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INTERACTION OF LARGE WATER BODIES WITH AQUIFER

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PREFACE

The management of groundwater resources of an area requires the assessment of each component of groundwater balance equation. Recharge from large water bodies is a crucial component in the sense that it has often been calculated as the residual of water balance equation, thus incorporating the uncertainties involved in other components of the water balance equation. Large water bodies are common features on land surface and are important in respect of various water uses. Keeping in view their importance in many hydrologic and economic fields, their hydrologic studies are of great use. Therefore, proper field and theoretical studies on interaction of large water bodies with aquifer are needed based on variability of flow pattern near the boundaries of the water body and beneath it.

The present study deals with the modelling the three dimensional flow pattern around water bodies, using a three dimensional finite difference groundwater flow model. The hypothetical system of boundaries has been assumed. Similarity approach has been employed to develop type curves which enables the assessment of the rate of recharge from water body. The effect of aquifer-anisotropy and size of the water bodies on the type curve has also been analyzed.

The present study has been carried out by Shri S.K. Singh, Scientist 'C' under the guidance of Dr. P.V. Seethapathi, Scientist 'F' and Technical Coordinator.

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Abstract

The transient analysis of flow around a large water body of square cross-section and having uniform depth has been analyzed. Hypothetical setting of boundaries were assumed, i.e., rivers on the lateral sides of the water body and no flow boundaries at the other two sides. Both type of boundaries were assumed to be at equal distance from the centre of the water body. A three dimensional groundwater flow model has been adopted for the analysis of flow from large water body. Similarity approach has been employed to develop type curves which enables the assessment of the rate of recharge from water body, knowing the variation in the observed head in an observation well situated within the influence area of the water body. Correlation between different type-curves pertaining to different sizes of water bodies and for different hydraulic conductivities have also been presented. The effect of variation of K_h/K_v (here, K_h and K_v are the hydraulic conductivities in horizontal and vertical direction respectively) on the type curve has been analyzed.

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1.0 INTRODUCTION

1.1 General

Large water bodies are common features on land surface and are important in respect of various water uses. Keeping in view their importance in many hydrologic and economic fields, it is necessary to pay attention to their water qualitv and existence. Their hydrologic studies are of great use. In spite of early recognition of its their water management studies, very few attempts were made to understand the interaction of large water bodies with aquifer. In most of the water balance studies, the quantification of seepage from large water bodies have been done as a residual of water balance equation, thus leading to erroneous values because the uncertainties in the estimation of other components get incorporated into seepage component. Therefore, proper field and theoretical studies on interaction of large water bodies with aquifer are needed based on variability of flow pattern near the boundaries of the water body and beneath it.

1.2 Scope of the Present Study

The present study deals with the theoretical analysis of the three dimensional flow pattern around water bodies, using a finite difference groundwater flow model. The hypothetical system of boundaries has been assumed and effort has been made towards the understanding of the groundwater system in conjunction with water body as a recharge source on one side and a river as a discharge source on the other side.

The study aims at developing type curves between non dimensional time and non-dimensional drawdown due to the seepage occurring from the water body. The aquifer has been considered homogeneous with different values K_h/K_v varying from 1 to 750 (here K_h and K_v are the hydraulic conductivities in horizontal and vertical direction respectively). The water body has been assumed to be of square cross-section having uniform depth. Two different sizes of the water body have been considered.

1.3 Further Scope

The further study is required to be carried-out to establish the relation for the influence of the following parameters on the interaction of water body with aquifer.

1. The boundary of the water body should be so assumed as to fit the natural shape of the water body.
2. Modelling of the clay layers present in the aquifer.
3. Effect of hydrograph of the water level fluctuation in water body on the rate of recharge from the water body.
4. Effect of comparative distances of no flow boundaries constant head boundaries on the rate of recharge from the water body.

2.0 REVIEW

Amongst the early investigators who stressed upon the groundwater flow pattern around a lake, are Mayboon(1966,67), Williams(1968) and Janquet(1976). Bergstrom and Hansen(1962), Skinner and Borman(1973) and Cartwrite et al.(1979) have computed the ground water flow from the lake Michigan, Bergstrom et al. calculated the groundwater component as residual term. Skinner and Borman(1973) also estimated the groundwater component as the residual of their water balance equation but Cartwrite et al.(1979) made direct measurement of hydraulic gradients in the southern part of the lake. The wide variations between their estimates in seepage rates indicate the need of refined methods for determining the groundwater flux from a larger lake.

Allred et al.(1971), Manson et al.(1968), Solan(1972) and Mc-Bride(1969) measured the groundwater flow from lake with the help of water level observation wells. Some insight into the groundwater regime of discharge estimates was provided by Winter (1976,1978), who used two and three dimensional steady state models applied to hypothetical groundwater lake systems.

Janquet(1976) did a study of Snake Lake, which was classified as flow through type, i.e, a lake which receives groundwater through part of the lake basin and recharges to the groundwater system over the rest of the lake basin. His data indicated that in the spring of 1973, the formation of groundwater mound on the downgradient side of the lake caused a reversal in flow direction. If the seasonal formation of groundwater mounds, stagnation points and prolonged seasonal reversals in the flow occurs commonly at flow through lakes, it would be important to study seasonal changes in head around a lake before taking certain type of planning decisions related to lake management.

McBride and Pfannkuch(1975) used a numerical model to evaluate the vertical component of groundwater flow into one side of a lake for a number of hypothetical settings. Winter(1976) used numerical simulation of vertical groundwater flow to examine the hydrological factors that control the interaction of lakes and groundwater along the entire lake bottom for a wide variety of hypothetical settings. He showed that the movement of groundwater to and from a lake depends on the continuity of the boundary separating the local groundwater flow system associated with lake, from the intermediate and regional flow systems passing at depth beneath the lake. Based on his simulation study, he suggested the field methods and new approaches to the study of the interaction of lakes and groundwater along with critique of commonly used approaches. Studies in which one or many wells are placed near a lake to determine the interaction of lakes and groundwater, must be scrutinized carefully, because placement and construction wells are critical to a proper understanding of the interrelationship between lakes and groundwater.

Rinaldo lee and Anderson(1980) conducted investigation into the cause of high levels for Bass Lake during the early 1970's in a groundwater dominant. lake in North Western Wisconsin. A groundwater flow model was used to determine whether increased recharge rates of the magnitude that probably occurred as a result of average precipitation in the early 1970's would be sufficient to account for the observed rise in the lake level or whether regulation of the water level in the reservoir would be expected to effect lake level. The model was used to investigate

(i) the expected magnitude of the change in lake level in response to increased recharge rates brought about by high precipitation and,

(ii) the effect on lake level of changes in the level of reservoir. The result of the simulation suggests that increased recharge brought about by above average precipitation in 1975 was sufficient to account for observed increase in lake level.

Anderson and Munter(1981) studied the development of groundwater mound near Snake Lake by means of two dimensional transient groundwater flow models. The areal flow model was used to demonstrate the effect of stagnation zone on the water budget of lake. The groundwater component of lakes water budget as measured by difference between total inflow of groundwater through the bottom of the lake and total outflow of groundwater through the lake bottom varied from -4.0×10^{-4} m^3/s at the beginning of the simulation to +0.21 ft^3/s 36 days later. Thus, the seasonal formation of stagnation zone can have a marked effect on the groundwater component of lakes water budget. They found that the primary factors governing the formation of mound and stagnation zone at snake lake are

1.Location of lake on groundwater divide.

2. The suspected occurrence of zones of low permeability northwest of the lake and/or locally high recharge from leaky storm severs in that area. Stross and Spangler(1980) reported the occurrence of deposit of low permeability on the down gradient side of several small lakes in Florida. They suggested that the mechanism for concentration is the flushing of fine-grained particles from the lake sediments into the aquifer down gradient of the lake. If this is the case then the presence of low permeability of Snake Lake would not be unexpected.

3. A seasonally high rate of recharge primary trigger the formation of the mound while other factors such as the reduced

thickness of the aquifer beneath the lake and a low regional hydraulic gradient also contribute to the development of the mound.

They concluded that the existence of a stagnation point is an indication that the groundwater seeping into the lake over most of the lake basin reflects a change in the groundwater flow pattern around a flow-through lake. Such a change in groundwater regime can have a marked effect on a lakes water and nutrient budgets. However, more field observations of seasonal variations in groundwater potential around flow-through lakes are needed before an assessment of the frequency with which seasonal stagnation points form at flow-through lakes is made.

Munter and Anderson(1981) showed that two and three dimensional groundwater flow models provide flexible and effective means of calculating flow rates in well defined but complex natural flow systems around lakes. The general conclusion drawn from the study are:

1. The Bass Lake model showed that the ratio of horizontal to vertical hydraulic conductivity K_h/K_v of the aquifer has significant effect on the simulated head distribution around Bass Lake, and on the magnitude and distribution of seepage from the lake. K_h/K_v can be estimated by model calibration of field measurements. The vertical hydraulic conductivity of littoral lake bed sediments exerts a strong influence on computed seepage rates. It has been recommended that the field studies on lake include mapping the thickness and distribution of different types of lake bed sediments particularly in littoral areas, and estimating their vertical hydraulic conductivity.

2. The models constructed for Nepco lake shows that two dimensional profile models are not always adequate or applicable for simulating flow systems around lakes. A three dimensional model however, although larger and more difficult to use is a realistic alternative where a complex three dimensional flow system actually exists. For lake seepage rate estimation both the two and three dimensional models are improvement over one dimensional Darcy's Law Method because of their ability to incorporate the heterogeneity and anisotropy of the aquifer as wells as hydraulic gradient data in more than one dimension, hence, it is suggested that at many lakes a combination of two-and/or three dimensional models could be used to estimate lake seepage rates. This may include the application of a two dimensional areal model (e.g. see Anderson and Munter,1981).

3. Two dimensional models are appropriate when the geology of a lake is well known and where vertical head data indicate the presence of a two dimensional flow system around part or all of a lake. Two dimensional models are also useful in a three dimensional flow system where a less accurate simulation is desired or where a sensitivity analysis is to be performed. The results of a two dimensional sensitivity analysis can be transferred readily to a similarly posed three dimensional model.

4. Three dimensional modelling should be considered when geometry of water-surface of surface water bodies and/or geological condition create a truly, three dimensional flow system, and where sufficient geological data and vertical and areal hydraulic head data are available to describe the flow system.

From the critical review of the past studies, it is observed that the interference of the various variables involved, on the recharge rate from depression storage have not yet been clearly understood and so far, no guidelines or methods have been evolved for the assessment of recharge from large depression storage. Hence, there is a great need to study the influence of various parameters on the recharge rate, which will be useful for the assessment of the quantity of seepage from a water body.

3.0 PROBLEM DEFINITION

The purpose of the present study is to quantify the recharge from a water body under given oundary conditions. In real life problem the shape and size of water bodies, extent of aquifer and its drainage boundaries vary much in different situations and do not conform to any fixed shape or relation. For the present analysis, simplified cases with hypothetical setting of boundaries are assumed. In this direction an attempt was made by Singh and Seethapathi(1986) in which type-curve was developed for the assessment of recharge from a lake assuming the aquifer to be homogeneous and isotropic and lake to be of square cross-section having uniform depth. The present study is an extension of the above work in the sense that the effect of anisotropy of the aquifer and varying sizes of water body have been considered. Thus, the type-curve developed for homogeneous and isotropic case for a particular size needs to be modified in order to include the effect of anisotropy and different sizes of the lake.

For present problem two different sizes i.e., 270 m x 270 m and 340 m x 340 m have been considered. In the both the cases the depth of the water body was taken to be the same, i.e., 9 m. The aquifer have been assumed to be homogeneous with value of K_h/K_v varying from 1 to 750. The total depth of aquifer to an impermeable bed was taken as 50 m. The water body is assumed at the centre of the aquifer in plan and the aquifer extends to a distance of 3 km from the centre of the water body in both x and y directions. The boundaries of the aquifer parallel to one side of water body are fully penetrating constant head boundaries and boundaries of the aquifer parallel to the other side of water body are no flow boundaries. Description of the problem is given in fig.3.1.

It is intended to develop type-curves which enables the assessment of recharge from a water body for different values of K_h/K_v under above mentioned setting of boundaries, using the record of water level fluctuations in an observation well situated within the influence area of the water body. It is also intended to find suitable location of the observation well for this purpose. The range of variables considered in the present analysis is given in table 3.1.

Table 3.1

Range of variables

Size of water body	K_h/K_v	ΔH
270mx270m	1	
	10	3m,5m,7m, and 9m
	100	for every value of
	250	K_h/K_v
	500	
	750	
340mx340m	1	
	10	3m,5m,7m, and 9m
	100	for every value of
	250	K_h/K_v
	500	
	750	

ΔH is the water level in the water body measured above the head in the constant head boundary. Hydraulic conductivity in horizontal direction K_h was taken equal to 10.0 m/d.

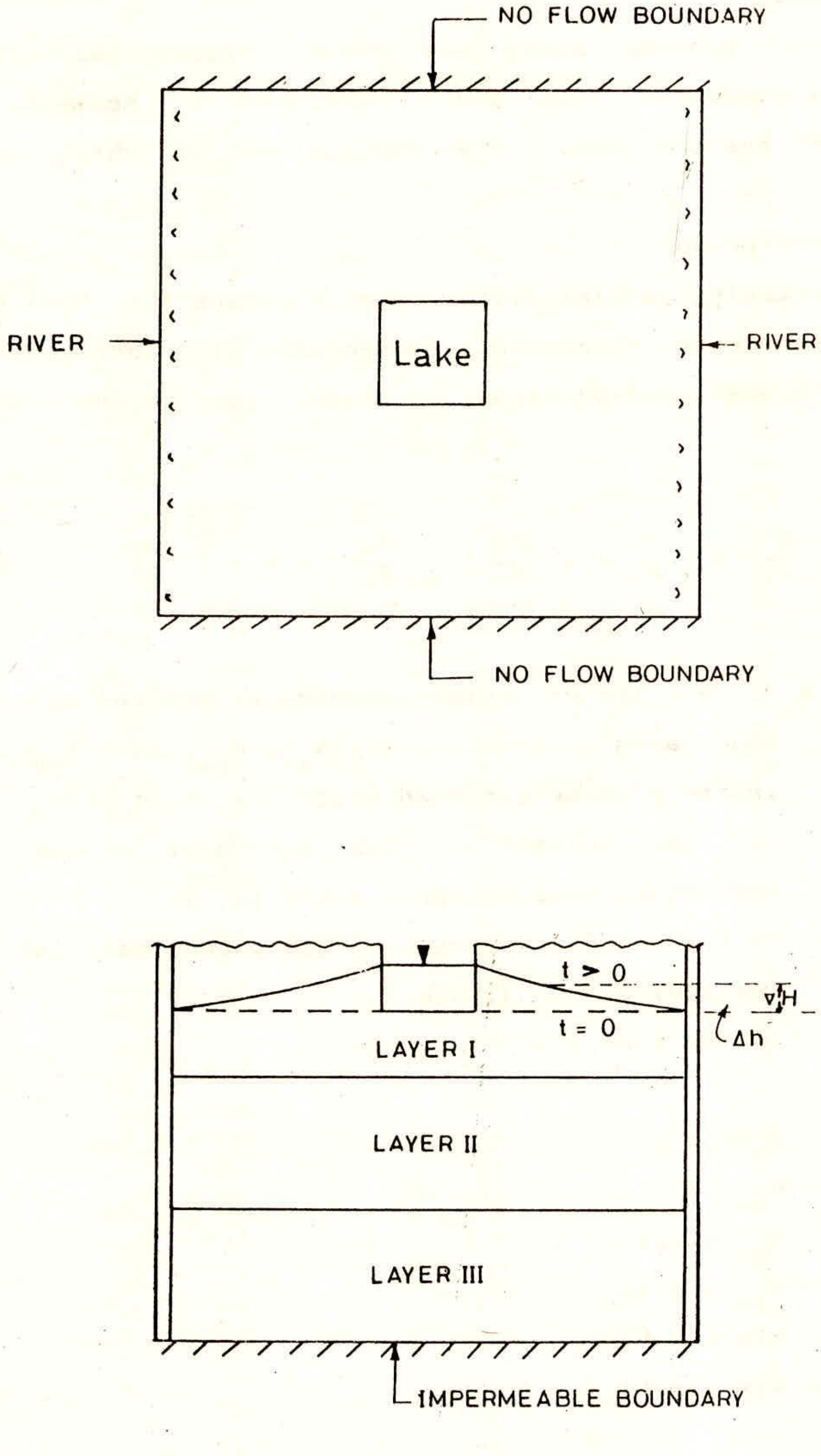


Fig. 3.1 Definition Sketch of the problem.

4.0 METHODOLOGY

For the present study the three dimensional finite difference groundwater flow model developed by Mcdonald and Harbaugh(1985) has been used, the description of which is as follows:

4.1 Model Description

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

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right) - W = S_s\frac{\partial h}{\partial t} \qquad ...(4.1)$$

where,

x,y,and z	are the cartesian coordinates aligned along the major axes of conductivity K_{xx}, K_{yy}, and K_{zz};
h	is the piezometric head (L);
W	is the volumetric flux per unit volume and represents source and/or sinks (T^{-1});
S_s	is the specific storage of the porous material of the aquifer (L^{-1}); and
t	is the time (T)

here,

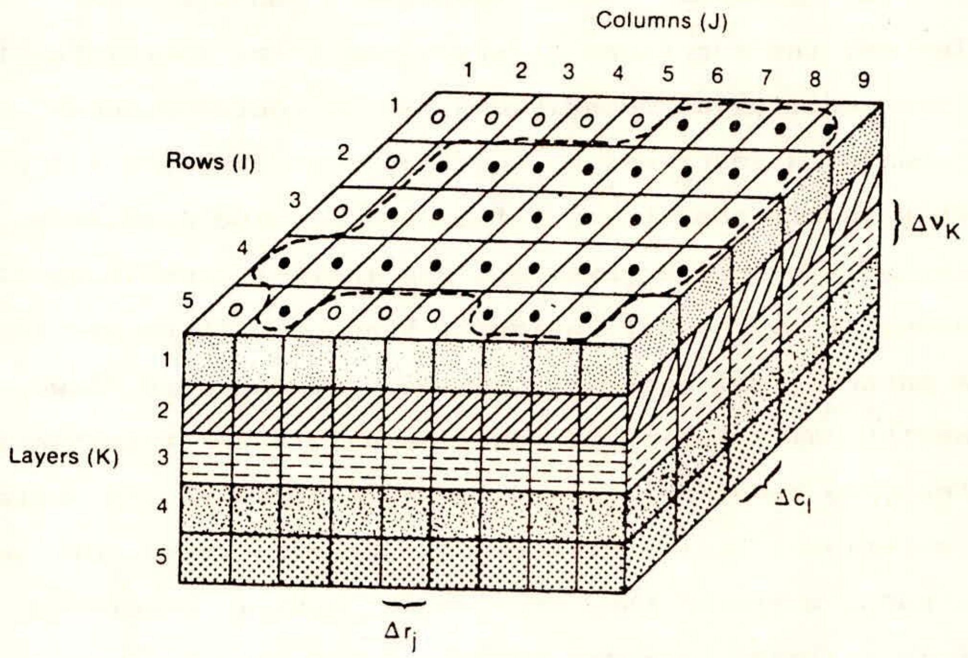
$$S_s = S_s(x,y,z)$$
$$K_{xx} = K_{xx}(x,y,z)$$
$$K_{yy} = K_{yy}(x,y,z)$$
$$K_{zz} = K_{zz}(x,y,z)$$
$$h = h(x,y,z,t)$$
$$W = W(x,y,z,t) \qquad ...(4.2)$$

Thus, in general 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 4.1. Equation 4.1 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 4.1 is not feasible for complex systems, so 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 Discretization convention

For the formulation of finite difference equations, the aquifer system needs 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.4.1. 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



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. 4.1 A Discretized Hypothetical Aquifer System.

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.4.1 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.4.1). Thus the cell with the coordinates of $(i,j,k) = (5,3,2)$ has a volume of $\Delta r_3.\Delta c_5.\Delta v_2$.

Configuration of cells:

There exist 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 a 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 a 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 are shown in fig.4.2.

4.3 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 in storage within the cell. Under the assumption that the groundwater is incompressible, the continuity equation (expressing the balance of flow) for a cell can be written as

$$\Sigma Q_i = S_s \frac{\Delta h}{\Delta t} . \Delta V \qquad \ldots(4.3)$$

where,

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

S_s is the 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});

ΔV is the volume of the cell (L^3); and

Δh is the change in head over a time interval Δt (L)

The right hand side of equation 4.3 represents the volume of water taken into the storage over a time interval Δt, given a change in head of Δh. Thus, equation 4.3 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.4.3 shows a cell i,j,k and six adjacent

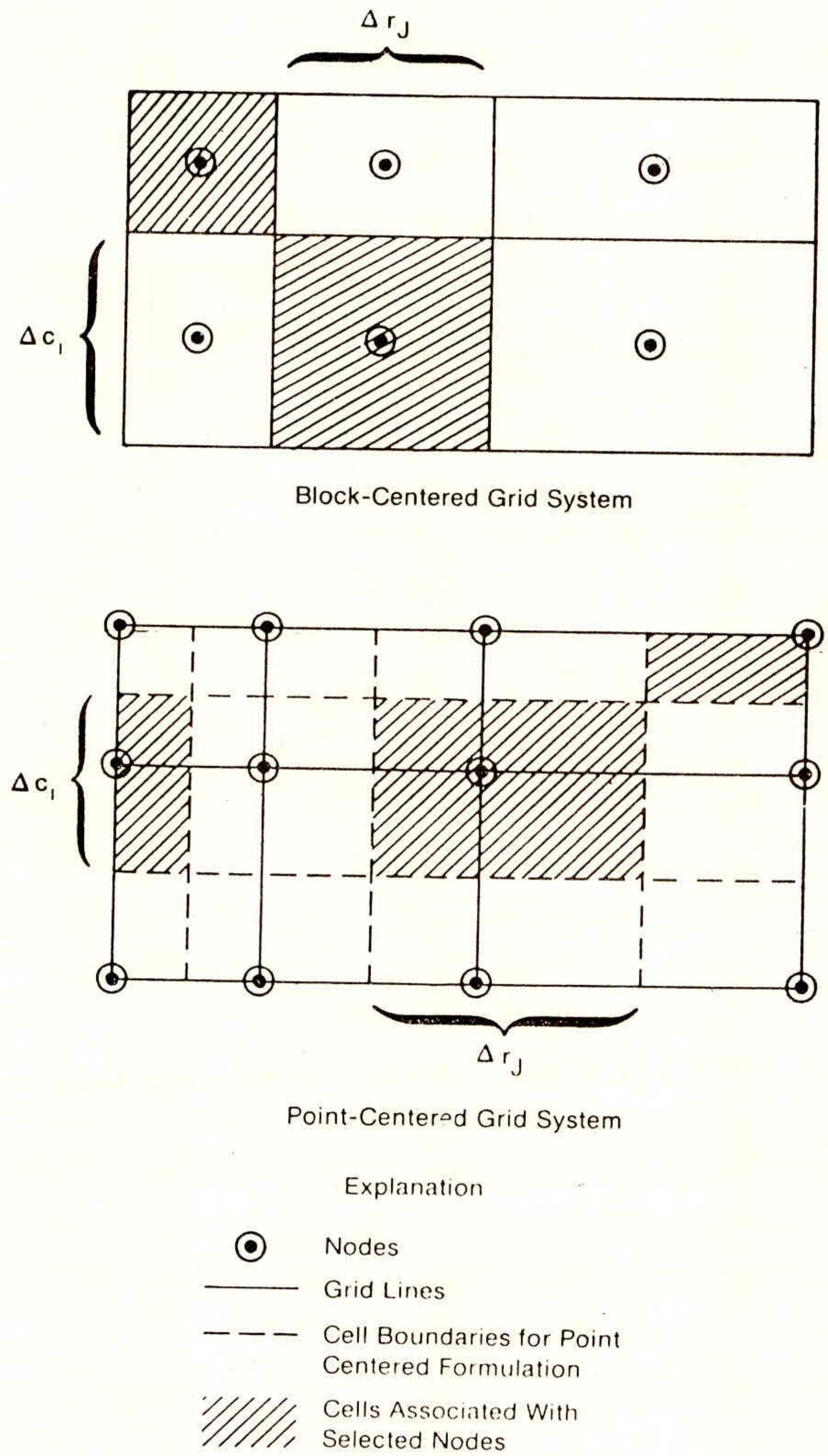


Fig. 4.2 Block-Centered and Point-Centered Grids.

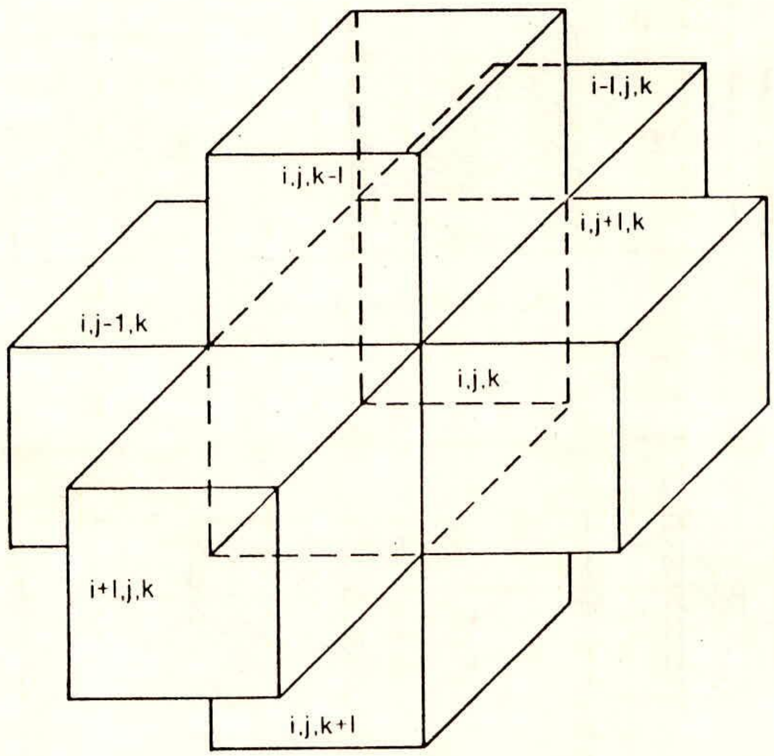


Fig. 4.3 Cell i,j,k and indices for the six Adjacent cells.

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 cell from six adjacent cells. Using Darcy's law, flow from each adjacent cell into the cell i,j,k can be obtained. Flow into the cell i,j,k in row direction from cell $i,j-1,k$ (fig.4.4) is given by

$$q_{i,j-1/2,k} = KR_{i,j-1/2,k} \, \Delta c_i \Delta v_k \frac{(h_{i,j-1,k} - h_{i,j,k})}{\Delta r_{j-1/2}} \qquad ...(4.4)$$

where,

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

$KR_{i,j-1,k}$ 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.4.4). 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 $i,j-1,k$.

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.

$$q_{i,j-1/2,k} = CR_{i,j-1/2,k}(h_{i,j-1,k} - h_{i,j,k}) \qquad ...(4.5)$$

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

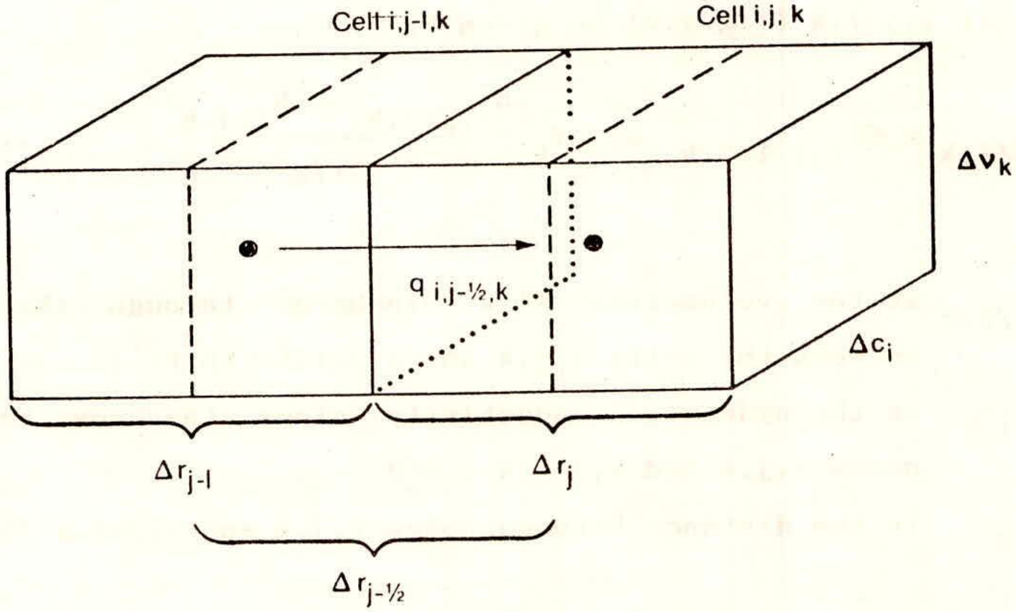


Fig. 4.4 Flow into Cell i,j,k from Cell i,j-1,k.

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.

$$q_{i,j+1/2,k} = CR_{i,j+1/2,k} (h_{i,j+1,k} - h_{i,j,k}) \qquad ...(4.6)$$

$$q_{i-1/2,j,k} = CC_{i-1/2,j,k} (h_{i-1,j,k} - h_{i,j,k}) \qquad ...(4.7)$$

$$q_{i+1/2,j,k} = CC_{i+1/2,j,k} (h_{i+1,j,k} - h_{i,j,k}) \qquad ...(4.8)$$

$$q_{i,j,k-1/2} = CV_{i,j,k-1/2} (h_{i,j,k-1} - h_{i,j,k}) \qquad ...(4.9)$$

$$q_{i,j,k+1/2} = CV_{i,j,k+1/2} (h_{i,j,k+1} - h_{i,j,k}) \qquad ...(4.10)$$

Equations 4.5 -4.10 represent the flow into the cell i,j,kfrom 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 4.1, may be expressed in general as

$$a_{i,j,k,n} = p_{i,j,k,n} h_{i,j,k} + q_{i,j,k,n} \qquad ...(4.11)$$

where,

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

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

$q_{i,j,k,n}$ is 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,

$$p_{i,j,k,1} = 0 \text{ ; and}$$
$$a_{i,j,k,1} = -q_{i,j,k,1} \quad \text{...(4.12)}$$

If the second external source (n=2) is taken to be seepage from river bed. Seepage 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,

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

or, $$a_{i,j,k,2} = -CRIV_{i,j,k,2}\, h_{i,j,k} + CRIV_{i,j,k,2}\, R_{i,j,k} \quad \text{...(4.13)}$$

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 4.11 . If there are N external sources or stresses affecting a single cell, the combined flow is expressed by

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

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

$$= P_{i,j,k} h_{i,j,k} + Q_{i,j,k} \qquad \ldots(4.14)$$

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 4.5-4.10 and the flow from the external sources into cell i,j,k is represented by equation 4.14 . Substituting these equations in equation 4.3, we get

$$\begin{aligned}
& CR_{i,j-1/2,k} (h_{i,j-1,k} - h_{i,j,k}) + \\
& CR_{i,j+1/2,k} (h_{i,j+1,k} - h_{i,j,k}) + \\
& CC_{i-1/2,j,k} (h_{i-1,j,k} - h_{i,j,k}) + \\
& CC_{i+1/2,j,k} (h_{i+1,j,k} - h_{i,j,k}) + \\
& CV_{i,j,k-1/2} (h_{i,j,k-1} - h_{i,j,k}) + \\
& CV_{i,j,k+1/2} (h_{i,j,k+1} - h_{i,j,k}) + \\
& P_{i,j,k} h_{i,j,k} + Q_{i,j,k} \\
& = SS_{i,j,k} (\Delta r_j \Delta c_i \Delta v_k)(\Delta h_{i,j,k} / \Delta t) \qquad \ldots(4.15)
\end{aligned}$$

where,

$\Delta h_{i,j,k}/\Delta t$	is a finite difference approximation for head change with respect to time $[LT^{-1}]$
$SS_{i,j,k}$	is the specific storage of cell i,j,k $[L^{-1}]$; and
$\Delta r_j \Delta c_i \Delta k_k$	is the 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.,

$$CR_{i,j-1/2,k}(h^m_{i,j-1,k} - h^m_{i,j,k}) + CR_{i,j+1/2,k}(h^m_{i,j+1,k} - h^m_{i,j,k})$$
$$+CC_{i-1/2,j,k}(h^m_{i-1,j,k} - h^m_{i,j,k}) + CC_{i+1/2,j,k}(h^m_{i+1,j,k} - h^m_{i,j,k})$$
$$+CV_{i,j,k-1/2}(h^m_{i,j,k-1} - h^m_{i,j,k}) + CV_{i,j,k+1/2}(h^m_{i,j,k+1} - h^m_{i,j,k})$$
$$+P_{i,j,k}h^m_{i,j,k} + Q_{i,j,k} = SS_{i,j,k}(\Delta r_j \Delta c_i \Delta v_k)\frac{(h^m_{i,j,k} - h^{m-1}_{i,j,k})}{t_m - t_{m-1}} \quad \ldots(4.16)$$

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, we are left with a system of 'n' equations with 'n' unknowns. Such a system of equations can be solved simultaneously.

4.4 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 boundaries. 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 4.16 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 for a no flow cell nor the equations for the adjacent cells containing a term representing flow from the no flow cell, are formulated. The use of no-flow cells and constant head cells to simulate boundary conditions is given in fig. 4.5. 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 4.16 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 mark the end of the first time step. Therefore, in equation 4.16, the subscript m is taken as 2 and thus the subscript m-1, which appear in only one head term, is taken as 1.

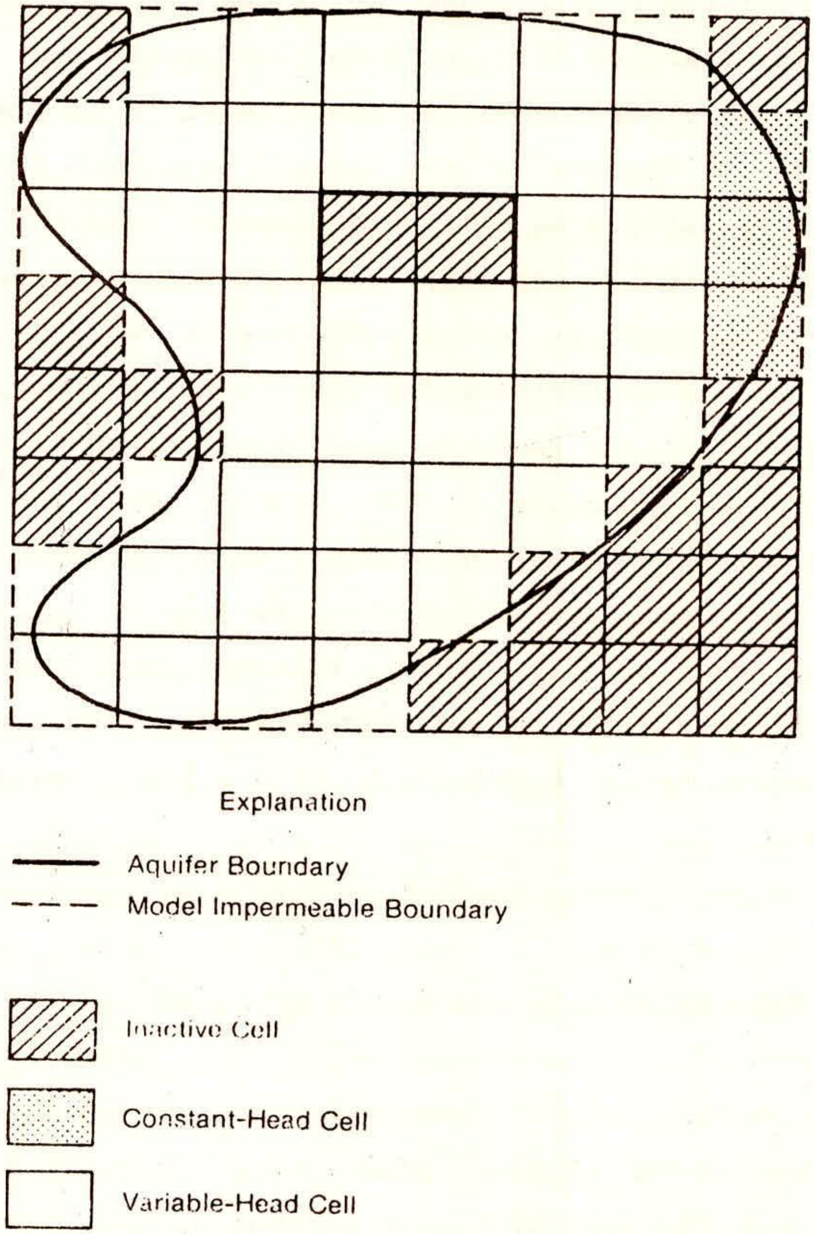
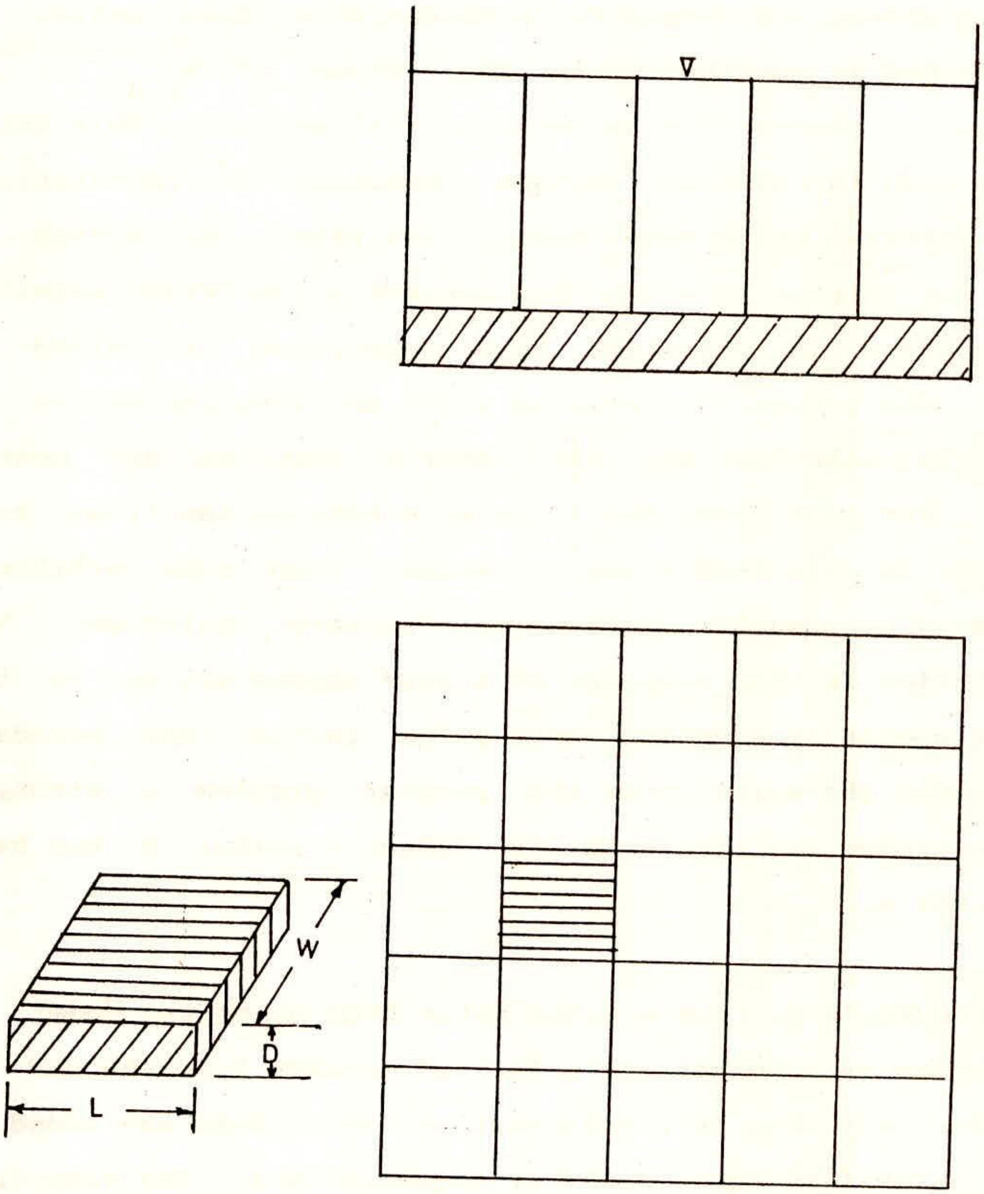


Fig. 4.5 Discritized Aquifer Showing Boundaries and Constant Head Cells.

Once such equations are formed for each variable head cells, an 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.

4.5 Modelling Recharge from a Large Water Body using the Model

In order to simulate the effect of leakage from the bed of a large water body, the term representing the leakage are added to the ground water flow equation, i.e., equation 4.16. The flow from any external source to the cell i,j,k, can be expressed in the term of $P_{i,j,k}$ and $Q_{i,j,k}$ as given by equation 4.14. Here, the external source is recharge occurring through the large water body. The bed of the large water body. The bed of the large water body may be discretized into a number of rectangular cells (fig. 4.6). The seepage from one of the such rectangular cell can be expressed as;



$$C = \frac{K\,L\,W}{D}$$

Fig. 4.6 Discretized bed of a Large Water Body

$$Q_S = \frac{K.L.W(H-Ha)}{D} \qquad \ldots(4.17)$$

where,

Q_S is the rate of seepage from the rectangular cell of the bottom of the large water body (L^3T^{-1});

K is the hydraulic conductivity of the bed material of water body contained in the rectangular cell (LT^{-1});

L is the length of the rectangular cell (L);

W is the width of the rectangular cell (L);

D is the thickness of ith rectangular cell (L);

H is the water level of the water body in the ith cell (L);

Ha is the head on the aquifer side of the water body (L).

The above equation can be expressed as;

$$Q_S = C(H - Ha) \qquad \ldots.(4.18)$$

where,

C is the conductance of the river bed (C = KLW/D).

The seepage Q_S depends upon the respective values of H and Ha. If Ha is below the bottom of the water body then Ha=Hbottom, (HBOTTOM being the elevation of the bottom of the water body). And the recharge to aquifer is given by;

$$Q_S = C\ (H - HBOTTOM\ ) \qquad \ldots.(4.19)$$

In this case the aquifer is recharged by the water body. If Ha is above the bottom but less than H, then the flow to the aquifer is given by eq. 4.18. If Ha>H, the flow to water body from the aquifer takes place. In this case the recharge to water body from aquifer is given by the following equation.

$$Q_S = C(Ha - H) \quad ....(4.20)$$

4.5.1 Discretization in Space

For the purpose of formulation of finite difference equations, the aquifer system defined in chapter 3, has been discretized in plan into 32 rows 32 columns by rectangular grid arrays and in the vertical into 3 layers forming 1024 cells. Variable grid spacing have ben assumed. Finner grids have been taken inside the water body and close to the water body while coarser grids have been taken away from the water body. Away from the centre of the water body the spacing of respective grids have been assumed same in both x and y directions. Fig.4.7 shows the space discretization, the top layer and the middle layer have been taken as 15m thick while bottom layer have been taken 20m thick (total 50m thickness of the aquifer has been modelled). The top layer has been considered unconfirmed with specific yield Sy as 0.15 and the middle and bottom layer confirmed with storage coefficient S as 0.01 and 0.001. An impermeable boundary has been assumed at the bottom. Fig. 3.1 shows the vertical discretization.

4.5.2 Time Discretization:

The total period of simulation have been taken to be 364 days consisting of 104 times steps. The head distribution and the volumetric budgets have been obtained at the end of every fourth time steps.

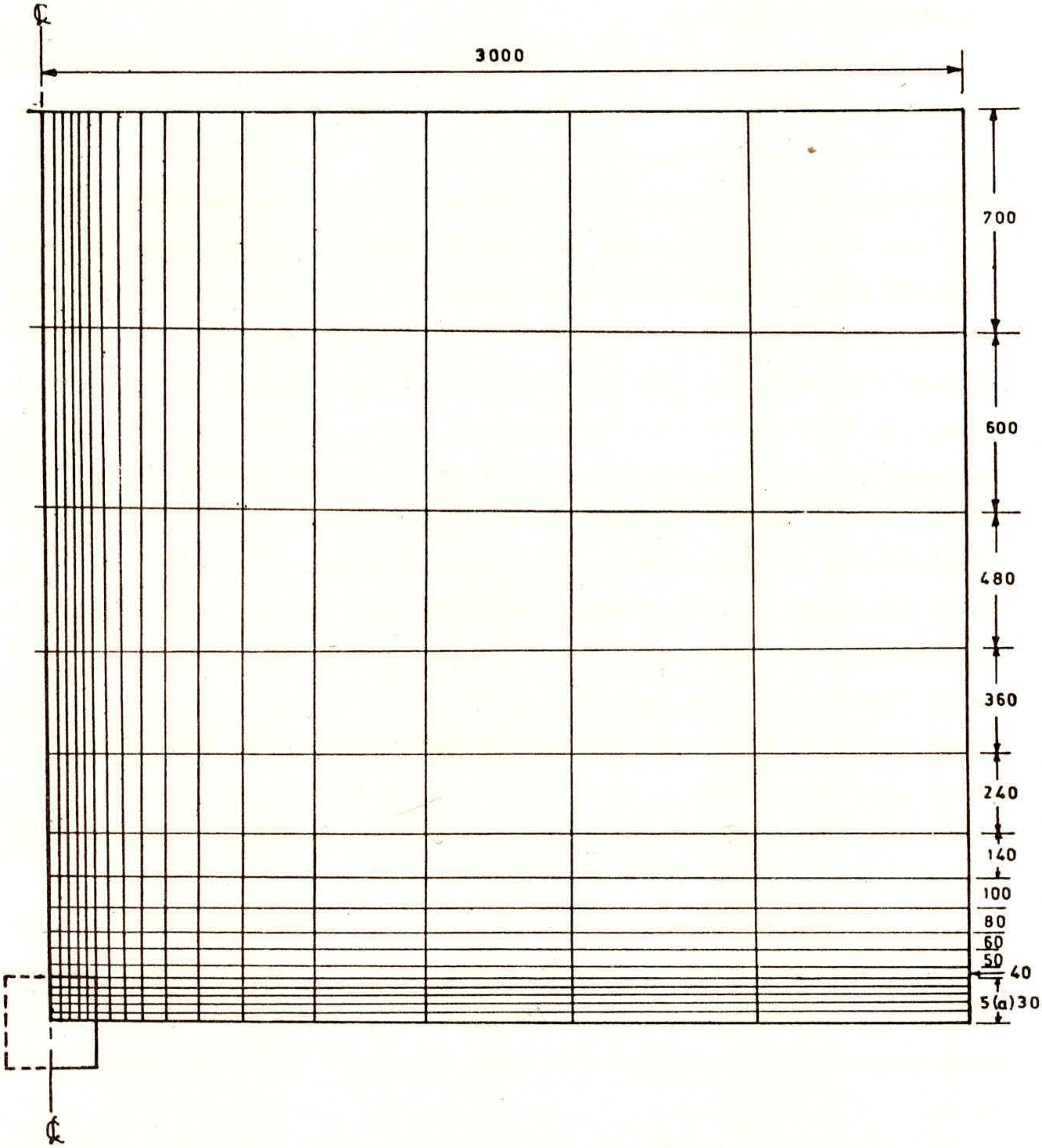


Fig. 4.7 Space Discretizaton

5.0 ANALYSIS OF RESULTS

For the hypothetical problem described earlier, simulation of transient flow from the water body to aquifer and subsequently to constant head boundary was carried out using a three dimensional finite difference groundwater flow model. The space and time discretization for model-runs have been discussed in the previous chapter. For each model-run, the head distribution in the aquifer system at discrete nodes at each time step over a period of time was obtained. The simulation was carried out for different values of head difference between the water level in the water body and the head in the constant head boundary, designed as ΔH. $\Delta H$was taken 3m,5m,7m, and 9m respectively for different simulations. The head in constant head boundary was kept constant while the water level in the water body was varied for different simulations. The hydraulic conductivity of the aquifer was assumed to be 10m/d while the specific yield was taken as 0.15. For a fixed value of K_h/K_v the simulation was carried out for different of ΔH. Different values of K_h/K_v, i.e., 1,10,100,250,500,750 were taken and the simulations were repeated for different values of ΔH.

All the above simulation runs were repeated for another size of the water body. In each simulation the difference between the head at discrete points (nodes) and the head in the constant head boundary was computed for each time-step and was designated as Δh.

Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t was observed for all values of t and for each simulation (ΔH = 3.0m,5.0m,7.0m and 9.0m; K_h/K_v=1,10,100,250,500,750; l/L=0.102 and 0.128; where l is the dimension of the side of the water body, X is the perpendicular distance from the lake to the observation well and L is the perpendicular distance between centre of the water body to the constant head boundary). Δh_s is the steady state value of Δh. The values of X/L and X for present discretization is given in Table 5.1. Analysis of the above plots show that with increasing time, the maxima of the curve shift in a positive X-direction and shift of the maxima for all value of t is ranged between X/L = 0.15 to 0.25 (this range of X/L was found to be the same for different values of ΔH, K_h/K_v and l/L).

Table 5.1

Values of X and X/L

Sl.No.	X(in meters)	X/L
1.	15	0.0056
2.	45	0.017
3.	75	0.028
4.	105	0.039
5.	135	0.051
6.	170	0.064
7.	215	0.08
8.	270	0.101
9.	340	0.128
10.	430	0.162
11.	550	0.207
12.	740	0.279
13.	1040	0.392
14.	1460	0.551
15.	2000	0.755
16.	2650	1.00

Figs. 5.1, 5.2 and 5.3 show such plots for K_h/K_v = 1, 100 and 500 respectively (the value of l/L is 0.102 for each plots)

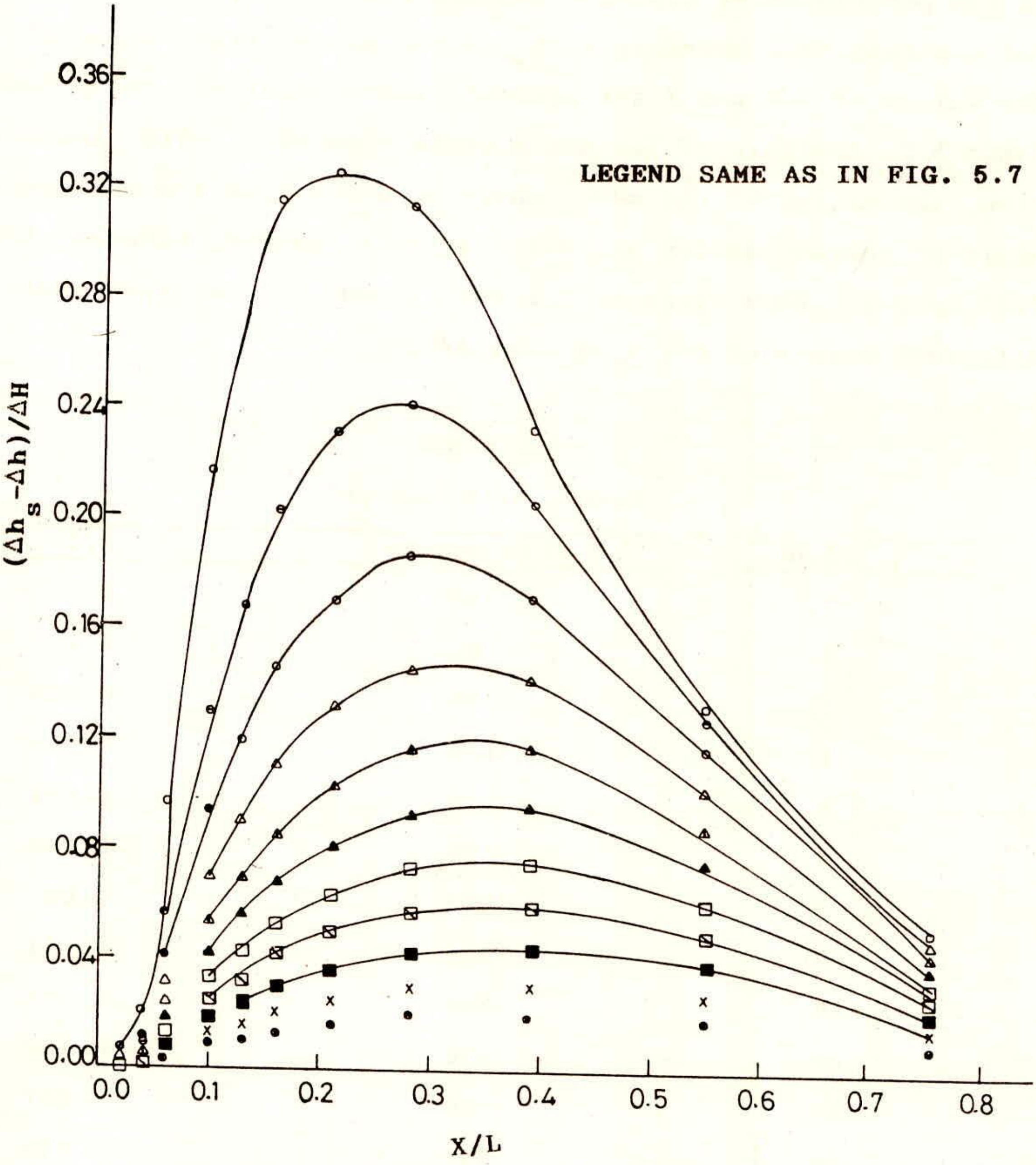


Fig. 5.1 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t (K_h/K_v=1 and l/L=0.102)

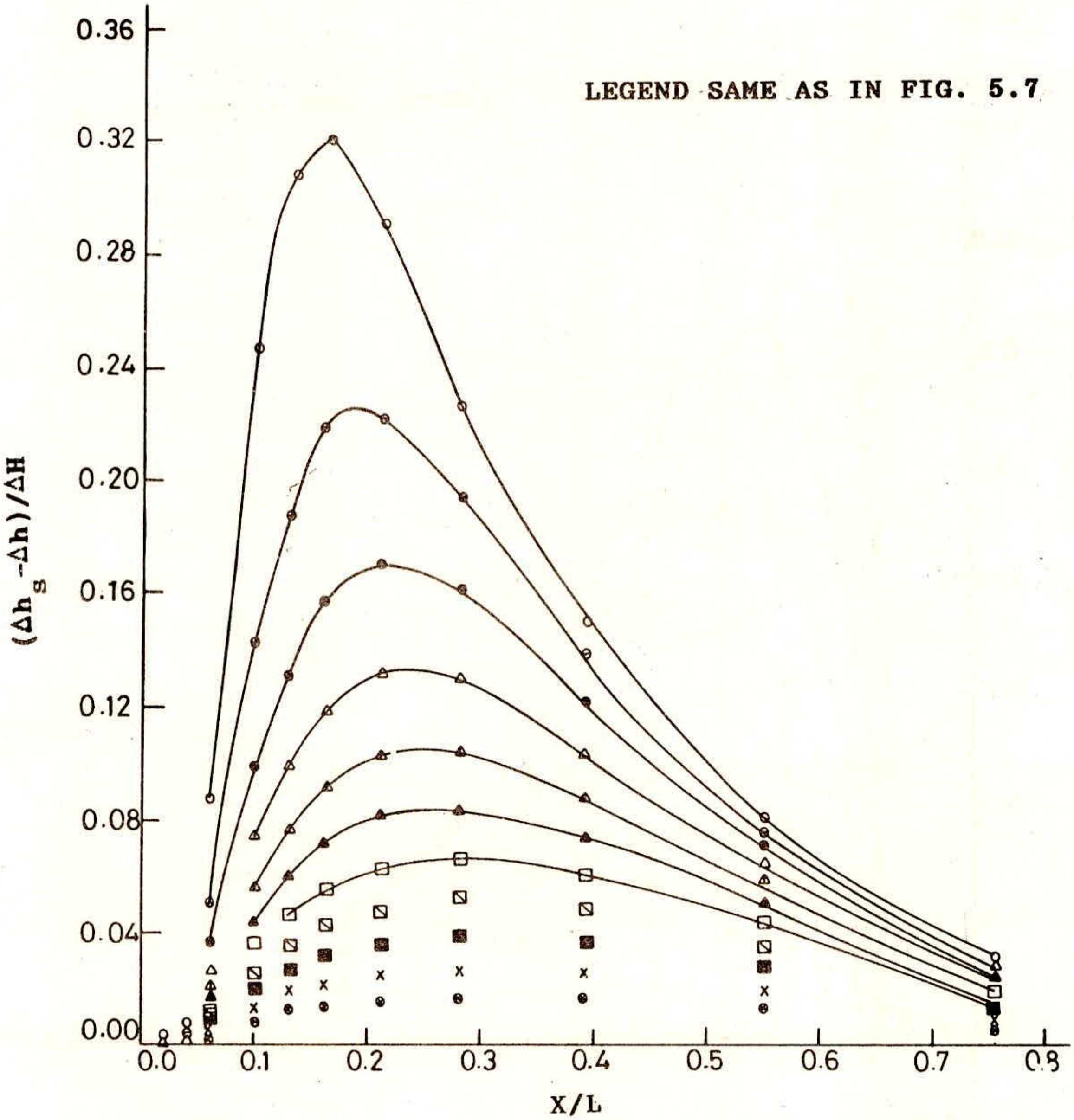


Fig. 5.2 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t

(K_h/K_v=100 and 1/L=0.102)

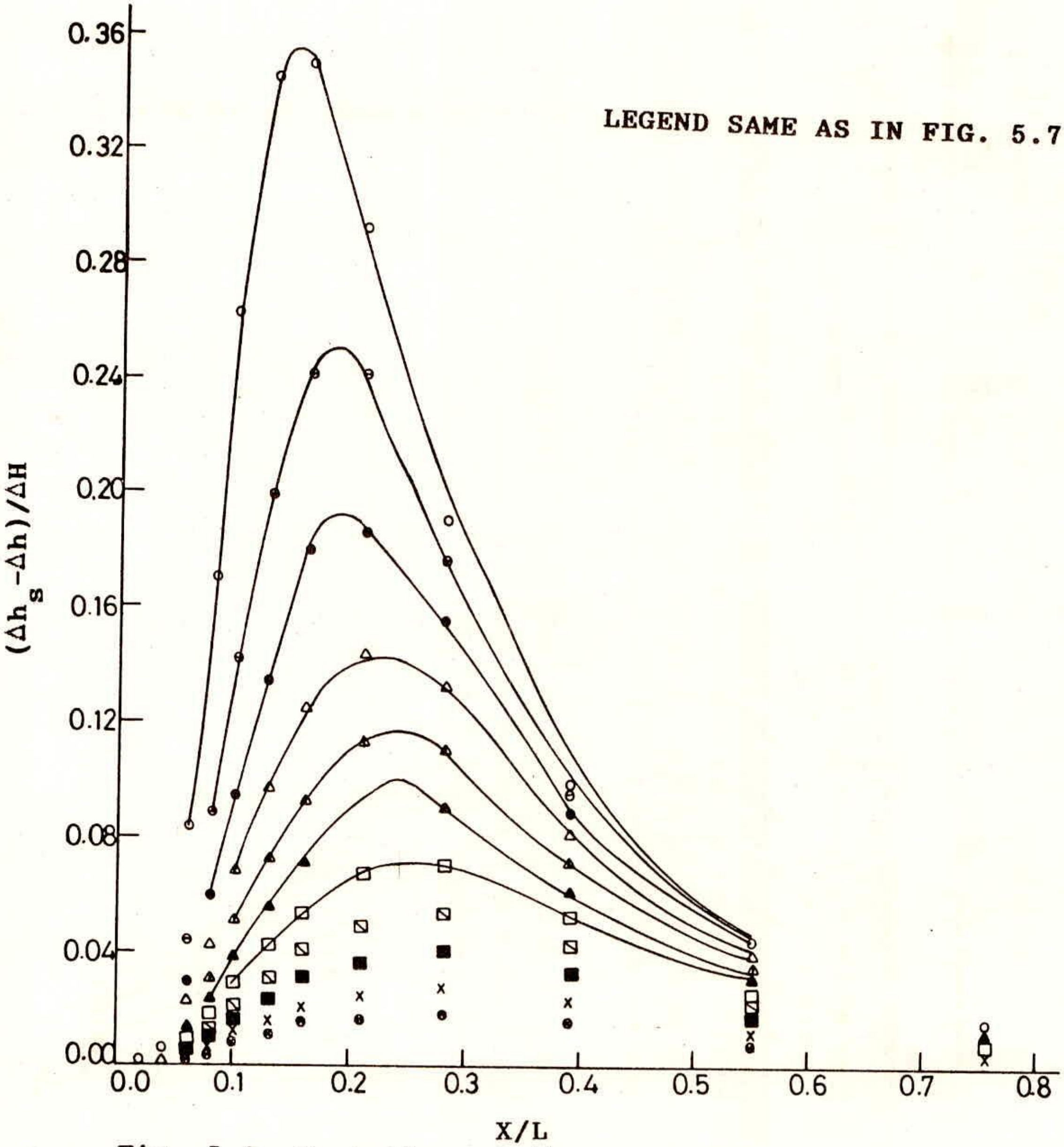


Fig. 5.3 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t

$(K_h/K_v=500$ and $l/L=0.102)$

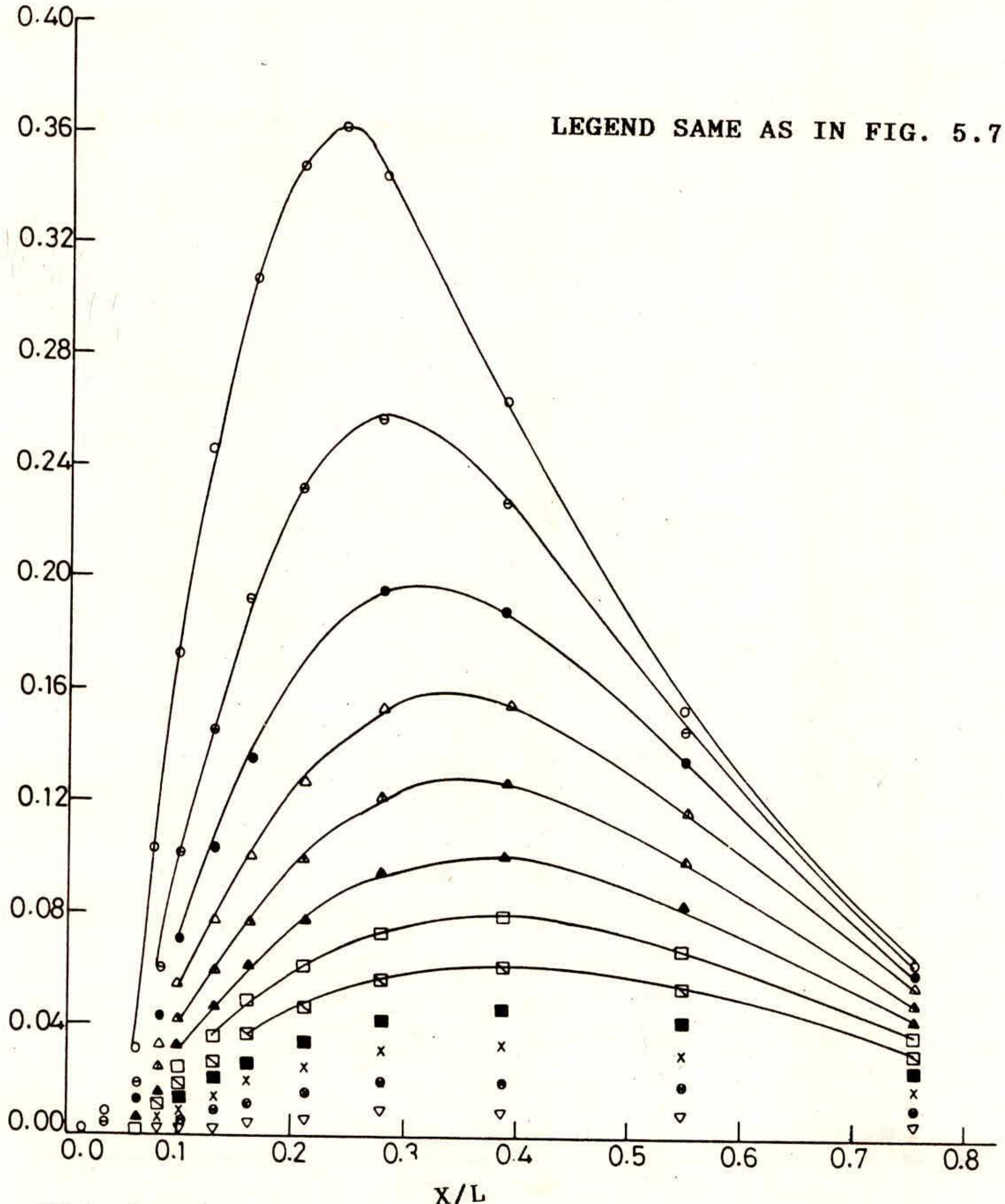


Fig. 5.4 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t_o

$(K_h/K_v=1$ and $l/L=0.128)$

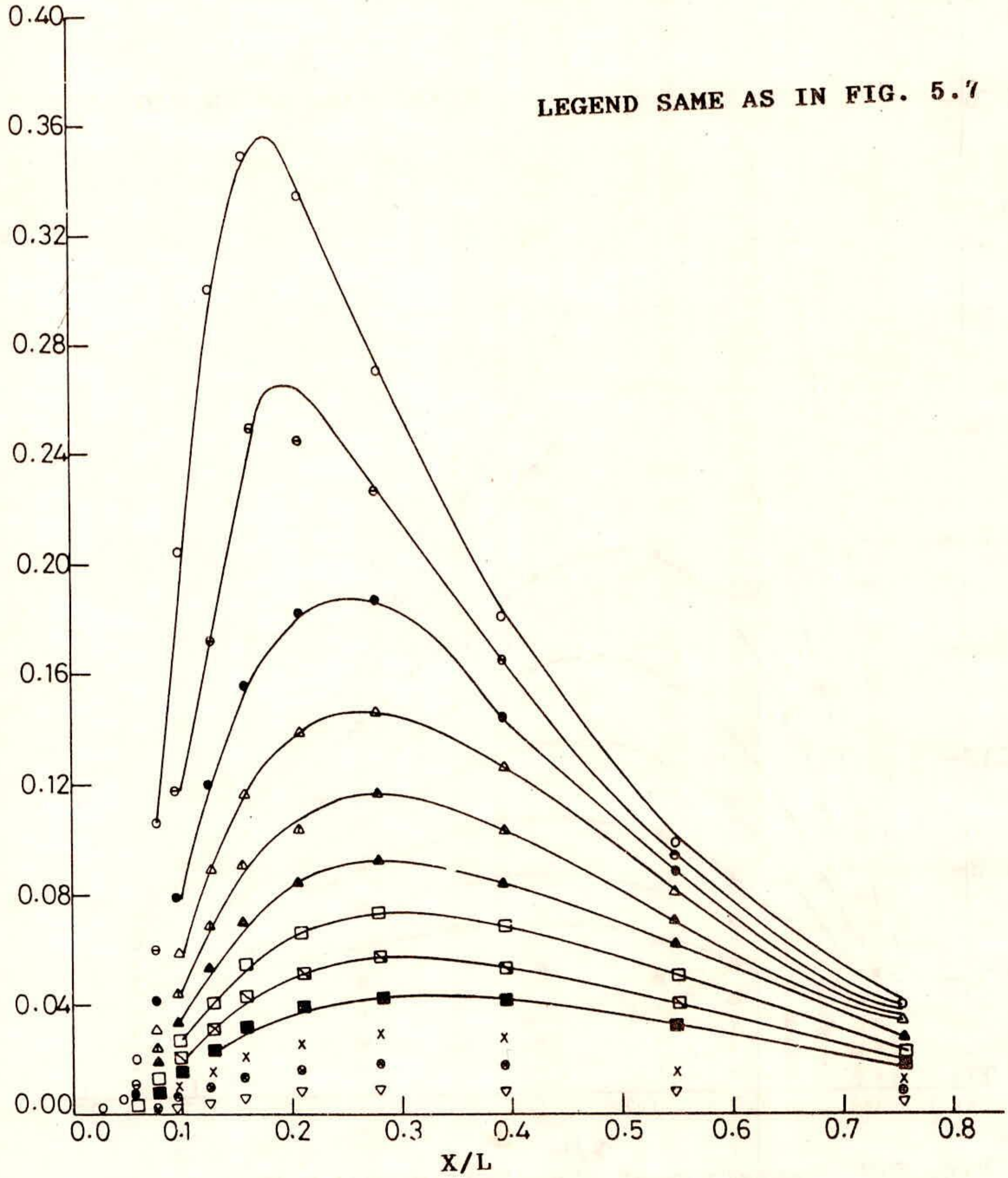


Fig. 5.5 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t

$(K_h/K_v = 100$ and $l/L = 0.128)$

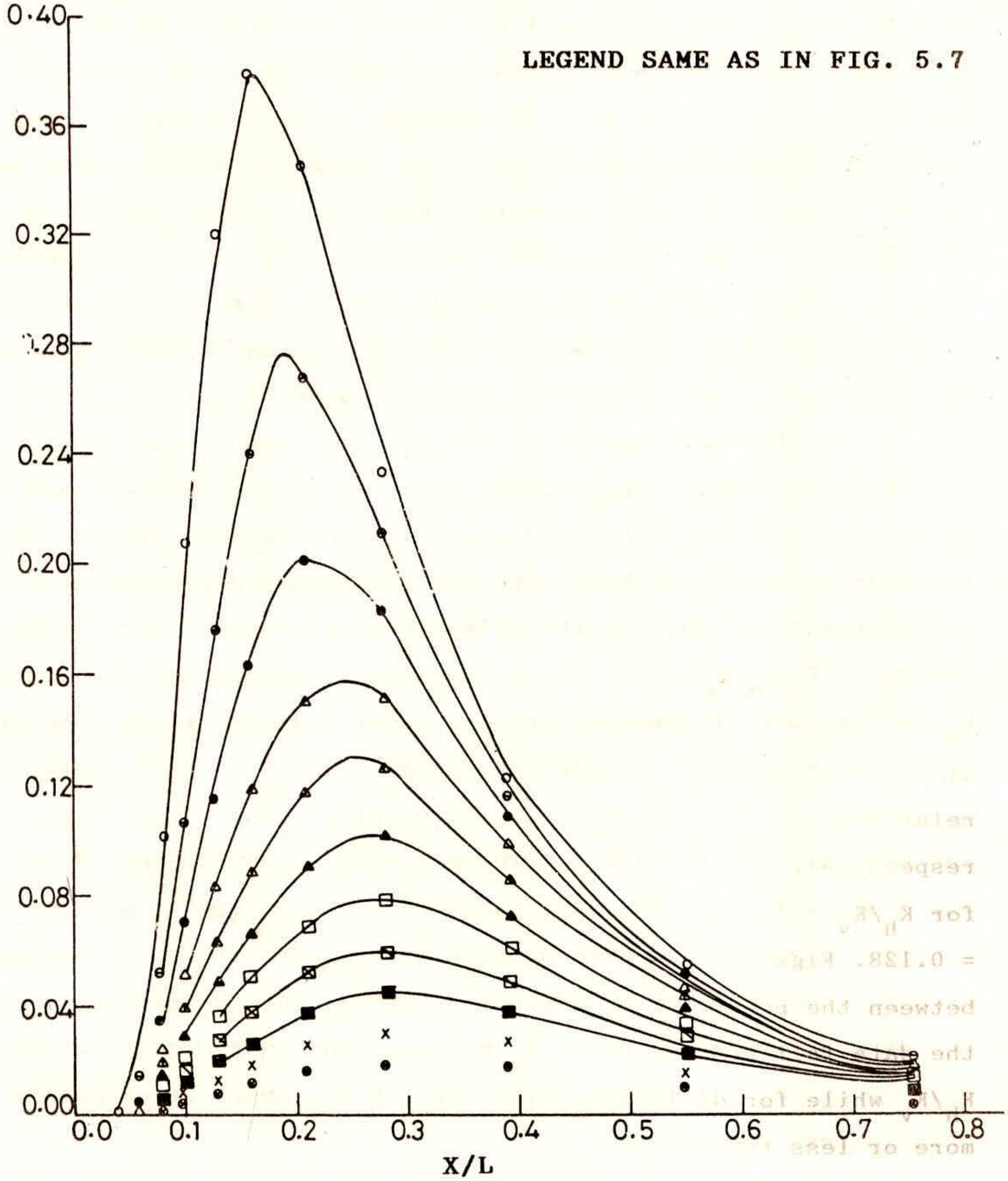


Fig. 5.6 Variation of $(\Delta h_s - \Delta h)/\Delta H$ with X/L and t

(K_h/K_v=500 and l/L=0.128)

and Figs. 5.3, 5.4 and 5.5 are for K_h/K_v = 1, 100 and 500 respectively; each plot has l/L = 0.128. Same range of X/L, i.e., 0.15 to 0.25 has also been reported by the Singh and Seethapathi(1988) in a separate similar study. Hence, it may be said that if an observation well is located within the above range, it will observe a comparatively rapid change of head and thus it will be more sensitive. Therefore, it can be concluded that the location of the observation well within X/L of 0.15 to 0.25 will be ideal as it will observe rapid head change and thus the observation recorded will be less liable to errors.

The further analysis of results has shown that the parameters $X^2S/T.t$ and $T\Delta h/Q_R$ are uniquely related and the relation was found to be the same for different values of ΔH (similar relation has been observed by Singh and Seethapathi(1986) in a separate study) but the relation is different for different values of K_h/K_v and l/L. Here, T is transmissivity of the aquifer, Q_R is the rate of seepage from the water body and t is the time since the start of simulation. Figs. 5.7 to 5.12 show such relationships for K_h/K_v = 1, 10, 100, 250, 500 and 750 respectively and for l/L = 0.102 and Figs. 5.13 through 5.18 are for K_h/K_v = 1, 10, 100, 250, 500 and 750 respectively and for l/L = 0.128. Figs. 5.7 to 5.18 shows similar trend of data agreement between the parameters $X^2S/T.t$ and $T.\Delta h/Q_R$ but the curves on which the data fall are not exactly the same for different values of K_h/K_v while for different values of l/L the above relationship is more or less the same.

The curves beetween $X^2S/T.t$ and $T\Delta h/Q_R$ for a fixed value of l/L and different values of K_h/K_v , show that shifting the axes of the graphs for $K_h/K_v>1.0$, all the curves merge together. Only early part of data show poor merging. Similarly for l/L = 0.128

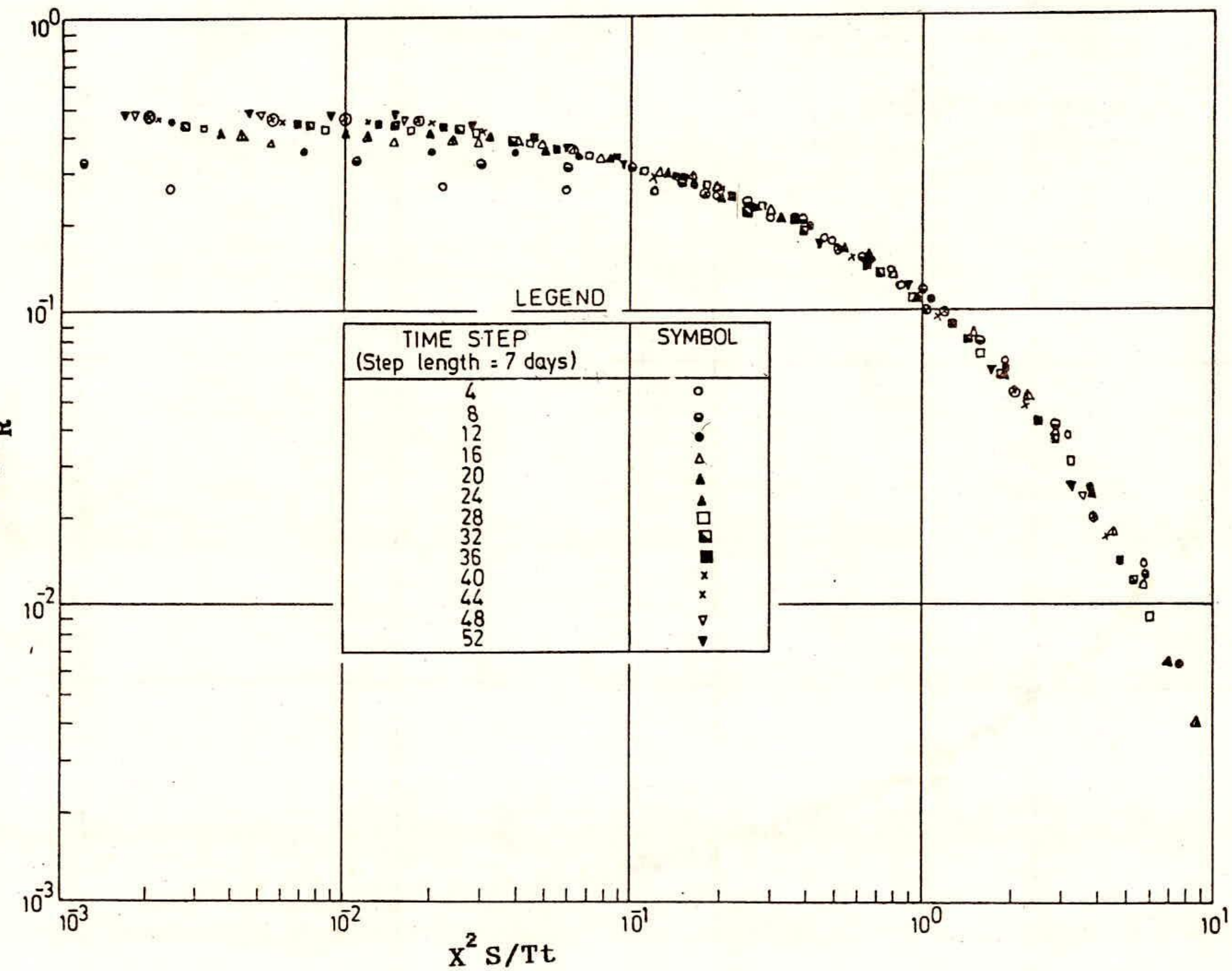


Fig. 5.7 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v =1 and 1/L=0.128)

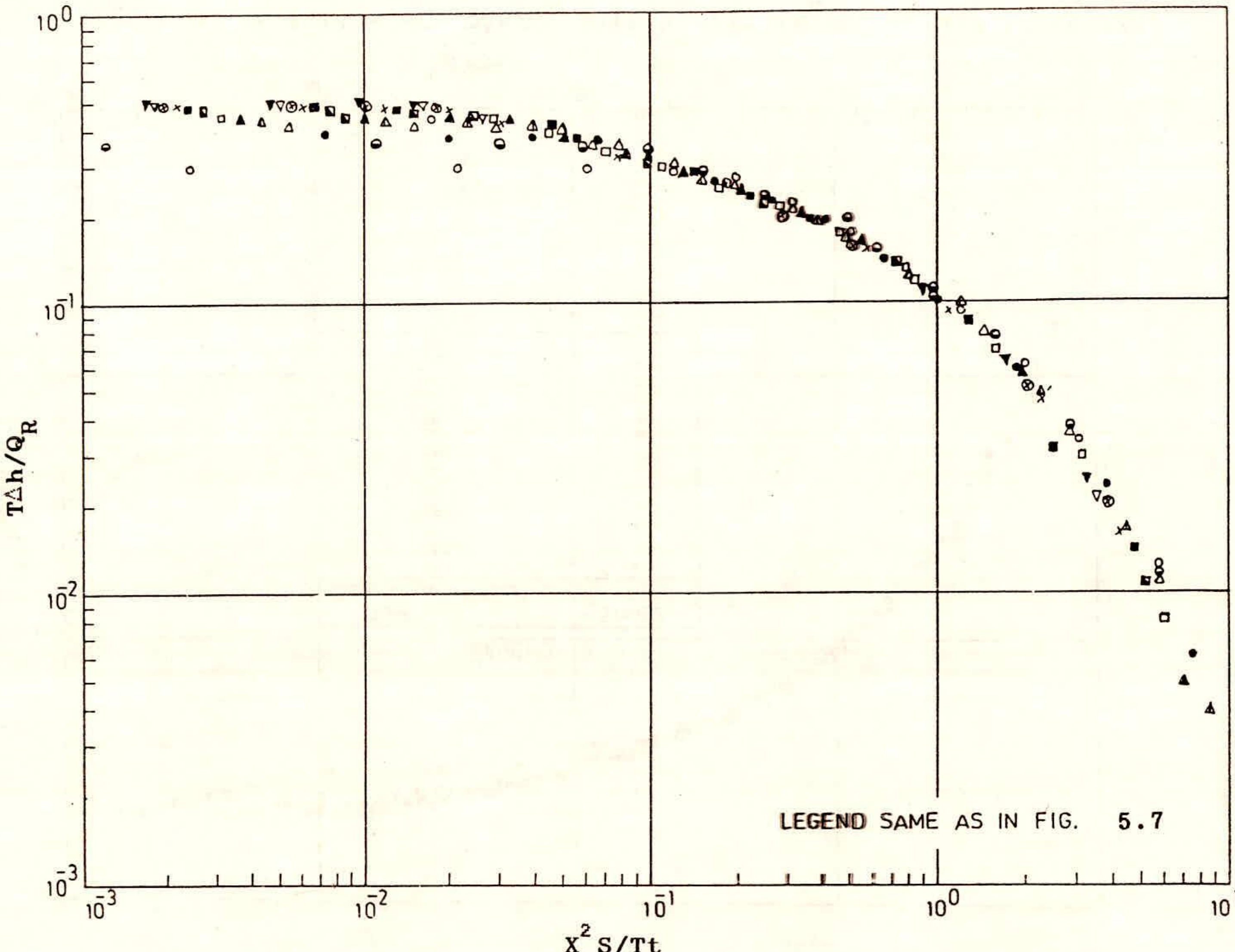


Fig. 5.8 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=10 and l/L=0.128)

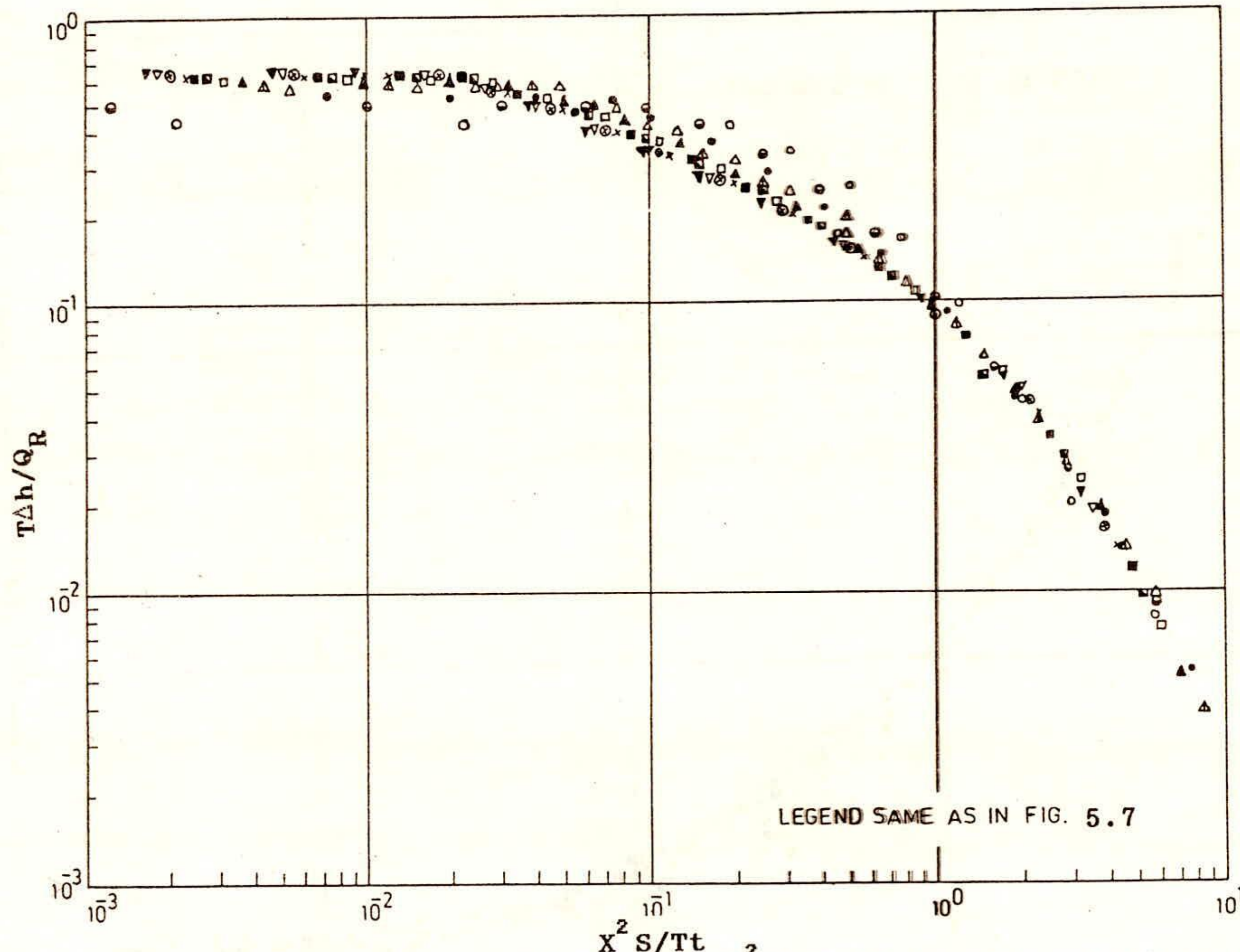


Fig. 5.9 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=100 and 1/L=0.128)

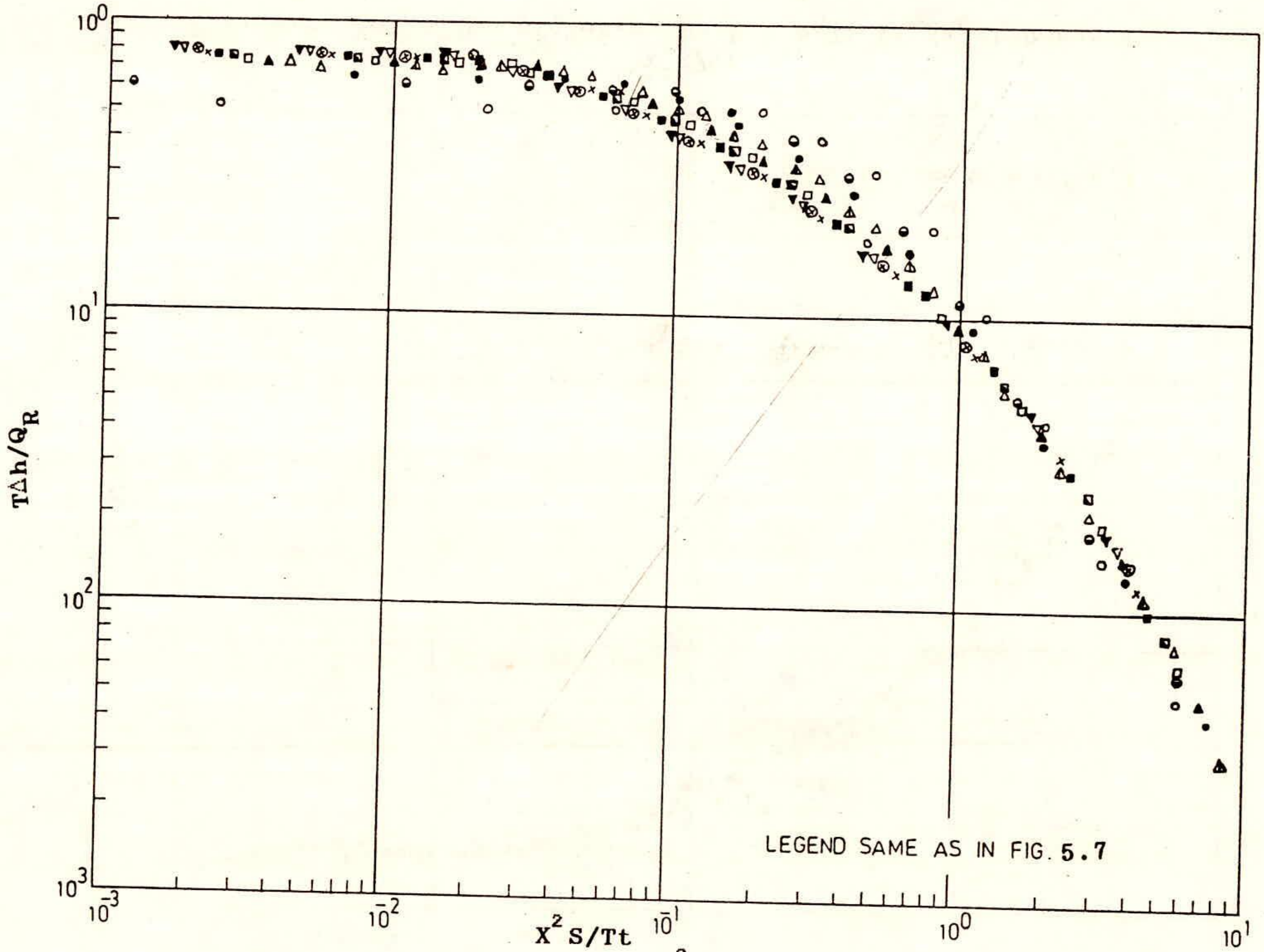


Fig. 5.10 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=250 and 1/L=0.128)

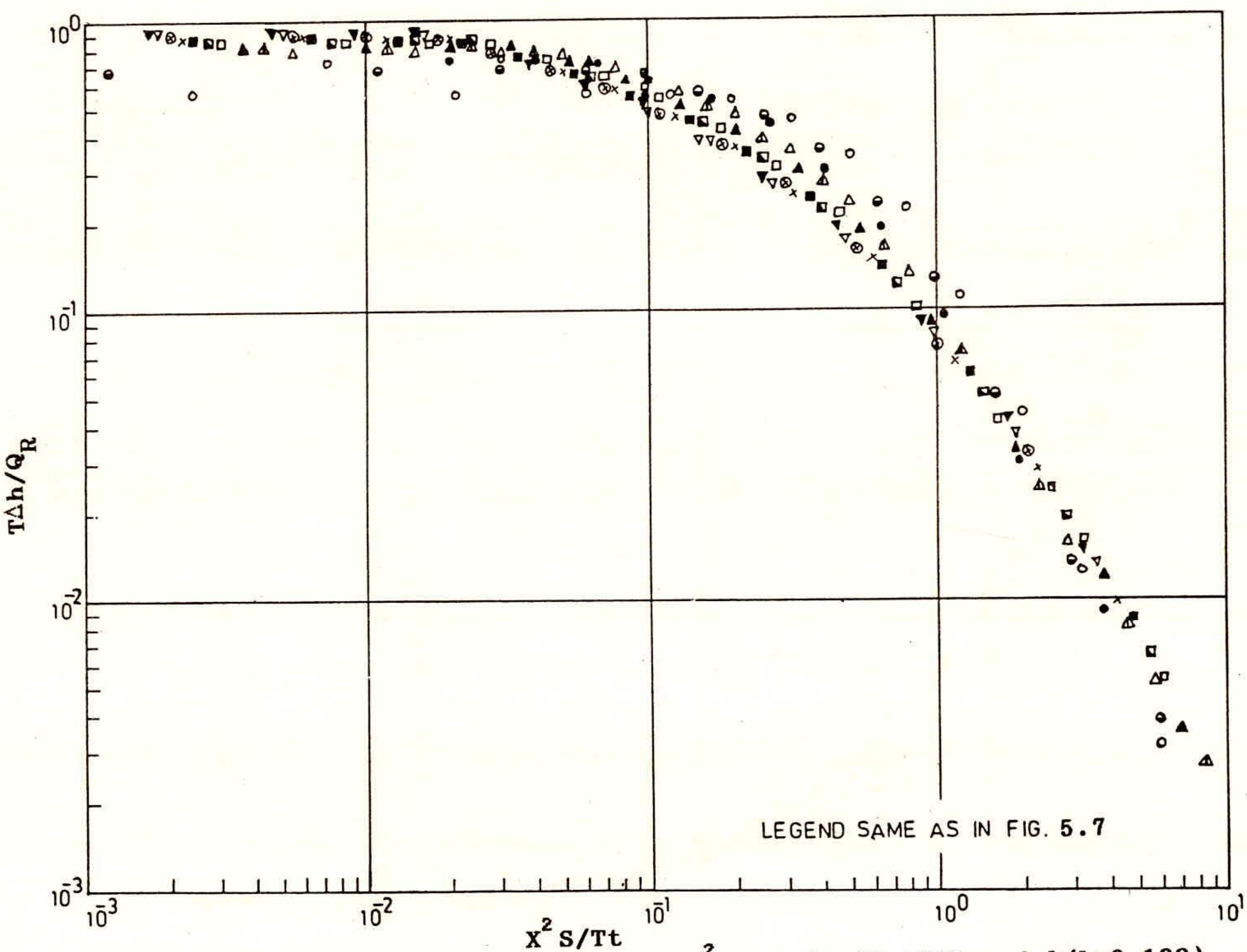


Fig. 5.11 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=500 and l/L=0.128)

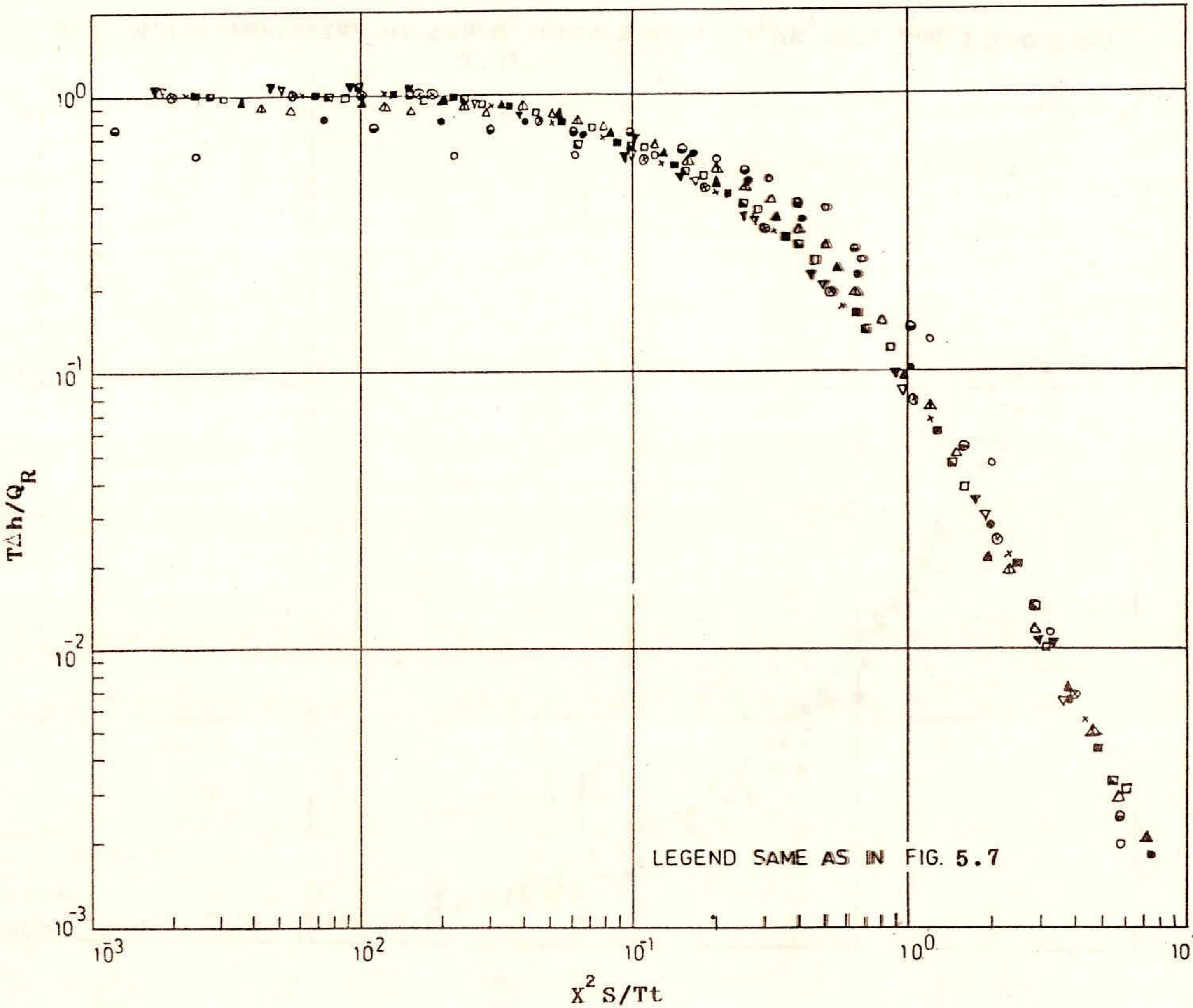


Fig. 5.12 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=750 and l/L=0.128)

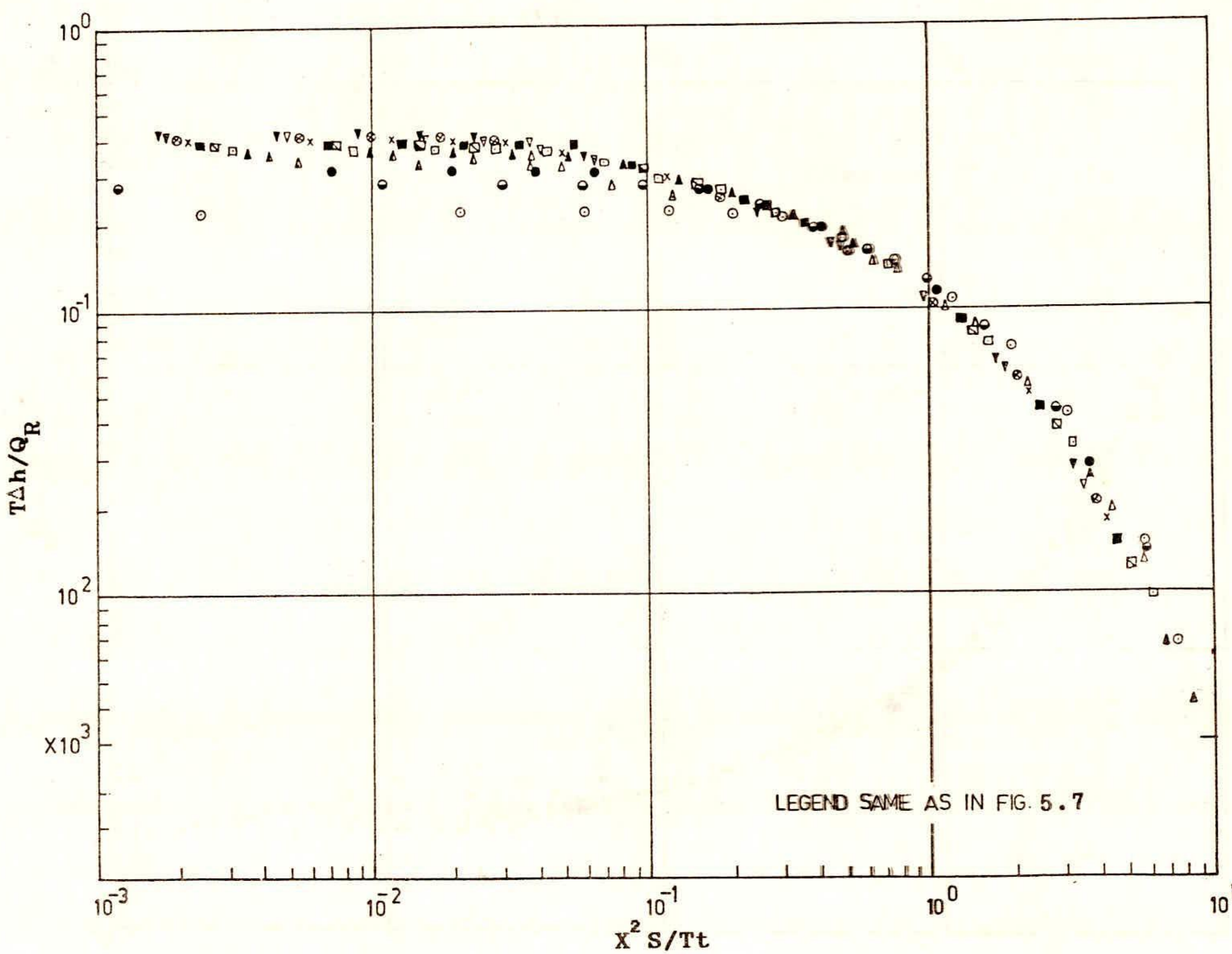


Fig. 5.13 Variation of TΔh/Q_R with $X^2 S/Tt$ (K_h/K_v =1 and 1/L=0.102)

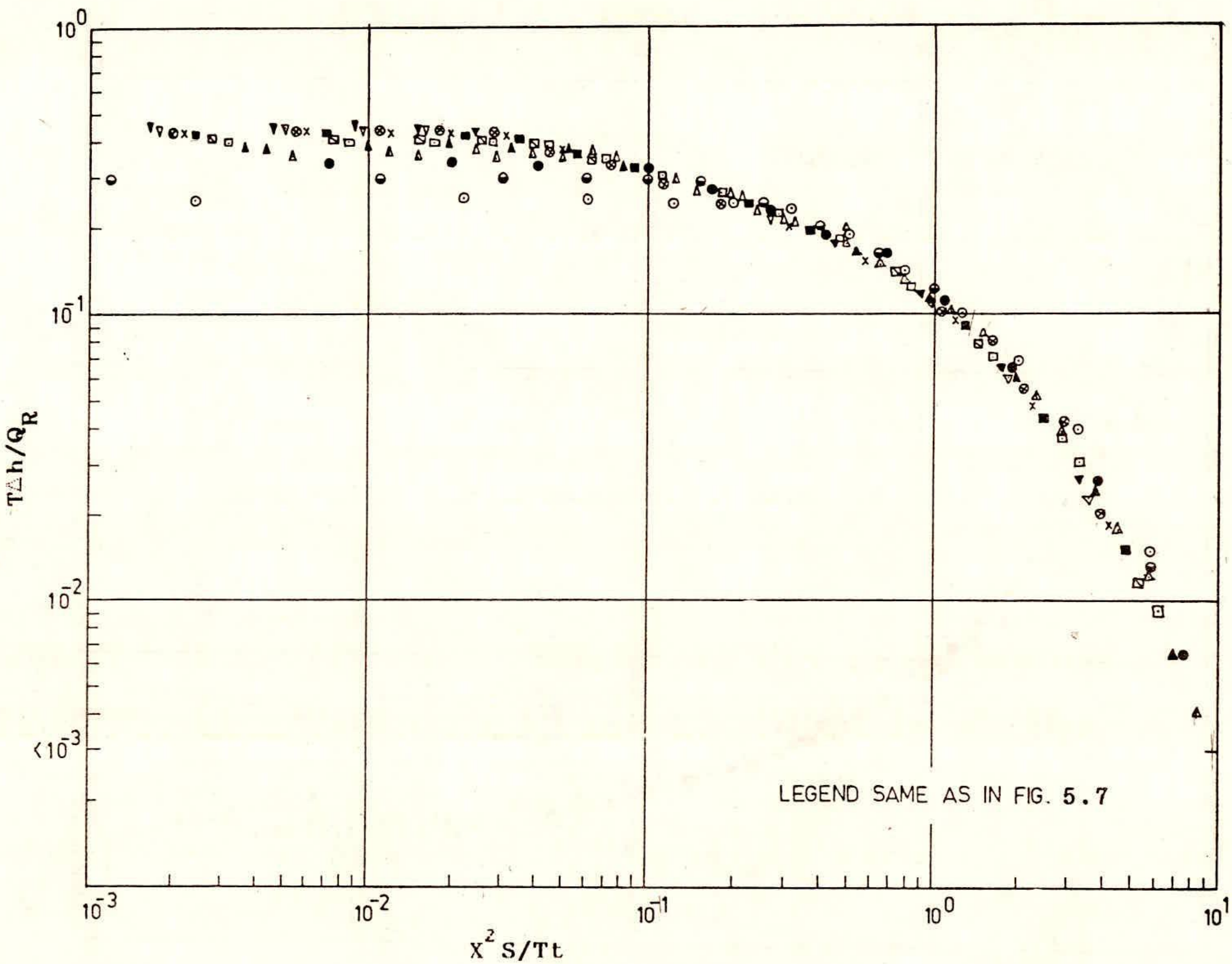


Fig. 5.14 Variation of $T\Delta h/Q_R$ with $X^2 S/Tt$ ($K_h/K_v=10$ and $l/L=0.102$)

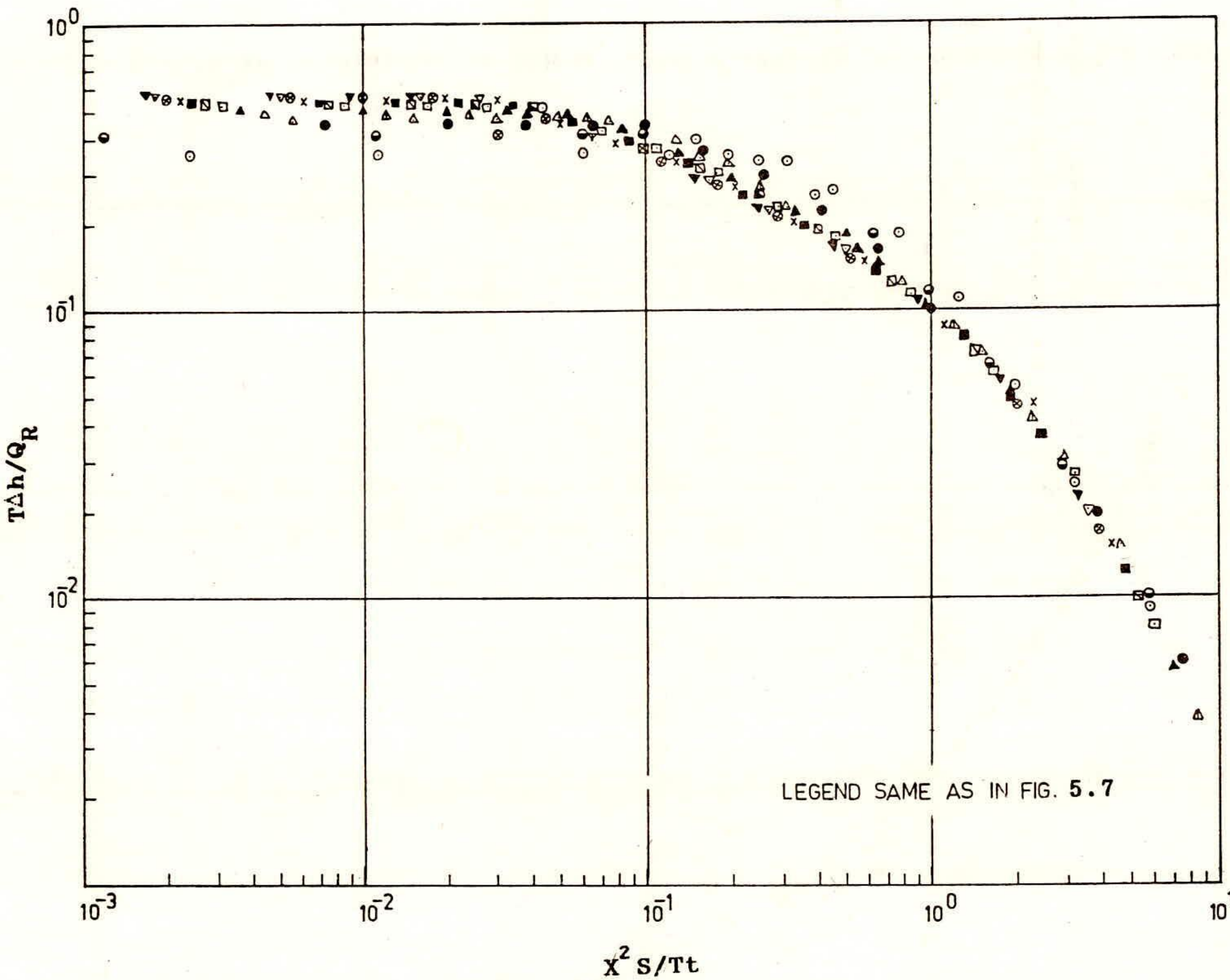


Fig. 5.15 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=100 and l/L=0.102)

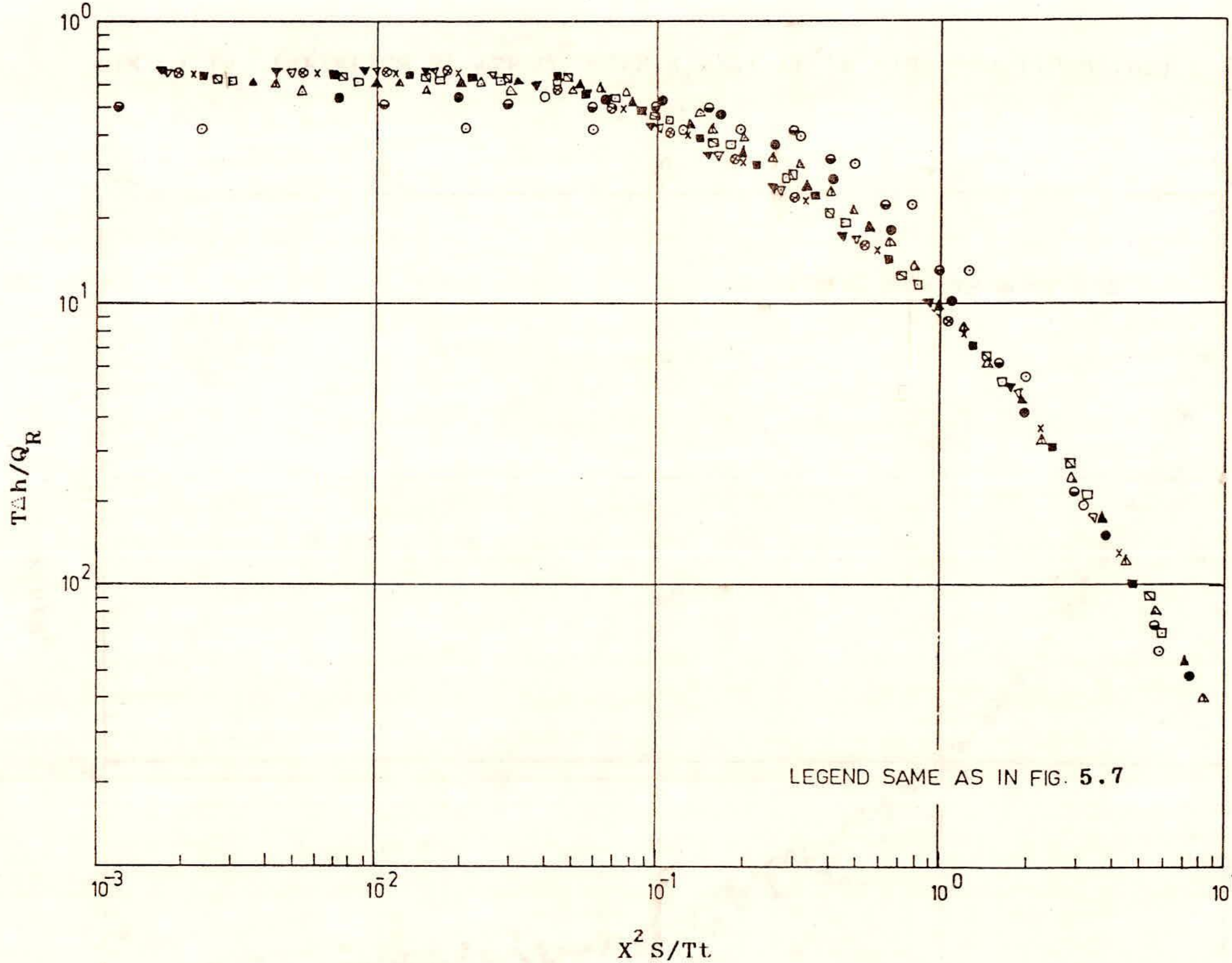


Fig. 5.16 Variation of $T\Delta h/Q_R$ with $X^2 S/Tt$ (K_h/K_v=250 and l/L=0.102)

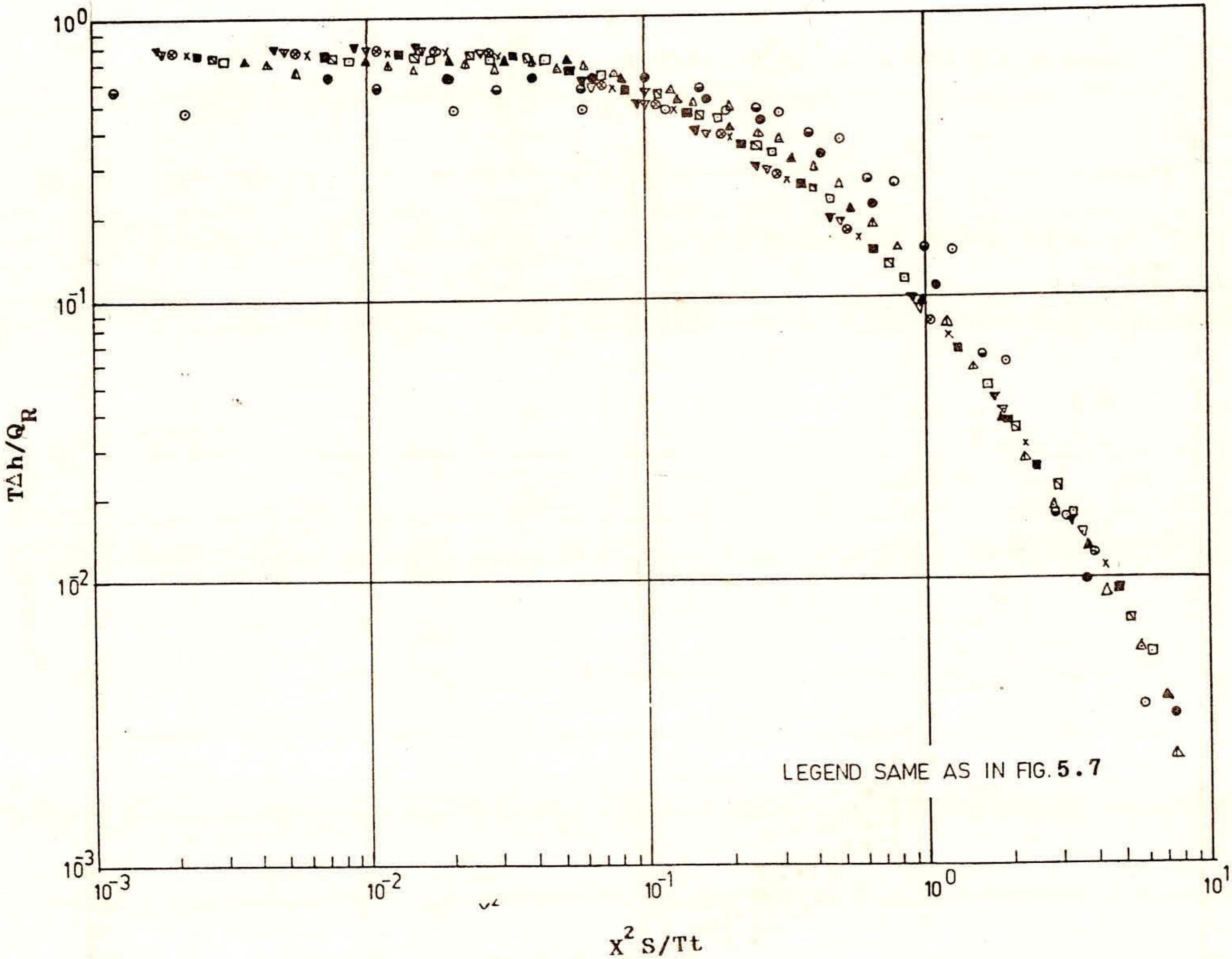


Fig. 5.17 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=500 and l/L=0.102)

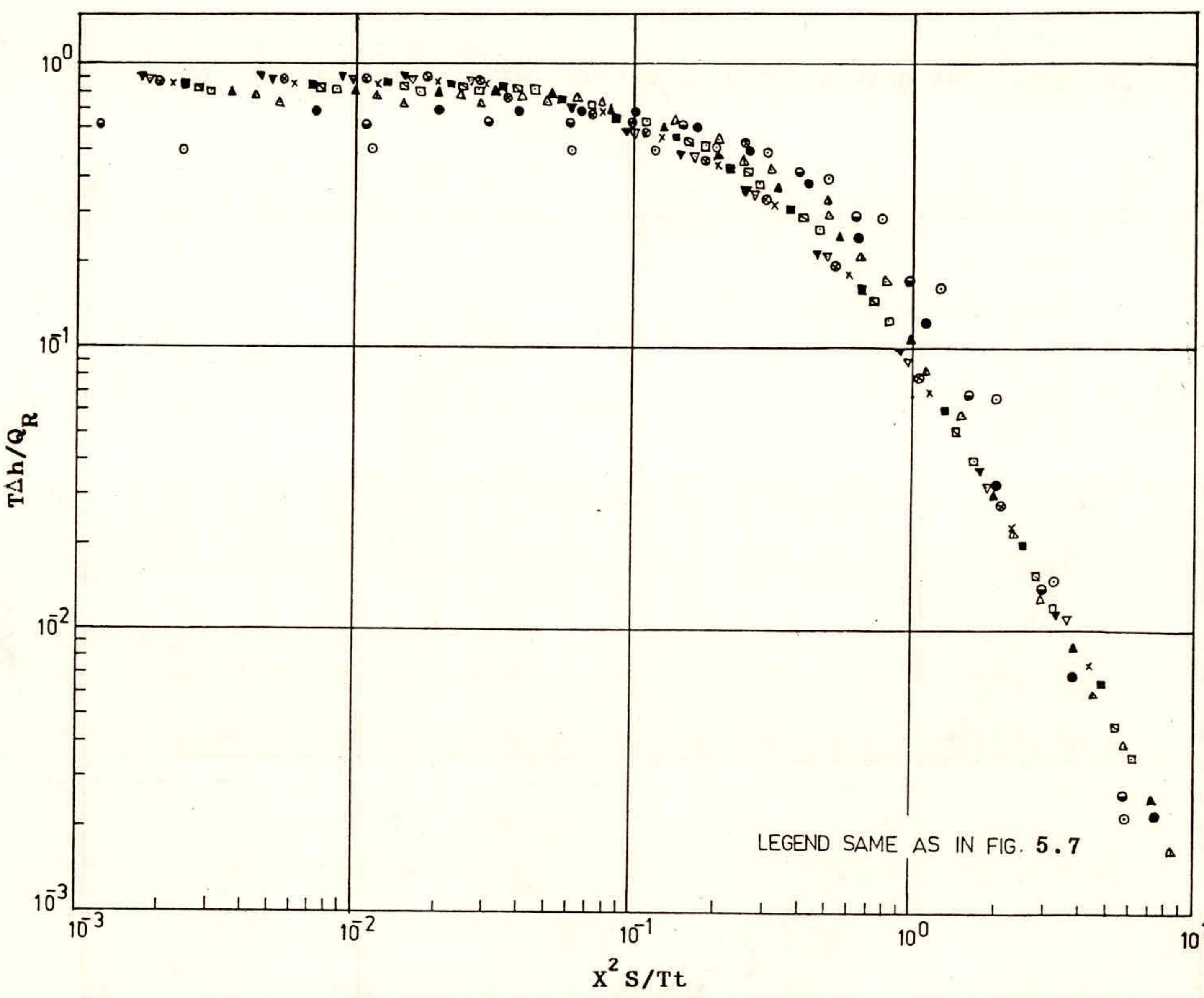


Fig. 5.18 Variation of $T\Delta h/Q_R$ with X^2S/Tt (K_h/K_v=750 and 1/L=0.102)

each of the curves for $K_h/K_v > 1.0$ with reasonable shift of axes reduces to the curve of agreement for $K_h/K_v = 1.0$. The above mentioned shifts in X and Y axis is the same in magnitude but opposite in direction (shift in X axis being in positive direction and that in Y axis being in negative direction). If the value of shift for any of the above curve is C, the, plot between the parameters $X^2 S/T.t\ .C$ and $T\Delta h/QR\ .1/C$ for that data set will fall on the curve between the parameters $X^2 S/T.t$ and $T.\Delta h/QR$ for $K_h/K_v = 1.0$. After determining the values of C for each curve if we plot between the parameters $(X^2 S/T.t)C$ and $(T.\Delta h/QR)/C$, the data from all sets, i.e., for different values of K_h/K_v, follow a unique curve, provided the data having $X^2 S/T.t < 10^{-2}$ is neglected. The values C for different values of K_h/K_v and 1/L is given in Table 5.2.

Table 5.2

Values of C for Different Values of K_h/K_v and 1/L

Sl.No.	1/L	K_h/K_v	C
1.	0.102	1	1.00
		10	1.05
		100	1.30
		250	1.70
		500	2.60
		750	3.20
2.	0.128	1	1.00
		10	1.05
		100	1.30
		250	1.70
		500	1.70
		750	3.20

Thus, we see that the value of C is independent of 1/L. Figs. 5.19 and 5.20 show the relationship between the parameters $(T.\Delta h/Q_R)/C$ and $C(X^2S/T.t)$ for 1/L=0.102 and 0.128 respectively. The curve of agreement (mean lines) for Figs. 5.19 and 5.20 are found to be same. Therefore, it can be concluded that the parameters $(T.\Delta h/Q_R)/C$ and $C(X^2S/T.t)$ are uniquely related irrespective of the values of K_h/K_v, 1/L and ΔH (the relation is shown in fig. 5.19 and fig. 5.20).

The variation of C with K_h/K_v is given in fig.5.21 which shows that C is linearly related to K_h/K_v, and can be represented by the following equation.

$$C = 0.997 + 2.74\times10^{-3}(K_h/K_v) \quad (5.1)$$

5.1 Procedure for Determining Recharge from Large Water Bodies

Data required are:

a) Aquifer Parameters T and S,

b) Anisotropy Factor K_h/K_v,

c) record of head changes in an observation well.

1. Calculate the value of C from equation 5.1 knowing the value of K_h/K_v.
2. Calculate the value of the parameter $C(X^2S/T.t)$, (values of S. T,X and t are known and the value of C is calculated at step 1).
3. Knowing the value of the parameter $C(X^2S/T.t)$find the corresponding value of the parameter $(T.\Delta h/Q_R)/C$ from the curve given in fig. 5.19 or in fig. 5.20.
4. Once the value of the parameter $(T.\Delta h/Q_R)/C$ is known, the rate of recharge from the water body, i.e., $Q_R$can be determined knowing the value of T, Δh and C.
5. Check for $X^2.S/T.t < 10^{-2}$.

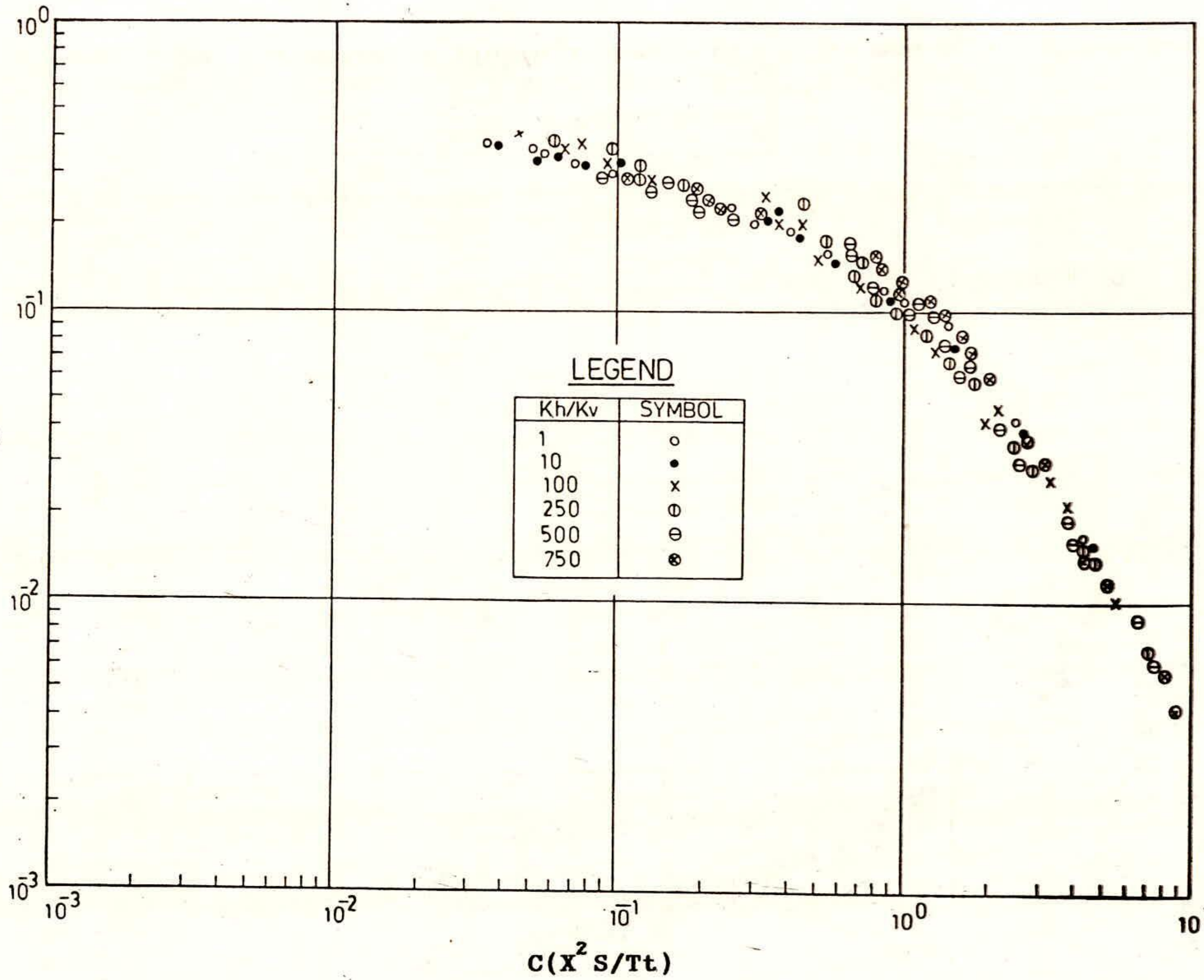


Fig. 5.19 Variation of $(T\Delta h/Q_R)/C$ with $C(X^2S/Tt)$ and K_h/K_v (1/L=0.128)

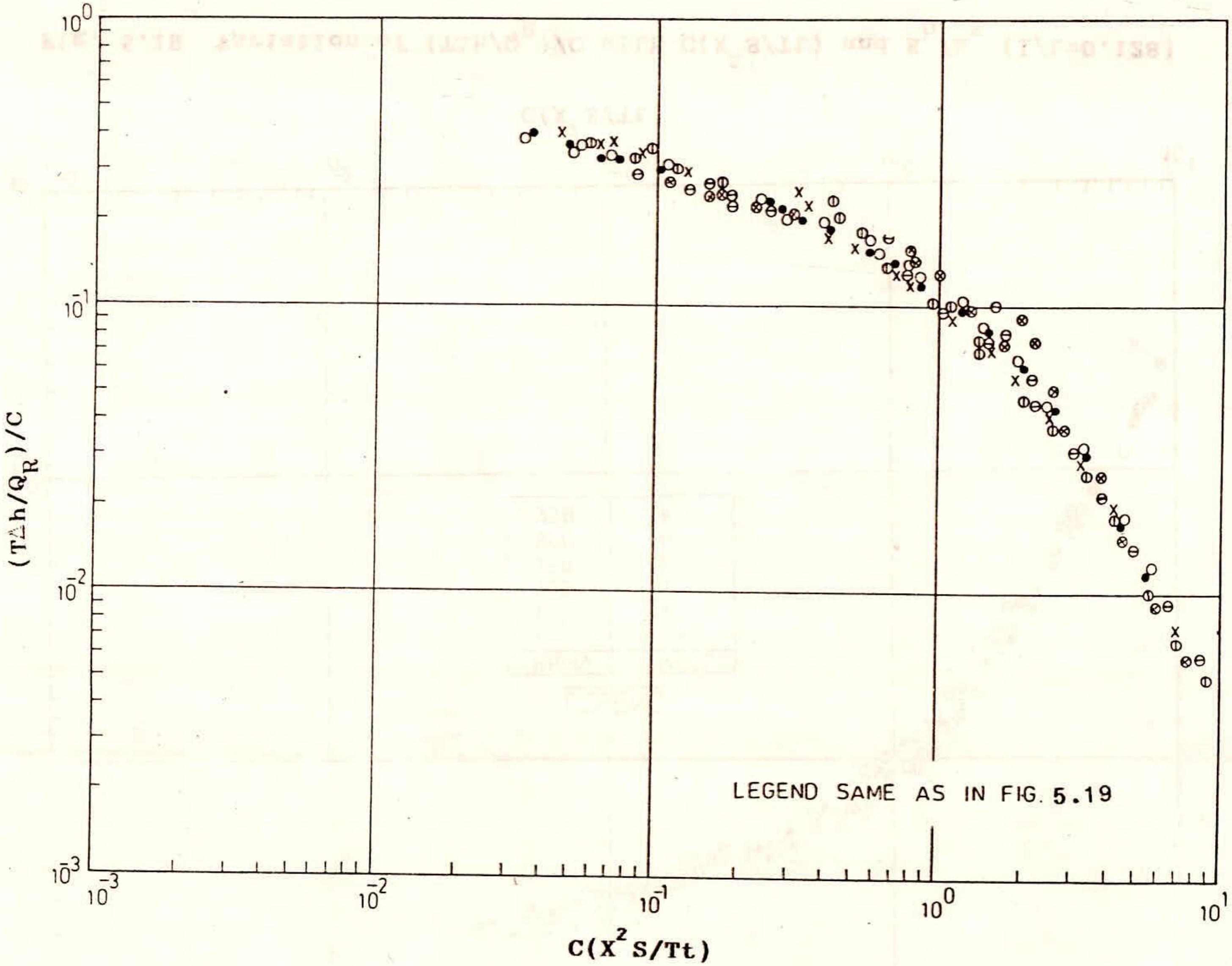


Fig. 5.20 Variation of $(T\Delta h/Q_R)/C$ with $C(X^2S/Tt)$ and K_h/K_v $(1/L=0.102)$

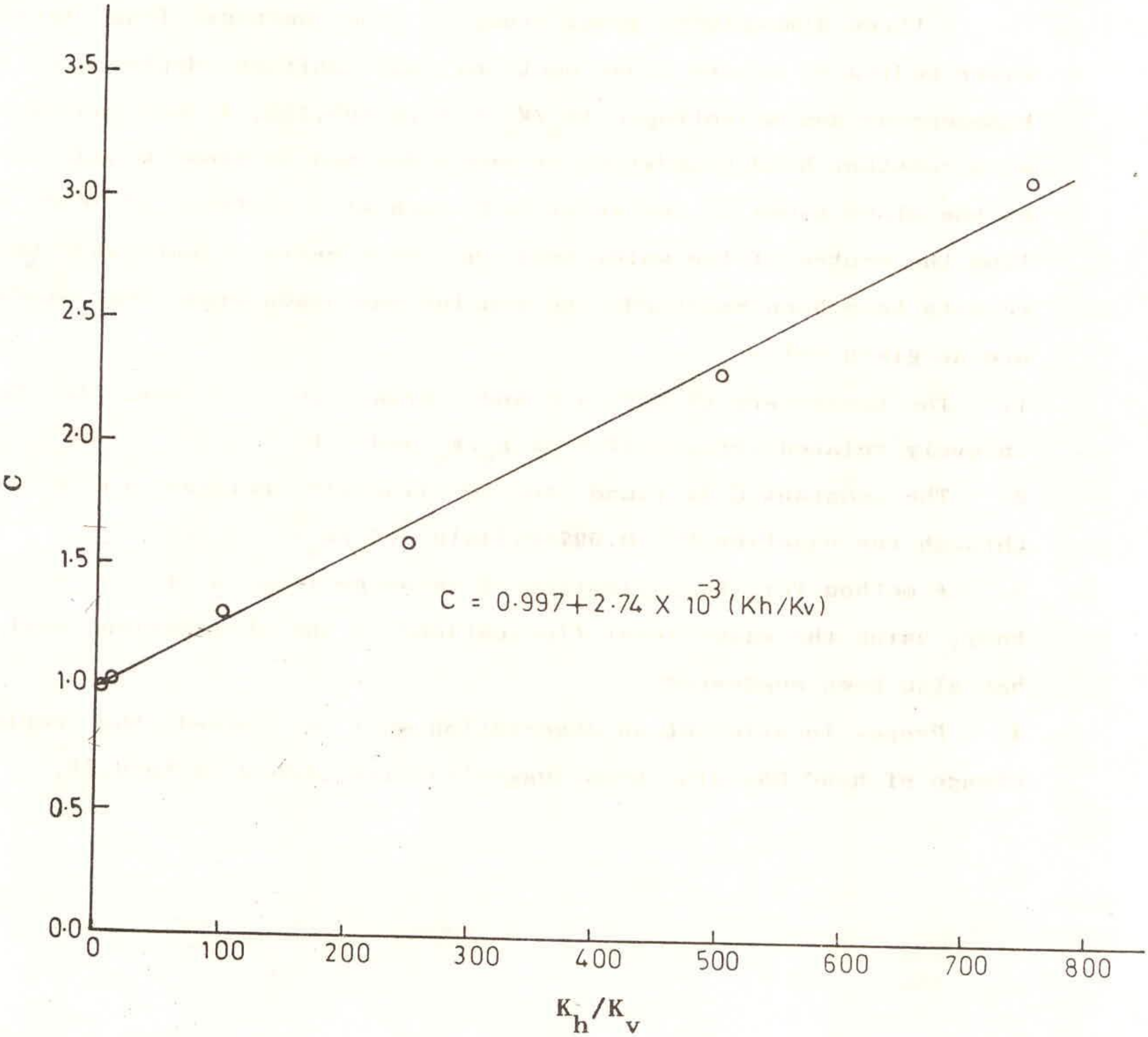


Fig. 5.21 Variation of C with K_h/K_v

6.0 CONCLUSIONS

A three dimensional model study of the seepage from large water bodies of square cross sections and uniform depths in a homogeneous and anisotropic (K_h/K_v = 1,10,100,250, & 750) aquifer with constant head boundaries on one sides and no flow boundaries on the other sides of the water body each at a distance of 2650 m from the centre of the water body, has been carried out and the results have been analysed. The conclusions drawn from the study are as given below:

1. The parameters $(X^2S/T.t).C$ and $T\Delta h/QR\ .1/C$ are found to be uniquely related irrespective of K_h/K_v and 1/L.
2. The constant C is found to be linearly related to K_h/K_v through the equation $C = 0.997+2.74\times10^{-3}(K_h/K_v)$.
3. A method for the estimation of recharge from a large water body, using the water level fluctuations in the observation well, has also been suggested.
4. Proper location of an observation well to record the rapid change of head has also been suggested, i.e.,X/L=0.15 to 0.25.

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ANNEXURE

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