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ANALYSIS OF FLOW TO A DUG-WELL IN HARD ROCK AREA IN AN UNCONFINED AQUIFER BY CELL THEORY

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PREFACE

The occurrence and the distribution of groundwater in an area depend on the characteristics of geological formations of the area. There are different hydrogeological conditions in different parts of the country. Groundwater regime of hard rocks assumes significant importance in our country in view of the fact that nearly 70% of the geographical area of our country is underlain by hard rocks. Groundwater is stored in the weathered and the fissured zones of hard rocks. Owing to the limited depth of weathered zone and its low transmissivity ranging from $1m^2/d$ to $100m^2/d$. Large diameter wells are ideally suited for groundwater abstraction in hard rock areas.

Dug wells are used extensively in hard rock areas for ground water abstraction. Development of ground water in these areas requires the knowledge of the dynamics of flow towards a dug well. Determination of aquifer parameters are prerequisite for planning and wise use of the groundwater resources in an area. Some times an aquifer test is conducted in an existing large-diameter well. Evaluation of aquifer parameters from the test data requires understanding of the flow towards a large-diameter well. The conventional method of analysis of test data based on Theis(1935) equation is not applicable for large diameter wells due to the significant contribution from the well storage during pumping.

In the present study, analysis of unsteady flow to a partially penetrating dug well has been made taking into account the well storage, partial penetration and seepage surface. Three dimensional groundwater flow model developed by Macdonald and

Harbaugh as an extension to the cell theory proposed by Bear has been used for the analysis. The study has been carried out for different well penetration. Set of type-curves have been presented using which aquifer parameters can be determined. The performance of the well, i.e., contribution of well storage to pumping, has been evaluated for different penetration. The component of the well loss for different penetration has been estimated. The results presented are useful for the design of diameter and depth of of a well for required pumping rate and schedule.

The present study has been carried out by Shri S.K. Singh, Scientist 'C' as per the work programme of the Ground Water Assessment Division under the technical guidance of Dr. G.C. Mishra, Scientist 'F' and Technical Coordinator of the Division.

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Abstract

Dug wells are used extensively in hard rock areas for ground water abstraction. Development of ground water in these areas requires the knowledge of the dynamics of flow towards a dug well. In the present study, analysis of unsteady flow to a partially penetrating dug well has been made taking into account the well storage. Three dimensional groundwater flow model develped by Macdonald and Harbaugh as an extension to the cell theory proposed by Bear has been used for the analysis. The study has been carried out for different well penetration. Set of type-curves have been presented using which aquifer parameters can be determined. The performance of the well, i.e., contribution of well storage to pumping, has been evaluated for different penetration. The component of the well loss for different penetration has been estimated. The results presented are useful for the design of diameter and depth of of a well for required pumping rate and schedule.

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(S.K. Singh)

Contents

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Groundwater reservoir is a reliable source of water supply for meeting irrigation and drinking water requirements. According to the statistics of 1983, 25.6 mha area was being irrigated by groundwater in India and 2.5 mha area was being added annually to it (Saxena,1983). Out of 9.5 million wells in India, 79% were large diameters wells, 18% were shallow tube-wells in hard rocks and soft rocks, and 3% were deep tube wells in alluvial areas (Baweja,1979). The groundwater has been given importance during past decade because of droughts. Therefore, development of groundwater for irrigation has become imperative.

The occurrence and the distribution of groundwater in an area depend on the characteristics of geological formations of the area. There are different hydrogeological conditions in different parts of the country. Nearly 65% of the total area of the country consists of hard rocks. The groundwater is stored in the weathered and the fissured zones of hard rocks. Generally, the weathered zone extends up to a depth ranging from 10m to 50m. Water is available under both confined and unconfined conditions. Owing to the limited depth of weathered zone and its low transmissivity ranging from $1m^2/d$ to $100m^2/d$ (Pathak,1984), the groundwater is tapped through large diameter wells in hard rock areas. The advantages of a large diameter dug wells are :

i) construction of large-diameter well is cheap and simple;

ii) large-diameter wells are ideally suited for shallow aquifers with low transmissivities ;

ii) the well loss in a large-diameter well is less than that in a well of small radius because a significant part of well discharge is withdrawn from the well storage during pumping;

in some hydro-geologic condition more water flows from the aquifer to the well after stoppage of pumping than during the pumping;

iii) when water is pumped from a large diameter well the energy consumption due to pumping is less than that in a tube well under similar condition.

Determination of aquifer parameters are prerequisite for planning and wise use of the groundwater resources in an area. Aquifer parameters are needed (i) to assess the available groundwater resources, (ii) to know the aquifer response to pumping, (iii) to judge the performance of a well, and (iv) to study the interference of wells.

Some times an aquifer test is conducted in an existing large-diameter well. Evaluation of aquifer parameters from the test data requires understanding of the flow towards a large-diameter well. The conventional method of analysis of test data based on Theis(1935) equation is not applicable for large diameter wells due to the significant contribution from the well storage during pumping.

If well of small diameter is pumped with a very small rate of pumping, a significant part of the water pumped is taken from well-bore storage. In this case the response of the well to pumping resembles to that of a finite diameter well and the Theis method of analysis, which is valid for a well with infinitesimal diameter, is not applicable (Booth,1988).

The transient flow towards a large diameter well has been analysed by many investigators. Foremost among them are, Papadopulos and cooper(1967), who have presented the solution for a fully penetrating large diameter well in a confined aquifer.

Ruston and Holt(1981) have analysed by numerical method the flow towards a large diameter well considering the seepage face. Modelling of the flow to a large diameter well has been done by Patel and Mishra(1983), and Mishra and Chachadi(1984,1985) using discrete kernel approach. Rajgopalan and Jose(1986) have presented a numerical method to analyse flow to a large-diameter well in an unconfined aquifer. Many investigators have presented the analysis of flow to a fully penetrating well of finite radius considering well storage (Boulton and Streltsova,1976; Sakthivadivel and Ruston, 1989)

From economy and construction point of view a large-diameter should be dug to a depth to provide the requisite yield. Only few attempts have been made in modelling the flow to a partially penetrating large diameter well. The present study is an effort in this direction. The modelling of transient flow to a partially penetrating large-diameter well has been carried out using numerical modelling. A three dimensional groundwater flow model developed by Mcdonald and Harbuogh(1984) has been adapted. Continuum approach has been used in modelling the hard rock aquifer. The well storage and the seepage face at the well (difference between well water level and the phreatic level at the well face) and the partial penetration of the well have been considered in the modelling. The results have been presented in the form of type curves and tables for both pumping and recovery periods.

2.0 REVIEW OF LITERATURE

In a transient groundwater flow towards a large-diameter well, the abstraction from aquifer storage during pumping is time-variant; it commences from zero and attains a maximum equal to the pumping rate after some time during continuous pumping. When pumping is stopped, the recovery of well storage starts. During reçovery, the aquifer contribution to well storage decreases with time. As water continues to flow into the well, the seepage face diminishes with time .

Analytical and numerical solutions to the problem of transient flow towards a large diameter well have been given by many investigators. In this chapter, a critical review of literature pertaining to the analysis of flow to a large-diameter well has been made. Various techniques for the analysis of pump test data on large diameter well have also been reviewed.

Using Laplace transform technique, Papadopulos and Cooper (1967) have obtained analytical solution to the transient flow to a well with storage and fully penetrating a confined aquifer. The solution determines drawdown both in the well and in the aquifer for a constant rate of abstraction. The well loss arising from the entrance resistance of the well screen has been assumed negligible. The equation for drawdown has been given as:

$$s_w = Q_p/(4\pi T)\ F(u_w,\alpha)$$

$$u_w = r_w^2\ \phi\ /(4Tt), \quad \text{and} \quad \alpha = \phi\ (r_w/r_c)^2 \qquad \ldots(2.1)$$

where,

s_w = drawdown in the well,

r_w = effective radius of the well screen,

r_c = radius of the well casing,

t = time since the start of the pumping,

ϕ = storage coefficient of aquifer,

T = transmissivity of the aquifer, and

Q_p = constant rate of pumping from the well.

They have given a family of type-curves for different values of α between well function $[s_w/(Q_p/4\pi T)]$ and $1/u_w$ on a double log paper. In order to determine the aquifer parameters, the drawdown versus time plot of pump test data is required to be compared with the family of type-curves drawn on the scale which is the same as that of the drawdown-time plot. The proposed family of type-curves consist of a straight line portion, and for different values of α, these straight lines are parallel. These straight line portions of the type-curves corresponds to the period when the aquifer contribution to the well discharge is negligible and almost the entire discharge is taken from the well storage. The effect of well storage continues up to a time t given by $t=25r_c^2/T$. For a short duration pumping test in a large diameter-well, when $t\leq25r_c^2/T$, the time drawdown curve matches with the straight line portion of any of the type-curves which pertains to different values of α. Therefore, for a short duration pumping test ($t\leq25r_c^2/T$), the storage coefficient(ϕ) can not be determined uniquely, though, a unique value of transmissivity can be determined. In order to determine the unique value of ϕ, the well should be pumped beyond the time $25r_c^2/T$ which is quite long for a shallow aquifer with low transmissivity. Pumping from a large diameter well in hard rock aquifer for such a long time is not practicable (Herbert and Kitching,1981). Hence, the other alternative to determine value of ϕ is to go for the analysis or the recovery data .The analysis proposed by Papadopulos and Coope assumes the rate of abstraction to be constant. In actual practice, constant rate of abstraction can not be maintained especially when centrifugal pumps are used.

Lai and Su(1974) have given an equation for the drawdown in and around a large diameter well in a leaky confined aquifer due to an arbitrary time-variant rate of pumping using Laplace transformation technique. They have observed that the effect of well storage on drawdown is more pronounced when the time of pumping is small or the aquifer diffusivity, i.e., ratio of the transmissivity and storage coefficient of the aquifer, is small. Their analysis assumes linearly and exponentially variable abstraction rates but often, it is not possible to represent satisfactorily the variation in the abstraction rate that actually occurs in practice. The method proposed by Lai and Su to determine the drawdown, requires numerical integration of improper integrals involving Bessel functions. Thus, the numerical integration involved therein requires large computations.

Zdankus (1974) has proposed a method for the analysis of pump-test data on dug wells in hard rock area the hydraulic conductivity of which varies linearly with depth. The hydraulic conductivity has been assumed maximum at the phreatic surface and zero at the bottom of the aquifer. He has developed approximate equations to determine the conditional radius of influence and the average hydraulic conductivity. These approximate equations make use of a drawdown function which depends on the drawdown in the aquifer at the well face and the saturated thickness of the aquifer. While calculating the conditional radius of influence from one of the approximate equation, the value of specific yield of the aquifer has to be assumed depending upon the type of rock at well site. The hydraulic conductivity and the conditional radius of influence can be determined by trial and error method using both the approximate equation proposed by Zdankus for each time interval. The accuracy of the estimated parameters is not high due to the use of approximate equations. The other problem is that the drawdown in the aquifer at the well face, is not feasible to be measured. However, this method is useful for the analysis of recovery data as the drawdown at the well face during recovery phase rapidly reduces to the drawdown in the well with the increase in time since the start of recovery and is almost equal to the drawdown in the well towards the later part of the recovery.

Slichter(1906) has suggested a formula based on specific capacity of well to judge the performance of recovery of large diameter wells. Sammel(1974) in his review of different methods of analysing pump-test data on large diameter well, has pointed out that performance of the recovery of large diameter wells can not be compared based on the values of specific capacity because specific capacity in Slitcher's formula depends only on the maximum value of drawdown in the well. Therefore, variation of specific capacity with time, can not be obtained by Slitcher's formula, however, this formula may be used to compare qualitatively the productivity of the wells provided the dimension of wells and discharge rates during pumping are identical.

Boulton and Streltsova(1976) have given a more realistic analytical model for the analysis of transient flow to a large diameter well in an unconfined aquifer taking into account the partial penetration of well and the anisotropy of the aquifer in respect of hydraulic conductivity and compressibility. Their method to determine the aquifer parameters is based on curve-matching. Since a large number of parameters are involved in the solution, it is not feasible to construct the whole sets of type-curves and hence, no complete set of type-curves is available. Therefore, the curve matching in this case becomes tedious because a large number of options are to be selected for curve matching. Unique value of the storage coefficient can not be obtained as the well function is non linear in ϕ.

Rao(1983) has critically analysed the well functions proposed by Boulton and Streltsova (1976) and presented a modified model incorporating relevant field conditions. The modified model allows a faster computation of the well function for specified values of parameters.

Fenske(1977) has presented mathematical derivations that consider the effect of storage in production well as well as in observation well and have extended the Theis equation to remove the requirement that the discharging well and the observation well have infinitesimal diameter.

Rushton and Holt (1981) have proposed a numerical approach for estimation of the aquifer parameters from abstraction and recovery data on a large diameter well tapping either a confined or an unconfined aquifer. They have considered the seepage face, variable abstraction rate and well loss in their digital model. Well storage is included by modifying the transmissivity and the storage coefficient of the portion of theaquifer occupied by the well. This is achieved by setting the transmissivity to a very high value to simulate the horizontal water level in the well, while the storage coefficient is set to unity, thereby representing the free water level in the well. If the casing radius is greater than the well radius, the storage coefficient is artificially increased to $1.0x(r_c^2/r_w^2)$. Their numerical model takes care of the presence of boundaries, variable transitivities within the aquifer and the variable discharge rate. While analytical method of Papadopulos and Cooper(1967) is not capable of coping with the above variations, numerical method can consider these variations. In the absence of the precise information about the relationship representing the decreasing abstraction rate, Ruston and Holt assumed that the abstraction rate decreases in proportion to the well drawdown. A satisfactory fit to observed data were obtained by introducing well losses in the abstraction phase by reducing the permeability for the mesh spacing adjacent to the well face. This reduction in the permeability represents the effect of seepage face and the partial penetration of the well. For ideal situations, the numerical method gave the same result as obtained from analytical solution (curve matching), for confined aquifer. But for unconfined aquifer where the decrease in saturated thickness is significant, the analytical method using curve matching technique is unreliable. In a confined aquifer, where the diameter of the dug portion of the well is much larger than the hole bored through the confined stratum, the aquifer response is found to be particularly sensitive to the ratio of these diameters. In an unconfined aquifer test, the important feature is the reduction in abstraction rate as the well is emptied. In both the cases the recovery data is critically important in analysing the pump-test data.

Herbert and Kitching(1981) have proposed approximate expressions for determining the transmissivity of an unconfined aquifer using recovery data. Since some of the wells require many days for the drawdown to recover fully, two expressions have been given , one for 50% recovery and other for 90% recovery of a large diameter partially penetrating well. Singh(1982) while using the expressions given by Herbert and Kitching to estimate the aquifer parameters did not found the reasonable estimate of the aquifer parameters. The estimated transmissivity was found in error by a factor of 2 which was either a multiplying or a dividing one.

Rajgopalan(1983) has developed a mathematical model for the analysis of recovery data in a large diameter well. It was assumed that the drawdown in large diameter well is linearly related to the partial derivative of hydraulic head with respect to the radius along the well face, and approximate equations for the drawdown during recovery phase have been derived. Unlike Slitcher's(1906) formula the expressions derived by Rajgopalan takes care of the effect of variable discharges on the rate of recovery and hence, provides a useful means of estimation of aquifer parameters from recovery data on large diameter wells.

Patel and Mishra(1983) have suggested a discrete kernel approach which is suitable for the analysis of pump-test data on large diameter well. In this method, a varying withdrawal rate from the aquifer can be represented as a series of step values. The aquifer withdrawal rate within a time step when added to the rate of release from the well storage equals the pumping rate from the well within that time step. The temporal variation of drawdown has been obtained at the well and at a point in the aquifer. These variations of drawdown compare well with the drawdown given by Papadopulos and Cooper (1967). The kernel function method proposed by Patel and Mishra is simple and involves inversion of a 2x2 matrix, while Papadopulos and Cooper method requires numerical integration of an improper integral involving Bessel's function, which involves large computations. The only draw back of this method is that the drawdown in the well has been assumed equal to the drawdown in the aquifer at the well face, i.e., no seepage face has been considered. This method can be used for both pumping and recovery phases.

Ruston and Singh(1987) have pointed out if the seepage face is ignored, both the transmissivity and storage coefficient are under estimated. Ruston and Singh have extended the kernel function technique proposed by Patel and Mishra to include the effect of seepage face which occurs when large diameter wells in unconfined aquifers are pumped.

Ruston and Singh(1983) have presented type-curves using numerical approach for both, constant and variable abstraction rates from large diameter wells. The variation of well discharge was assumed to vary linearly with drawdown which may result in errors because discharge variations in the field may not be according to this assumption. However, it has been stated that the estimation of storage coefficient by this approach is not reliable.

Chachadi and Mishra(1985) have extended the applicability of discrete kernel approach proposed by Patel and Mishra (1983) to the analysis of recovery data on a large diameter well in a confined aquifer of infinite areal extent pumped at a uniform rate up to a certain time. The drawdown in the well casing was assumed equal to the drawdown in the aquifer at the well face, i.e., seepage face has not been considered. Based on this approach, a set of type-curves have been provided using which a reliable estimate of aquifer parameters can be made.

Chachadi and Mishra(1986) have proposed expressions for the drawdown which occurs due to the transient flow to a large diameter well in a confined aquifer of unlimited areal extent for variable abstraction rates. A quadratic relationship between pumping rate and drawdown has been assumed. The drawdown computed for an average pumping rate when compared with those computed for variable pumping rate showed considerable difference. Hence, it was suggested that an average pumping rate can not substitute the variable abstraction rate.

Singh and Gupta(1986) have presented a method based on numerical modelling for estimation of aquifer parameters from pump test data on large diameter wells. The method takes care of the variable pumping rate and the recovery of well drawdown. The variation in the pumping rate has been taken care of by splitting the total time of pumping into a number of small time step which has uniform pumping rate but the pumping rate vary from one time step to another. In order to find the solution, principle of superposition has been utilized. It has been assumed that there is no loss due to entry resistance around the well, the seepage face is negligible and the aquifer response is instantaneous.

Rajgopalan and Josh(1986) have developed a digital simulation model for the solution of the transient radial flow to a large diameter dug well perforating the full saturated thickness of an unconfined aquifer. The numerical solution is based on the finite difference approach. The time variant discharge from the aquifer and the reduction in aquifer thickness due to water withdrawal during both the abstraction and recovery period have been suitably incorporated in the simulation model. It has been observed that even during recovery period (after the pump has been stopped) the hydraulic head in the aquifer continues to decline for a while before it starts rising. It has also been noticed that beyond a certain radial distance from the well, the hydraulic head continues to decline even during the recovery phase. This is because during the recovery period the aquifer continues to discharge appreciable amount of water which accumulate in the well diameter. The results indicate that the model response of hydraulic heads in the aquifer is mainly sensitive to variations in lateral permeability and specify yield alone. Hence, while matching the model response with field response in a parameter adjustment procedure, attention can be focussed on these two parameters.

Sakthivadivel and Ruston(1989) have presented a methodology based on numerical analysis of large diameter wells with a dynamic seepage face for estimating the aquifer parameters from pumping tests in large diameter wells fully penetrating an unconfined aquifer of infinite areal extent. From the analysis of the field data the following equation for the seepage face has been proposed.

$$s_w = F_w . s_a \quad ; \text{and} \quad F_w = (1 + aQ_a)\exp[b(t_t - t)/t_t] \qquad ...(2.2)$$

where,

s_w = drawdown in the well (L)

s_a = drawdown in the aquifer at the well face (L)

Q_a = aquifer contribution to well discharge (L^3T^{-1})

t_t = total duration of test (T)

t = since start of the test (T)

a,b= constants

Other equations are

$$Q_p = Q_w + Q_a \quad ;\text{and} \quad Q_w = A_w \frac{dw}{dt} \qquad ...(2.3)$$

where,

Q_p = rate of pumping

Q_w = well contribution to rate of pumping

A_w = the cross-sectional area of well

Using the above equations with assumed initial value of Q_a, e.g., $Q_a = 0.001Q_p$ and $s_w = 0.0$, the variation of s_w, s_a and Q_a with time can be obtained by repetitive trials. Since time t_t can be varied, it is not fully justified in the equation describing seepage face.

Sridharan et al.(1990) have proposed a numerical analysis of flow to a dug well in an unconfined aquifer taking into account well storage, elastic storage release, gravity drainage, anisotropy, partial penetration and seepage surface at the well face. The pumping rate was assumed constant. The degree of anisotropy is found to be most significant factor influencing the development of seepage face with a pronounced seepage face for relatively low vertical hydraulic conductivity. The degree of anisotropy and well penetration have found to affect flow characteristic significantly. The effect of change in specific yield and elastic storage coefficient are less significant than those of anisotropy or well penetration. The applicability of the method for estimation of the aquifer parameters has not been discussed, under various parameters involved.

3.0 PROBLEM DEFINITION

The purpose of the present study is to analyse the unsteady flow due to pumping of a large diameter well partially penetrating an unconfined aquifer by cell theory. The definition sketch of the problem is shown in figure 3.1. A large diameter well partially penetrates an unconfined aquifer infinite in areal extent. The aquifer is homogeneous and isotropic and has an impervious boundary at the bottom. The sides as well as the bottom of the well are open.

Initially, the phreatic surface is horizontal that coincides with the well water level. The well is pumped at a uniform rate. In order to study the behavior of aquifer during recovery, the pumping is stopped after certain hours of pumping. It is required to find the following considering the well storage and the seepage surface at the well face:

i) The aquifer response in abstraction phase as well as in recovery phase.

ii) The temporal variation of aquifer contribution to the well discharge in abstraction phase and variation of aquifer contribution to well storage with time during recovery phase,

iii) The effect of aquifer parameters, diameter of well and partial penetration on aquifer response during both pumping and recovery phases,

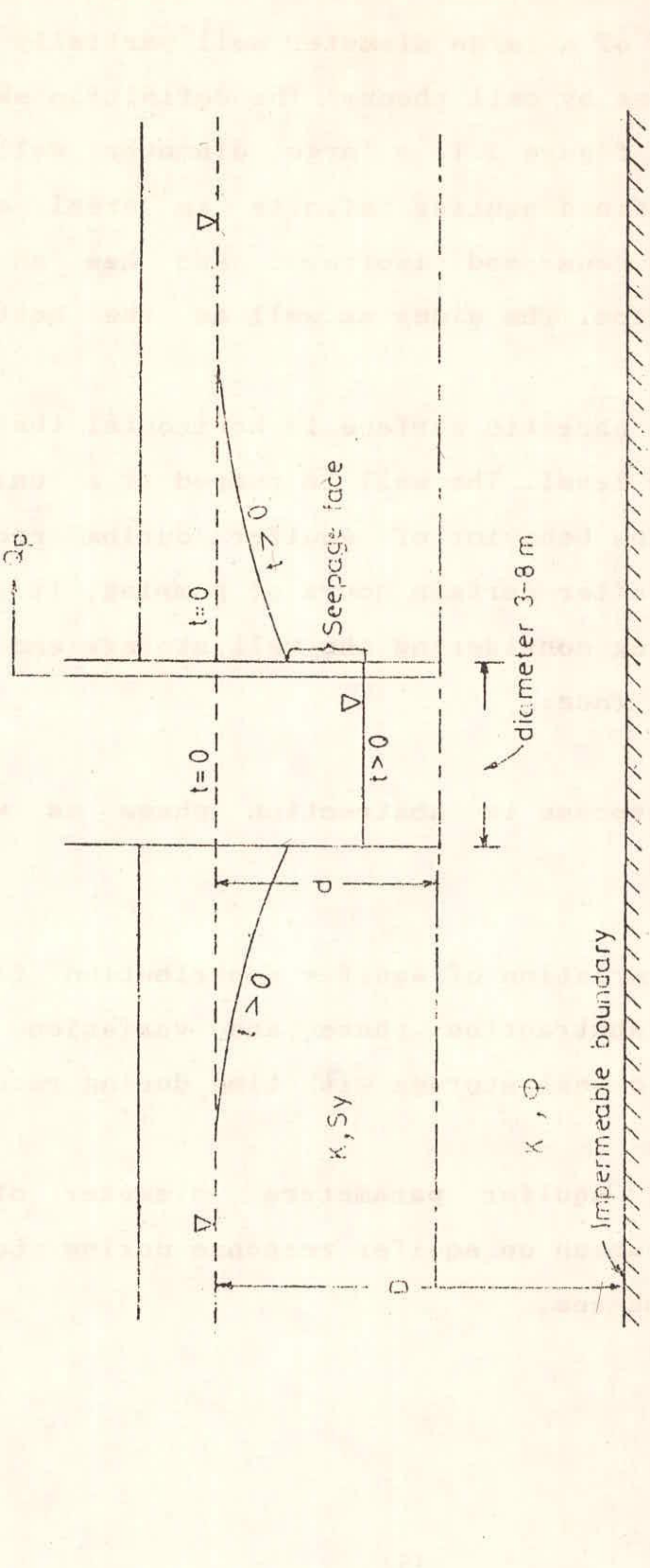


FIG.3.1 – DEFINITION SKETCH OF THE PROBLEM

The ranges of parameters involved in the present study are as given below:

(a) transmissivity :	24-120 m2/d
(b) storage coefficient :	0.1-0.01
(c) pumping rate :	24-120 m3/d
(d) diameter of well :	3-8 m
(e) recovery after :	2.25, 4.5, and 9.0 hours of pumping with uniform rate
(f) partial penetration :	0.1, 0.2, 0.4, 0.6, 0.8, 1.0
(g) thickness of aquifer :	50 m

4.0 METHODOLOGY

Modelling the flow to a large diameter partially penetrating well is a complex problem because of the following factors:

1. Storage in the well :

When a large diameter well having storage is pumped, a part of the well-discharge is taken from the well-storage and the remaining part of it is taken from the aquifer. At any time the sum of aquifer-contribution and the well-storage contribution is equal to the pumping rate. Initially, the aquifer-contribution to well discharge is insignificant and almost the entire water pumped is taken from the well-storage. The aquifer-contribution increases with time, it is zero at the start of pumping and attains a maximum value equal to the rate of pumping after a sufficiently long time. On the other hand, the well-storage contribution to well-discharge decreases with time being maximum, i.e., equal to the pumping rate at zero time and zero at a very large time. The storage in the well acts as a buffer. During recovery, the aquifer- discharge to well-storage recoup the well-storage.

2. Seepage face:

In the beginning of pumping the aquifer contribution is less than the pumping rate and part of well discharge is taken from the well storage. Depending upon the aquifer parameters, the well storage, and the pumping rate, the water level in the well will be below the phreatic level in an unconfined aquifer at the well face and thus, a seepage face is developed at the well face (Fig.3.1). The development of seepage face enhance the aquifer discharge to well storage. The seepage face changes with time. The seepage face developed during abstraction phase is responsible for a fast recovery of drawdown in the well after stoppage of pumping.

3. Partial penetration of the well:

Partial penetration of a well reduces the aquifer contribution. If the well is not sealed at the bottom, flow also occures from the bottom. Therefore, the flow problem is to be treated as a three dimensional one. A numerical method has been adopted and is discussed in the subsequent paragraphs.

4.1 CELL THEORY

4.1.1 Single Cell Model

Cell theory is an approach for regional modelling of aquifers. This approach is also termed as cell models. The simplest model is, perhaps, the one which visualizes an entire basin as a single cell. Average water table elevation and average conditions are assumed to describe the behaviour of this aquifer cell. The groundwater quantity balance for a cell of horizontal area A, bounded by impervious boundaries, for the period from t to $t+\Delta t$ can be written as:

$$\Delta t[A\{N+R-P\}-Q] = A\,\phi(\bar{h}_{t+\Delta t}-\bar{h}_t) \qquad \ldots(4.1)$$

where,

ϕ = storage coefficient of the aquifer (dimensionless),

N = natural replenishment (LT^{-1}),

R = artificial recharge (LT^{-1}),

P = rate of pumping (LT^{-1}),

Q = rate of spring discharge (L^3T^{-1}) ,and

$\bar{h}_t$ = average water table elevation in the cell at time t.

The cell model assumes that N, R, P and Q remain constant during Δt. If their average values vary over Δt, the variations have to be taken into account. Spring discharge Q, sometimes is taken to be a function of average head $\bar{h}$(water table elevation).

The unknown aquifer parameters (ϕ and N) are determined through calibration of the model. Sometimes $Q = Q(\bar{h})$ is to be determined. The calibration is based on the unknown values of $\bar{h}$, R and P for a number of periods in the past. The average value of water table elevation $\bar{h}$ over the cell are obtained from the contour maps at t and $t+\Delta t$. If the area A is large, it is convenient to subdivide it into areas (ΔA) for which the h_i are easily determined. Then $\bar{h}=\Sigma h_i (\Delta A)_i / A$. The period for which the balance is done, i.e., Δt, is usually chosen as 2 months, half year or a year. Knowing the value of aquifer parameters, Δt and $\bar{h}_t$, $\bar{h}_{t+\Delta t}$ can be found from equation 4.1.

When the aquifer area under consideration is not bounded by impervious boundaries, the inflow and outflow through cell boundaries have to be taken into account. Left hand side of equation 4.1 represents net inflow to the cell and right hand side represents net increase in groundwater storage that is reflected by change in water table elevation. Hence, the total volume of inflow into the cell through the boundaries minus that of outflow from it through boundaries during Δt is added to the left hand side of equation 4.1. In order to calculate net inflow to the cell through boundary, cell boundary is divided into a number of segments and inflow through each segment is calculated. Thus,

New inflow to the cell = $\Delta t \, \Sigma \, W_i \, \bar{T}_i \, \bar{I}_i$

where,

W = length of the segment,

$\bar{T}$ = average transmissivity over the segment,

$\bar{I}$ = average normal gradient over the segment, and

i = subscript used for i^{th} segment.

Depending upon the type of problem, wherever necessary, other terms expressing inflows or outflows may be added to the water balance of a single cell aquifer model; these includes

i) Evapotranspiration,

ii) Recharge through influent streams,

iii) Return flow from effluent streams,

iv) Drainage,

v) Leakage from a water supply system ,

vi) Return flow from irrigation

vii) Leakage into or out of the considered aquifer through overlaying and/or underlaying semipervious layers.

Single cell model is simple and very useful, especially in early stages of development and management when data are scarce and the planner is interested in an overall picture (average water levels) of the area, and in making decisions (e.g., on pumping) related to the area as a whole.

4.1.2 Multiple Cell Model:

The basic idea underlaying the multi-cell approach is the same as in a single cell model. In this approach, the aquifer is divided into a relatively small number of cells, usually of rectangular shape. The water-balance equation similar to equation 4.1 is written for each of the cells in the model. Fig. 4.1 shows a number of rectangular cells in a multicell aquifer model. Water balance equation for the cell i,j is

$$\Delta t \left[Q_x \Big|_{i-1/2,j} - Q_x \Big|_{i+1/2,j} + Q_y \Big|_{i,j-1/2} - Q_y \Big|_{i,j+1/2} + R_{i,j} - P_{i,j} + N_{i,j} \right] = \phi_{i,j} \, \Delta X_i \Delta y_i (h_{i,j}^{t+\Delta t} - h_{i,j}^{t}) \quad \ldots(4.2)$$

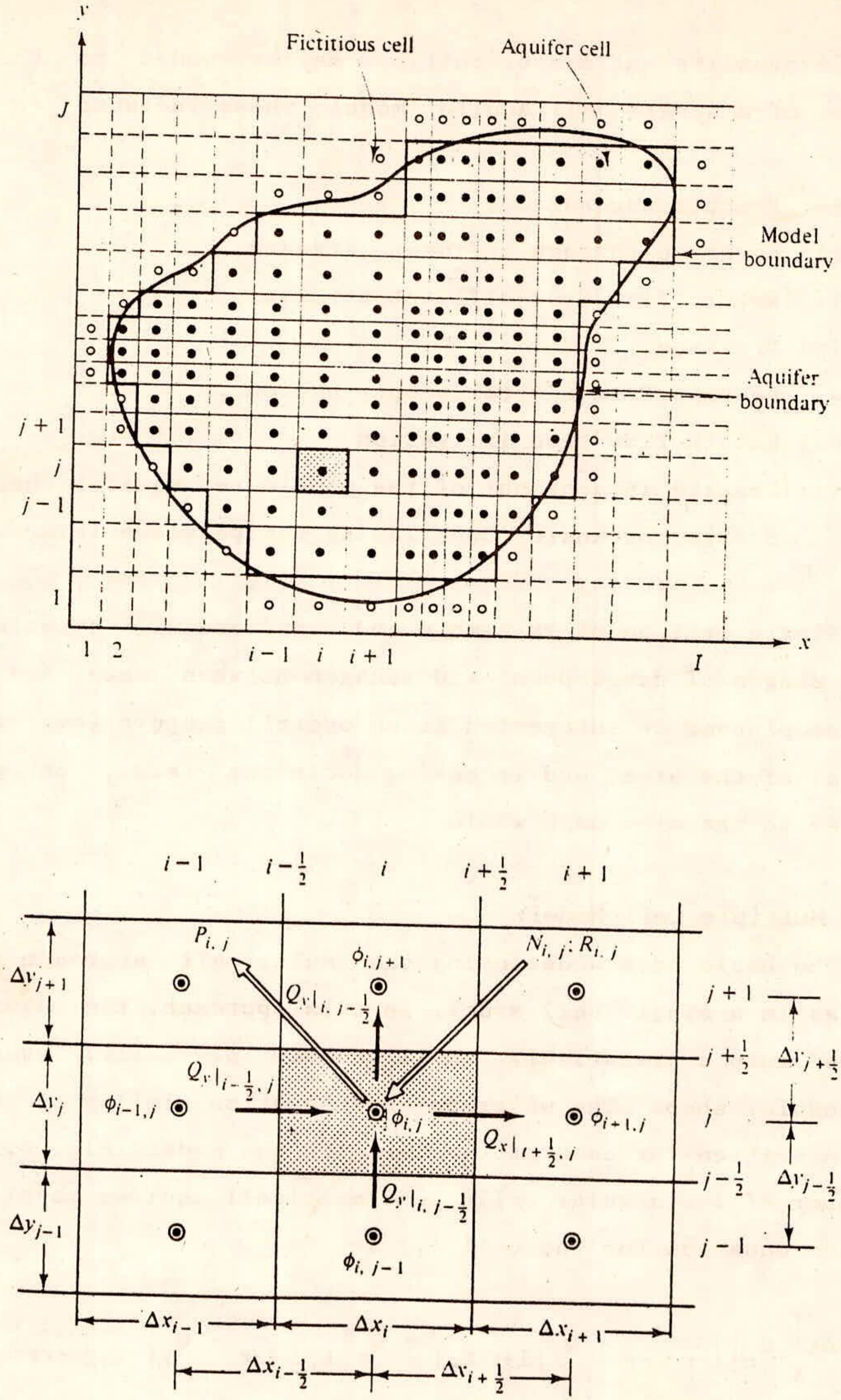


Fig. 4.1 Rectangular Cells in a Multicell Aquifer model.

where,

Q_x and Q_y =average rate of flow through the cell boundaries during time t to $t+\Delta t$ (L^3T^{-1}),

$(R_{i,j}+N_{i,j})$=total recharge rate (artificial and natural respectively) in the cell i,j during Δt (L^3T^{-1}),

$P_{i,j}$ =pumping rate in the cell i,j during Δt (L^3T^{-1}),

$\phi_{i,j}$ =average aquifer storativity in cell i,j, and

$h^t_{i,j}$ =piezometric head in cell i,j at time t (L).

In multicell approach the aquifer properties are generally assigned to each cell. For computing Q_x and Q_y the value of T on the boundary between the cell is required. These values may be calculated by either of the following equations.

i) Arithmetic Mean:

$$T_{i+1/2,j} = (T_{i+1,j} + T_{i,j})/2 \qquad ...(4.3)$$

ii) Harmonic Mean:

$$T_{i+1/2,j} = (\Delta x_i + \Delta x_{i+1})/(\Delta x_i/T_{i,j} + \Delta x_{i+1}/T_{i+1,j}) \qquad ...(4.4)$$

Harmonic mean is more accurate because it takes into account flow through two different transitivities. Similar expressions can be written for T on the other boundaries. Expressing Q_x and Q_y by Darcy's law, equation 4.2 is written as

$$T_{i-1/2,j}\,\Delta y_j\,\frac{h^t_{i-1,j} - h^t_{i,j}}{(\Delta x_i + \Delta x_{i-1})/2} + T_{i+1/2,j}\,\Delta y_j\,\frac{h^t_{i+1,j} - h^t_{i,j}}{(\Delta x_i + \Delta x_{i+1})/2}$$

$$+ T_{i,j-1/2}\,\Delta x_i\frac{h^t_{i,j-1} - h^t_{i,j}}{(\Delta y_j+\Delta y_{j-1})/2} + T_{i,j+1/2}\,\Delta x_i\frac{h^t_{i,j+1} - h^t_{i,j}}{(\Delta y_j+\Delta y_{j+1})/2} + N_{i,j}$$

$$+R_{i,j} - P_{i,j} = \phi_{i,j}\Delta x_i \Delta y_j\,\frac{h^{t+\Delta t}_{i,j} - h^t_{i,j}}{\Delta t} \qquad ...(4.5)$$

An equation of the above type is written for each cell. For a N-cell model N equations similar to equation 4.5 are written containing N unknowns (heads in N cells). These equations can be solved satisfying the conditions for the boundary cells.

A multi-cell model is useful particularly for making management decisions (say on rate of pumping and recharge) for certain specified subregions. In this approach the number of cells into which the aquifer is subdivided should be small in order to make clear distinction between this approach and that reflected in numerical methods such as finite difference method. Solution of the present problem is not feasible with this method taking only a few cells because this is a particular problem of analysis of flow towards a dugwell and is not a water balance problem of a region/sub-region for taking management decisions. Therefore, for the present problem, large number of cells of smaller dimensions should be required to achieve required accuracy. For this purpose three dimensional finite difference groundwater flow model developed by Mcdonald and Harbagh(1985) has been used, the description of which is as follows:

4.2 Model Description:

4.2.1 Mathematical Model:

The governing partial differential equation for the three dimensional unsteady (transient) movement of incompressible groundwater through heterogeneous and anisotropic medium may be described as

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right) - W = S_s\frac{\partial h}{\partial t} \quad \ldots(4.6)$$

where,

x,y,z = cartesian coordinates aligned along the major axes of conductivity K_{xx}, K_{yy}, K_{zz},

h = piezometric head (L);

W = volumetric flux per unit volume and represents source and/or sinks (T^{-1});

S_s = specific storage of the porous material of the aquifer (L^{-1}); and

t = time (T).

Here,

$$S_s = S_s(x,y,z),$$
$$K_{xx} = K_{xx}(x,y,z),$$
$$K_{yy} = K_{yy}(x,y,z),$$
$$K_{zz} = K_{zz}(x,y,z),$$
$$h = h(x,y,z,t), \text{ and}$$
$$W = W(x,y,z,t) \qquad ...(4.7)$$

The specific storage and the conductivities may the functions of space and time. Therefore, the flow under non-equilibrium conditions in a heterogeneous and anisotropic medium is described by equation 4.6. Equation 4.6 when combined with boundary conditions (flow and/or head conditions at the boundaries of the aquifer system) and initial condition (in case of transient flow, specification of head conditions at $t = 0$), constitute a mathematical model of transient groundwater flow.

The analytical solution of equation 4.6 is not feasible for complex systems, so various numerical methods must be employed to obtain approximate solutions. Finite difference approach is one of such numerical methods, wherein the continuous system described by equation 4.6 is replaced by a set of discrete points in space and time, and the partial derivatives are replaced by finite differences between the functional values at these points. Thus, the process leads to a systems of simultaneous linear algebraic difference equation and their solution yields values of head at specific points and time. These values are an approximation to the time varying head distribution that would be given by an analytical solution of the partial differential equation of flow.

4.2.2 Discretization convention

For the formulation of finite difference equations, the aquifer system need to be discretized into a mesh of points termed nodes, forming rows, columns, and layers. Such spatial discretization of an aquifer system is shown in fig.4.2. To conform with computer array convention, an i,j,k coordinate system is used. If an aquifer system consists of 'nrow' rows , 'ncol' columns and 'nlay' layers, then

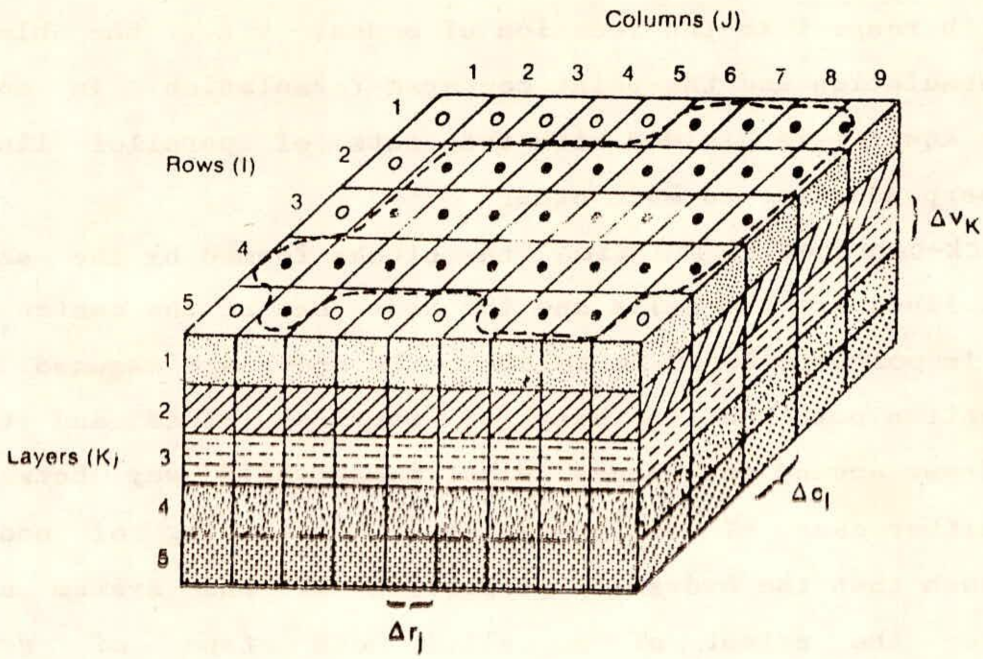
i is the row index, $i = 1,2,\ldots$ nrow;

j is the column index, $j = 1,2,\ldots$ncol;

k is the layer index, $k = 1,2,\ldots$nlay.

For example, fig.4.2 shows a system with nrow=5, ncol=9 and nlay=5. With respect to cartesian coordinate system, points along a row are parallel to x-axis, points along a column are parallel to the y-axis, and points along vertical are parallel to z-axis. In spatial discretization, nodes represents prisms of porous material termed cells in conceptual sense. Within each cell the hydraulic properties are constant so that any value associated with a node applies to or is distributed over the extent of a cell.

The width of cells along rows is designated as Δr_j for the j^{th} column; the width of cells along columns are designated as ΔC_i for i^{th} row; and the thickness of layers in vertical are designated as ΔV_k for the K^{th} layer (Fig.4.2). Thus the cell with the coordinates of $(i,j,k) = (5,3,2)$ has a volume of $\Delta r_3 . \Delta C_5 . \Delta V_2$.



Explanation

---- Aquifer Boundary

• Active Cell

0 Inactive Cell

Δr_J Dimension of Cell Along the Row Direction. Subscript (J) Indicates the Number of the Column

Δc_I Dimension of Cell Along the Column Direction. Subscript (I) Indicates the Number of the Row

Δv_K Dimension of the Cell Along the Vertical Direction. Subscript (K) Indicates the Number of the Layer

Fig. 4.2 A Discretized Hypothetical Aquifer System.

Configuration of cells:

There exists two conventions for defining the configuration of cells with respect to the location of nodes, viz., the block centered formulation and the point centered formulation. In both systems the aquifer is divided with two sets of parallel lines which are perpendicular to each other.

In block-centered formulation, the blocks formed by the sets of parallel lines are the cells and the nodes are at the centre of the cells. In point-centered formulation the nodes are assumed at the intersection points of the sets of parallel lines and the cells are drawn around the nodes with faces half way between nodes. In either case of configuration, the spacing of nodes should be such that the hydraulic properties of the system are uniform over the extent of a cell. Both type of grid configurations have been shown in fig.4.3.

4.2.3 Finite Difference Equation

The following development of finite difference equation holds good for both type of grid configuration described earlier. The groundwater flow equation may be written in finite difference form applying continuity equation. Thus, the sum of all flows into and out of cell must be equal to the rate of change of storage within the cell. The continuity equation for mass balance for a cell can be written as

$$\Sigma Q_i = S_s \frac{\Delta h}{\Delta t} . \Delta V \qquad \ldots(4.8)$$

where,

Q_i = flow rate into the cell ($L^3 t^{-1}$),

S_s = specific storage defined as the ratio of volume of water which can be injected per unit volume of aquifer material per unit change in head (L^{-1}),

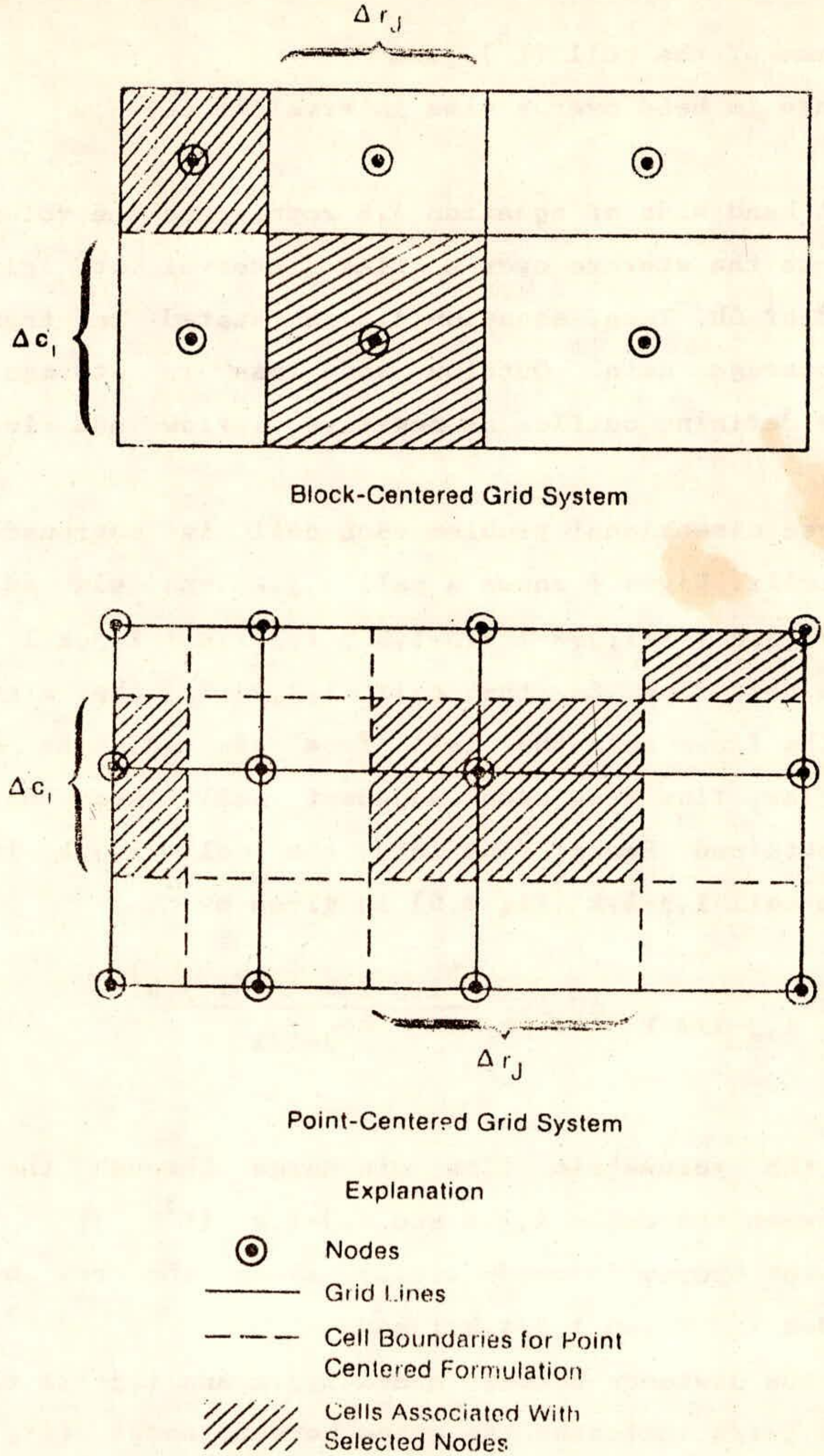


Fig. 4.3 Block-Centered and Point-Centered Grids.

ΔV = volume of the cell (L^3), and

Δh = change in head over a time interval Δt (L).

The right hand side of equation 4.8 represents the volume of water taken into the storage over a time interval Δt, given a change in head of Δh. Thus, equation 4.8 is stated in terms of inflow and storage gain. Outflow and loss in storage are represented by defining outflow as negative inflow and loss as negative gain.

For a three dimensional problem each cell is surrounded by six adjacent cells. Fig.4.4 shows a cell i,j,k and six adjacent cells, i.e.,i-1,j,k ; i+1,j,k ; i,j-1,k ; i,j+1,k ; i,j,k-1 ; and i,j,k+1 . Thus net flow to the cell i,j,k is the algebraic summation of the flows into the fell from six adjacent cells. Using Darcy's law, flow from each adjacent cell into the cell i,j,k can be obtained. Hence, flow into the cell i,j,k in row direction from cell i,j-1,k (fig.4.5) is given by

$$q_{i,j-1/2,k} = KR_{i,j-1/2,k} \; \Delta c_i \Delta v_k \frac{(h_{i,j-1,k} - h_{i,j,k})}{\Delta r_{j-1/2}} \qquad \ldots(4.9)$$

where,

$q_{i,j-1/2,k}$ is the volumetric flow discharge through the face between the cells i,j,k and i,j-1,k (L^3t^{-1});

$KR_{i,j-1,k}$ is the hydraulic conductivity along the row between nodes i,j,k and i,j-1,k ; and

$\Delta r_{j-1/2}$ is the distance between nodes i,j,k and i,j-1,k (L)

The index j-1/2 indicates the space between nodes (fig.4.5). It does not indicate a point exactly half way between nodes. For example, $KR_{i,j-1/2,k}$ represents average hydraulic conductivity in the entire region between nodes i,j,k and i,j-1,k.

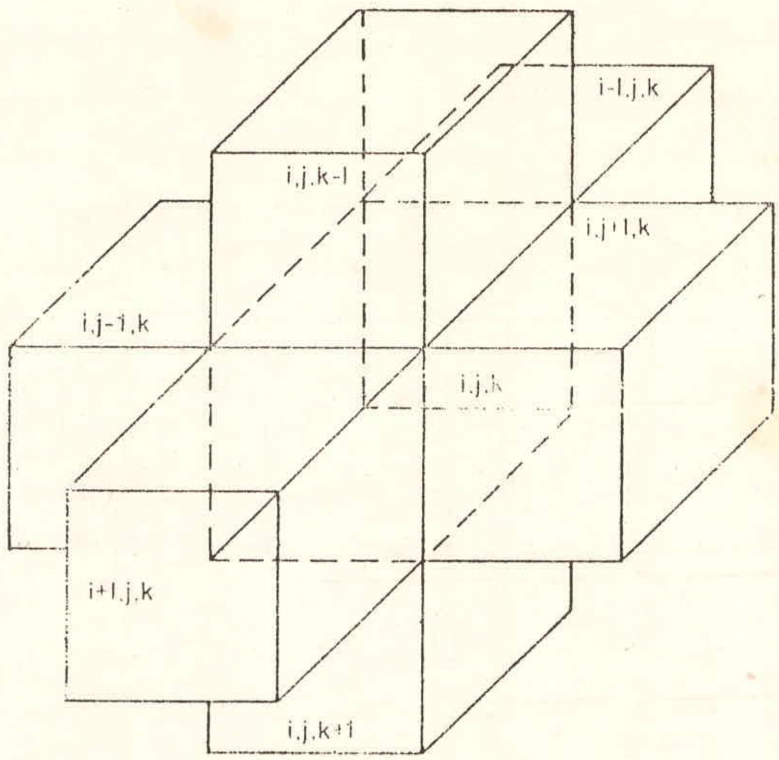


Fig. 4.4 Cell i,j,k and indices for the six Adjacent cells.

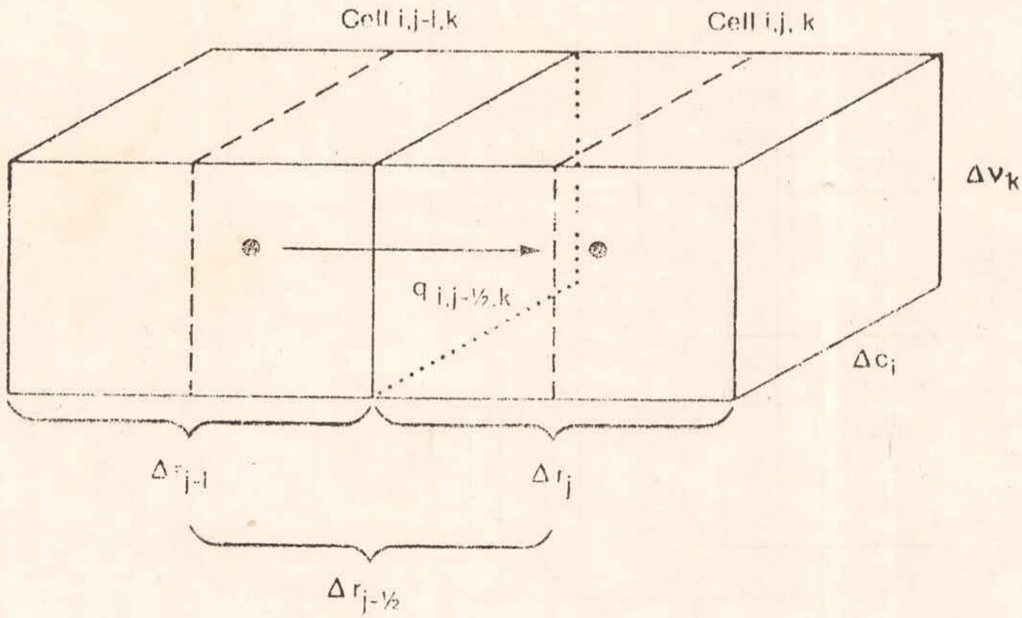


Fig. 4.5 Flow into Cell i,j,k from Cell i,j-1,k.

Since the grid dimensions and hydraulic conductivity remain constant throughout the solution process, the above equation may be rewritten by combining the constants into single constant called hydraulic conductance or simply 'conductance' of the cell.

$$q_{i,j-1/2,k} = CR_{i,j-1/2,k}(h_{i,j-1,k} - h_{i,j,k}) \qquad \ldots(4.10)$$

where, $CR_{i,j-1/2,k} = KR_{i,j-1/2,k}\Delta c_i \Delta v_k / \Delta r_{j-1/2}$

$CR_{i,j-1/2,k}$ is the conductance in i^{th} row and k^{th} layer between nodes i,j-1,k and i,j,k $[L^2 t^{-1}]$. Thus conductance is the product of hydraulic conductivity and cross-sectional area of flow divided by length of flow path; in this case, the distance between the nodes. Here, C represents the conductance and R represents in row direction.

Similar expressions can be written approximating the flows into or out of the cell i,j,k through the remaining five faces. Such expressions are as written below.

$$q_{i,j+1/2,k} = CR_{i,j+1/2,k} (h_{i,j+1,k} - h_{i,j,k}) \qquad \ldots(4.11)$$

$$q_{i-1/2,j,k} = CC_{i-1/2,j,k} (h_{i-1,j,k} - h_{i,j,k}) \qquad \ldots(4.12)$$

$$q_{i+1/2,j,k} = CC_{i+1/2,j,k} (h_{i+1,j,k} - h_{i,j,k}) \qquad \ldots(4.13)$$

$$q_{i,j,k-1/2} = CV_{i,j,k-1/2} (h_{i,j,k-1} - h_{i,j,k}) \qquad \ldots(4.14)$$

$$q_{i,j,k+1/2} = CV_{i,j,k+1/2} (h_{i,j,k+1} - h_{i,j,k}) \qquad \ldots(4.15)$$

Equations 4.10 -4.15 represent the flow into the cell i,j,k from six adjacent cells. Seepage from the stream beds, drains, areal recharge, evapotranspiration and flow from wells are taken care of by additional terms which accounts for flow into the cell from outside the aquifer. These flows may depend on the head in the receiving cell but are independent of the heads in all other cells of the aquifer or they may be entirely independent of head in receiving cell. Flow from outside the aquifer which is represented by W in equation 4.6, may be expressed in general as

$$a_{i,j,k,n} = p_{i,j,k}\, h_{i,j,k,n} + q_{i,j,k,n} \qquad \ldots(4.16)$$

where,

$a_{i,j,k,n}$ = flow from the n-th external source into cell i,j,k $[L^3T^{-1}]$

$p_{i,j,k,}$ = a constant $[L^2T^{-1}]$

$q_{i,j,k,n}$ = a constant $[L^3T^{-1}]$

For example, let cell i,j,k represents a well and $q_{i,j,k}$ represents discharge. $q_{i,j,k,1}$ is the discharge being pumped. In this case, the discharge from the well is assumed to be independent of head. Hence,

$$p_{i,j,k,1} = 0 \text{ ; and}$$

$$a_{i,j,k,1} = -q_{i,j,k,1} \qquad \ldots(4.17)$$

If the second external source (n=2) is taken to be seepage from river bed, it is proportional to the head difference between the river stage ($R_{i,j,k}$) and head in the receiving cell i,j,k ($h_{i,j,k}$). Hence,

$$a_{i,j,k,2} = CRIV_{i,j,k,2}\,(R_{i,j,k} - h_{i,j,k})$$

$$\text{or,}\quad a_{i,j,k,2} = -CRIV_{i,j,k,2}\, h_{i,j,k} + CRIV_{i,j,k,2}\, R_{i,j,k} \qquad \ldots(4.18)$$

where,

$CRIV_{i,j,k,2}$ is the conductance of the river bed in cell i,j,k $[L^2T^{-1}]$

The conductance $CRIV_{i,j,k,2}$ corresponds to $p_{i,j,k,2}$ and the term $CRIV_{i,j,k,2}\ R_{i,j,k}$ corresponds to $q_{i,j,k,2}$. Similarly, all other external sources or stresses can be represented by an expression of the form of equation 4.16 . If there are N external sources or stresses affecting a single cell, the combined flow is expressed by

$$QS_{i,j,k} = \sum_{n=1}^{N} a_{i,j,k,n}$$

$$= \sum_{n=1}^{N} p_{i,j,k,n}\ h_{i,j,k} + \sum_{n=1}^{N} q_{i,j,k,n}$$

$$= P_{i,j,k}\ h_{i,j,k} + Q_{i,j,k} \qquad \ldots(4.19)$$

where, $P_{i,j,k} = \sum_{n=1}^{N} p_{i,j,k,n}$

and $Q_{i,j,k} = \sum_{n=1}^{N} q_{i,j,k,n}$

While writing the continuity equation of the form given by equation 4.8, for cell i,j,k, the term ΣQ_i consists of flow to the cell from six adjacent cells, and all other external flow rate to the cell. The flow from six adjacent cells into cell i,j,k is given by equations 4.10-4.15 and the flow from the external sources into cell i,j,k is represented by equation 4.19 . Substitution of these equations in equation 4.8 yields

$$CR_{i,j-1/2,k}(h_{i,j-1,k}-h_{i,j,k}) + CR_{i,j+1/2,k}(h_{i,j+1,k}-h_{i,j,k}) +$$

$$CC_{i-1/2,j,k}(h_{i-1,j,k}-h_{i,j,k}) + CC_{i+1/2,j,k}(h_{i+1,j,k}-h_{i,j,k}) +$$

$$CV_{i,j,k-1/2}(h_{i,j,k-1}-h_{i,j,k}) + CV_{i,j,k+1/2}(h_{i,j,k+1}-h_{i,j,k}) +$$

$$P_{i,j,k}\, h_{i,j,k} + Q_{i,j,k}$$

$$= SS_{i,j,k}\, (\Delta r_j \Delta C_i \Delta V_k)(\Delta h_{i,j,k} / \Delta t) \qquad \ldots(4.20)$$

where,

$\Delta h_{i,j,k}/\Delta t$ = a finite difference approximation for head change with respect to time $[LT^{-1}]$,

$SS_{i,j,k}$ = specific storage of cell i,j,k $[L^{-1}]$, and

$\Delta r_j \Delta C_i \Delta V_k$ = volume of cell i,j,k $[L^3]$.

The above equation can be written in backward difference form by specifying flow term at t_m, the end of the time interval, and approximating the time derivative of head over the interval t_{m-1} to t_m, i.e.,

$$CR_{i,j-1/2,k}\,(h^m_{i,j-1,k} - h^m_{i,j,k}) + CR_{i,j+1/2,k}(h^m_{i,j+1,k} - h_{i,j,k})$$

$$+CC_{i-1/2,j,k}(h^m_{i-1,j,k} - h^m_{i,j,k}) + CC_{i+1/2,j,k}(h^m_{i+1,j,k} - h^m_{i,j,k})$$

$$+CV_{i,j,k-1/2}(h^m_{i,j,k-1} - h^m_{i,j,k}) + CV_{i,j,k+1/2}(h^m_{i,j,k+1} - h^m_{i,j,k})$$

$$+P_{i,j,k}h^m_{i,j,k} + Q_{i,j,k} = SS_{i,j,k}(\Delta r_j \Delta C_i \Delta V_k)\frac{(h^m_{i,j,k} - h^{m-1}_{i,j,k})}{t_m - t_{m-1}}$$

$$\ldots(4.21)$$

An equation of the above type can be written for each of the 'n' cells in the system; and, since there is only one unknown head for each cell, 'n' equations with 'n' unknowns are available. Such a system of equations can be solved simultaneously.

4.2.4 Provision for Boundary Conditions and Initial Condition

The type of boundaries that may be imposed in the model include constant head, no-flow, constant flow, and head dependent flow. These various types of boundaries are represented by the difference cell types. Cells can be designated by three types viz, inactive cell; constant head cell; and, variable head cell. Variable head cells are those in which head vary with time. Therefore, an equation of the type of equation 4.21 is required for each variable head cell. Head remains constant with time in constant head cells and these cells do not require an equation, however, the adjacent variable head cells will contain non-zero conductance terms representing flow from the constant head cell. 'No flow cells' are those to which there is no flow from adjacent cells. Neither an equation is formulated for a no flow cell nor the equations for the adjacent cells contain a term representing flow from the no flow cell. The use of no-flow cells and constant head cells to simulate boundary conditions is given in fig. 4.6. Constant-flow and head-dependent flow boundaries can be represented by a combination of no-flow cells and external sources.

In most cases, the actual number of equations of the form of equation 4.21 will be less than the total number of model cells. This is because the number of equations is only equal to the number of 'variable head cells'. The objective of transient simulation is to predict the head distribution at successive times with the given initial head distribution and the boundary conditions. The initial head distribution consists of a value of $h^{1}_{i,j,k}$ at each point in the mesh at time t_1, the beginning of the first of the discrete time steps into which the time axis is divided in the finite difference process. The first step in the solution process is to calculate values of $h^{2}_{i,j,k}$,i.e., head at time t_2 which marks the end of the first time step. Therefore, in equation 4.21, the subscript m is taken as 2 and thus the subscript m-1, which appear in only one head term, is taken as 1. Once such equations are formed for each variable head cells, an

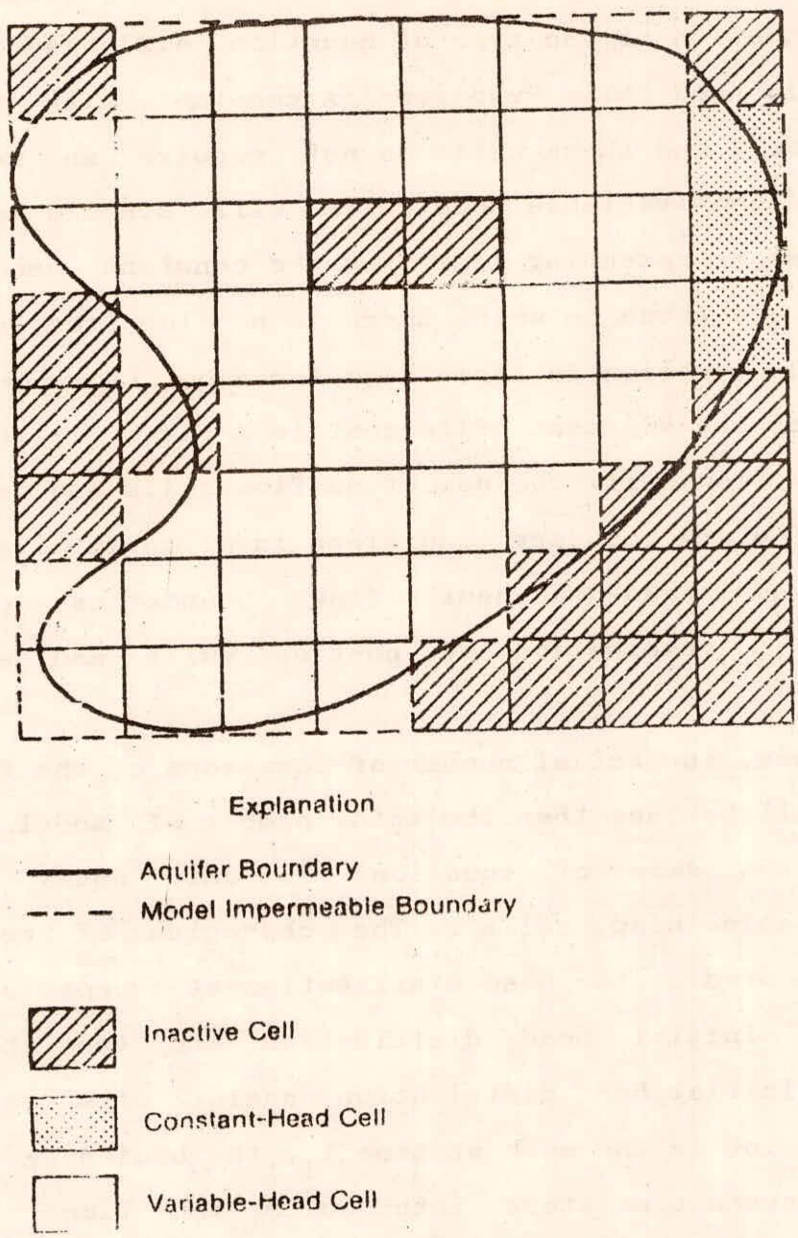


Fig. 4.6 Discritized Aquifer Showing Boundaries and Constant Head Cells.

iterative method is used to obtain the values of $h^2_{i,j,k}$. An iterative method starts with an initial trial solution. This trial solution is used to calculate through a procedure of calculation, an interim solution which more nearly satisfies the system of equations. The interim solution then becomes a new trial solution and the procedure is repeated. Each repetition is called an 'iteration'. The process is repeated until an iteration occurs in which the trial solution and the interim solution are nearly equal, i.e., for each node, the difference between the trial head value and the interim head value is smaller than some arbitrary established value, usually termed as 'closure criterion'. The interim solution is then regarded as a good approximation to the solution of system of equation under given initial and boundary conditions. For the solution of the present problem a strongly implicit programme has been used. The 'closure criterion' has been taken as 0.001 m

4.3 APPLICATION OF THE MODEL FOR THE PRESENT STUDY

4.3.1 Space Discretization

For the purpose of present study of the flow to a large diameter well, the well has been assumed to be located at the centre of an unconfined aquifer bounded by impermeable boundaries from all the four sides. The depth of aquifer has been taken as 50m. The bottom of the aquifer has been taken to be an impermeable boundary. The aquifer has been discretized in plan by rectangular grid arrays forming 29 rows and 29 columns and in vertical, in two layers. The well has been assumed to penetrate the top layer only with screen provided in full depth of top layer. The well has been considered open from the bottom. The aquifer has been considered homogeneous, i.e., hydraulic conductivity in row and column direction (K_h) and in vertical direction (K_v) have been taken to be the same for both the layers. The top layer has been taken to be unconfined and the bottom layer to be confined.

The discaretization in plan and in vertical are shown in fig. 4.7 and fig. 3.1 respectively. The central grid has been considered to be the well and away from this grid, the spacing of the grids have been taken to be the same for both positive and negative x and y directions. This spacing of grids away from the well (central grid)are 1,1,1,2,4,8,16,32,64,128,256,512,1024 and 1 m. The first and the last rows and columns have been assumed to be no flow boundaries, each having 1m width. The boundaries have been assumed at such a distance away from the well as not to affect the drawdown curve of well due to pumping.

4.3.2 Modelling the Flow to a Large Diameter Well:

In the foregoing discussions, we have seen that the well has been assumed to be of square cross-section (Central grid). But in pracatice, circular wells are commonly used. The model restriction is that only well with square cross-section can be modelled. If we want to model a circular well, transformed square cross-section of well with transformed aquifer properties will have to be taken into account such that the drawdown distributions due to pumping the transformed well is the same as would have been obtained by pumping the actual circular well using actual aquifer properties. This equality of flow to actual circular well and to transformed square well can be obtained only if;

(i) Flow from the bottom of both the circular and the transformed square well is the same,

(ii) Flow from the sides of both the circular and transformed well is the same.

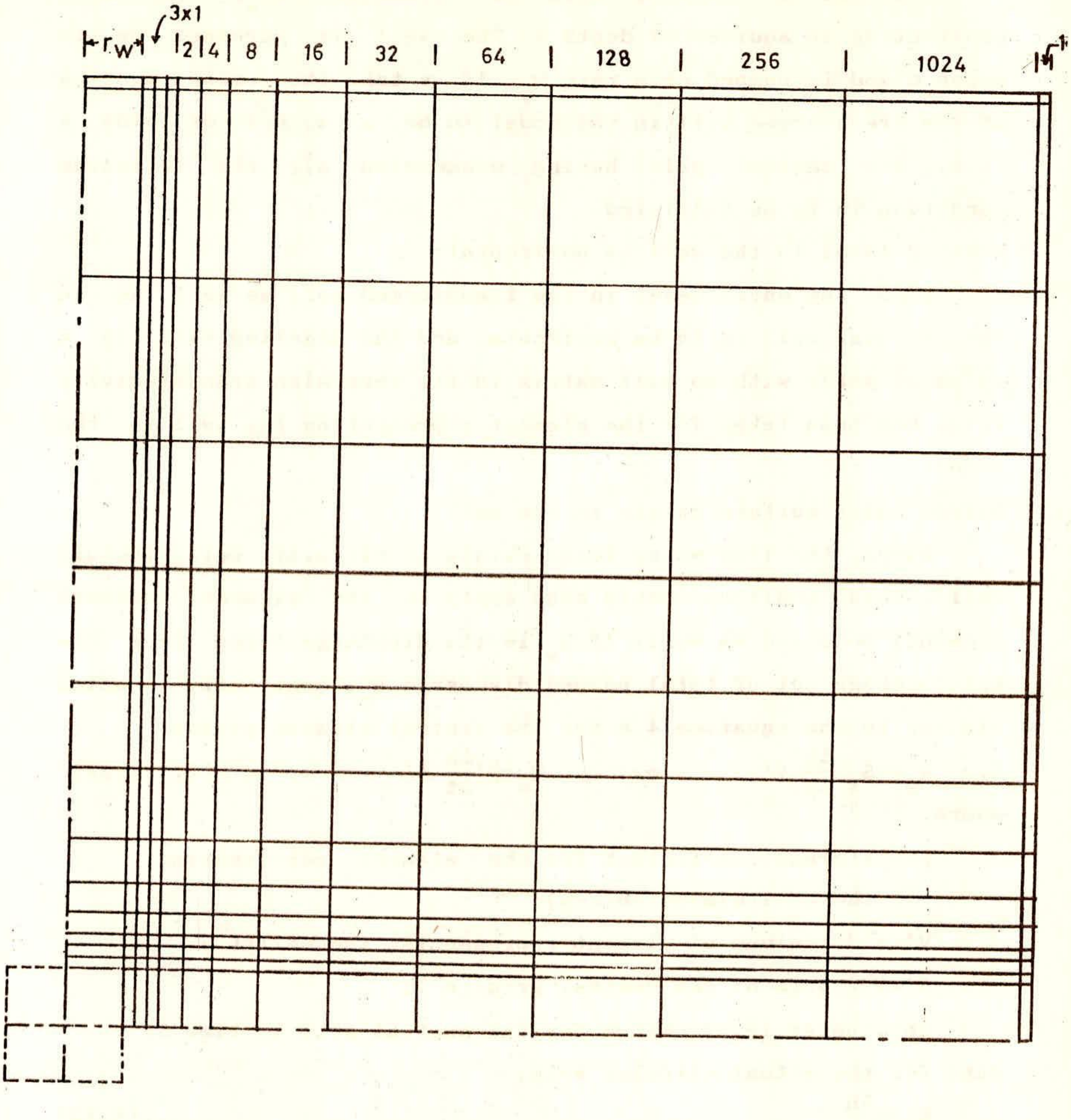


Fig. 4.7 Plan View of the Problem.

Consider a circular well of diameter $2r_w$, partially penetrating an aquifer of depth D. The well is screened up to depth d and is pumped at a rate Q_p. If we take the cross section of the transformed well in the model to be a square of side a (i.e., the central grid having diamension a), the following condition is to be fulfilled.

a) Water level in the well is horizontal:

Since the water level in the transformed well as well as in the circular well is to be horizontal and the modelled well is a prism of water with no soil matrix in it, very high transmissivity value has been taken for the element representing the well in the model.

b) Free water surface exists in the well:

Since, the free water level exists in circular well (actual well), this condition should also apply to the element (central element) modelled as well. If Q_w is the discharge taken from the well storage out of total pumped discharge Q_p, continuity equation similar to the equation 4.8 for the central element yields

$$Q_w = S_s \frac{\Delta h'}{\Delta t} V' \; ; \qquad \text{or,} \qquad Q_w = \phi' \frac{\Delta h'}{\Delta t} A' \qquad \text{....(4.22)}$$

where,

ϕ' = storage coefficient for the element representing the well (dimensionless);

$V' = a^2 d$ = volume of element representing the well (L^3);

$A' = a^2$ = area of the central grid (a^2);

$\Delta h'$ = water level change for the central grid in time Δt

But, for the actual circular well,

$$Q_w = A \frac{\Delta h}{\Delta t} \qquad \text{...(4.23)}$$

where,

$A = \pi r_w^2$ = cross-sectional area of the well (L^2);

Δh = change in water level of the well in time Δt (L).

Since, the flow through the bottom of the actual circular well and the modelled square well should be the same (i.e. A=A') and the value of the parameter $\Delta h/\Delta t$ should be the same for the actual and the modelled well, we have,

$$A=A' \quad ...(4.24)$$

$$\Delta h=\Delta h' \quad ...(4.25)$$

From equations 4.22 to 4.25, we can conclude that $\phi'=1.0$. Therefore, the central element representing the well has been assigned storage coefficient value equal to unity. From equation 4.24, we get,

$$\pi r_w^2 = a^2 \quad ;or, \quad a=\pi^{1/2} r_w \quad ...(4.26)$$

The above equation gives the equivalent size of square grid (central grid) for modelling a circular well of radius r_w.

In order to equate the flow from the sides of the two wells, the hydraulic gradient at the face of circular well and at the face modelled square well should be equal.

$$\left.\frac{\partial h}{\partial r}\right|_{r=r_w} = \left.\frac{\partial h}{\partial r}\right|_{r=a/2} \quad ...(4.27)$$

For the purpose of axial symmetry, let the modelled square well be taken to be a circular well of radius a/2, then equating the total flow from the sides of the wells, we get,

$$2\pi r_w T\left.\frac{\partial h}{\partial r}\right|_{r=r_w} = 2\pi (a/2)T'\left.\frac{\partial h}{\partial r}\right|_{r=a/2} \quad ...(4.28)$$

Combining equations 4.27 and 4.28, we get, $T=[a/(2r_w)]T'$. Substituting the value of a from equation 4.26 in this relaton,

$$T=0.5\pi^{1/2} = 0.886\ T' \quad ....(4.29)$$

Since , T=Kd and T'= K'd, where K is the actual hydraulic conductivity and K' is the hydraulic conductivity of the adjoining grids of modelled square well and d is the penetration of the well. Thus, the grids adjoining the central grids (well) have been assigned hydraulic conductivity value equal to 1.128 times the actual hydraulic conductivity value(keeping the same penetration as in actual for the transformed well).

4.3.3 Range of variables:

The values of different variables considered are given below.

T = 1, 2, 4, and 5 m^2/hr

ϕ = 0.1, 0.05, 0.01, 0.005, and 0.001

S_y = 0.25, 0.1, 0.05, 0.025, and 0.01

S_y/r_w = 10, 50, 100, 500, 1000

r_w = 3, 5, 7, and 9 m

Q_p = 2 and 4 m^3/h

5.0 ANALYSIS OF RESULTS:

The variation of the parameter $4\beta t/r_w^2$ ($\beta=T/S_y$) and $s_w/(Q_p/4\pi T)$ for fully penetrating large diameter well is shown in fig. 5.1. Here, r_w has been taken equal to r_c ,therefore, $\alpha=S_y$. The curves for different value of α are different and are exactly same as obtained by Papadopulos and Cooper(1967). It has been observed that the parameter ϕ/S_y (varied from 10 to 100) does not affect these curves. The type-curves for each value of α has a straight line portion in the beginning, then it deviates from straight line and ultimately merges with the Theis curve. The straight line portion of the curves for different value of α are parallel. The analysis of these curves shows that the time up to which straight line is observed is given by $t_s=0.05\ r_c^2/T$ and the time after which the curve merges with Theis curve, is given by $t_T=25r_c^2/T$.

When time of pumping $t\le t_s$, the aquifer contribution to well discharge is negligible. The value of ϕ can not be determined uniquely using the pump test data, however, the value of T can be determined. The relationship between the drawdown, $s_w(t)$, in the well and the pumping discharge, Q_p,is given as:

$$Q_p = \pi r_w^2 \frac{ds_w(t)}{dt} \qquad ...(5.1)$$

In the above equation $ds_w(t)/dt$ is the initial slope of the time-drawdown curve. Therefore, with the known value of r_w, the above equation gives the value of discharge value, Q_p. Sen, Zakai(1986) have found that this method for calculation of discharge from early drawdown data on large diameter-wells, gives practically reliable results(maximum error 10% and average error 3%).

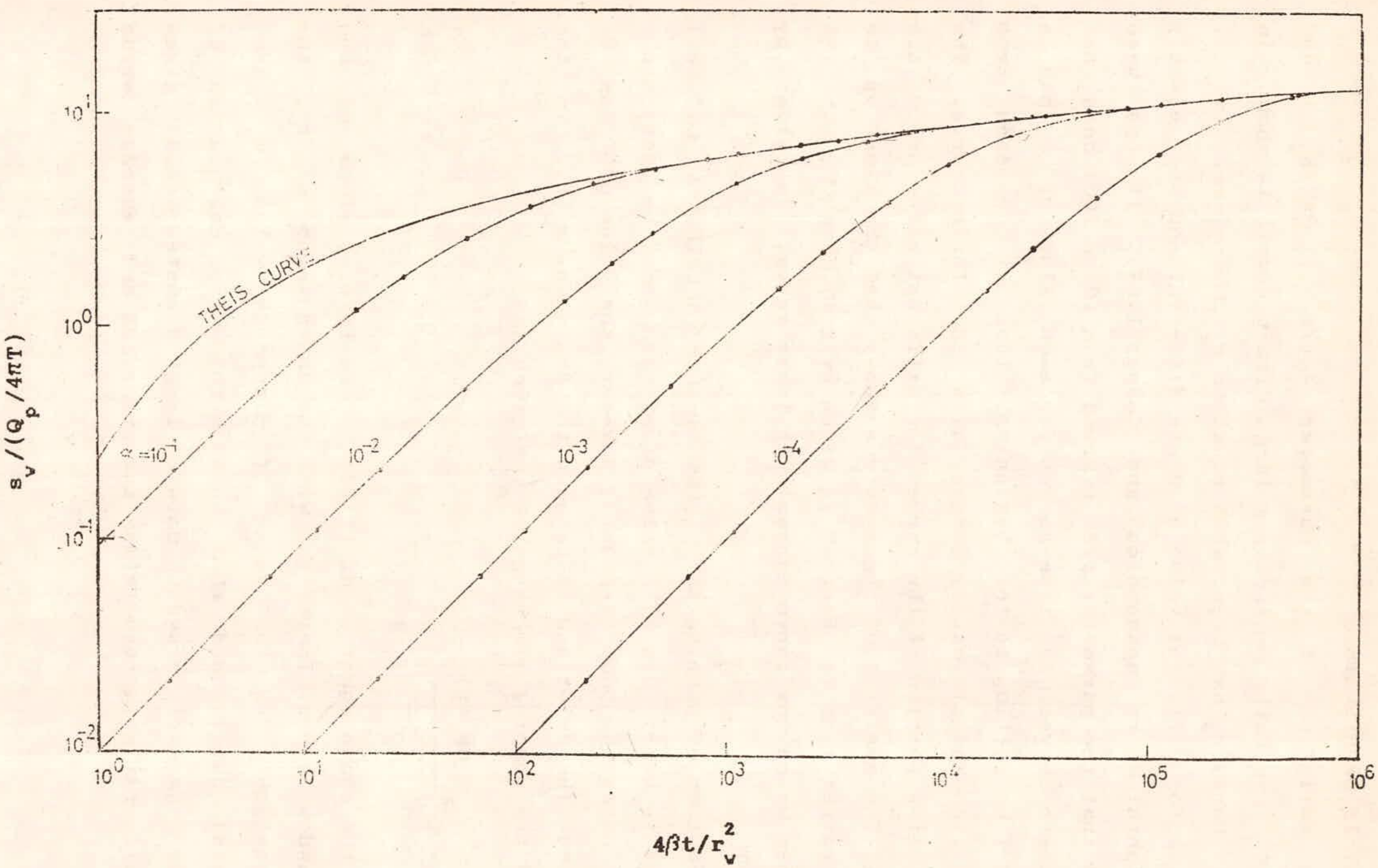


Fig. 5.1 Variation of $s_v/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α.

When $t_s < t < t_T$, the aquifer storage and the well storage both contribute to well discharge. The value of both T and ϕ can be determined uniquely using the pump test data.

When $t \geq t_T$, the contribution from well storage to well discharge is negligible. Theis type-curve can be used to determine the values of the aquifer parameters(T and ϕ).

Fig.5.2 through fig.5.8 show the variation of the parameter $4\beta t/r_w^2$ and $s_{w\eta}/(Q_p/4\pi T)$ with α for η (η=penetration ratio, i.e., ratio of the penetrated thickness and total thickness of the aquifer). The curves are different for different value of α for each value of η and it consists of a straight line portion which occurs for low value of $4\beta t/r_w^2$. These straight line portions for different value of α are parallel for each value of η. The curves deviates from straight line at larger value of $4\beta t/r_w^2$ and at very large value of $4\beta t/r_w^2$ the curves for different value of α are the same for each value of η. For a particular value of $4\beta t/r_w^2$, the value of the parameter $s_{w\eta}/(Q_p/4\pi T)$ decreases with increase in η for each value of α, when $t > t_s$. When $t \leq t_s$, the curves are independent of η for each value of α. These figures can be used as type-curves for estimation of aquifer parameters through curve matching, using pump test data on partially penetrating large diameter well. For short duration pumping, the drawdown and time plot on a log-log graph paper falls only on the straight line portion of the type curve, hence, the ϕ value can not be determined uniquely, however, unique value of T can be determined.

These figures show that the time up to which straight line is observed, i.e., $t_{s\eta}$ is different for different value of η and can be expressed by the following equation.

$$t_{s\eta}=K(\eta)\ r_w^2/T \qquad \ldots(5.2)$$

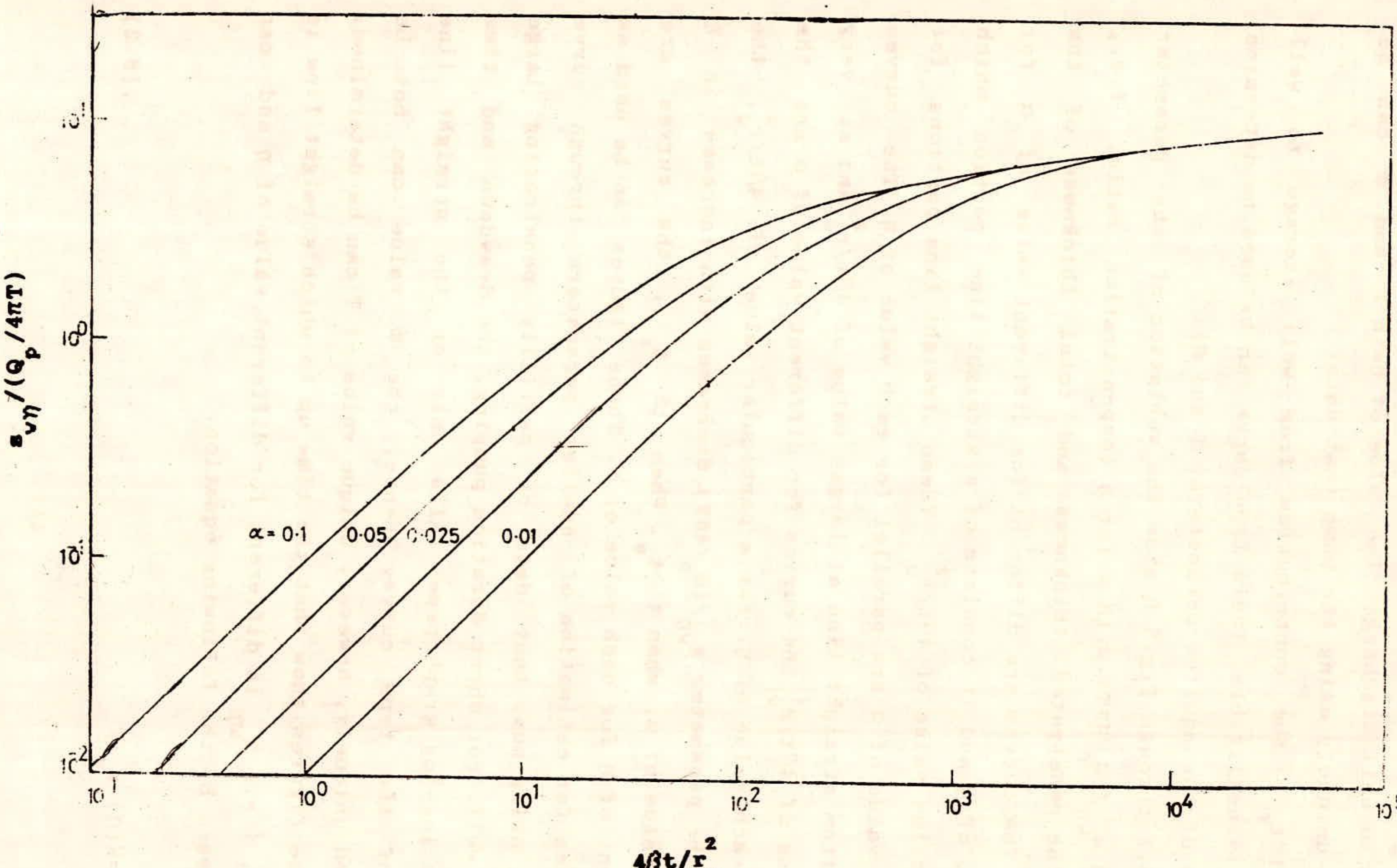


Fig. 5.2 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=1.0.

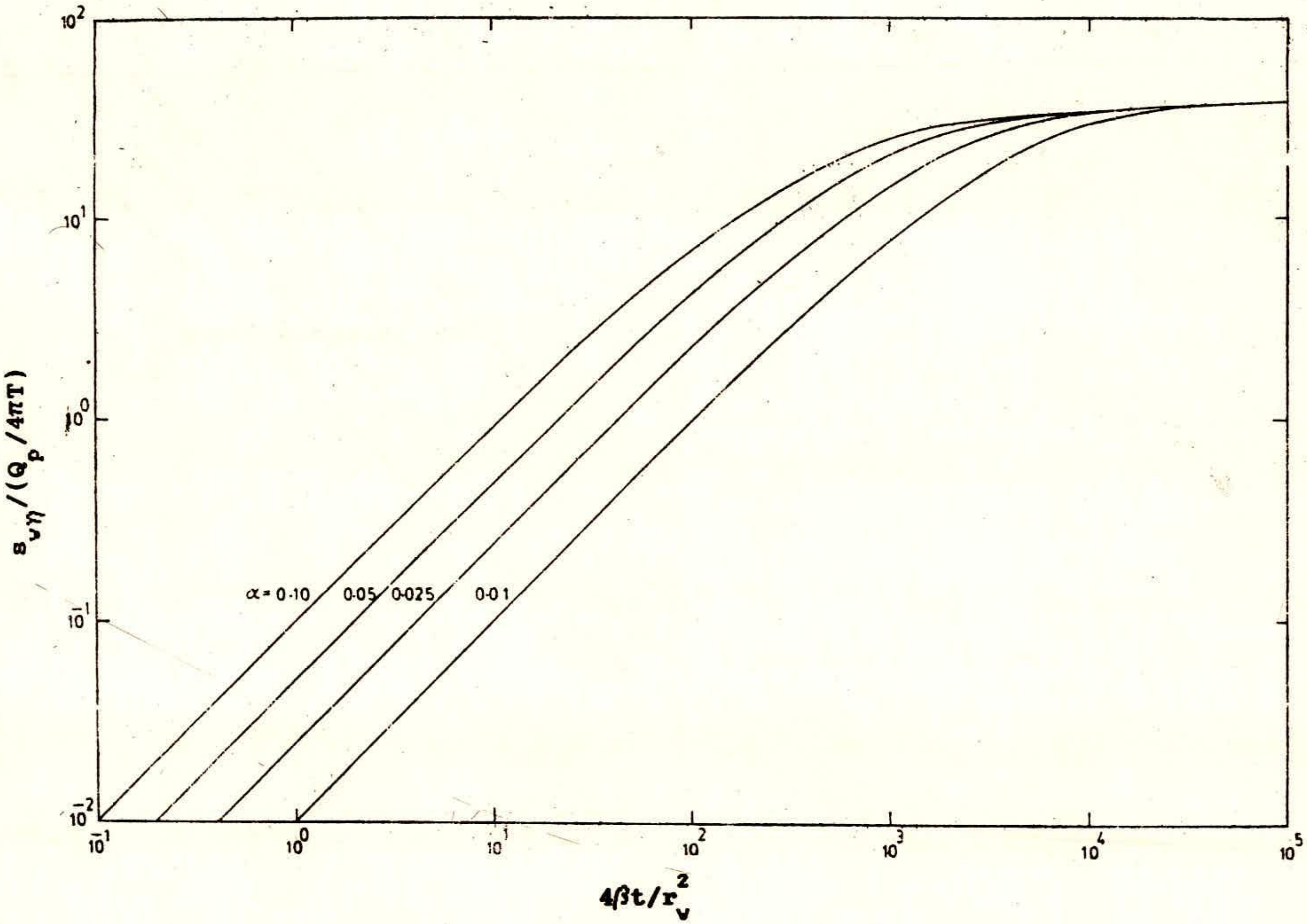


Fig. 5.3 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.1.

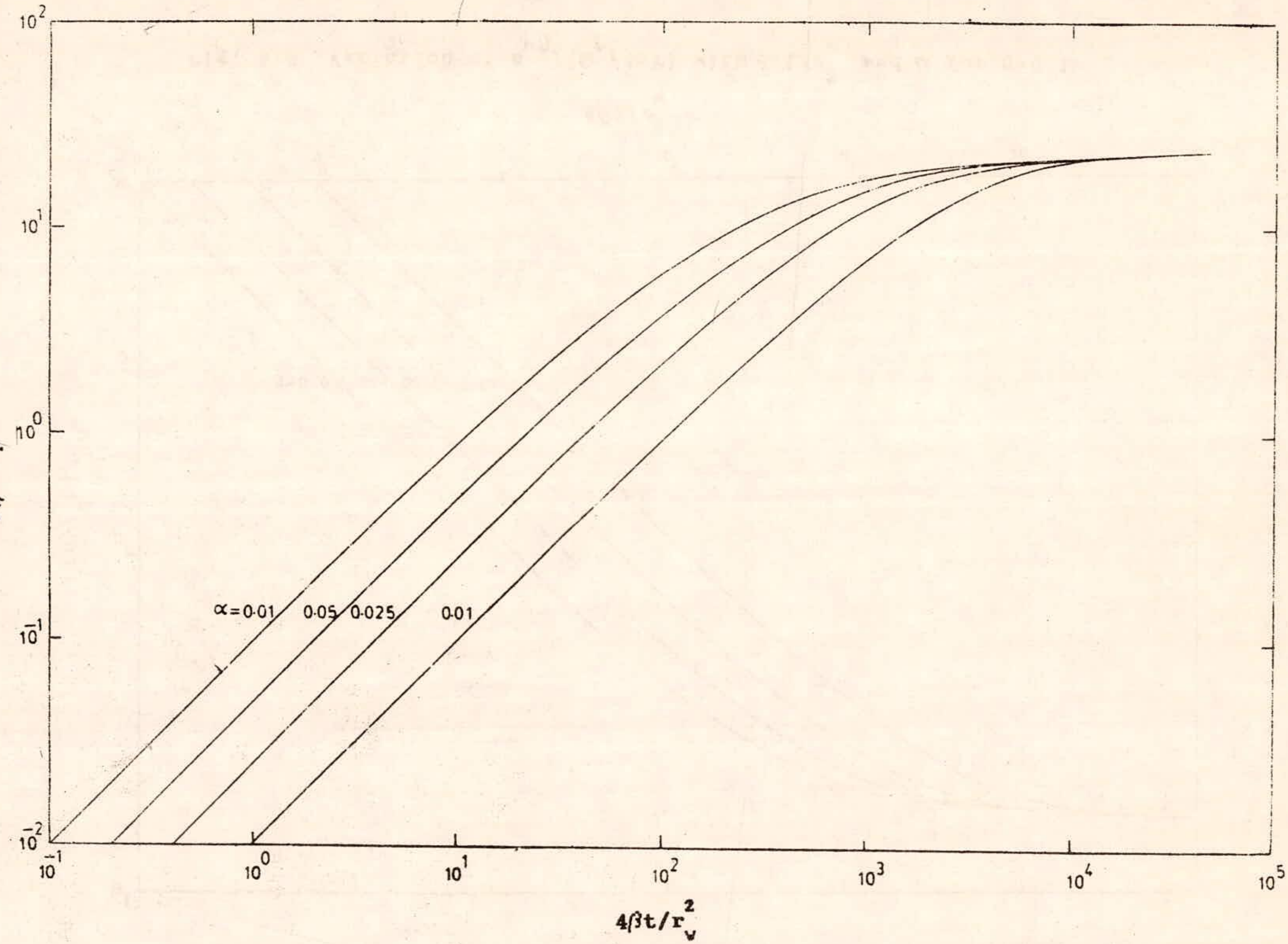


Fig. 5.4 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.2.

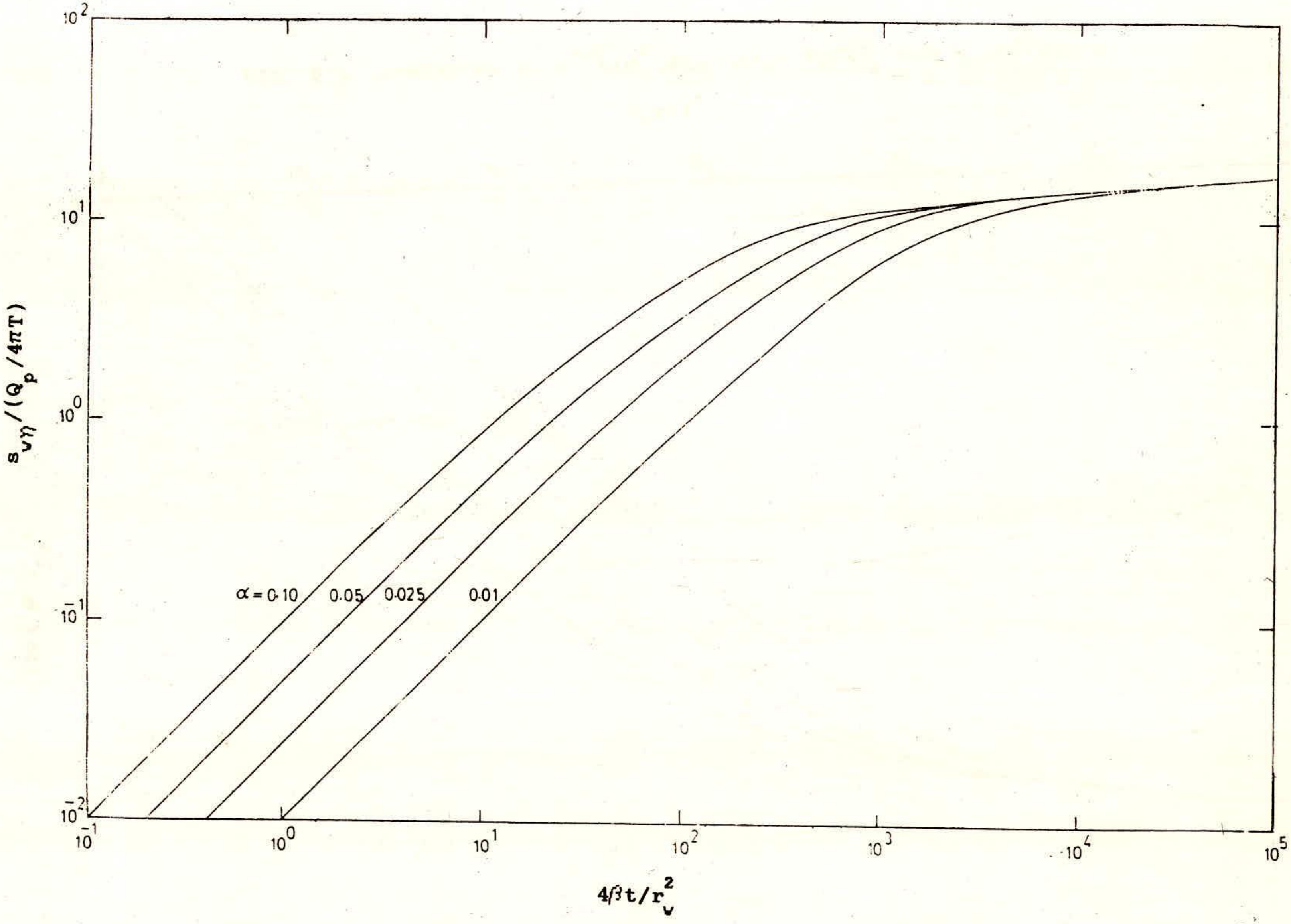


Fig. 5.5 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_w^2$ and α for $\eta=0.4$.

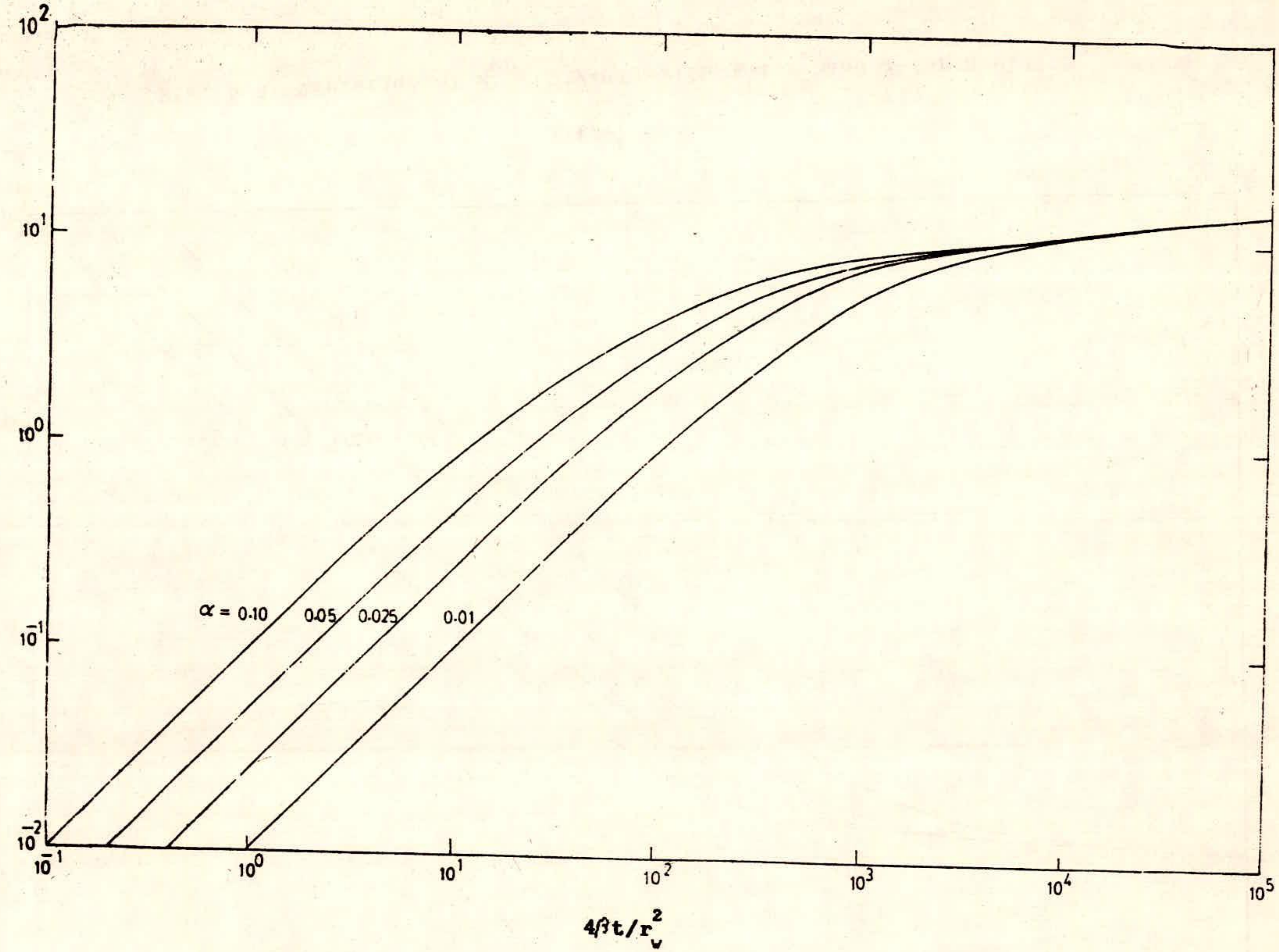


Fig. 5.6 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.6.

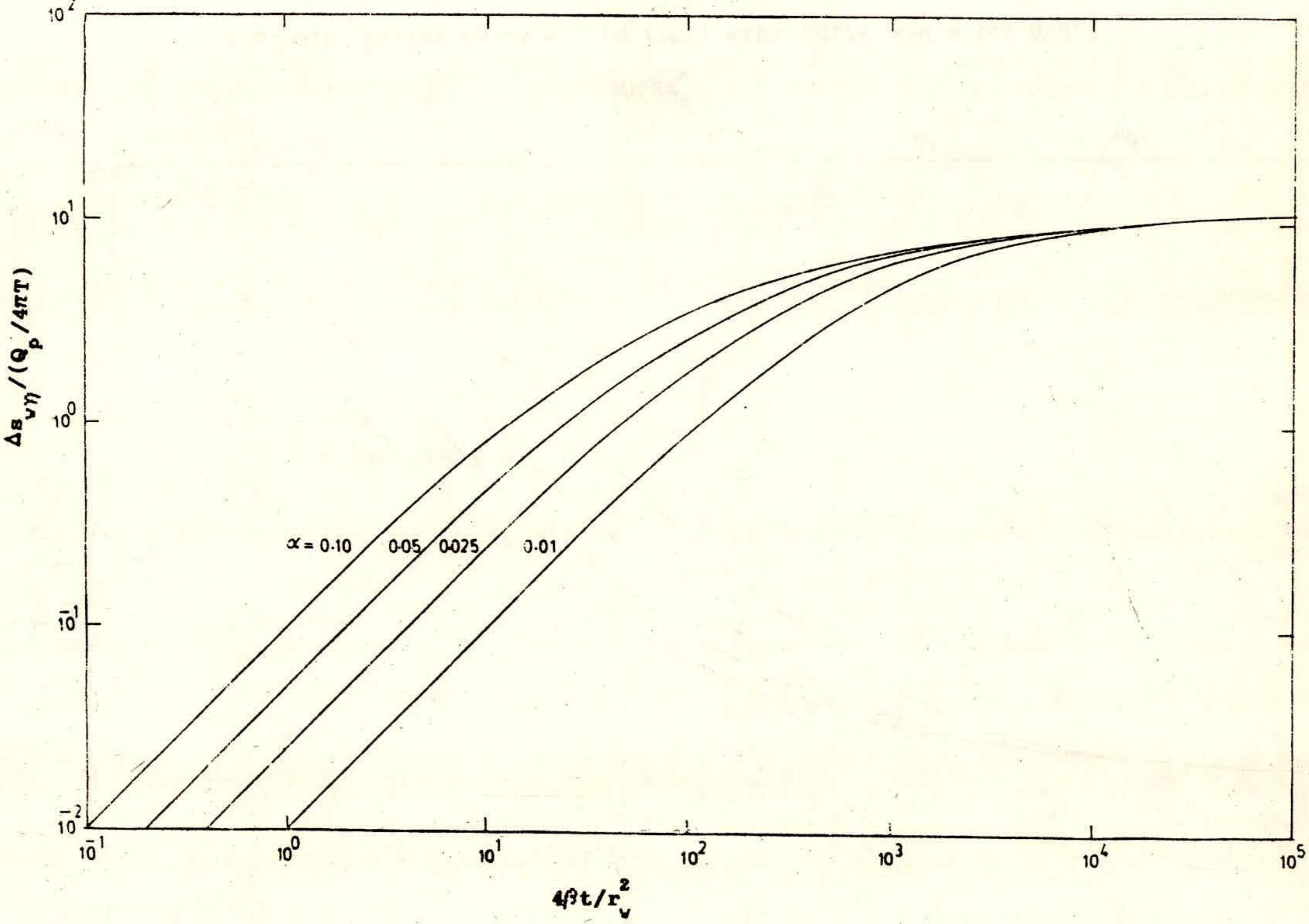


Fig. 5.7 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.8.

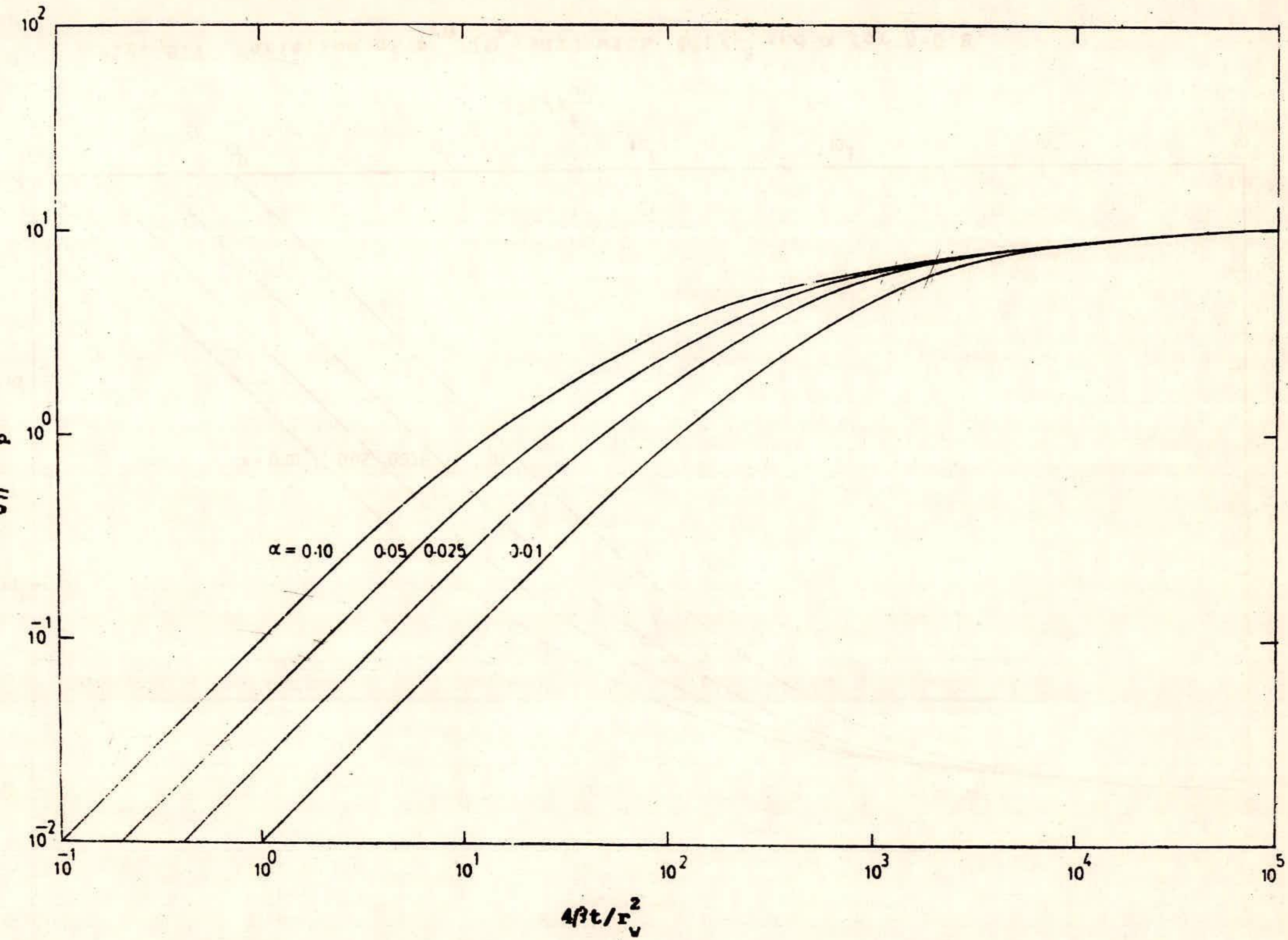


Fig. 5.8 Variation of $s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.9.

Where, $K(\eta)$ is constant which varies with η. The values of the parameter $4Tt/r_w^2$ up to which the straight line occurs and the values of K for different values of η are given in the table 5.1.

Table 5.1

VALUES OF PARAMETER $4Tt_{s\eta}/r_w^2$ AND K FOR DIFFERNENT VALUE OF η

Sl. No.	η	$4Tt_{s\eta}/r_w^2$	K
1.	0.1	5.0×10^{-1}	0.125
2.	0.2	4.2×10^{-1}	0.105
3.	0.4	3.2×10^{-1}	0.080
4.	0.6	2.6×10^{-1}	0.065
5.	0.8	2.2×10^{-1}	0.055
6.	0.9	2.1×10^{-1}	0.0525
7.	1.0	2.0×10^{-1}	0.0500

Variation of K with η is shown in fig. 5.9. In a partially penetrating large diameter well, when time of pumping is less than $t_{s\eta}$, the aquifer contribution to well discharge is negligible In this case the relationship between the drawdown in the well, $s_{w\eta}$ and the pumped discharge Q_p can be approximated by the following equation.

$$Q_p = \pi r_w^2 \frac{ds_{w\eta}(t)}{dt} \qquad ...(5.3)$$

In the above equation $ds_{w\eta}(t)/dt$ is the initial slope of the time-drawdown curve. Therefore, with the known value of r_w, the above equation can be used to determine the discharge value, Q_p, using early time-drawdown data on partially penetrating large diameter well.

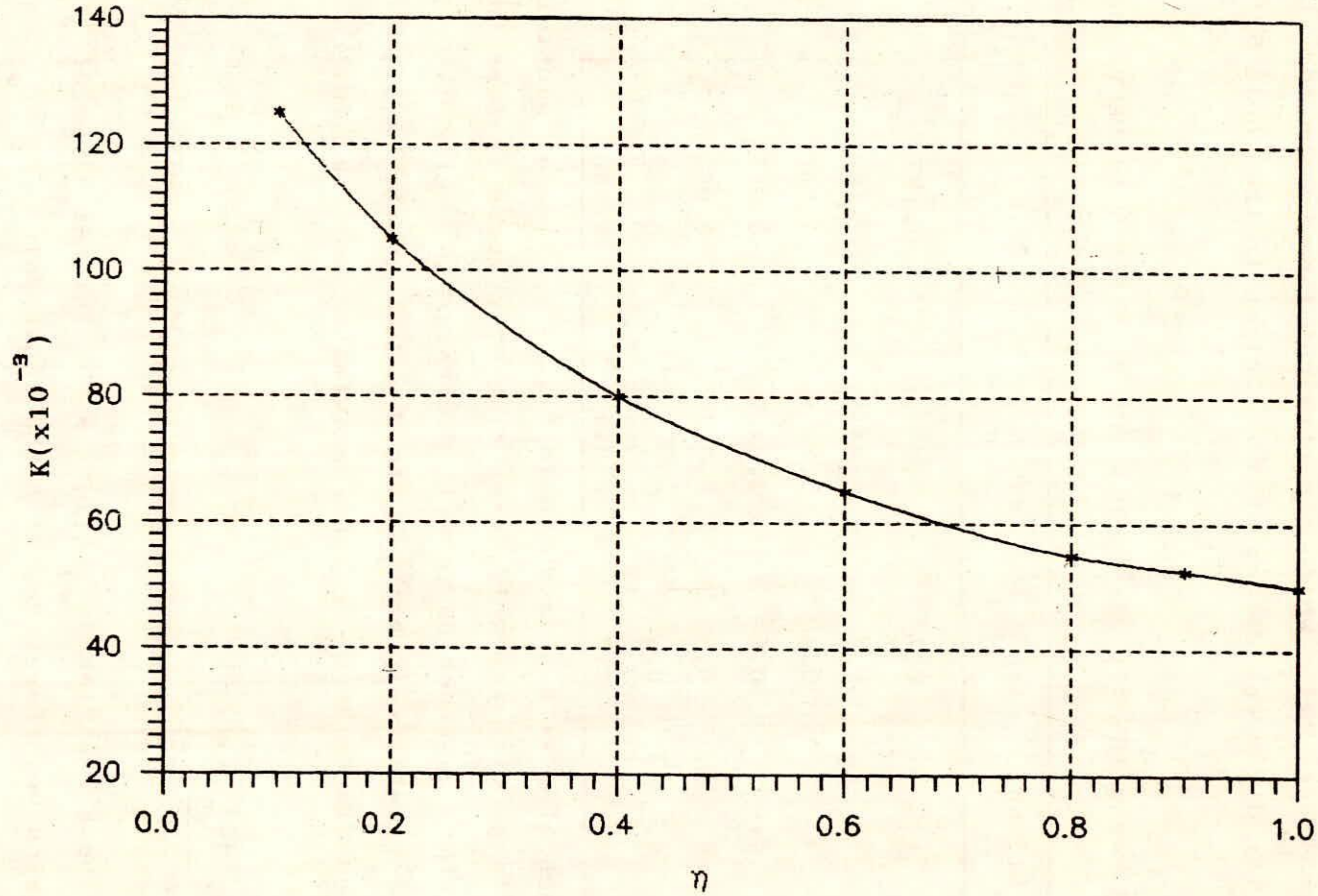


Fig. 5.9 Variation of K with η.

When time of pumping is more than $t_{s\eta}$, both the aquifer parameters T and ϕ can be determined through curve matching, using the type-curves for appropriate value of η from the figs. shown above.

Figs. 5.10 through 5.15 show the variation of $\Delta s_{w\eta}/(Q_p/4\pi T)$ with $4\beta t/r_w^2$ and α for different value of η. Here, $\Delta s_{w\eta}$ = well loss due to partial penetration = $s_w - s_{w\eta}$. These figures show that for each value of η, the curves are different for different value of α. When the time of pumping is less than t_s, $\Delta s_{w\eta}$ is negligible. For fixed values of η and $4\beta t/r_w^2$,the value of the parameter $\Delta s_{w\eta}/(Q_p/4\pi T)$ increases with increase in α and for fixed value of α and η, the value of the parameter $\Delta s_{w\eta}/(Q_p/4\pi T)$ increases with increase in the value of the parameter $4\beta t/r_w^2$. When $t>t_s$, for a fixed value of $4\beta t/r_w^2$, the value of the parameter $s_{w\eta}/(Q_p/4\pi T)$ decreases with increase in η for each value of α. At very large value of $4\beta t/r_w^2$ the curves for different value of α are the same for each value of η. These figures can be used as type-curves for estimation of well loss due to pumping a partial penetrating large diameter well through curve matching, using pump test data.

Figs. 5.16 and 5.17 shows variation of Q_a/Q_p with $4\beta t/r_w^2$ and α for each value of η. These curves show that the fraction of pumped discharge released by aquifer increases with increase in the value of the parameter $4\beta t/r_w^2$ for fixed η for each α. At very large value of $4\beta t/r_w^2$, Q_a/Q_p approaches unity for each value α and η. The value of Q_a/Q_p increases with α for fixed values of $4\beta t/r_w^2$ and η. The above figures can be used as type-curves for determining aquifer contribution to the pumped discharge at any time provided the aquifer parameters are known. The time at which Q_a/Q_p=0.95 ,i.e. , $t_{0.95Q}$ can be expressed by the following equation.

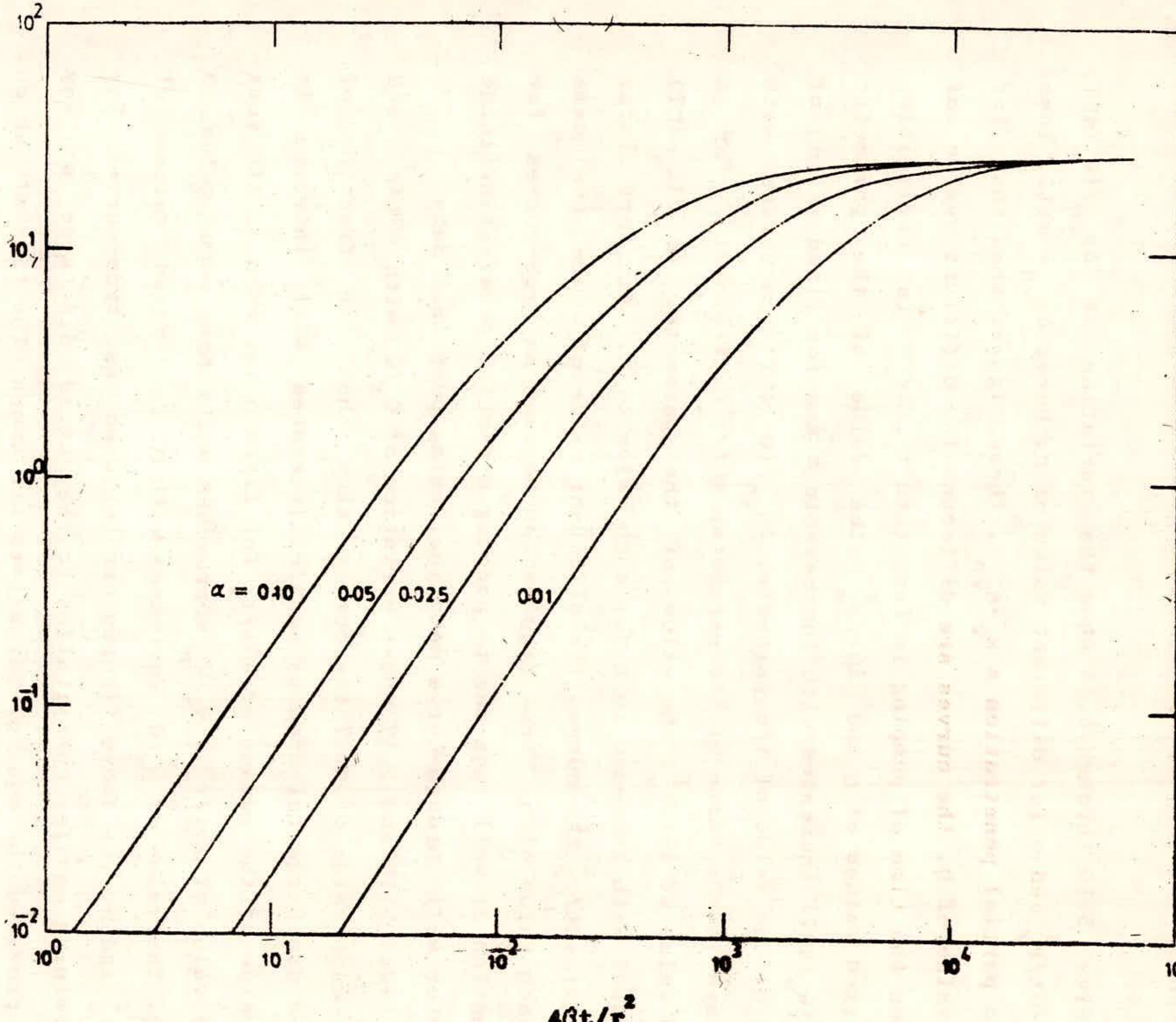


Fig. 5.10 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.1.

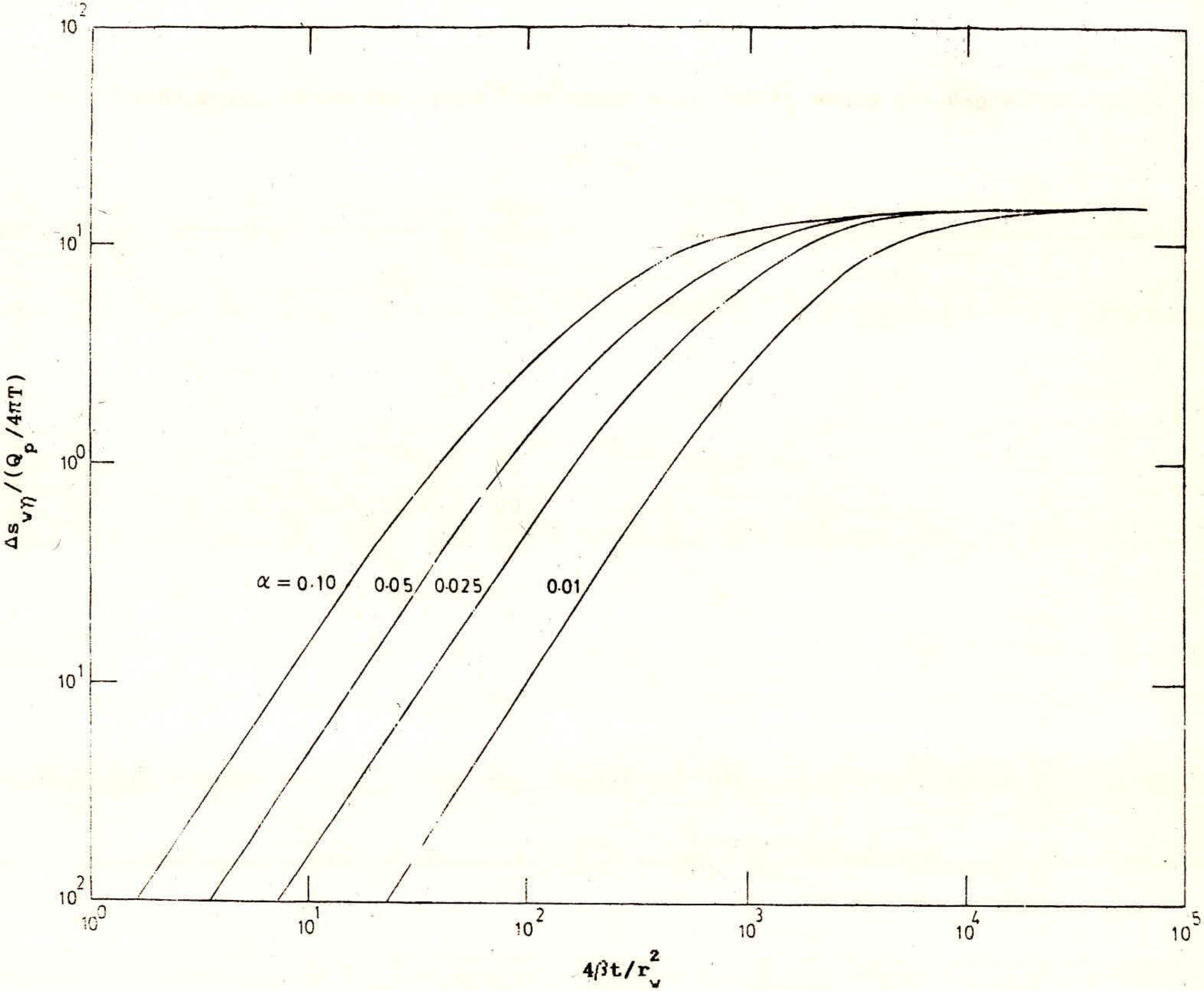


Fig. 5.11 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_w^2$ and α for η=0.2.

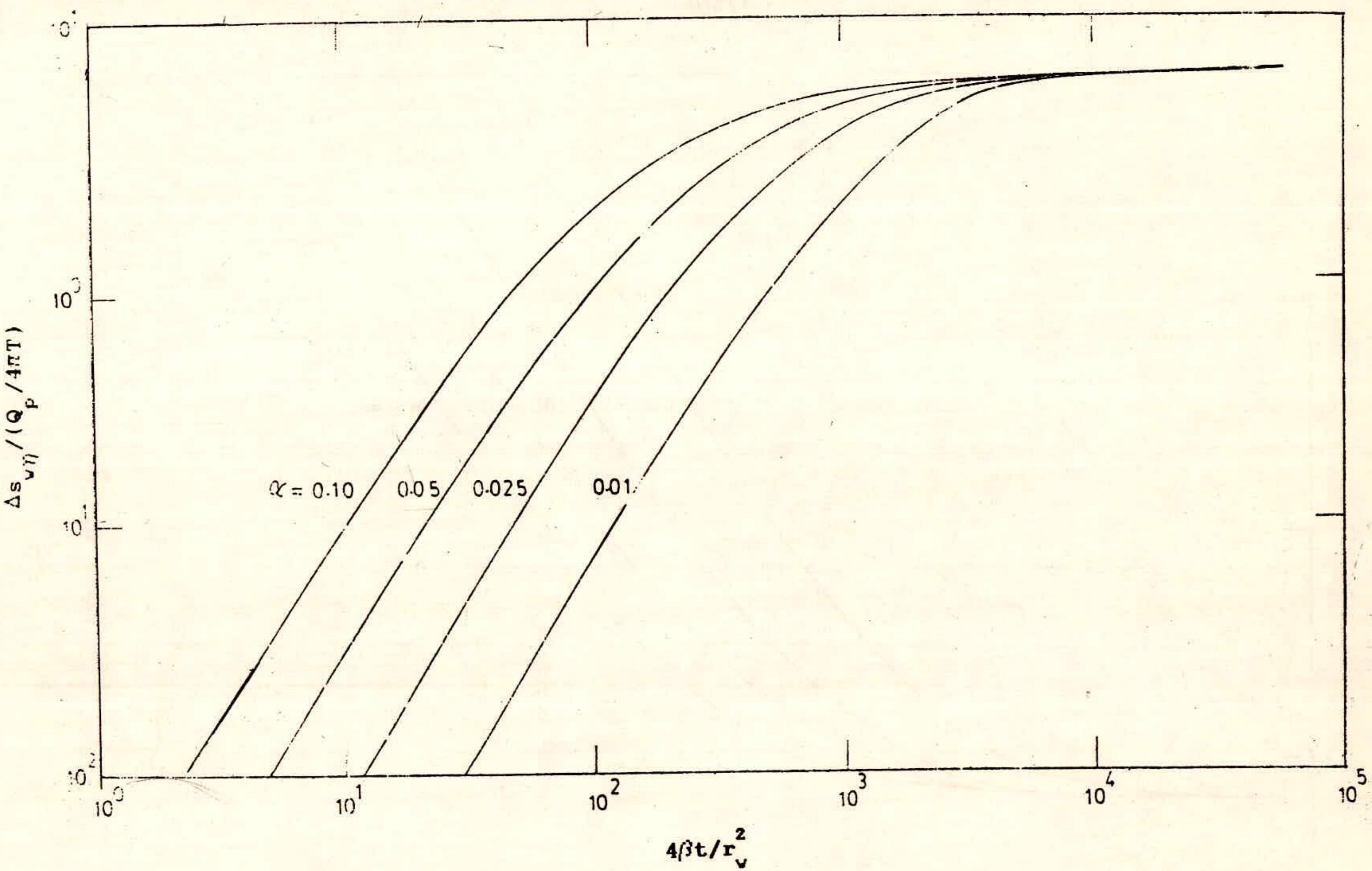


Fig. 5.12 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.4.

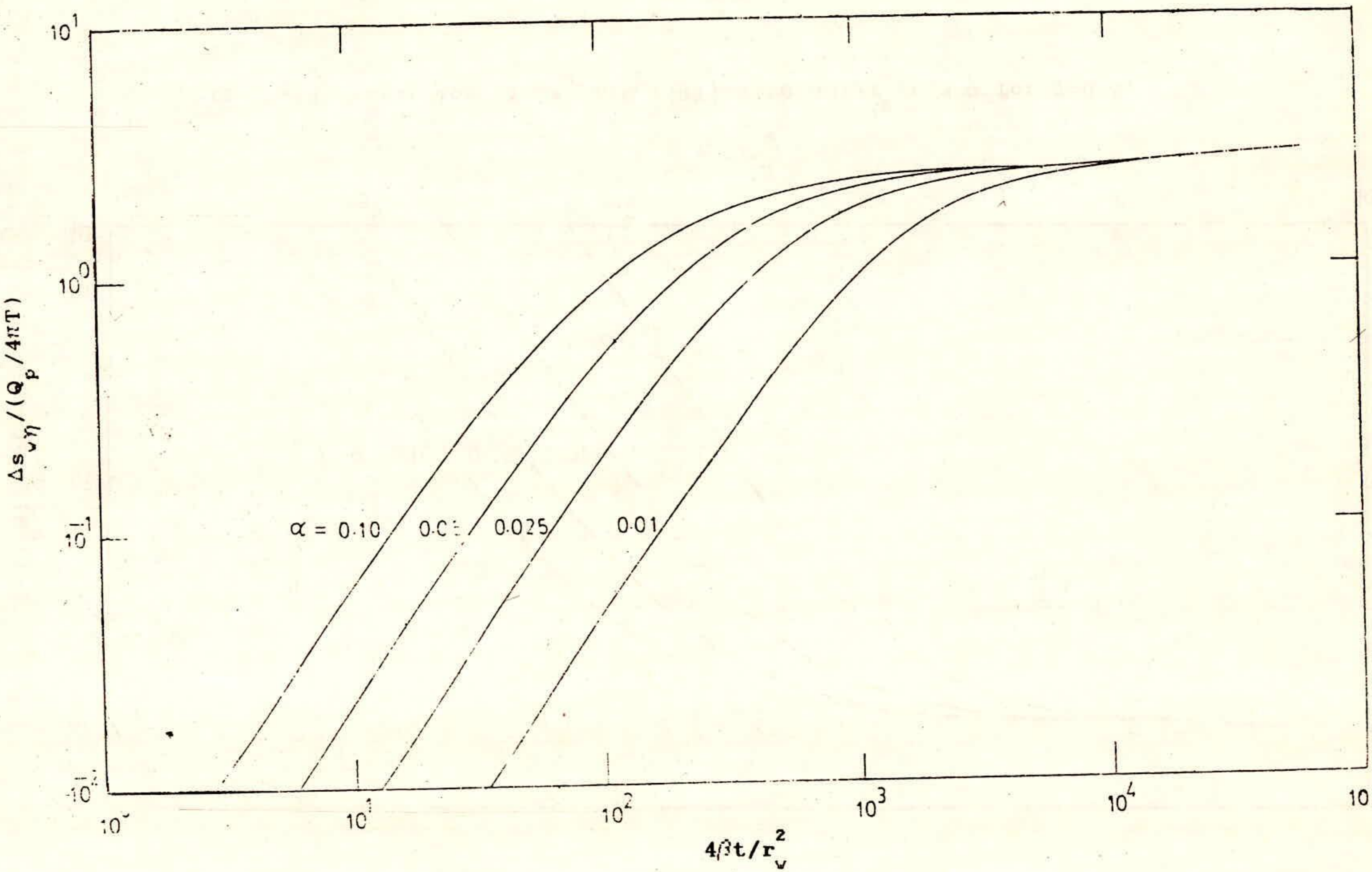


Fig. 5.13 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for $\eta=0.6$.

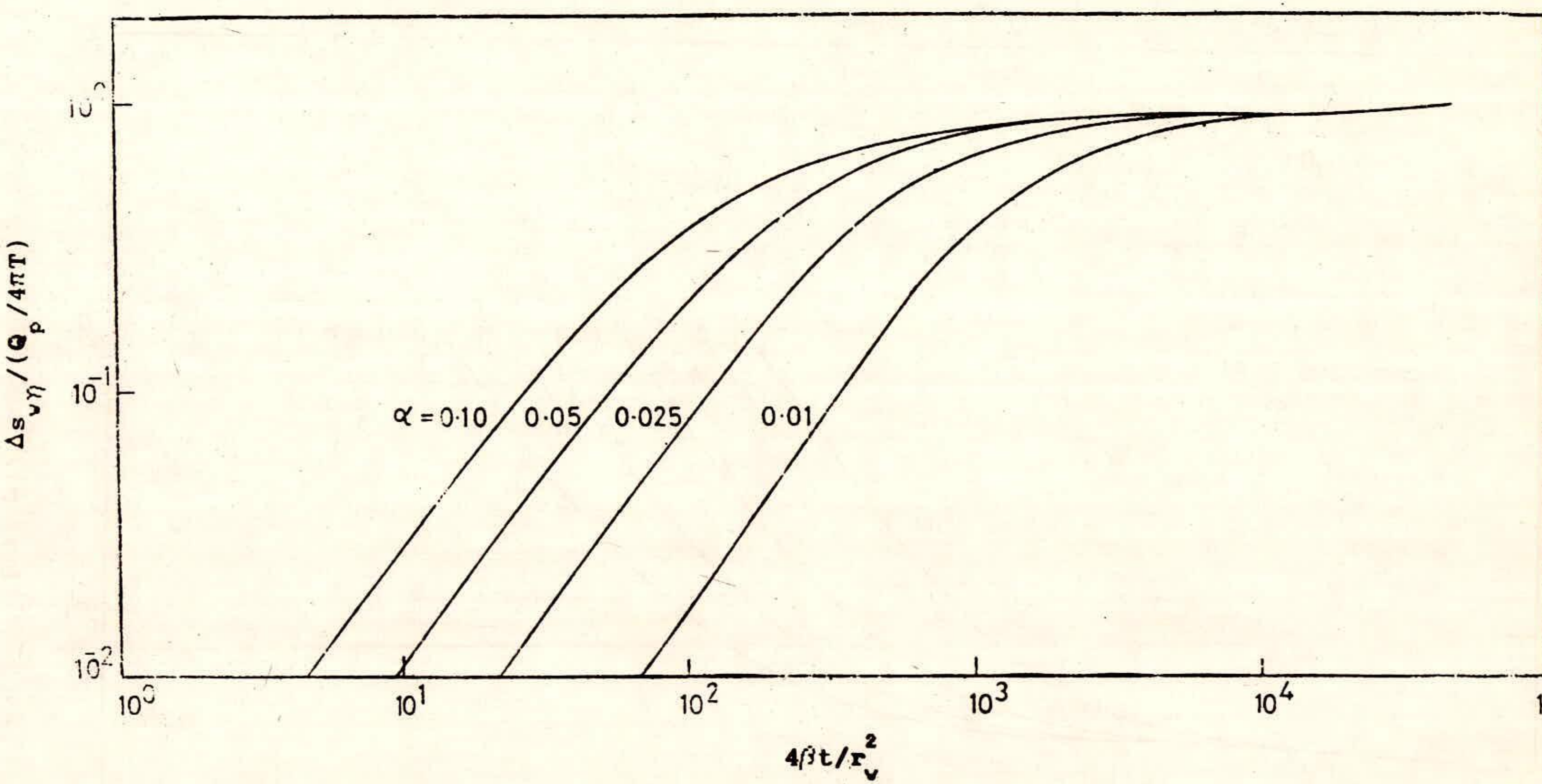


Fig. 5.14 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for $\eta=0.8$.

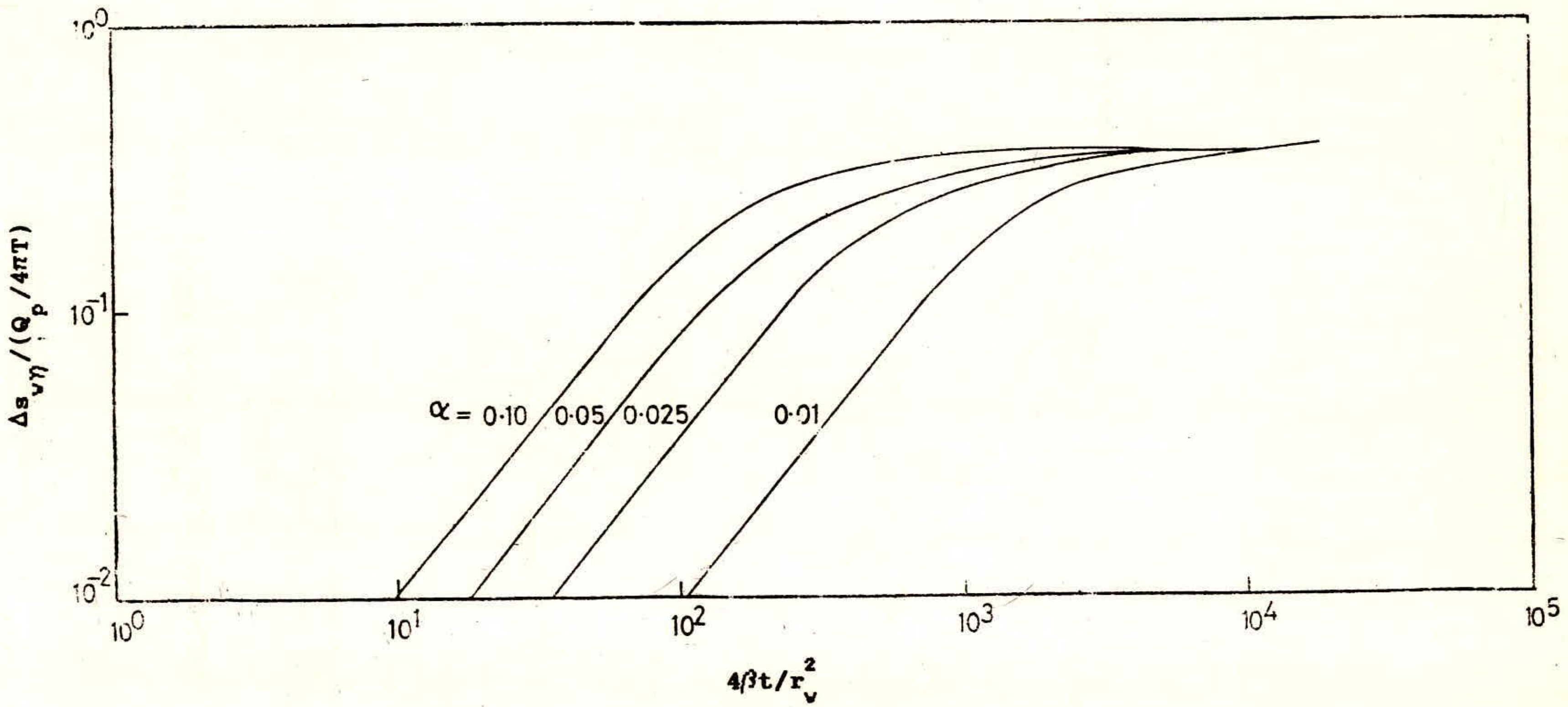


Fig. 5.15 Variation of $\Delta s_{v\eta}/(Q_p/4\pi T)$ with $4\beta t/r_v^2$ and α for η=0.9.

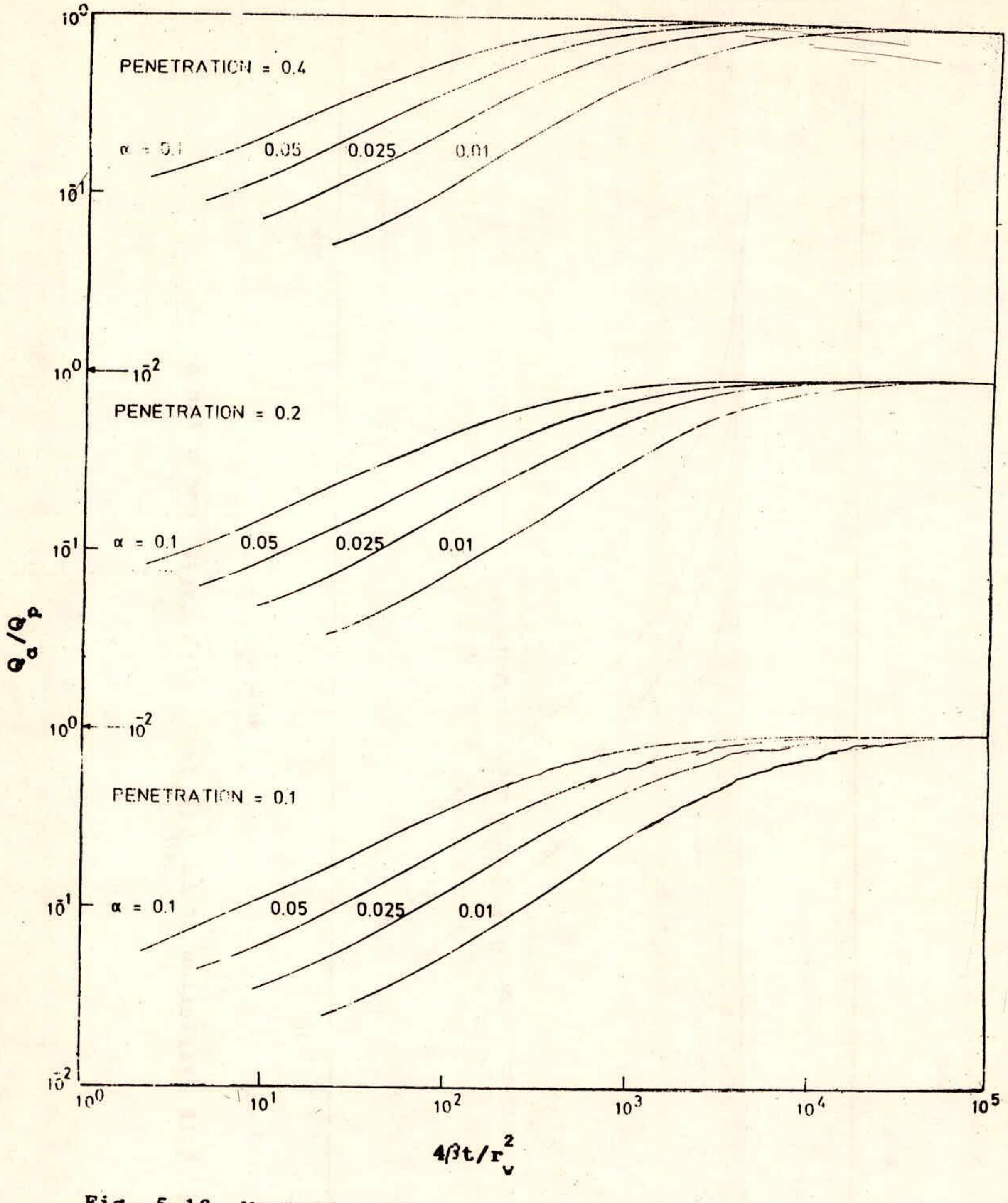


Fig. 5.16 Variation of Q_α/Q_p with $4\beta t/r_w^2$ and α.

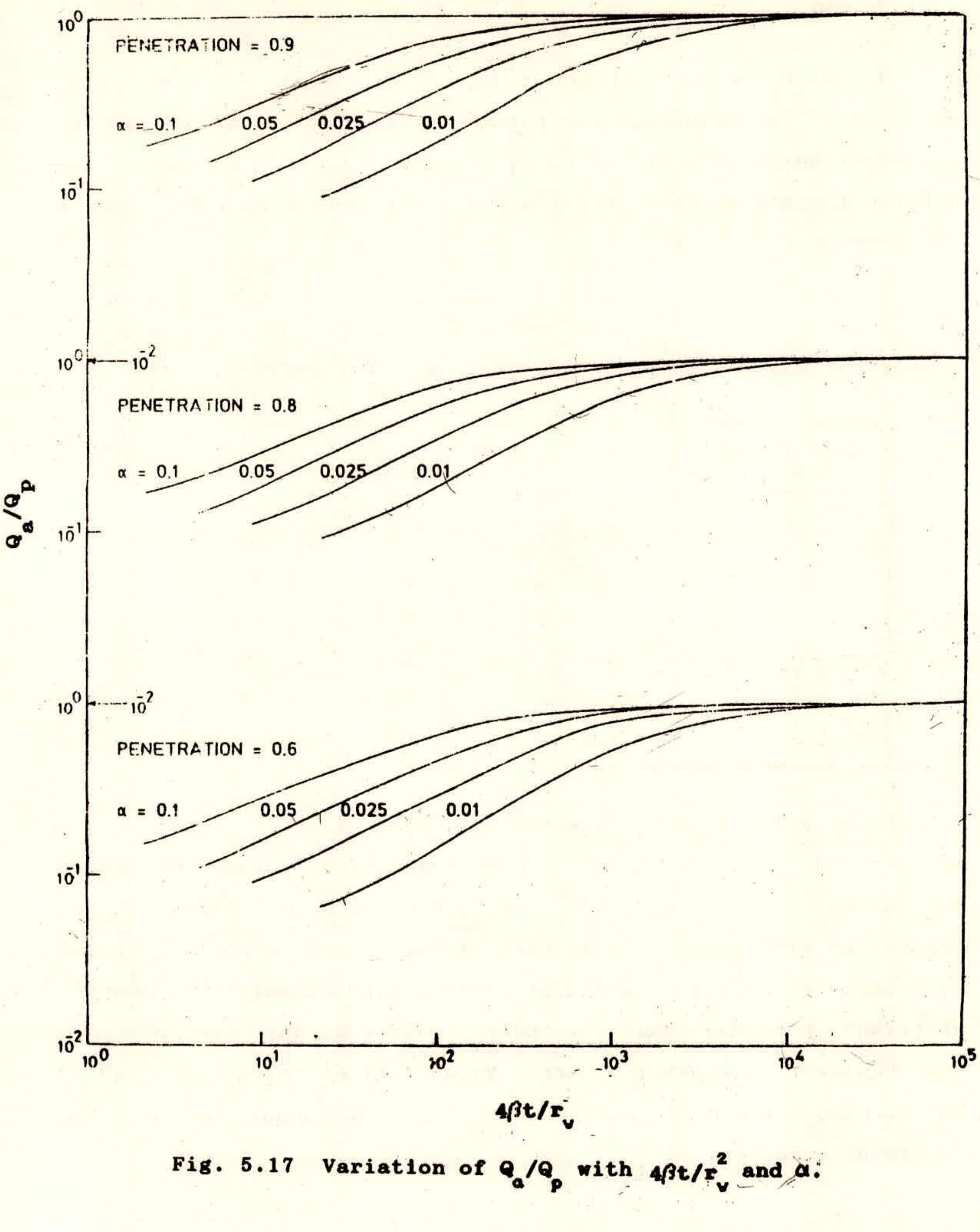


Fig. 5.17 Variation of Q_a/Q_p with $4\beta t/r_v^2$ and α.

$$t_{0.95Q} = K_1(\eta)\ r_w^2/T \qquad \ldots(5.4)$$

Where, $K_1(\eta)$ is constant which is different for different value of η. The values of the parameter $4Tt/r_w^2$ at which 95% of the pumped dicharge is contributed by aquifer and the values of K_1 for different η are given in the table 5.2. The variation of K_1 with η is shown in fig. 5.18.

Table 5.2

VALUES OF PARAMETER $4Tt_{0.95Q}/r_w^2$ AND K_1 FOR DIFFERNENT VALUE OF η

Sl. No.	η	$4Tt_{0.95Q}/r_w^2$	K_1
1.	0.1	150	37.50
2.	0.2	90	22.50
3.	0.4	55	13.75
4.	0.6	40	10.00
5.	0.8	33	8.25
6.	0.9	30	7.50

Figs. 5.19 and 5.20 show the variation of ratio of cumulative aquifer contribution, $\Sigma Q_a(t)\delta t$ and cumulative volume of water pumped, $\Sigma Q_p(t)\delta t$, with $4\beta t/r_w^2$ and α for each η. These curves follow the similar pattern as that of figs. 5.15 and 5.16 . Figs. 5.19 and 5.20 can be used for determining cumulative aquifer contribution to the cumulative pumped volume at any time provided the aquifer parameters are known. The time at which $\Sigma Q_a(t)\delta t/\Sigma Q_p(t)\delta t=0.95$,i.e., $t_{0.95V}$ can be expressed by the following equation.

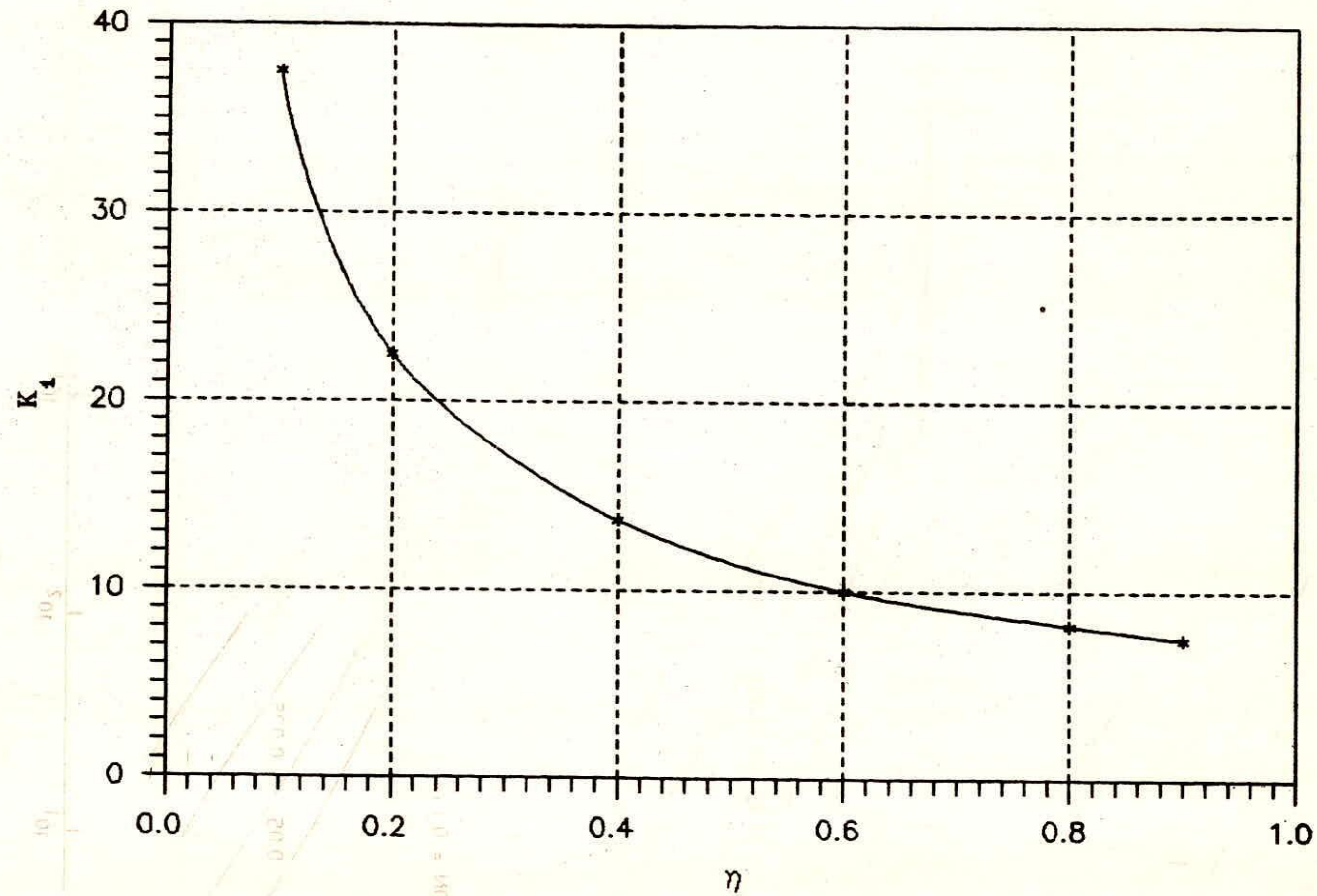


Fig. 5.18 Variation of K_1 with η.

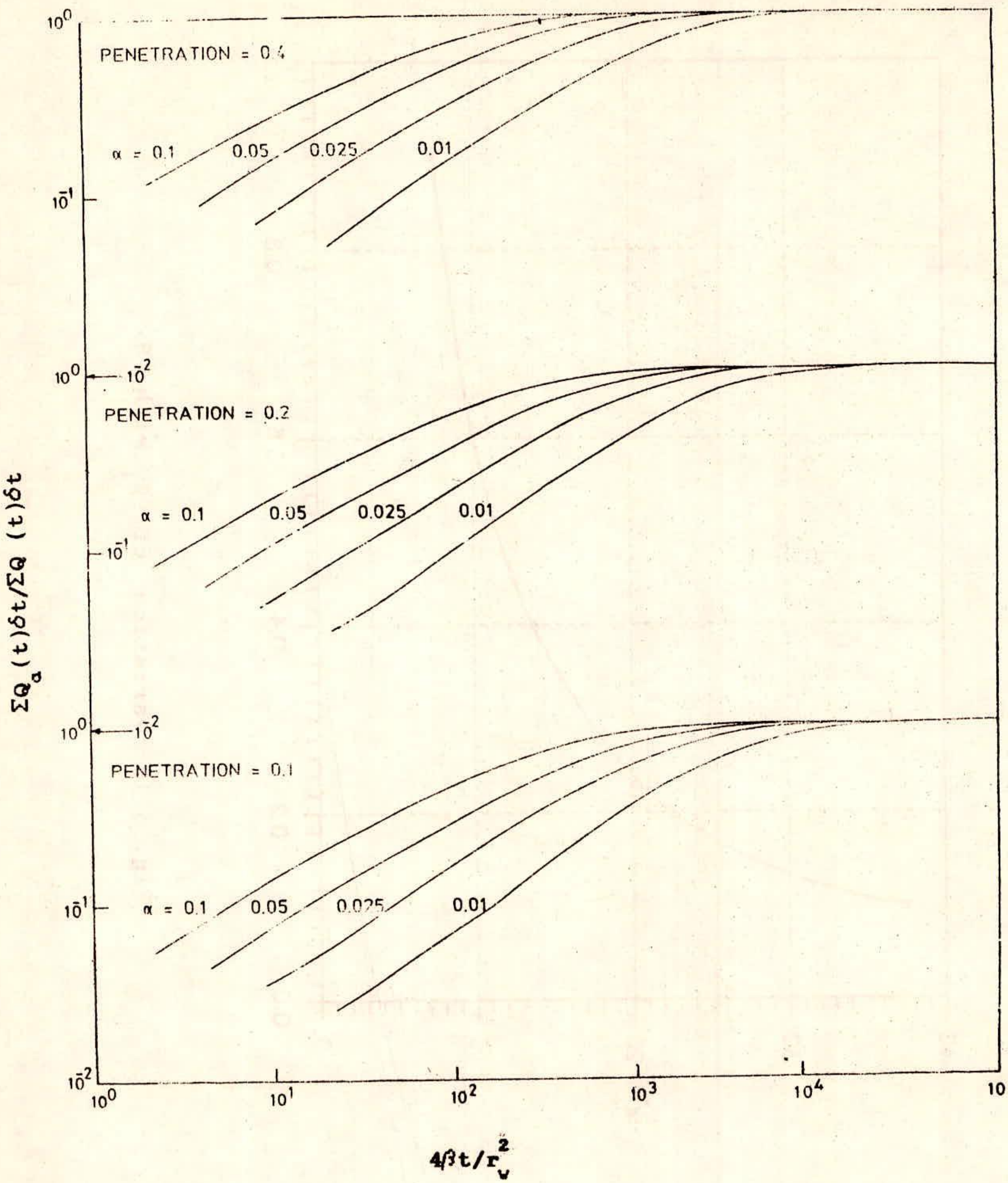


Fig. 5.19 Variation of $\Sigma Q_{\alpha}(t)\delta t/\Sigma Q\ (t)\delta t$ with $4\beta t/r_w^2$ and α.

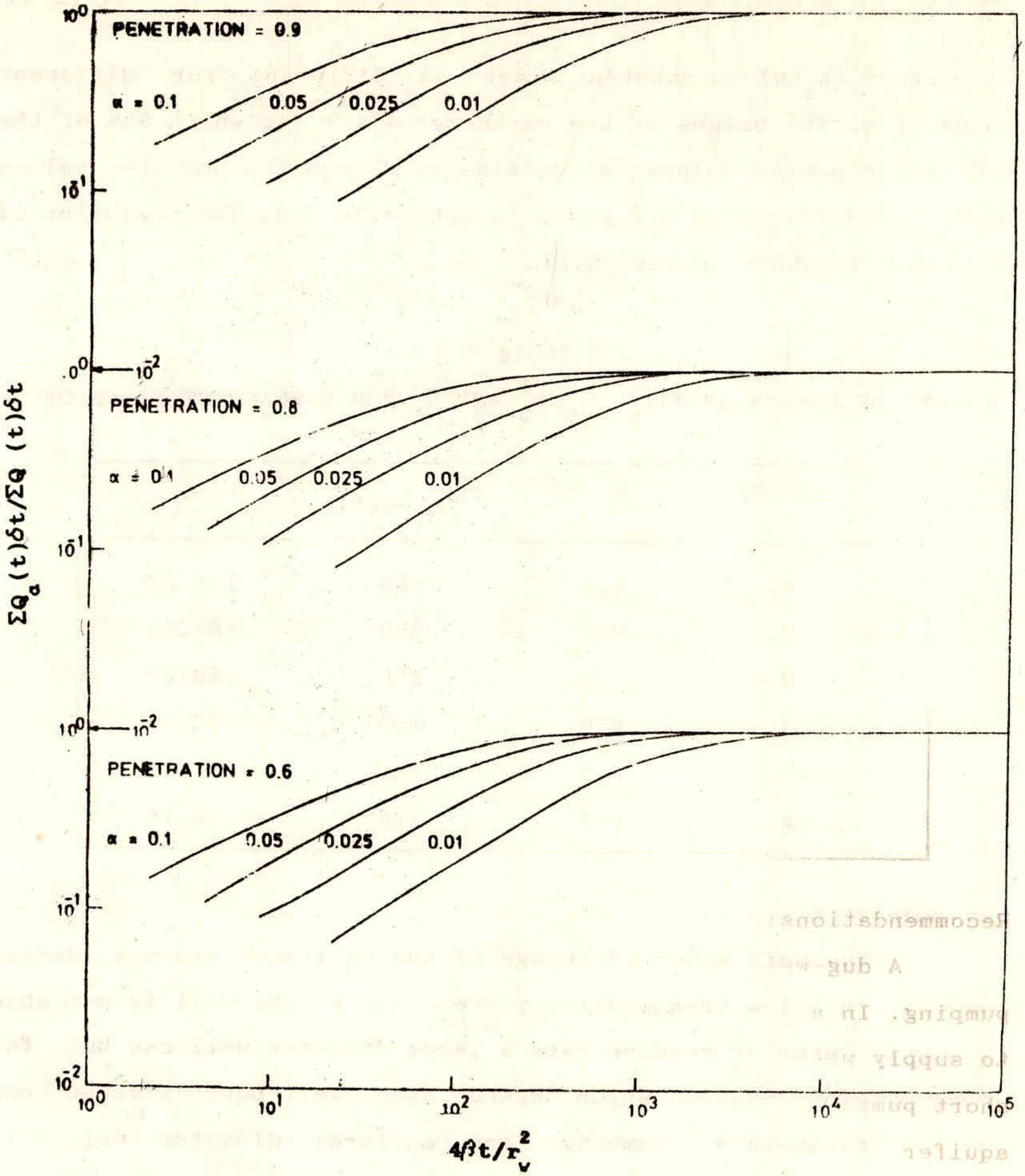


Fig. 5.20 Variation of $\Sigma Q_a(t)\delta t/\Sigma Q(t)\delta t$ with $4\beta t/r_v^2$ and α.

$$t_{0.95V} = K_2(\eta)\, r_w^2/T \qquad ...(5.5)$$

Where, $K_2(\eta)$ is constant which is different for different value of η. The values of the parameter $4Tt/r_w^2$ at which 95% of **the** cumulative pumped volume is contributed by aquifer and the values of K_2 for different η are given in the table 5.3. The variation of K_2 with η is shown in fig. 5.21.

Table 5.3

VALUES OF PARAMETER $4Tt_{0.95V}/r_w^2$ AND K_2 FOR DIFFERNENT VALUE OF η

Sl. No.	η	$4Tt_{0.95V}/r_w^2$	K_2
1.	0.1	450	112.50
2.	0.2	350	87.50
3.	0.4	200	50.00
4.	0.6	150	37.50
5.	0.8	125	31.25
6.	0.9	115	28.75

Recommendations:

A dug-well makes advantage of the well-bore storage during pumping. In a low transmissivity area when a tube-well is not able to supply water at require rate a large diameter well can but for short pumping duration which depends upon well-bore storage and aquifer parameters. pumping from a large diameter well is economical as initially up to a certain time, almost all the pumped water comes from the well-storage ,thus making the well loss component negligible. Even if the large diameter well is pumped for a longer time only a part of the discharge is taken

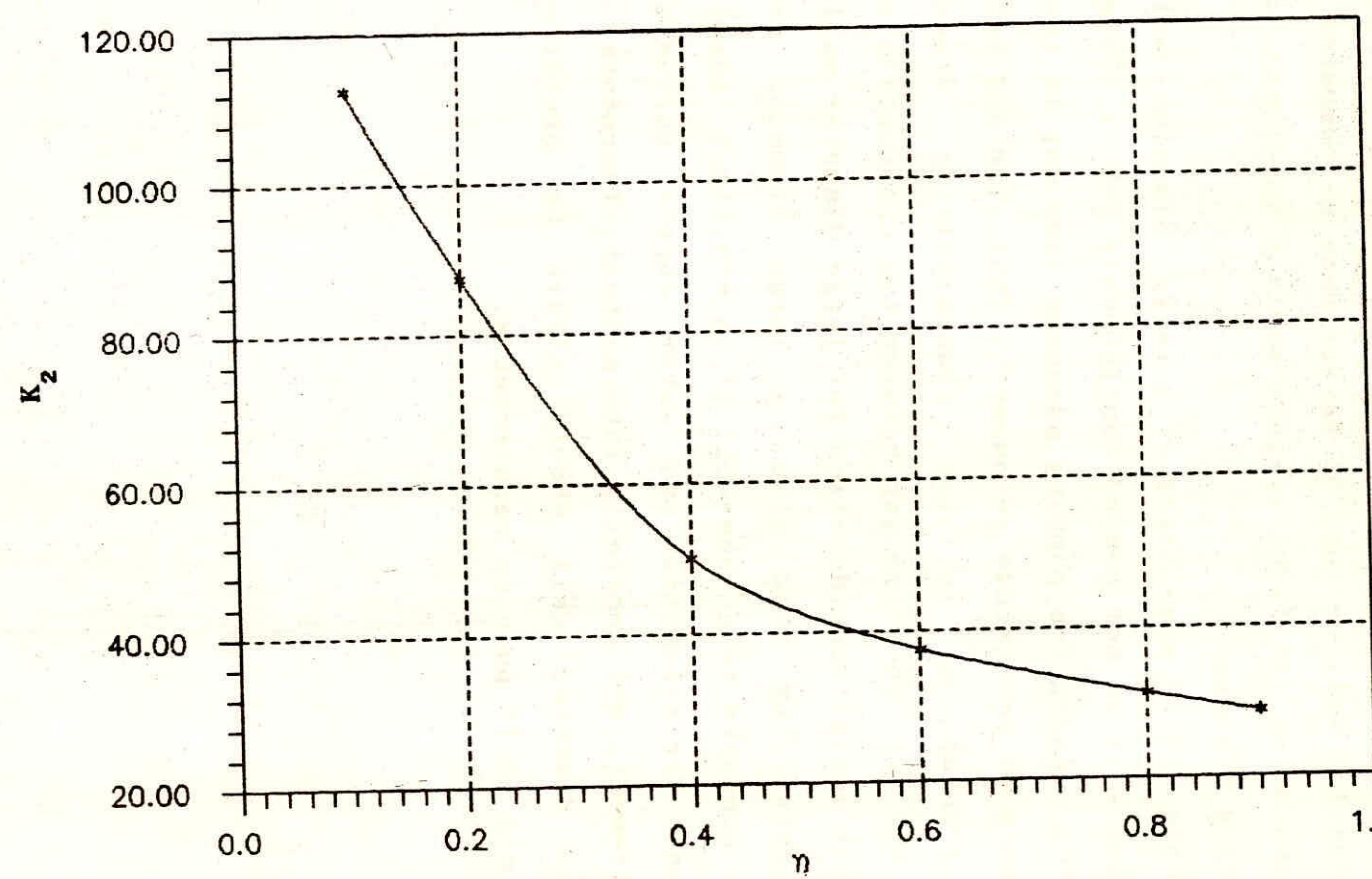


Fig. 5.21 Variation of K_2 with η.

from the aquifer, thus reducing the well-loss component substantially. If pumping is stopped after sometime, the drawdown in a large diameter well recoups at faster rate as compared to a tube-well, because of the large seepage surface developed at the well-face during pumping.

The above listed advantages of a large diameter well are obtained if the well is not pumped continuously for a long time and between two consecutive pumping adequate time gap is there to facilitate the good percentage recoupment. Thus, the dug well are very much advantageous in low transmissivity areas for intermittent pumping. The time gap between two consecutive pumping is an important design consideration for large diameter wells. In a high transmissivity area also, a large diameter well is economical as compared to a tube-well if intermittent pumping is the requirement. For irrigation and water supply purposes, the pumping requirements are generally intermittent. Therefore, a well designed large diameter well should always be preferred for intermittent and small pumping requirements.

6.0 CONCLUSION

In the present study, the unsteady flow to a partially penetrating dug well has been analyzed taking into account the well storage, seepage surface, partial penetration and ratio of specific yield to storage coefficient using a three dimensional groundwater flow model developed by Macdonald and Harbaugh. Set of type-curves between non-dimensional draw-down and non-dimensional discharge, have been presented for different penetration. The curves showing the contribution of well storage to pumping and the well loss component for different penetration, have also been given.

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