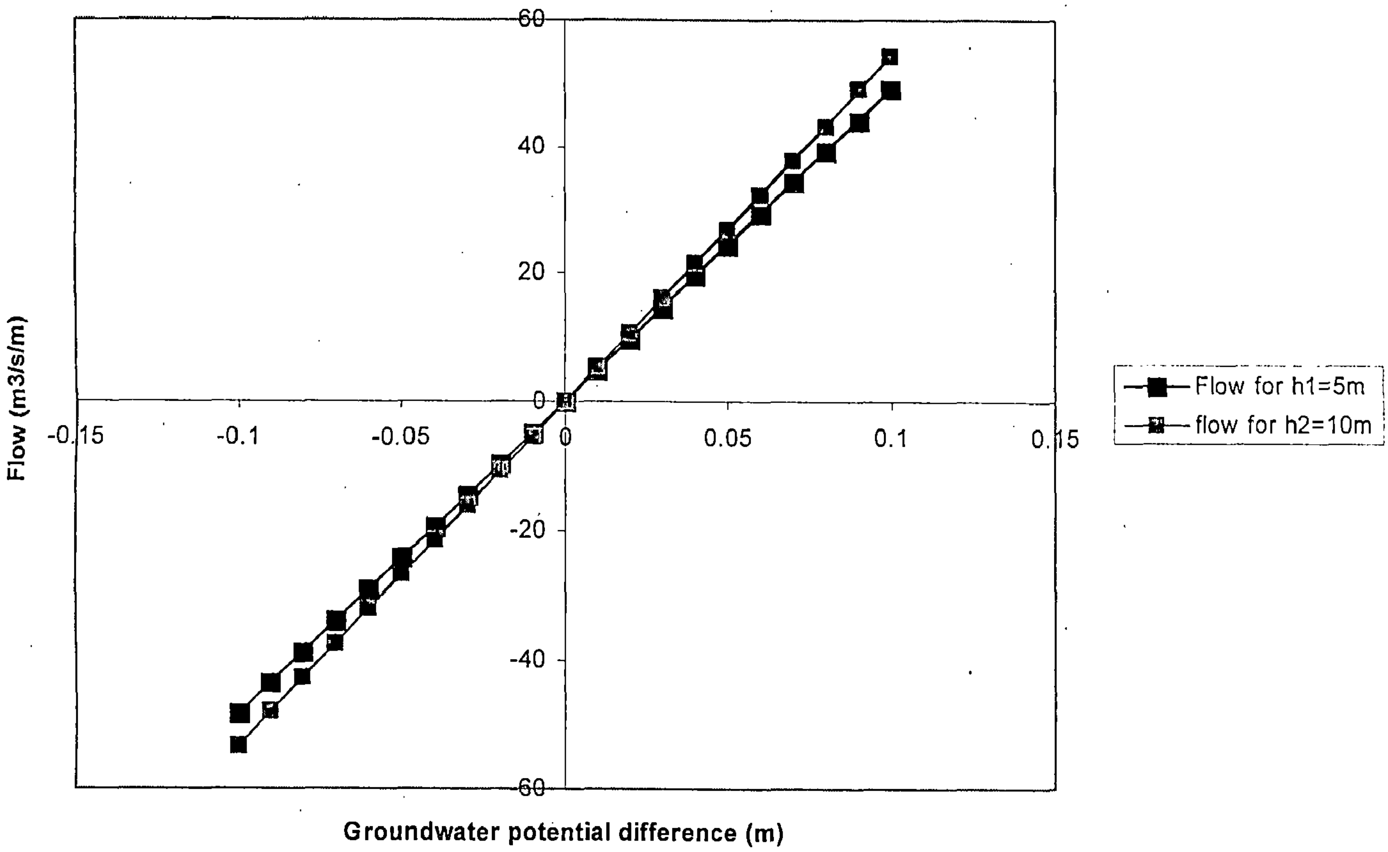


**Fig 3.3.3 Flow from aquifer to river or vice-versa for B/D =0.2**

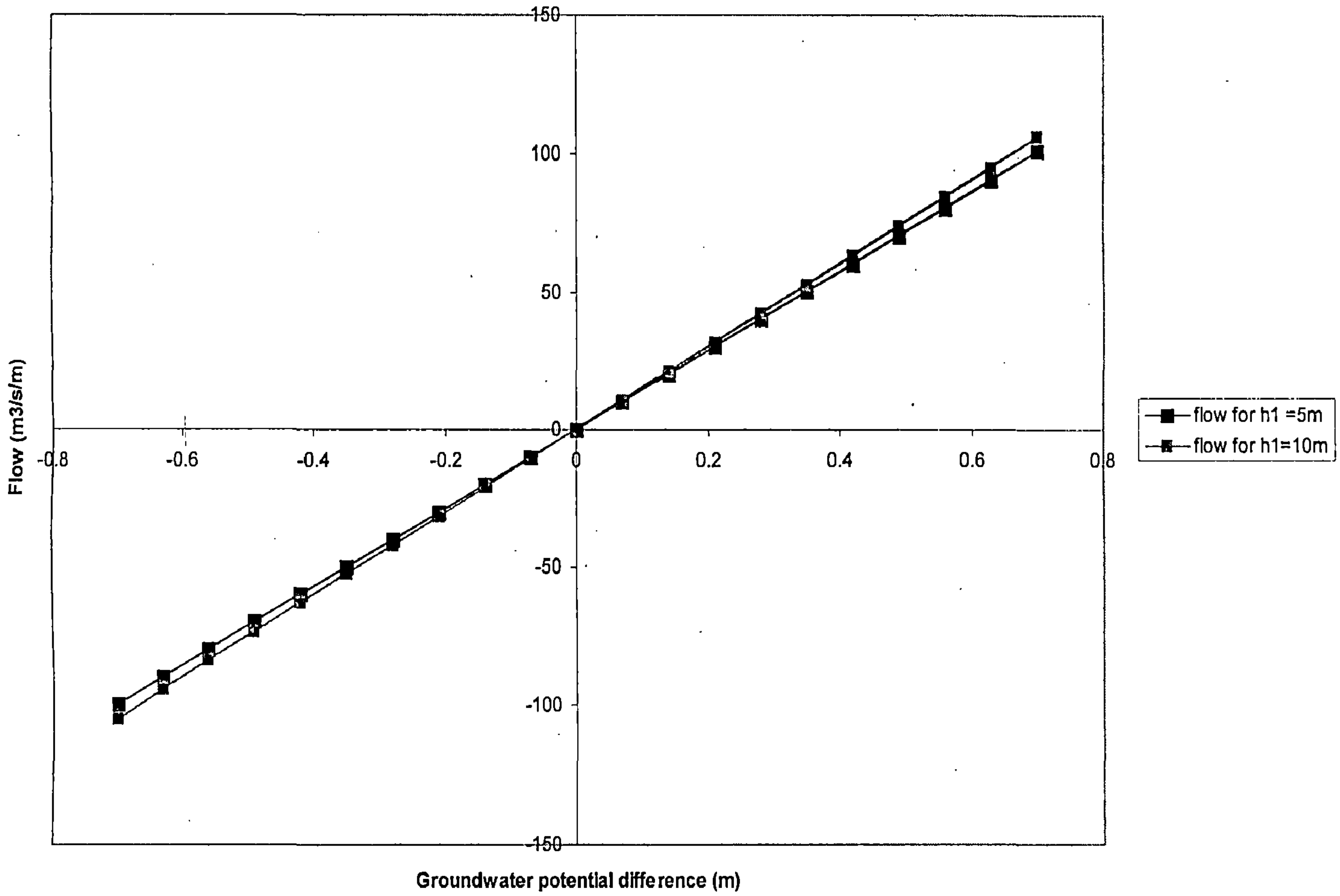


Fig 3.3.4 Comparison of flow using Dupuit's Method and the method of fragments for B/D =1

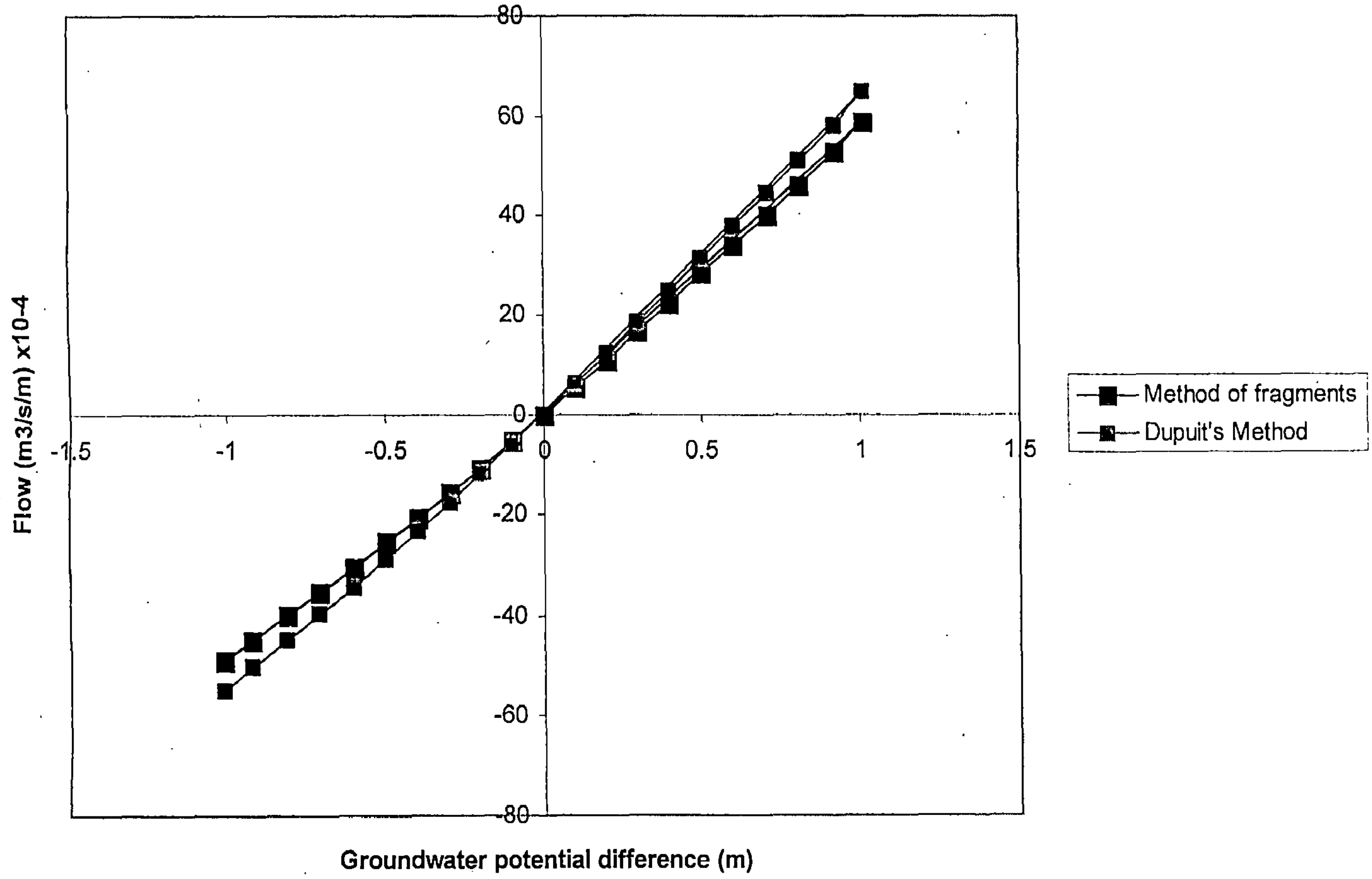


Fig 3.3.5 Comparison of flow using the method of fragments and Dupuit's Assumption for B/D=2

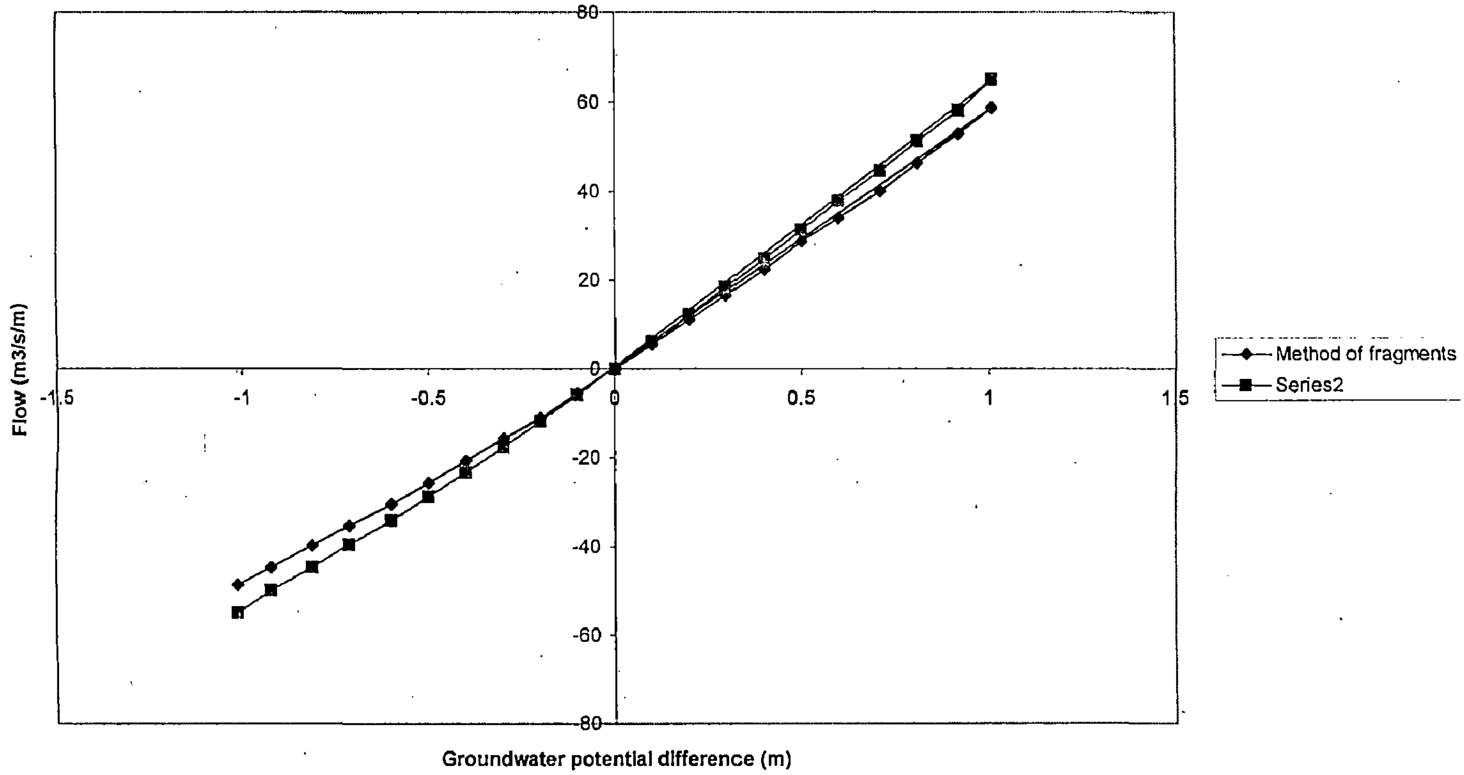


Fig 3.3.6 Comparison of Dupuit's Method and the Method of fragments for B/D =5

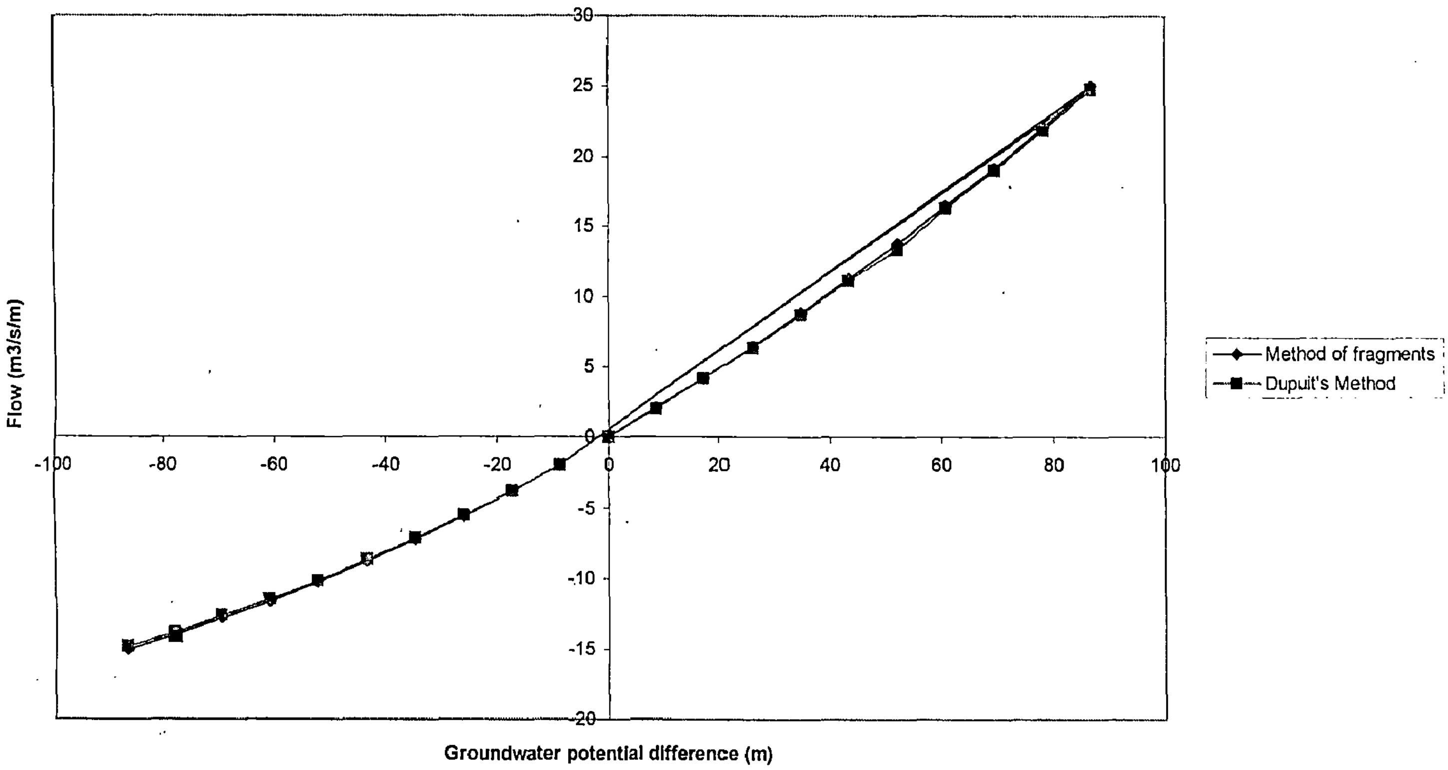


Fig 3.3.7 Relationship between B/D and q/k(h2-h1)

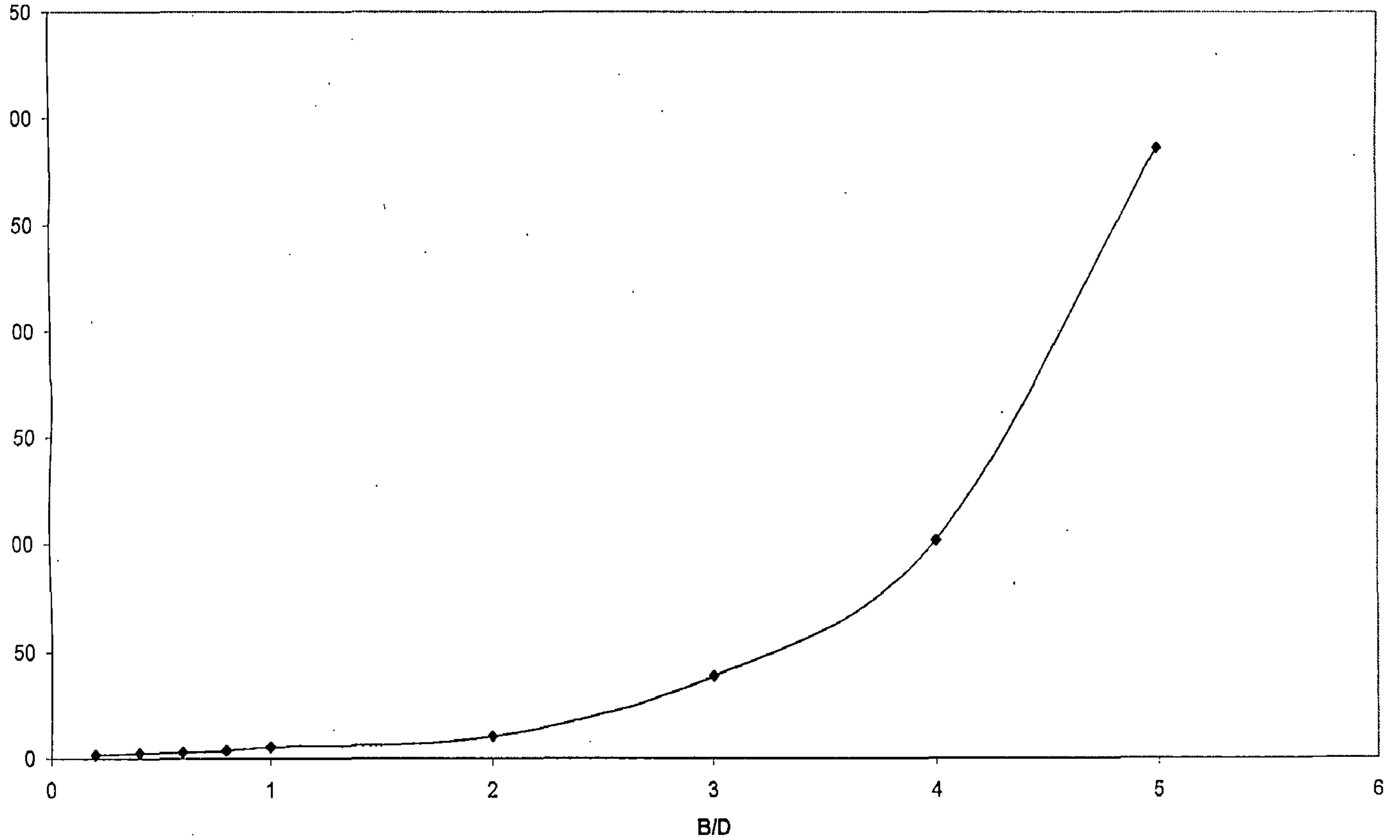


Fig 3.3.8 Variation of Rushton's Constant C1 with B/D ratio for h1=10m and h2-h1 =0.01m

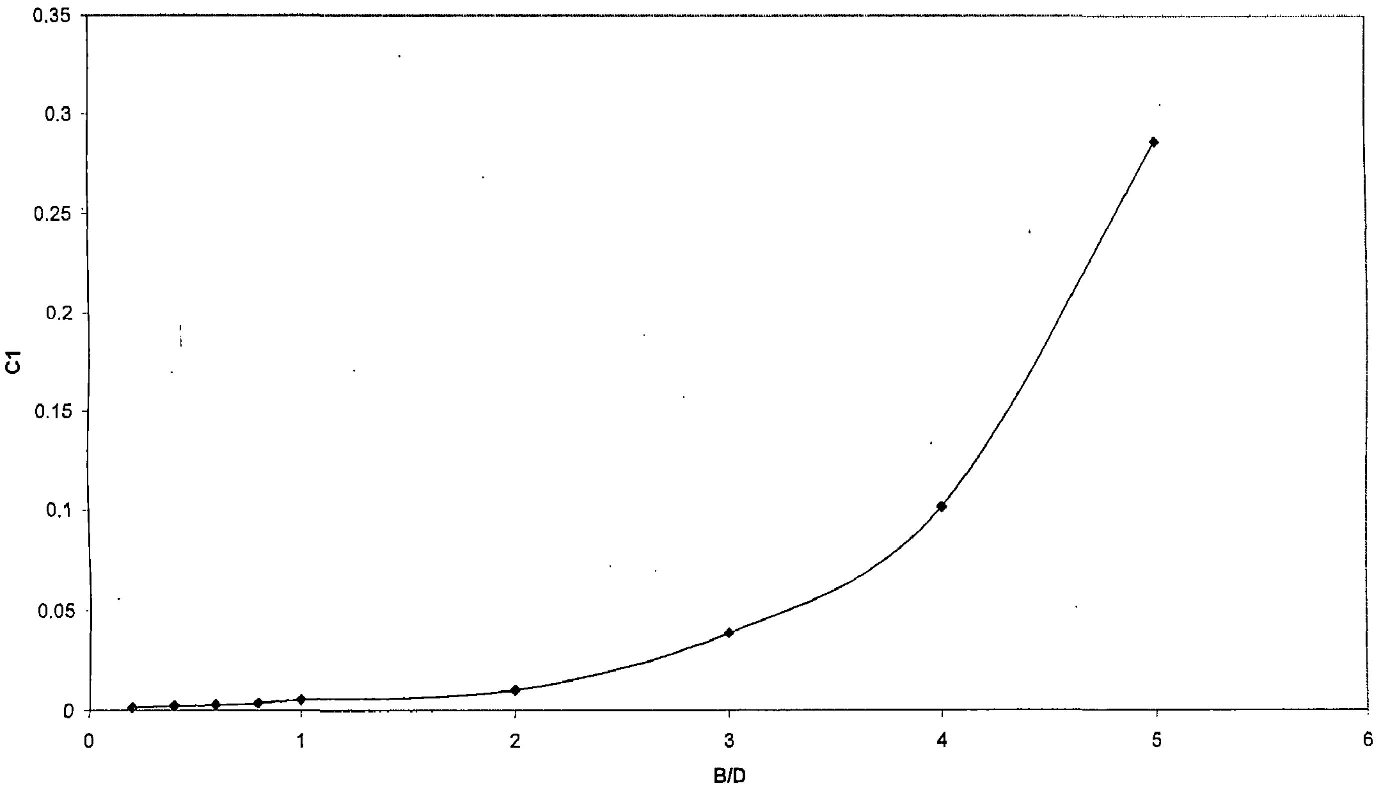


Fig 3.3.9 Variation of Rushton's Constant C1 with h1 for B/D =1

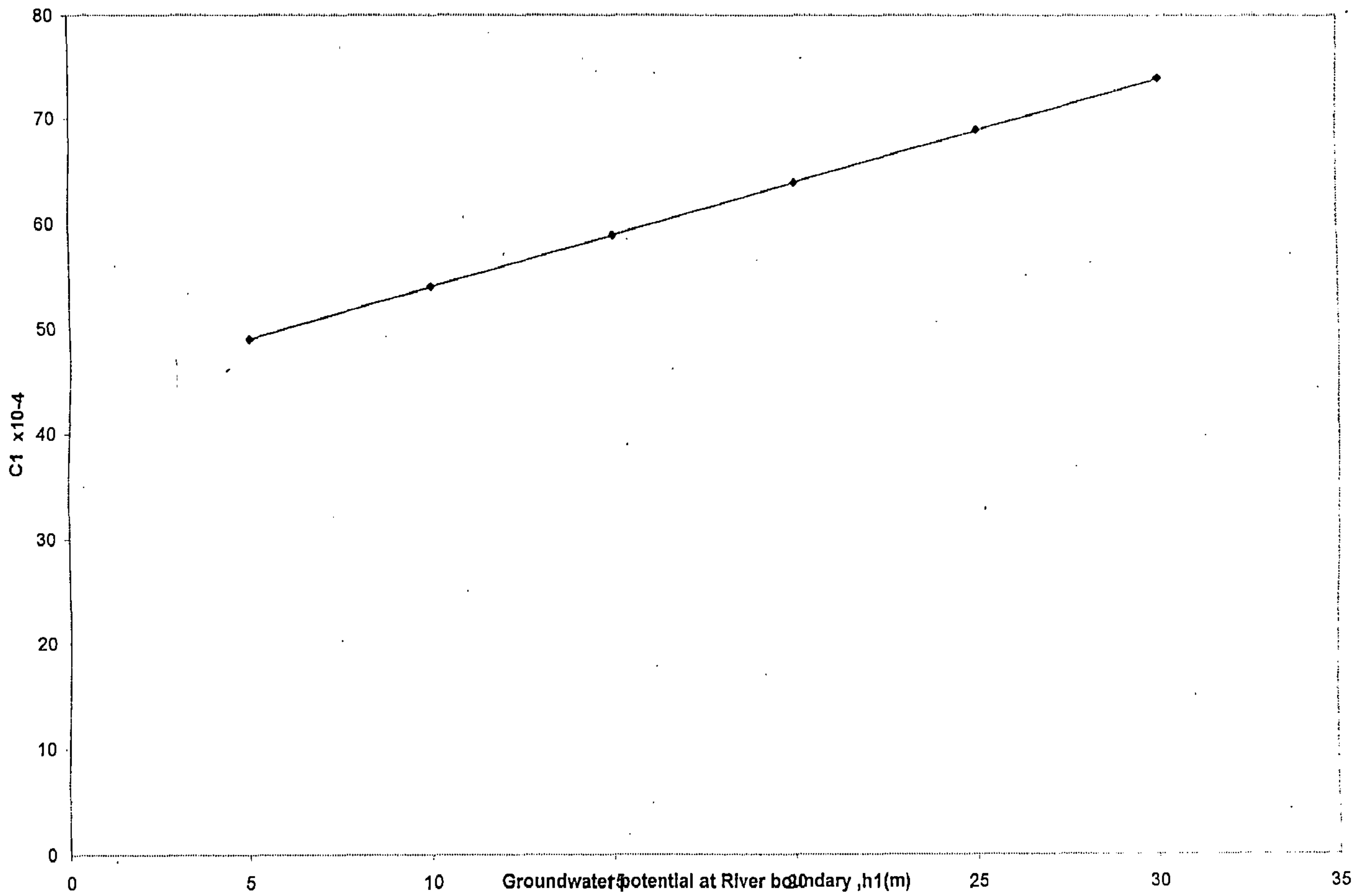
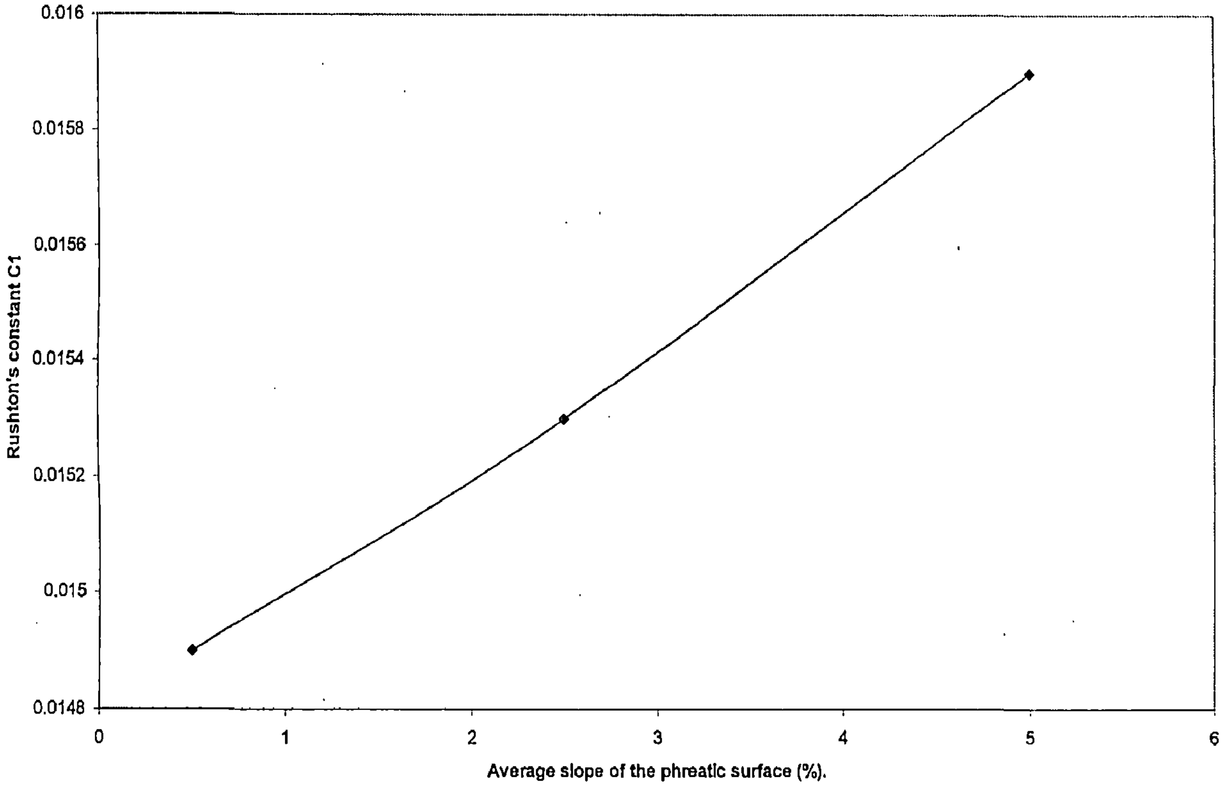


Fig 3.3.10 Variation of Rushton's constant C1 with average slope of the phreatic surface for B/D =2 and h1 =10m



### 3.4 Results and Discussion

It is found that the exponential relationship, which is suggested by Rushton based on field evidence, comes into picture, when the ratio of bed width of the river to depth of the aquifer below the riverbed becomes larger. In cases which are considered presently, that is, when $B/D \leq 5$ it is observed that linear trend can fairly represent the relationship of groundwater potential difference and flow. On the other hand, it is seen that Dupuit's assumption and the method of fragments yielded approximately close results (with an error of 10% for $B/D = 1$).

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# Seepage from Rectangular Channel to an unconfined aquifer.

## 4.1 General

If groundwater levels are below water levels in streams, canals, lakes, or reservoirs, water will seep into the ground from these surface waters .The rate of seepage from streams or canals depends on channel geometry, conductivity of bottom material and underlying soil layers, and depth of groundwater table at some distance from the channel. Flow velocity in the channel has no direct effect on seepage (Bouwer,1978) , but it could affect seepage indirectly because fine particles and other sediments have more chance to accumulate on the bottom of stagnant or slow-flowing channels than in rapid streams.

## 4.2 Statement of the Problem.

A rectangular channel in isotropic and homogeneous porous medium is assumed to feed the aquifer. Symmetrical conditions are assumed on both sides of the stream. It is required to compute the flow from the channel to an aquifer. Then making use of the computed values the Rushton constants are determined for different ratios of width of the river to depth of the aquifer below the river bed.

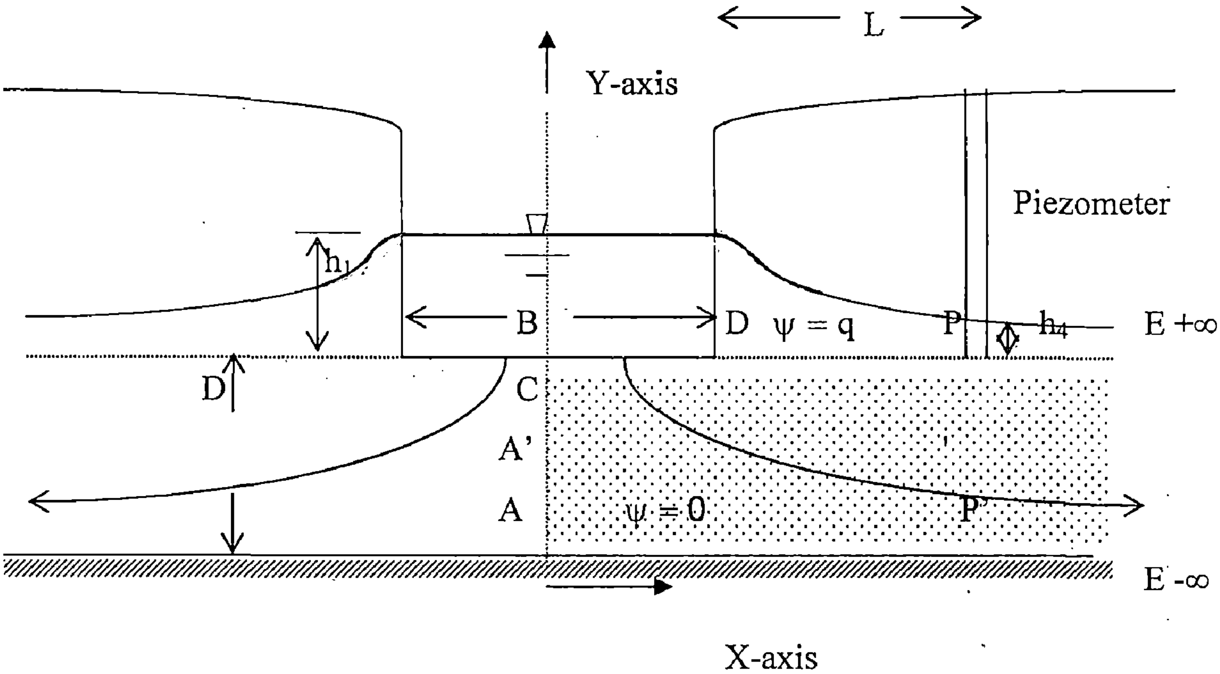


Fig 4.2.1 Physical flow domain Z-Plane.

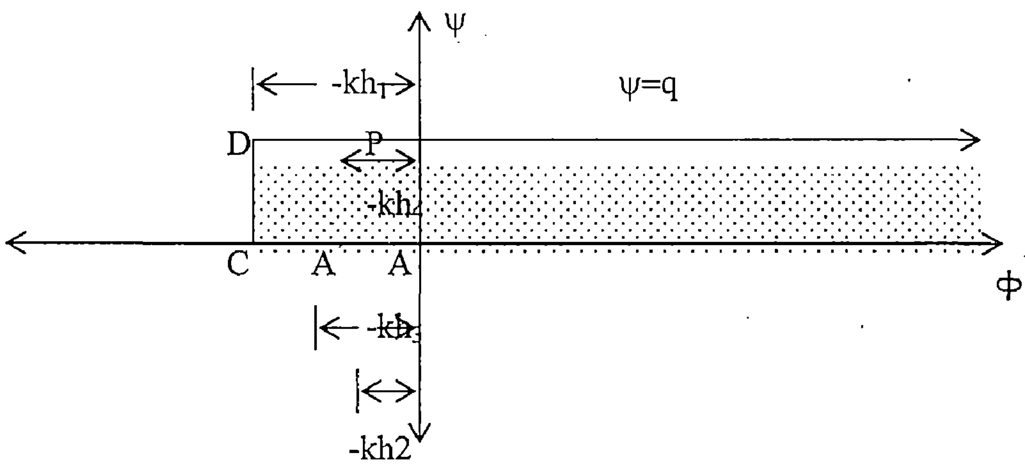


Fig 4.2.2 complex potential plane.

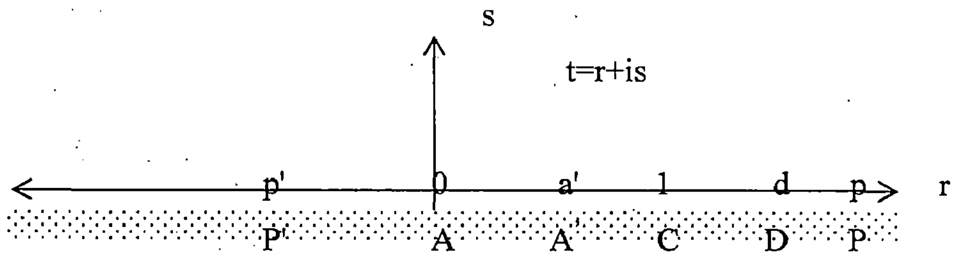


Fig 4.2.3 Auxiliary Plane.

### 4.3 Analysis

The problem is tackled by splitting the original problem into three segments. Dupuit-Forchheimer assumption is used for the portion of flow above the riverbed. That portion of flow, which is below the riverbed, is considered as confined flow and the method of conformal mapping is used for this portion. For some cases Dupuit-Forchheomer assumption is used for the whole flow system and the result is compared with the previous method. The analysis is similar to the reverse case therefore for figures chapter three can be referred.

Using the Schwarz-christoffel transformation, the mapping of the complex potential onto the lower half of the auxiliary plane is,

$$\frac{dw}{dt}=\frac{M}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}}$$

After integrating, $w = M\int\frac{dt}{(t-1)^{\frac{1}{2}}(t-d)^{\frac{1}{2}}} + N$, Where, M and N are arbitrary complex constants.

Substituting $t = Re^{i\theta}$ and $dt = Re^{i\theta}\, i d\theta$,

$$W = M\int\frac{Re^{i\theta}\, i d\theta}{\left(Re^{i\theta}-1\right)^{\frac{1}{2}}\left(Re^{i\theta}-d\right)^{\frac{1}{2}}} + N$$

As one traverse on t-plane along a radius of infinity from $\pi$ to $2\pi$, the corresponding change in w-plane is iq

Thus, $iq = M\int_{\pi}^{2\pi}\frac{Re^{i\theta}\,id\theta}{\left(Re^{i\theta}-1\right)^{\frac{1}{2}}\left(Re^{i\theta}-d\right)^{\frac{1}{2}}}$

$$\lim_{R\to\infty} M\int_{\pi}^{2\pi}\frac{Re^{i\theta}\,id\theta}{\left(Re^{i\theta}-1\right)^{\frac{1}{2}}\left(Re^{i\theta}-d\right)^{\frac{1}{2}}}$$

$$=M\int_{\pi}^{2\pi}\frac{Re^{i\theta}\,d\theta}{Re^{i\theta}\left(1-\frac{1}{Re^{i\theta}}\right)^{\frac{1}{2}}\left(1-\frac{d}{Re^{i\theta}}\right)^{\frac{1}{2}}}$$

$$=Mi\int_{\pi}^{2\pi}d\theta = Mi\theta \Big| = Mi\pi$$

Therefore, $M=\frac{q}{\pi}$

Then, $w=\frac{q}{\pi}\int_{d}^{t}\frac{dt}{(t-1)\frac{1}{2}(t-d)^{\frac{1}{2}}}-kh_1-iq$

For point P where t=p and w = $-kh_4$,

$$-kh_4-iq=\frac{q}{\pi}\int_{d}^{p}\frac{dt}{(t-1)\frac{1}{2}(t-d)^{\frac{1}{2}}}-kh_1-iq$$

Hence, $k(h_1-h_4)=\frac{q}{\pi}\int_{d}^{p}\frac{dt}{(t-1)\frac{1}{2}(t-d)^{\frac{1}{2}}}=\frac{2q}{\pi}\left\{\ln\left[\sqrt{t-1}+\sqrt{t-d}\right]_{d}^{p}\right.$

$$=\frac{2q}{\pi}\left\{\ln\left[\frac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]\right\}$$

Then, $q = \dfrac{\pi k(h1 - h4)}{\ln\left[\dfrac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]}$

Therefore, total confined flow equals $2q = 2\dfrac{\pi k(h_1 - h4)}{\ln\left[\dfrac{\sqrt{p-1}+\sqrt{p-d}}{\sqrt{d-1}}\right]}$

Similarly, $\dfrac{dz}{dt} = M\dfrac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}} \quad = M\dfrac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}}$

Substituting $t = \sin^2\theta$ and $dt = 2\sin\theta\cos\theta d\theta$,
$\sqrt{t} = \text{Sin}\theta$ and $\theta = \sin^{-1}t$

$Z = M\int_0^{t'}\dfrac{2\sin\theta\cos\theta d\theta}{\sin\theta\cos\theta} \quad = 2M\theta = 2M\sin^{-1}\sqrt{t}$

For point C, Z=iD and t=1. Hence, $iD = 2M\dfrac{\pi}{2}$ and $M = \dfrac{iD}{\pi}$

Thus for $0 \le t \le 1$, $Z = \dfrac{i2D\sin^{-1}\sqrt{t}}{\pi}$

For $t \ge 1$ $\quad Z = M\int_1^{t'}\dfrac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}} + iD$

For point D, $Z = \dfrac{B}{2} + iD$ and t= d. Hence, $\dfrac{B}{2} + iD = \dfrac{iD}{\pi}\int_1^d\dfrac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}} + iD$

Then, $\dfrac{B}{2} = \dfrac{iD}{\pi}\int_1^d\dfrac{dt}{t^{\frac{1}{2}}(1-t)^{\frac{1}{2}}} \quad = \dfrac{D}{\pi}\int_1^d\dfrac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}}$

$= \dfrac{D}{\pi}\left\{2\ln\left(t^{\frac{1}{2}} + (t-1)^{\frac{1}{2}}\right)\right\}_1^d$

$$= 2\ln\left(d^{\frac{1}{2}} + (d-1)^{\frac{1}{2}}\right) - 2\ln(1+0)$$ . Then, $$\frac{B\pi}{4D} = \ln\left(d^{\frac{1}{2}} + (d-1)^{\frac{1}{2}}\right)$$

Therefore, $$d = \frac{1 + 2e^{\frac{B\pi}{2D}} + e^{\frac{B\pi}{D}}}{4e^{\frac{B\pi}{2D}}}$$

Similarly at point P where $Z = \frac{\mathbf{B}}{\mathbf{2}} + \mathbf{L} + \mathbf{iD}$ and $t = p$

$$\frac{\mathbf{B}}{\mathbf{2}} + \mathbf{L} + \mathbf{iD} = \frac{D}{\pi}\int_1^p \frac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}} + iD \quad = \frac{D}{\pi}\int_1^p \frac{dt}{t^{\frac{1}{2}}(t-1)^{\frac{1}{2}}}$$

$$= \frac{D}{\pi}\left\{\ln\left(t - \frac{1}{2} + (t^2 - t)^{\frac{1}{2}}\right)\right\}_1^p \quad = \frac{D}{\pi}\left\{\ln\left(p - \frac{1}{2} + (p^2 - p)^{\frac{1}{2}}\right) - \ln\left(\frac{1}{2}\right)\right\}$$

$$= \ln\left(\frac{p - \frac{1}{2} + \sqrt{p^2 - p}}{\frac{1}{2}}\right) \quad = \ln\left(2p - 1 + 2\sqrt{p^2 - p}\right)$$

Therefore, $$p = \frac{1 + 2e^{\frac{\pi}{D}\left(\frac{B}{2}+L\right)} + e^{\frac{2\pi}{D}\left(\frac{B}{2}+L\right)}}{4e^{\frac{\pi}{D}\left(\frac{B}{2}+1\right)}}$$

Numerical Values are assumed for the hydraulic conductivity and bed width of the river and the results are tabulated below.

**Table 4.3.1 Calculation of flow for B/D= 5 and $h_1$ =5m.**

| B/D =5 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) x10-5 | h2-h1 (m) x $10^{-5}$ |
|---|---|---|---|---|---|---|---|
| | 4.9 | -0.1 | 9.78 | 4.95 | 1.47 | 8.68 | 8.68 |
| | 4.8 | -0.2 | 19.57 | 9.8 | 2.94 | 17.4 | 17.4 |
| | 4.7 | -0.3 | 29.35 | 14.6 | 4.4 | 26 | 26 |
| | 4.6 | -0.4 | 39.13 | 19.2 | 5.83 | 34.7 | 34.7 |
| | 4.5 | -0.5 | 48.92 | 23.8 | 7.27 | 43.4 | 43.4 |
| | 4.4 | -0.6 | 58.7 | 28.2 | 8.69 | 52.1 | 52.1 |
| | 4.3 | -0.7 | 68.48 | 32.6 | 10.1 | 60.7 | 60.7 |
| | 4.2 | -0.8 | 78.27 | 36.8 | 11.5 | 69.4 | 69.4 |
| | 4.1 | -0.9 | 88.05 | 41 | 12.9 | 78.1 | 78.1 |
| | 4 | -1 | 97.84 | 45 | 14.3 | 86.8 | 86.8 |

**TAble 4.3.2 Calculation of flow for B/D =5 and $h_1$=10m.**

| B/D =5 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) x10-5 | h2-h1 (m ) x$10^{-5}$ |
|---|---|---|---|---|---|---|---|
| | 9.9 | -0.1 | 9.78 | 9.95 | 1.97 | -8.68 | -8.68 |
| | 9.8 | -0.2 | 19.57 | 19.8 | 3.94 | -17.4 | -17.4 |
| | 9.7 | -0.3 | 29.35 | 29.6 | 5.89 | -26 | -26 |
| | 9.6 | -0.4 | 39.13 | 39.2 | 7.83 | -34.7 | -34.7 |
| | 9.5 | -0.5 | 48.92 | 48.8 | 9.77 | -43.4 | -43.4 |
| | 9.4 | -0.6 | 58.7 | 58.2 | 11.7 | -52.1 | -52.1 |
| | 9.3 | -0.7 | 68.48 | 67.6 | 13.6 | -60.7 | -60.7 |
| | 9.2 | -0.8 | 78.27 | 76.8 | 15.5 | -69.4 | -69.4 |
| | 9.1 | -0.9 | 88.05 | 86.0 | 17.4 | -78.1 | -78.1 |
| | 9.0 | -1 | 97.84 | 95 | 19.3 | -86.8 | -86.8 |

**Table 4.3.3 Calculation of flow for B/D =1 and $h_1$ =5m.**

| B/D =1 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 5+ | h2-h1 (m) |
|---|---|---|---|---|---|---|---|
| | 4.9 | -0.1 | 43.75 | 4.95 | 4.87 | -.01 | -.01 |
| | 4.8 | -0.2 | 87.5 | 9.8 | 9.73 | -.02 | -.02 |
| | 4.7 | -0.3 | 131 | 14.6 | 14.6 | -.03 | -.03 |
| | 4.6 | -0.4 | 175 | 19.2 | 19.4 | -.04 | -.04 |
| | 4.5 | -0.5 | 218 | 23.8 | 24.2 | -.05 | -.05 |
| | 4.4 | -0.6 | 262 | 28.2 | 29.0 | -.06 | -.06 |
| | 4.3 | -0.7 | 306 | 32.6 | 33.9 | -.07 | -.07 |
| | 4.2 | -0.8 | 350 | 36.8 | 38.8 | -.08 | -.08 |
| | 4.1 | -0.9 | 394 | 41 | 43.5 | -.09 | -.09 |
| | 4 | -1 | 438 | 45 | 48.3 | -.10 | -.10 |

**Table 4.3.4 Calculation of flow for B/D =1 and $h_1$ =10m.**

| B/D =1 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 10+ | h2-h1 (m ) |
|---|---|---|---|---|---|---|---|
| | 9.9 | -0.1 | 43.75 | 9.95 | 5.37 | -.01 | -.01 |
| | 9.8 | -0.2 | 87.5 | 19.8 | 10.7 | -.02 | -.02 |
| | 9.7 | -0.3 | 131 | 29.6 | 16.1 | -.03 | -.03 |
| | 9.6 | -0.4 | 175 | 39.2 | 21.4 | -.04 | -.04 |
| | 9.5 | -0.5 | 218 | 48.8 | 26.7 | -.05 | -.05 |
| | 9.4 | -0.6 | 262 | 58.2 | 32.0 | -.06 | -.06 |
| | 9.3 | -0.7 | 306 | 67.6 | 37.4 | -.07 | -.07 |
| | 9.2 | -0.8 | 350 | 76.8 | 42.7 | -.08 | -.08 |
| | 9.1 | -0.9 | 394 | 86.0 | 48 | -.09 | -.09 |
| | 9.0 | -1 | 438 | 95 | 53.3 | -.10 | -.10 |

**Table 4.3.5 Calculation of flow for B/D =0.2 and $h_1$ =5m**

| B/D =0.2 and h1 =5m | h4 (m) | Groundwater potential difference (h4-h1) (m) | Confined flow ($m^3$/s/m) x $10^{-5}$ | unconfined flow (m3/s/m ) x10-6 | Total flow (m3/s/m) x10-5 | $h_2$ (m) 5+ | $h_2$-$h_1$ (m) |
|---|---|---|---|---|---|---|---|
| | 4.9 | -0.1 | 9.5 | 4.95 | 9.99 | -.07 | -.07 |
| | 4.8 | -0.2 | 19 | 9.8 | 20.0 | -.14 | -.14 |
| | 4.7 | -0.3 | 28.5 | 14.6 | 30.0 | -.21 | -.21 |
| | 4.6 | -0.4 | 38 | 19.2 | 39.9 | -.28 | -.28 |
| | 4.5 | -0.5 | 47.5 | 23.8 | 49.9 | -.35 | -.35 |
| | 4.4 | -0.6 | 57 | 28.2 | 59.8 | -.42 | -.42 |
| | 4.3 | -0.7 | 66.5 | 32.6 | 69.8 | -.49 | -.49 |
| | 4.2 | -0.8 | 76 | 36.8 | 79.7 | -.56 | -.56 |
| | 4.1 | -0.9 | 85.5 | 41 | 89.6 | -.63 | -.63 |
| | 4 | -1 | 95 | 45 | 99.5 | -.70 | -.70 |

**Table 4.3.6 Calculation of flow for B/D 0.2 and $h_1$ =10m.**

| B/D =0.2 and h1 =10 m | h4 (m) | Groundwater potential difference (h4-h1) (m) | confined flow (m3/s/m) x10-6 | unconfined flow (m3/s/m) x10-6 | Total flow (m3/s/m) x10-5 | h2 (m) 10+ | h2-h1 (m ) |
|---|---|---|---|---|---|---|---|
| | 9.9 | -0.1 | 9.5 | 9.95 | 10.5 | -.07 | -.07 |
| | 9.8 | -0.2 | 19.0 | 19.8 | 20.9 | -.14 | -.14 |
| | 9.7 | -0.3 | 28.5 | 29.6 | 31.5 | -.21 | -.21 |
| | 9.6 | -0.4 | 38.0 | 39.2 | 41.9 | -.28 | -.28 |
| | 9.5 | -0.5 | -47.5 | 48.8 | 52.4 | -.35 | -.35 |
| | 9.4 | -0.6 | 57.0 | -58.2 | 62.8 | -.42 | -.42 |
| | 9.3 | -0.7 | 66.5 | 67.6 | 73.3 | -.49 | -.49 |
| | 9.2 | -0.8 | 76.0 | 76.8 | 83.7 | -.56 | -.56 |
| | 9.1 | -0.9 | 85.5 | 86.0 | 94.1 | -.63 | -.63 |
| | 9.0 | -1 | 95.0 | 95 | 105 | -.70 | -.70 |

## 4.3 Results and Discussion

It is found that the exponential relationship which is suggested by Rushton based on Field evidence comes into picture, when the ratio of bed width of the river to depth of the aquifer below the river bed becomes larger. In the cases which are considered presently, that is, when B/D$\leq$ 5 it is observed that linear trend can fairly represent the relationship of groundwater potential difference and flow. Moreover, it is seen that Dupuit's assumption and the method of fragments yielded approximately close results (with an error of 10% for B/D =1).

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## Conclusion

It has been observed that different values of Rushton constants can be obtained for a given channel geometry and dimensions .The main factor upon which the values of these parameters depend is the ratio of bed width of the channel to depth of the impervious layer below it. Besides this, depth of water in the channel is the other condition which affects these parameters .On the other hand it is found that the non-linear relationship of flow and groundwater potential difference becomes significant when the slope of the phreatic surface is higher. There is also a tendency of non-linearity for higher water depth for a given slope of the phreatic surface. Moreover, real field situations are complex and there may be so many factors which affect the flow from river to aquifer or vice-versa in different ways. The most notable are non-homogeneity, anisotropy, irregular geometry of river cross-section and sediment deposition in river bed.

It is also found that the flow rates computed by Dupuit-Forchheimer assumption and the method of fragments were approximately same (there is an error of 10% between Dupuit's assumption and the method of fragments for B/D =1)

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