INVESTIGATION OF RESOURCES OF A LIMESTONE AQUIFER USING A DIGITAL TECHNIQUE

BY

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Dissertation submitted in partial
fulfilment of requirments for
the degree of Master of Engineering
in Hydrology and Water Resources.

University of Moratuwa,
Sri Lanka.

March 1986.

This dissertation has not been submitted to any University or an Institution for the purpose of obtaining a degree.

V.W. de Silva.
20th March, 1986.

PREFACE

The study described in this report was carried out as a part of the course requirements for a Master's degree in Hydrology and Water Resources Engineering. The course was sponsered by the UNESCO and conducted at the University of Moratuwa, Sri Lanka. The course coordinator was Prof. V.C. Kulaindaswamy, UNESCO advisor to the dept. of Civil Engineering. Prof. Kulaindaswamy assisted by a few visiting lecturers handled the entire teaching work in the course. The auther completed the examination requirements for the Master's degree by January 1981 but could not start on a research project for some time as there was no supervisor to guide him at the time. When Dr. D.C.H. Senarath came back to the department after his sabbatical leave in March 1981 he kindly agreed to supervise this study.

After a few months of works in data collection, debugging of computer program etc. the auther had to suspend his work due to the break down of the University computer system. Sebsequently the computation work was completed while the auther was studying at the International Institute for Hydraulic and Environmental Engineering(I.H.E.) Delft, the Netherlands.

In the study no attempt has been made on the development of mathematical aspect of the model. All the emphasis was placed on studying the behaviour of the aquifer with different sets of data which were compiled on the basis of available records as well as on a number of assumptions.

The historical data available was not long enough even for proper calibration of the model. Therefore the verification of the model could not be attempted.

ACKNOWLEDGEMENTS

The auther wishes to place his deep gratitude towards Prof. V.C.Kulaindasamy for his untiring efforts to teach in the Post Graduate course which the auther had followed in the field of Hydrology. Dr. D.C.H.Senarath spent his valuble time in supervising the project, without his continuous guidance the auther would not have been able to complete this work.

Mr. M.W.P.Wijesinghe, the former Deputy General Manager, Water Resources Board of Sri Lanka has authorised the use of their records for the study. He has made valuble suggestions for the study and the auther has benefited much from his wide experience in the field of Groundwater Hydrology. Mr. J. Davis, consultant and Mr. W.P.Rodrigo, technical assistant of the Water Resources Board provided help in collecting data.

The staff of the computer centres at University of Moratuwa and I.H.E. Delft provided much assistance in carrying out computations.

Dr. K.R.Rushton of Birmingham University and Mr. W. Spaans of I.H.E.Delft made valuble comments to improve this work.

The help and moral encouragements extended by Prof. Willie Mendis, Vice Chancellor, University of Moratuwa and Prof. B.L.Tennekoon, Head of Civil Engineering department are deeply appreciated. Thanks are extended to all the staff members of the Department of Civil Engineering who in numerous ways helped to carry out this work.

SUMMARY

Modelling of a limestone aquifer in the north west of Sri Lanka is attempted. The Vanathavillu basin situated 16 km. north of Puttalam, covering an area of approximately 60 sq.km. has been studied using a mathematical model based on an implicit finite difference scheme. The study area situated in the dry zone of Sri Lanka, recieves a seasonal rainfall of about 900 mm/year, the most of which falls during the months of October to December.

In the Vanathavillu basin there are essentially two water bearing formations:

- the miocene sedimentary strata.
- the quarternary deposits which overlie the miocene strata. (referred as Moongil Aru formation)

Moongil Aru formation consists of a series of clays and silts which partly confines the miocene formation. The piezometric levels in the miocene aquifer is lower than the phreatic surface by up to 30 m in the central parts of the region. In the north the piezometric levels are slightly higher than the phreatic surface. The two water bearing formations are interdependant as leakage takes place in and out of the miocene formation. The model was developed only for the miocene formation and the water table elevations in the Moongil Aru formation assumed constant.

The miocene formation is bounded in the east by basement rock outcrops which are relatively impermeable. To the west a fault exists which runs along the coast line. This fault restricts the flow in westerly direction. It is believed that two minor faults exists along two drainage paths of Kala Oya and Moongil Aru. The piezometric levels in the north suggest that the aquifer discharges into Kala Oya which could be treated as a constant head boundary. In the south the flow direction is entirely towards north.

The area has been studied by the Irrigation Dept. and the Water Resources Board of Sri Lanka. On the basis of these investigations aquifer parameters, recharge and abstraction from the limestone aquifer have been estimated. Development plans have been prepared

on the basis of these estimates. The purpose of the present model invest-igation was to assess the reliability of these estimates and also to provide a tool for planning future development and management of this valuble water resource .

A number of model runs with different sets of data represnting aquifer parameters, boundaries and flows were made. The results were compared with an available two year record of piezometric levels in the limestone aquifer. Computations were made with one layer model as well as with a simple two layer model.

A single layer model with constant inflows or a simple two layer model with water table elevation treated as a constant adequately describe the behaviour of the aquifer under the present level of abstraction. But the behaviour of the aquifer with highly increased abstraction can only be modelled adequately by a two layer model representing both unconfined and semiconfined aquifers together.

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CHAPTER ONE: INTRODUCTION

1.1 Location

Ground water development in the Vanathavillu area has been studied by several organisations for more than fifteen years. This area in the North West part of Sri Lanka is situated 18km. North of Puttalam. The area covers about 80 sq.km. in extent and is bounded by Kala Oya a perennial stream in the North, Puttalam lagoon in the West, the basement rock outcropping in the East and by a small intermittent stream Moongil Aru to the South. (see location map Fig.1.1)

1.2 Topography

The study area is low lying with elevations close to sea level in the South West and North to about 55m above m.s.l. in the East. The North West corner of the project area has an elevation of more than 65m above m.s.l. This feature is due to a major geological fault, as a result the Miocene limestone outcrops on the surface from where it is been quarried for manufacturing cement. (see fig. 1.2)

1.3 Geology

The pre cambrian crystalline complex which outcrops in the Eastern boundary slopes Westerly with dips of 10 deg to 20 deg West. This formation is reached at depths 250m to 300m below surface near the coast. The sedimentary sequence of Jurasic sandstone, Miocene limestone and quarternary unconsolidated sediments are laid down over the basement crystalline rock in a wedge form. These sediments become thiner towards the East and disappears near the boundary of the study area.

The geometry of the limestone formation is complicated by the presence of a series of faults along the coast line in the North West. There the miocene deposits could be found from the surface to a depth of about 50m below mean sea level. Boreholes close to the North boundary reveal a relatively minor fault running parallel to Kala Oya.

These features are indicated in geological sections along East West and North South directions. (see Fig.1.3 and 1.4)

1.4 Water bearing formations

The more recent quarternary deposits overlying limestone vary in thickness from 10m to 70m. These deposits consist of marine clays, silts, sands and ferrogenous gravels and of hetrogeneous nature. A large number of shallow dug wells tap water from this formation which is referred to as Moongil Aru formation in the related litereture. The combined thickness of the limestone formation which underlies the Moongil Aru formation is 20m in the east and 70m in the west. The limestone layer is partly confined by series of clays and silts thus separating the water table aquifer from the underlying limestone aquifer. There is leakage taking place from water table aquifer to the aquifer below.

1.5 Hydrological systems

The rainfall which is the only form of precipitation in the region is around 900m per year, the greater part of it falls during the months November to January due to North East monsoons. During the other months of the year the rainfall is neglegible. It is assumed that rain recharges the water table aquifer and leakage takes place from water table aquifer to the limestone aquifer below. This assumption was made due to delayed rise in piezometric head in the limestone aquifer following rainfall.(see Fig. 1.5)

In the water table aquifer the outflow is mainly to the lagoon and to Kala Oya in the North as seen from the water table contours. (Fig. 1.7) Abstractions from this aquifer are neglegible as water is tapped only from shallow dug wells. Water is also lost by evaporation from places where the water table intercepts the ground surface. It is believed that water leaks from this water table aquifer to the underlying limestone aquifer, but the extent of this leakage is not establishsd with any degree of accuracy due to non-availability of sufficient data. The amount of leakage taking from upper to lower aquifer is one of the problems to be analysed using the present model.

The lower aquifer consisting mainly of miocene limestone and jurassic sandstaone deposits carry water under pressure. This pressure is sufficient to take water upto the ground level in Northern and Western boundaries but at the centre of the area the piezometric level is more than 30m below ground level, whereas the water table at the same place is only about 5m below ground level. This large head difference in the two aquifers casts doubts about any significant leakage place from upper to lower aquifer.

The piezometric contours (fig. 1.6) indicate the flow direction as North and North-West. Water is assumed to discharge to the lagoon and to Kala Oya although the limestone formation is not in contact with any of them. The Eastern boundary of the aquifer can be taken as the basement outcrop, but the other boundaries are not well defined. Modelling of the regional ground water flow has become a necessity in order to understand the ground water system including its recharge, outflow and boundaries.

1.6 The need for modelling

As outlined above the ground water system in the Vanathavillu area is very complicated consisting of inter dependent aquifer systems. The entire livelihood of the inhabitants of this area depends on the availability of ground water as there is no other source of water. More over, the state land is alienated among farmers with the hope that ground water does exist for their use. Since there is a possibility in large scale exploitation of ground water an overall plan is necessary for its development; hence the system has to be studied completely.

Due to inter-dependance of water table aquifer and the lower limestone aquifer a mathematical model can be a useful tool to study the total system as a whole. No classical methods of analysis exists for this type of problems. A mathematical model can be used to understand the flow mechanism, in the aquifer. Inflows and outflows, aquifer parameters and the controlling boundary conditions can be varried in the model to study the system in detail. Once a model is set up and calibrated it can be used as a management tool. Further, the steep drawdown observed around the only large scale abstraction point at the cement corporation quarry casts doubts about the magnitude of recharge into the limestone aquifer.

A mathematical model involves the description of flow pattern by a set of mathematical equation. These equations are solved at a finite number of points representing the aquifer. The solution of these equations requires initial and boundary conditions. The complex flow phenomena are represented by simple boundary and initial conditions amenable to mathematical formulation.

The model to be developed is aimed primarily at establishing the boundaries and the recharge of the aquifer. The emphasis will be mainly placed upon the limestone aquifer, and the water table aquifer will be considered only so far as it effects the lower aquifer.

1.7 The history of investigation

Systematic investigation of the Vanathavillu ground water basin was initiated in 1967 by the Irrigation department of Sri Lanka which was responsible for all the water resources development at that time. They were assisted by a team of experts from Israel. Subsequently the investigation work became the responsibility of the Water Resources Board of Sri Lanka which started a fresh investigation with the assistance of Overseas Development Agency of United Kingdom in 1978. From 1967 to 1978 very small attention was paid to the area and most of the useful work carried out by the Irrigation Department was lost as no records were maintained about abstraction rates or any other hydrological events.

1.8 The scope of the study

In this report the author will restrict himself to present a very approximate mathematical model to simulate the ground water behaviour in the area which will provide the ground work for any future detail studies. This limitation was necessitated for two reasons : firstly the nature of the data available is not adequate to formulate a comprehensive model, secondly this is one of the first of this kind of work to be carried out in Sri Lanka and the basic facilities such as high speed computers are not easily available for this purpose.

The progress of this work was interrupted on a number of occasions due to breakdown of the University computer system. However, the author firmly believes that with the facilities geared to this kind of work and with the aquisition of the necessary expertise, a more comprehensive model could be developed, on the basis of the present work, which will help future ground water resource investigation in Sri Lanka.

1.9 The source of data

All the data used for analysis has been taken from the Water Resources Board of Sri Lanka both published and unpublished.

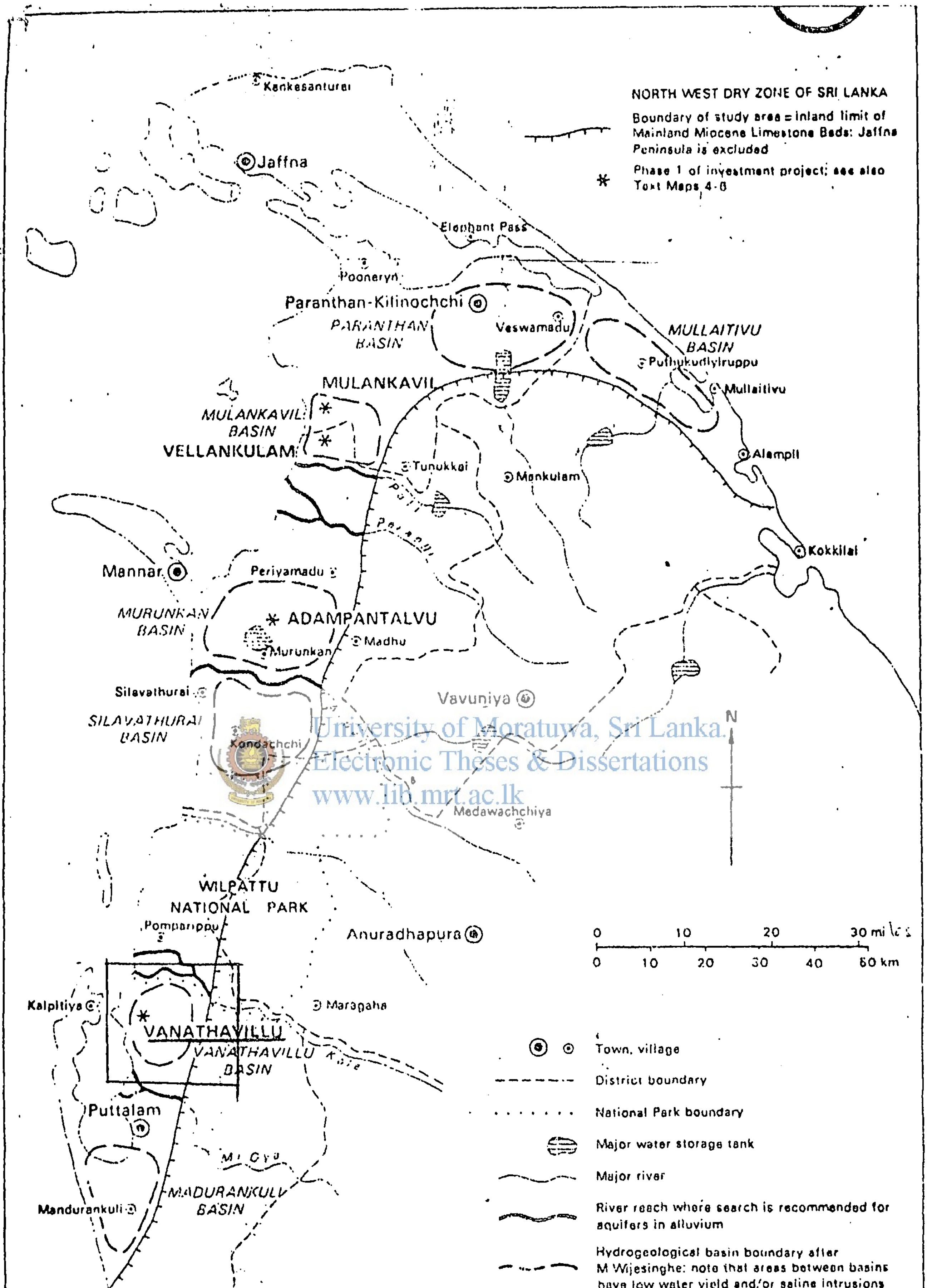


Fig.1.1 Location map.

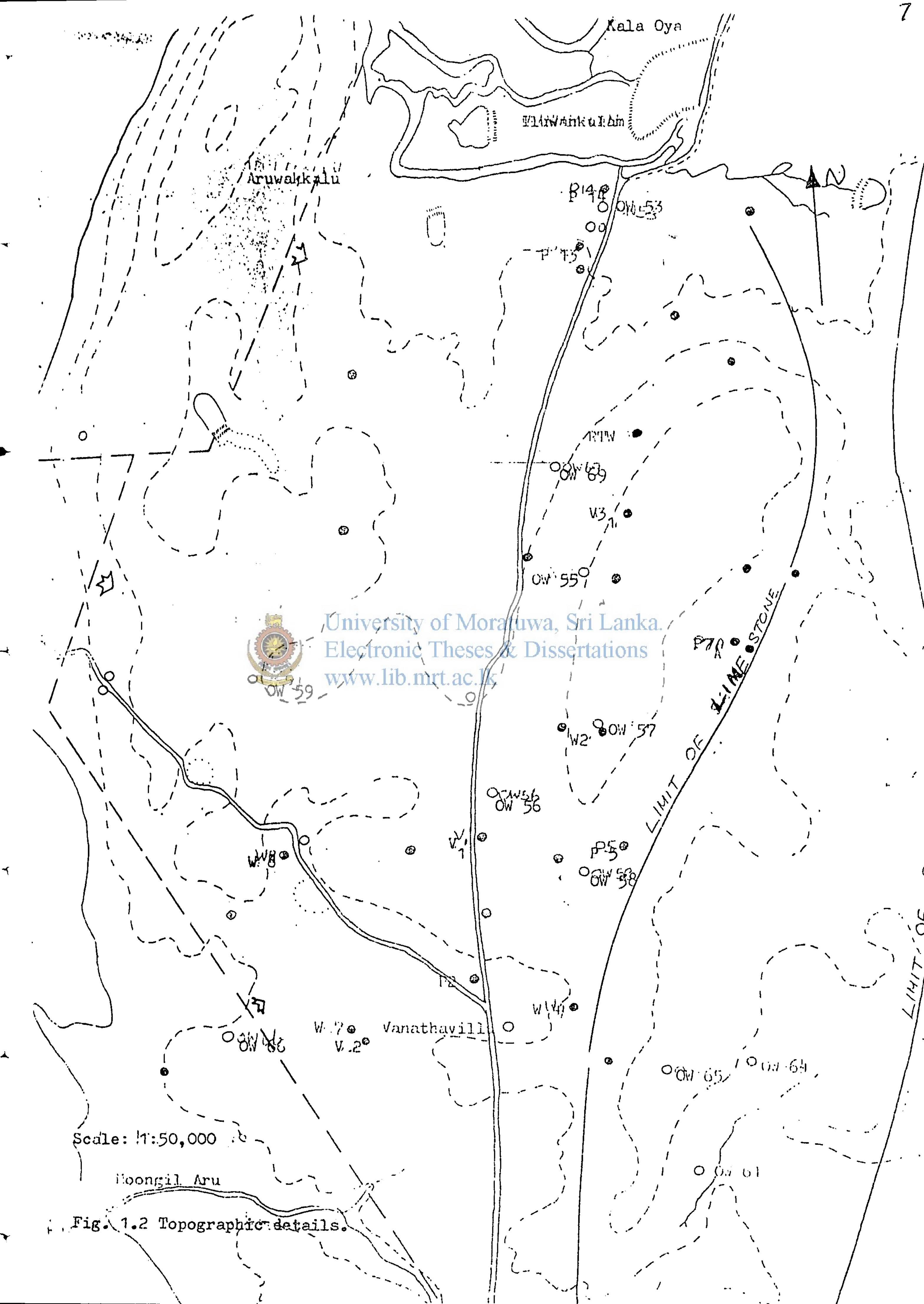
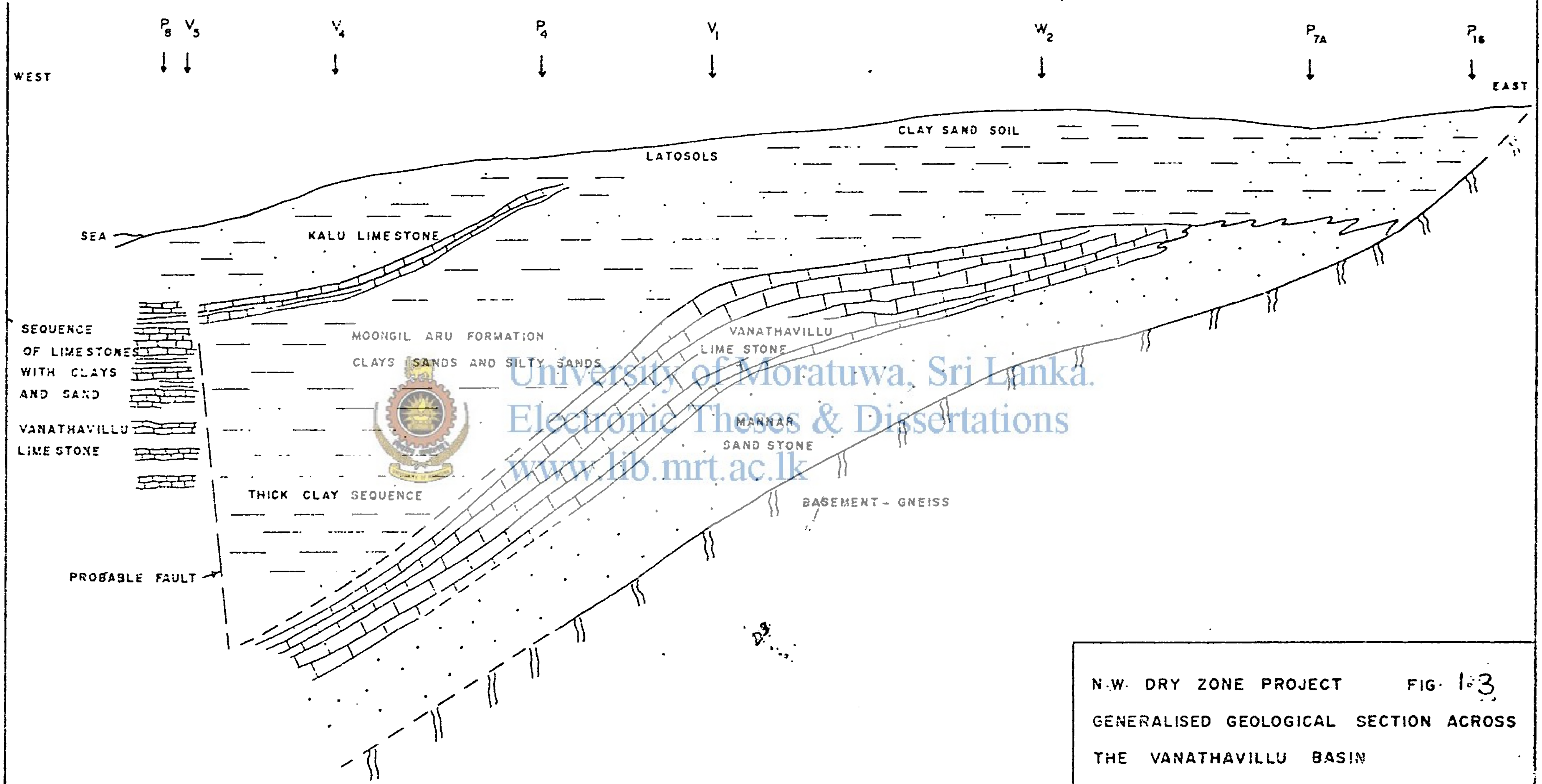


Fig. 1.2 Topographic details.



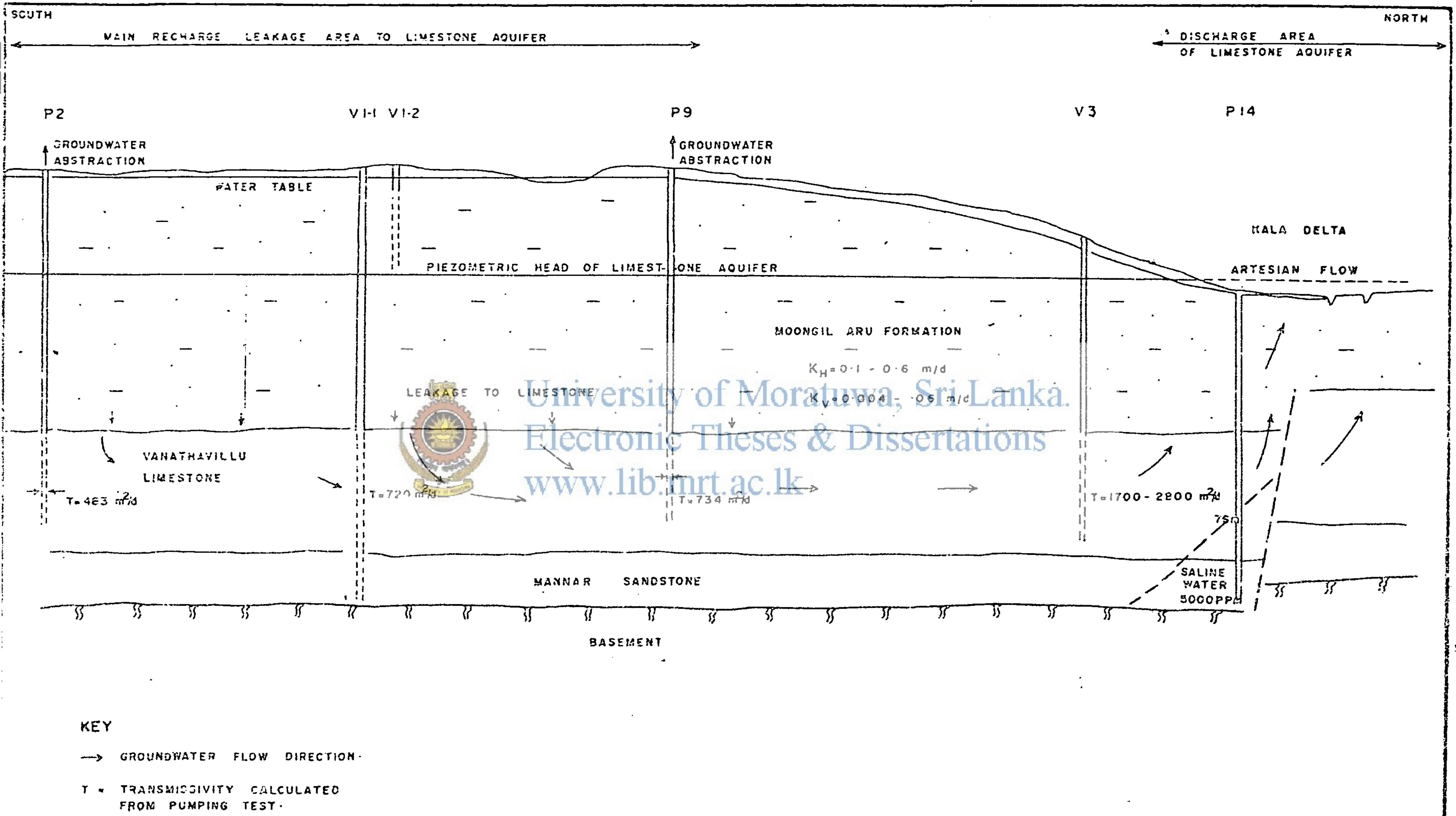
N.W. DRY ZONE PROJECT FIG. 1.3

GENERALISED GEOLOGICAL SECTION ACROSS THE VANATHAVILLU BASIN

SCALE :-

HORIZONTAL - 1 : 50,000

VERTICAL - 1 : 2,000



N.W. DRY ZONE PROJECT.

FIG. 1.4 HYDROGEOLOGICAL SECTION IN VANATHAVILLU (SOUTH/NORTH).

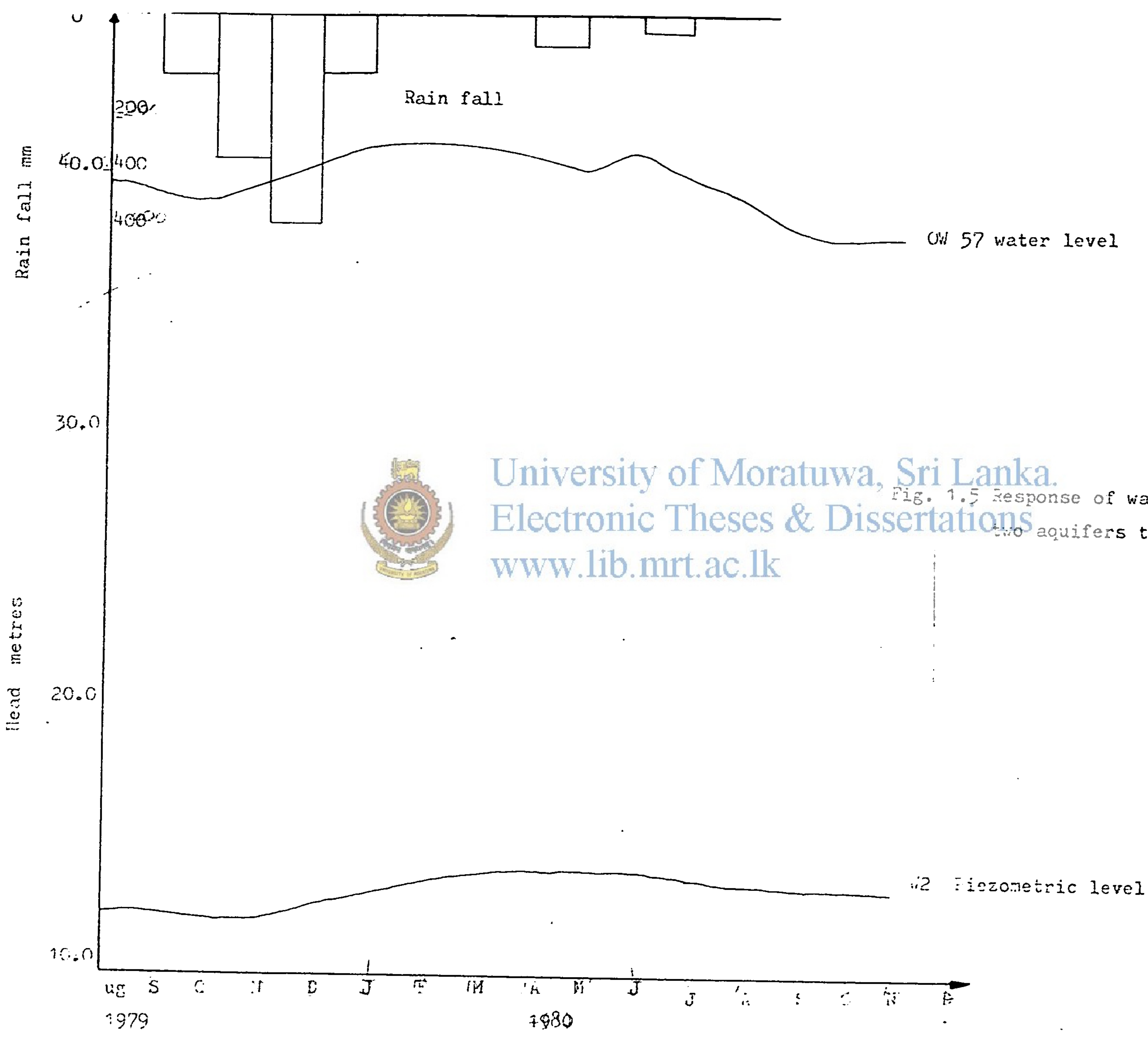
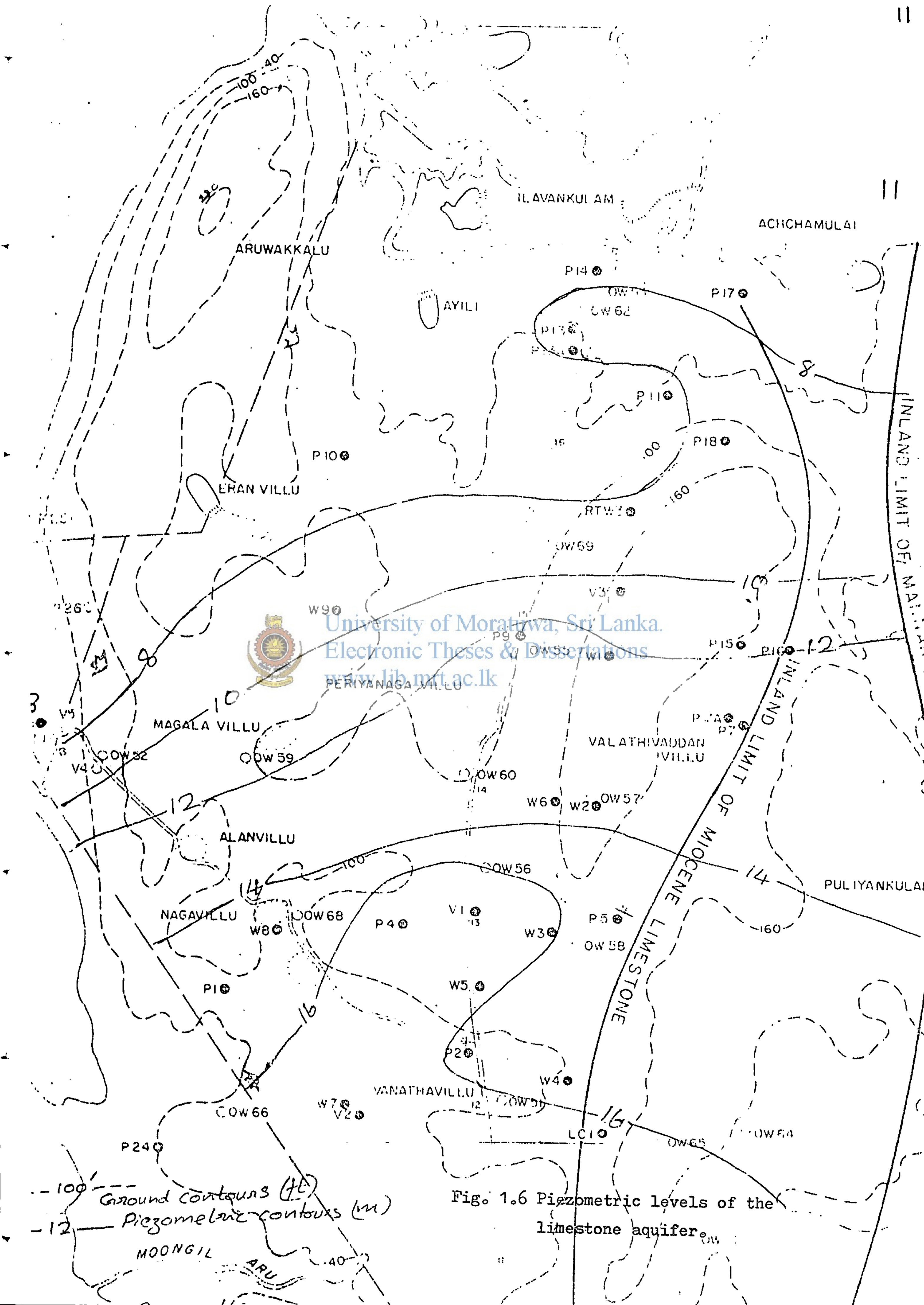


Fig. 1.5 Response of water levels in the two aquifers to rain fall.



100 Ground contours (ft)

12 Piezometric contours (m)

Fig. 1.6 Piezometric levels of the limestone aquifer.

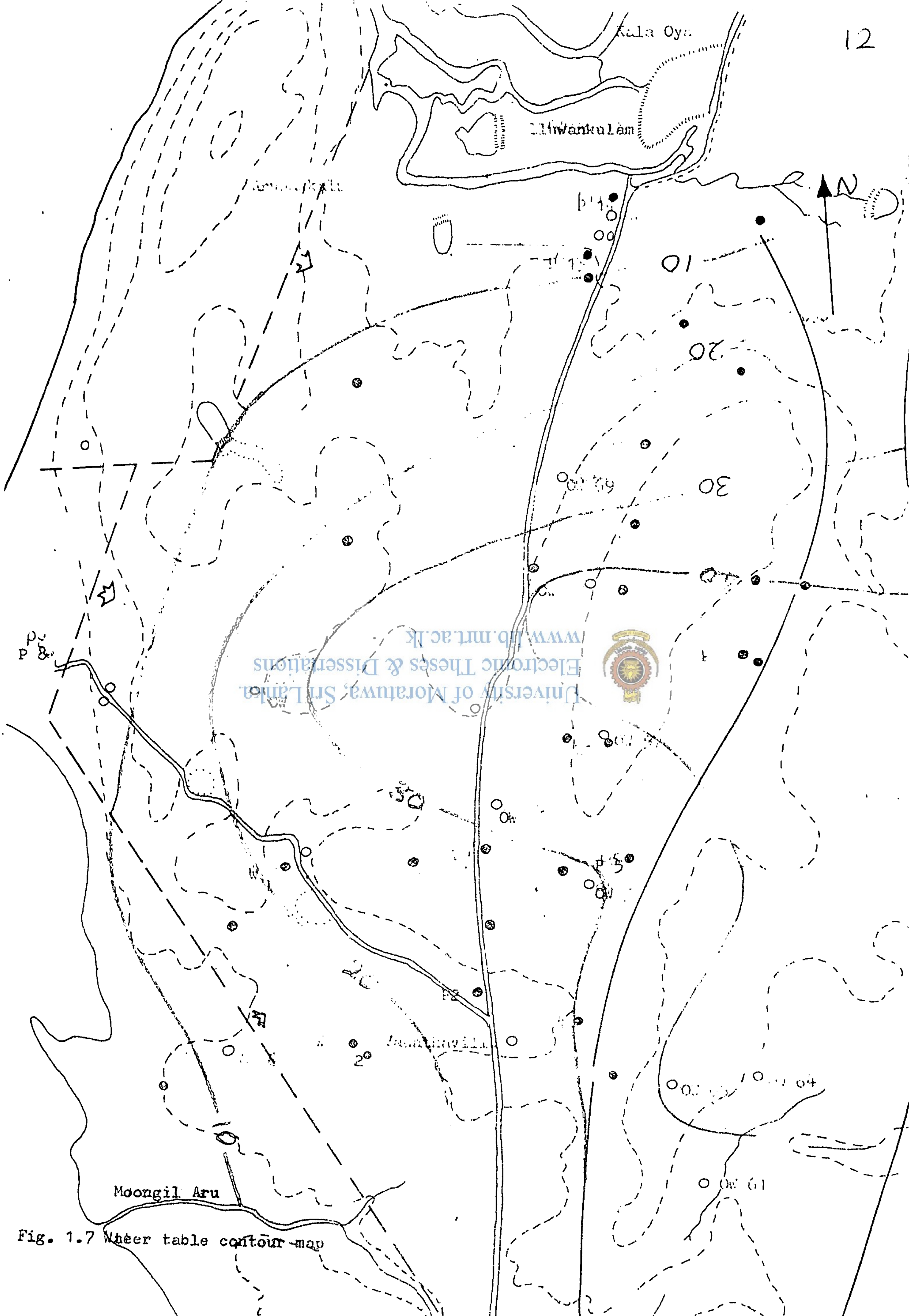


Fig. 1.7 Water table contour map

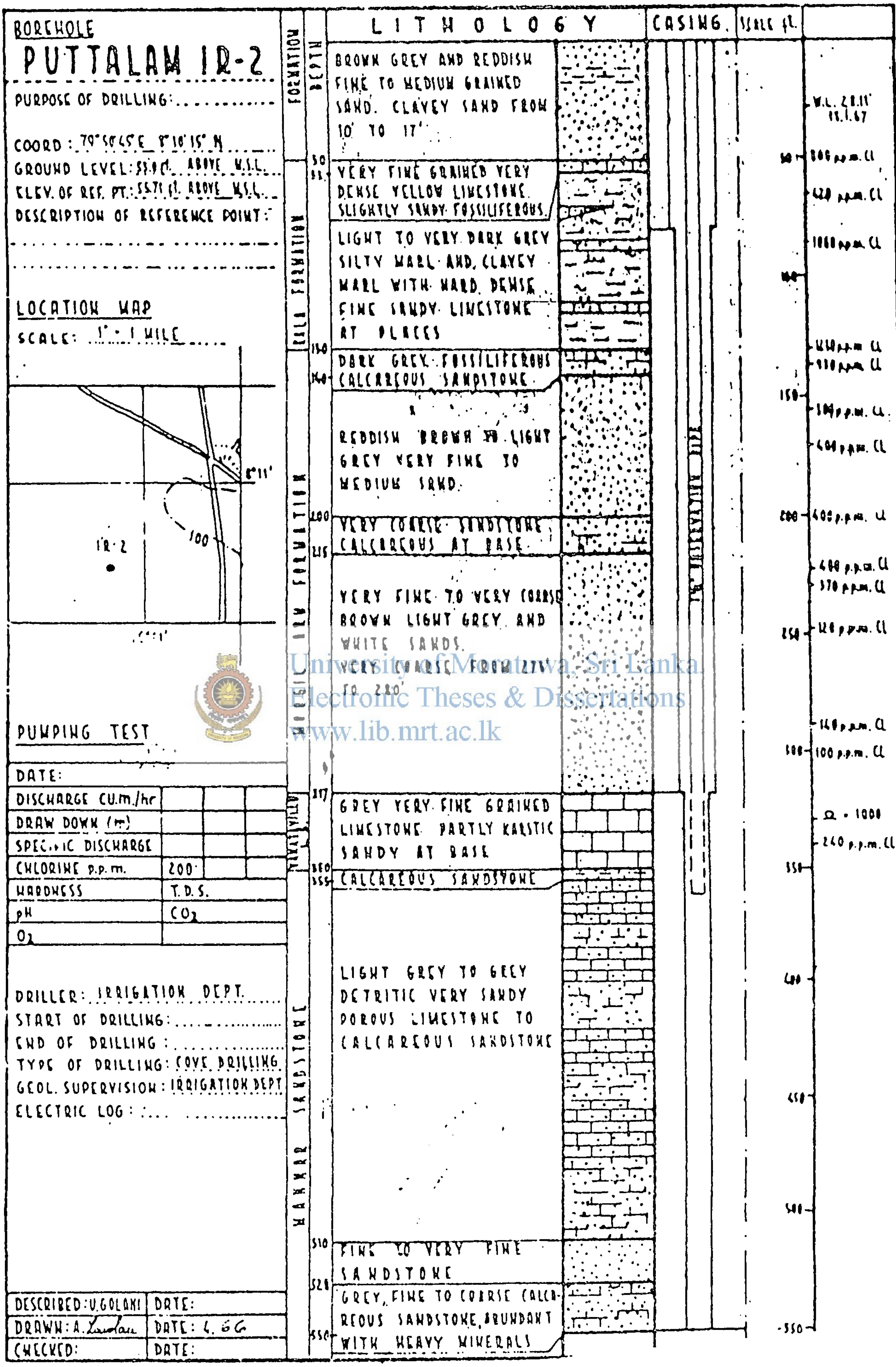


Fig. 1.8 Typical borehole log in the Vanathavillu aquifer

CHAPTER TWO: USE OF MATHEMATICAL MODELS TO INVESTIGATE GROUND WATER SYSTEMS.

2.1 General

The use of mathematical models to study ground water flow systems is now widely accepted method for its simplicity, low cost and reliability. Mathematical models have been successfully used in the early sixties to study ground water flow problems in California coastal plain (Tyson and Webber 1963) and Varmin plain of Iran (De Rider 1968). Both the above authers used an arithmetical and algebriac relations along with non mathematical logical processes.

A more comprehensive method involving descretisation based on finite difference approximations was developed by Rushton (1974). The method has the advantage that it can be applied to analyse a single well and other places where large drawdowns take place in addition to analysing regional ground water flow.

Mathematical modelling aims at a better understanding of a combination of elements which forms a complex that can be designated as a system. System simulation is the process of designing a model of a real system and conducting experiments with this model for the purpose of understanding the behaviour of the system or egaluating of various strategies for operating the system. Simulation of a complex system essentially requires the use of a digital, or under certain conditions an analogue computer.

Darcy's Law in combination with mass equation leads to a model of ground water flow. In Dracy's Law the complex phenomenon of flow of water through a porous medium is represented such that one can compute average velocities from which the rate of flow may be estimated. Darcy's law does not give the actual velocity of the liquid through the pores.

2.2 Types of models used in Hydrologic systems

A Hydrologic system can be defined as a set of physical, chemical and/or biological expressions which act upon input variables. In this definition a variable is understood to be a characteristic of a system which may be measured and which assumes different values when measured at different times. A parameter means a property of the system under investigation which is constant with time. For example the transmissivity of an aquifer is a parameter while the recharge may be a dependant variable. Modelling of a Hydrologic system means the study of the behaviour of variables resulting from the above mentioned processess acting upon input variables for different sets of parameters which are introduced to describe the above process.

Different types of models are available in the study of hydrological phenomena.

1. Physical scale models
2. Analogue models
3. Mathematical models.

2.3 Physical scale models

Using physical scale models the real world situation is scaled down to workable proportions simulating the actual situation. The scale models are suitable for studying local phenomena in detail but require sophisticated measuring equipments and hence the cost is very high and time consuming. Further more due to scale effects the interpretation of some of the results must be handled with care. For studying hydrogeological phenomena this type of models is seldom used.

2.4 Analogue methods

This catagory of models is very often used in studying ground water flow phenomena. Here the conductivity, rate of flow and hydraulic head are represented by an electrical resistance, current and a voltage respectively based on the Ohms Law.

Thus:

$V = R.I$

Where,

V = Drop in voltage representing change in the hydraulic head.

R = Resistance representing the reciprocal of hydraulic conductivity.

I = Ampereage representing the rate of flow.

The storage is introduced by means of capacitors. As the design, implementation, calibration and operation involved are rather time consuming and need an expertise in Electronic Engineering these models have a value only as a demonstrating tool but are of little practical utility.

Another type of electrical analogue model used in ground water flow is the construction of equipotential lines on a conducting paper having the shape of the porous body. This is possible due to mathematical similarity of voltage and electrical current to hydraulic head and flow rate respectively. The seepage through earth dams can easily be computed by this method. The disadvantage is that only steady flow problems can be analysed using this procedure.

2.5 Mathematical models (Analytical)

The principles of mass conservation and Darcy's law have been used to obtain solutions to ground water flow problems. However, the partial differential equations derived for ground water flow problems can only be solved analytically for very simple cases of steady flow phenomena. When dealing with either time dependent problems or with complicated geometry these equations cannot be solved analytically. To overcome this difficulty various types of numerical tehcniques are used.

2.6 Mathematic l models (Numerical)

These methods are extremely useful in one two and three dimensional modelling with varying hydrological parameters and boundary conditions. Specially they become advantageous in studying time dependent phenomena. The main disadvantages are:

1. The solution is only defined at descrete points in time and space.

2. The solution is an approximate one depending on the assumptions regarding hydrological and geometric parameters.

3. The accuracy depend on the size of the numerical grid chosen for computations.

With the availability of micro computers these methods have become extremely popular.

The numerical methods used in studying ground water flow problems can be classified into three types.

1. Finite difference methods based on Cartesian or polar co-ordinate system.

2. Finite element methods.

3. Polygon based difference methods.

Although apparently different in approach the basic concept is the solution of a set of partial differential equations based on Darcy's law and continuity principle at pre determined descrete points in space and time.

Each of the above methods leads to comparable results, the main difference being the covering of boundaries. The choice of which method is best is mainly determined by the availability or the understanding of the basic concepts on which the computer program is based.

2.7 Finite Difference Methods

The basis of the finite difference methods is that the partial derivatives of a variable with respect to x and y in two dimensional problem can be written as,

$$\frac{\partial h}{\partial x} = \frac{h(x+\Delta x) - h(x)}{\Delta x} \quad ; \quad \frac{\partial h}{\partial y} = \frac{h(y+\Delta y) - h(y)}{\Delta y}$$

For this purpose the area considered is covered by a rectangular grid with mesh size Δx and Δy. At each grid point or node hydrogeological parameters like transmissivity and storage coefficient are specified together with external flows like recharge abstraction etc. Approximating the governing basic equations by finite differences the hydraulic head at each grid point can be calculated. Othe finite difference methods describe the parameter heads and other parameters in the centre of the mesh the so called cells.

This method will be described in more detail in the next chapter.

2.8 Finite Element Methods

These methods are based on the principle that for any geometrical shape of elements an interpolation function can be found which within the element expresses the unknown hydraulic head in terms of hydraulic heads at corner points. The considered region is sub-divided into a number of elements, for example the most widely used triangular shaped elements. With these elements the often irregular shaped boundary and varying conditions within the area can be fitted quite easily, while triangles have simple interpolating functions. Within each element the hydrogeological parameters are specified average over the elements and assumed constant.

2.9 Polygon method

This method is based on the mass conservation law which is applied for each polygon shaped element of the considered region. Within the region and along the boundaries a number of nodal points are selected which can be considered representative for that part of the region. The nodes are located such, that their inter connections form triangles with acute angles. Around each node an impact area is constructed by drawing perpendicular bisectors of the sides of the triangles, quite similar to the well known Thiessen approach.

Similar to finite element methods the hydraulic head is defined at the nodes. For each internal node a mass balance is formulated related to the area of that node. This result in a set of linear independant equations which can be solved. (ref Spaans 1984) either by standard procedures for solving a set of a equations or by iterative procedures. The latter ones use less computer storage but generally take more computer time.

CHAPTER THREE : FORMULATION OF THE FINITE DIFFERENCE SCHEME

3.1 Basic concepts.

Any ground water flow problem is solved by two basic principles, namely that of conservation of mass and that of conservation of momentum in the form of Darcy's law.

Darcy's law.

Even though the flow in an aquifer takes place through pores, fissures or solution channels it is assumed that the overall effect can be described by Darcy's law. The law only holds for low velocities. In general ground water velocities are low, but in the vicinity of wells the velocities may be too high so that Darcy's law may not hold exactly.

This law can be written as

$$q = - k\, i \quad \ldots\ldots \qquad 3.1$$

Where,

k = permeability of the aquifer (m/s)

i = hydraulic gradient (m/m)

q = the specific discharge (m/s)

Note that the specific discharge has the dimensions of velocity, but does not really represent the velocity of water through the porous medium. The specific discharge is defined as the quantity of flow through a unit area.

The hydraulic gradient i is defined as,

$$i = \frac{\partial h}{\partial s} \quad \ldots\ldots\ldots \qquad 3.2$$

where,

∂h is the difference of head over a distance ∂s along the flow path.

3.2 Governing equations

The equations governing ground water flow can be derived by considering an element in three dimensional Cartesian space. Let specific discharge at the centre of the element be q_x, q_y and q_z respectively in x, y, and z directions. (see fig. 3.1). The net flux outflow from the element across the plane y,z, is

$$\frac{\partial q_x}{\partial x} \; dx \; dy \; dz.$$

Considering the other two planes as well, the total flux flowing out through the element is

$$\left[\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z}\right] dx \; dy \; dz$$

This should be equal to the rate of change of storage within the element.

For a fully saturated confined or semi-confined aquifer the following derivation may be adopted.

When the pressure is released from the aquifer due to lowering of head a certain quantity of water will be released from storage due to compressibility of porous medium and of water. The volume thus released from storage per unit decrease of head defined as the specific storage coefficient S_c is accordingly represented as,

$$S_c = \frac{\Delta V}{V} \frac{1}{dh} \qquad 3.2$$

The rate at which water is taken into storage is given by,

$$(dx \; dy \; dz) \; S_c \frac{dh}{dt}$$

This is equal to the rate of outflow from the element. Therefore,

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z} = S_c \frac{\partial h}{\partial t} \qquad 3.3$$

Incorporating Darcy's law,

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial h}{\partial z}\right) = S_c \frac{\partial h}{\partial t} \qquad 3.4$$

3.3 Idealisation of regional ground water flow.

A common assumption for regional ground water flow problems is that the vertical flow component is sufficiently small compared with other components so that it can be neglected. Also it is assumed that flow is entirely in the horizontal direction. Therefore removing the term $\frac{\partial(k_z \frac{\partial h}{\partial z})}{\partial z}$ and multiplying the equation by saturated thickness of the aquifer m, we get,

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) + Q_E = S \frac{\partial h}{\partial t} \qquad 3.5$$

where,

Q_E = External flow (m/day)

T_x = Transmissivity in x direction (m^2/day)

T_y = Transmissivity in y direction(m^2/day)

S = mS_c = Storage coefficient

3.4 Finite difference equations.

There are various finite difference approximations that can be used for the solution of equation 3.5. Each of these method differs in conditions of stability, convergence, computational effort, memory storage required by the computer and the simplicity of understanding. Rushton(1979) classifies the most common methods into following catogaries.

1. Forward difference explicit method.
2. Backward difference implicit method.
3. Alternating direction implicit.
4. Alternating direction explicit.
5. Modified alternating direction implicit.

In the forward and backward difference methods the differential equation is descretised in one step in both X and Y directions simultaneously. This results in a large set of linear equations which have to be solved simultaneously when an implicit scheme is adopted. But in alternating direction methods the time step is split into two levels where the equations for X and Y directions are solved one after the other. The latter method requires less computer storage and is much faster when compared with forward or backward difference methods. However certain shapes of boundaries produce instabilities in computations which is not the case in backward differences. In this report a backward difference method is described.

3.5 Backward difference formulation.

The principle of digital computer solution of equation 3.5 is that assuming the head at some time level n is known the computer generates the solution at next time step, time level n+1. For this purpose the time and space derivatives of the variable h is written in terms of h values at pre-determined space and time grid. Thus equation 3.5 is written in finite differences as,

$$L\left[\frac{\partial}{\partial x}\left(T_x\frac{\partial h}{\partial x}\right)^{n+1} + \frac{\partial}{\partial y}\left(T_y\frac{\partial h}{\partial y}\right)^{n+1}\right] + M\left[\frac{\partial}{\partial x}\left(T_x\frac{\partial h}{\partial x}\right)^{n} + \frac{\partial}{\partial y}\left(T_y\frac{\partial h}{\partial y}\right)^{n}\right] = S\,\frac{h^{n+1} - h^{n}}{\Delta t} \qquad 3.6$$

where L+M = 1. The methods can be classified by the values used for L and M as follows.

(a) L= 0, M = 1: this is a forward difference approximation which leads to an explicit formulation.

(b) L = 1, M = 0: this is a backward difference approximation which leads to an implicit scheme. The resulting set of linear equations are solved by eitherby an iterative procedure such as successive over relaxation technique or by modified alternative direction implicit procedure.

i (c) L = 0.5, M = 0.5: this is a central difference approximation. Both alternating direction explicit and alternating implicit methods use a central difference approximation.

In the forward difference method the space derivatives appear in the left hand side of the equation expressed by the known values at time level n. The unknown h values at time level n+1 appears as a single unknown variable in the right hand side of the equation. Therefore this single unknown can be determined at each grid point seperately by solving one linear equation at a time. For this reason these are classified as explicit methods. Usually the stability condition for these methods require very small time steps so that the computational effort required for solving a practical problem becomes too high.

In the backward and central difference formulations the space derivatives of variable h are represented at both time level n and time level n+1. Therefore the equation contains more than one unknown at time level n+1 necessitating the solution of a set of linear equations instead of one equation at a time. Very often this requires a larger computer storage and more complicated programming techniques than for explicit methods. However the solution procedures such as successive over relaxation, Gauss Seidel iteration and alternating direction procedures reduce the amount of computational effort required by implicit methods so that they can be fruitfully utilised for solving practical problems at a lower cost than with explicit methods. The main advantage of the implicit methods is their numerical stability for larger time steps, often resulting in much reduced computational costs.

The equation 3.6 can be written in finite difference as follows:

$$\frac{2}{X_{i+1}-X_i}\left[\frac{Tx_{i,j}\left(h^{n+1}_{i+1,j}-h^{n+1}_{i,j}\right)}{X_{i+1}-X_i}+\frac{Tx_{i-1,j}\left(h^{n+1}_{i-1,j}-h^{n+1}_{i,j}\right)}{X_i-X_{i-1}}\right]$$

$$\frac{2}{Y_{j+1}-Y_j}\left[\frac{Ty_{i,j}\left(h^{n+1}_{i,j+1}-h^{n+1}_{i,j}\right)}{Y_{j+1}-Y_j}+\frac{Ty_{i,j-1}\left(h^{n+1}_{i,j-1}-h^{n+1}_{i,j}\right)}{Y_j-Y_{j-1}}\right]$$

$$= S_{i,j}\left[\frac{h^{n+1}_{i,j}-h^{n}_{i,j}}{\Delta t}\right] - Q_{i,j} \qquad \ldots\ 3.7$$

The subscripts i,j refer to the grid point i,j in space and the superscripts n and n+1 refer to time level n and n+1 respectively.

It is convenient to write this equation in the form,

$$A_{i,j}\, h^{n+1}_{i+1,j} + B_{i,j}\, h^{n+1}_{i,j+1} + C_{i,j}\, h^{n+1}_{i-1,j} + D_{i,j}\, h^{n+1}_{i,j-1}$$

$$+ E_{i,j}\, h^{n+1}_{i,j} = F_{i,j}\, h^{n}_{i,j} - Q_{i,j} \qquad 3.8$$

where,

$$A_{i,j} = \frac{2Tx}{(X_{i+1} - X_{i-1})(X_{i+1} - X_i)}$$

$$B_{i,j} = \frac{2Ty}{(Y_{j+1} - Y_{j-1})(Y_{j+1} - Y_j)}$$

$$C_{i,j} = \frac{2Tx_{i-1,j}}{(X_{i+1} - X_{i-1})(X_i - X_{i-1})} \qquad 3.9$$

$$D_{i,j} = \frac{2Ty_{i,j-1}}{(Y_{j+1} - Y_{j-1})(Y_j - Y_{j-1})}$$

$$E = A + B + C + D + F$$

$$F = \frac{S_{i,j}}{\Delta t}$$

Equation 3.8 can be we- written as,

$$h^{n+1} = \left[Ah^{n+1}_{i+1,j} + Bh^{n+1}_{i,j+1} + Ch^{n+1}_{i-1,j} + Dh^{n+1}_{i,j-1} + Fh^{n}_{i,j} + Q \right] / E$$

3.10

Note that the subscripts of the coefficients A,B,C,D,E and F are dropped out here for convenience.

The equation 3.10 is solved by an iterative procedure in which unknown variables at i,j are continuously updated by the last calculated values after each iteration. In the iterative process the h value given by the equation 3.10 will be an under estimation and an over relaxation factor w is introduced to improve the convergence.

Assuming that the m th iteration is completed and another iteration m+1 is being calculated. The change in the head predicted by the equation 3.10 is,

$$\Delta h_{i,j}(m+1,m) = \left[\quad \right] / E - h_{i,j}(m) \qquad 3.11$$

where $\left[\quad \right]$ signifies the similar bracketed terms in equation 3.10.

With the over relaxation factor w the next value of h is given by,

$$h_{i,j}(m+1) = (1-w)\, h_{i,j}(m) + w \left[\quad \right] / E \qquad 3.12$$

An over relaxation factor of 1.6 has been found to be satisfactory in solution of ground water flow equations.

3.6 Convergence criteria.

The simplest method of testing the convergence is to check the accuracy of satisfying the finite difference form of the governing equations at each nodal point. This is achieved by substituting current values of heads into equation 3.6 and determinning the error (or residual) within the equation. This error has the dimensions of quantity per unit area. Knowing the quantity passing through the aquifer it is possible to specify the magnitude of the permissible error. Usually the permissible error is taken as 0.1 % of the average recharge of the aquifer.

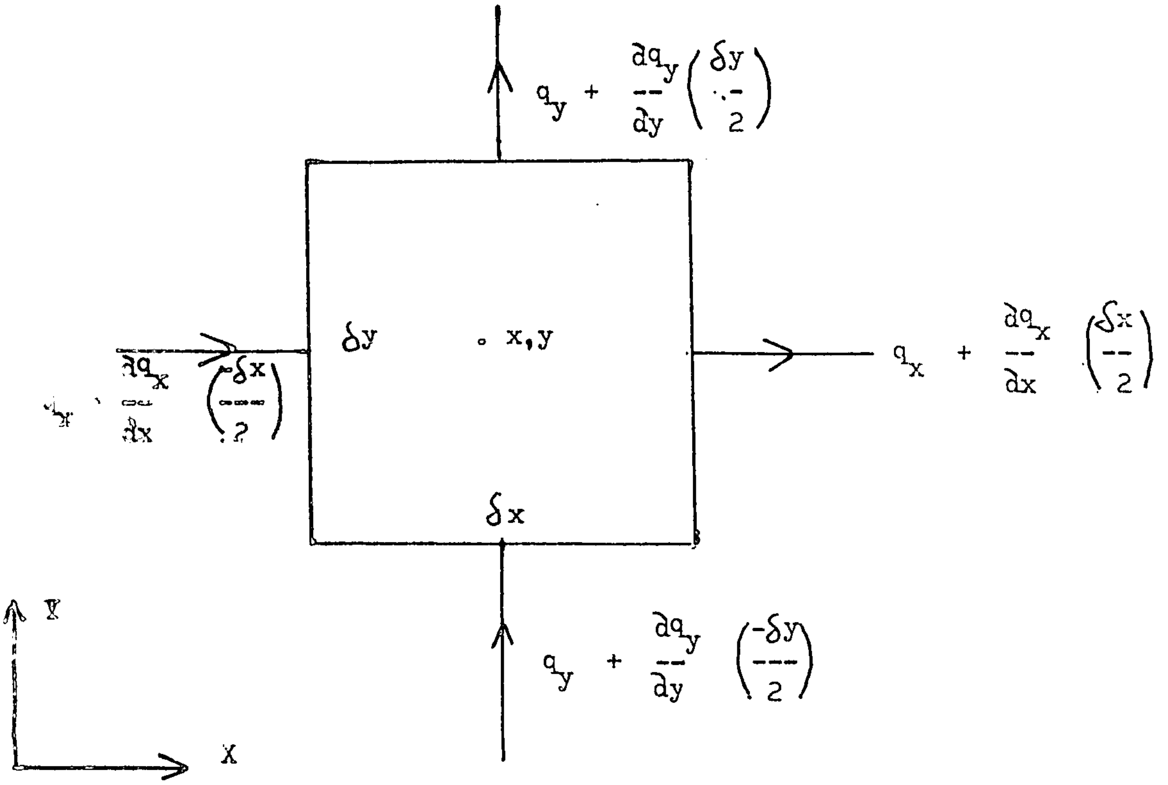


Fig. 3.1 An element of the aquifer. Derivation of governing equations.

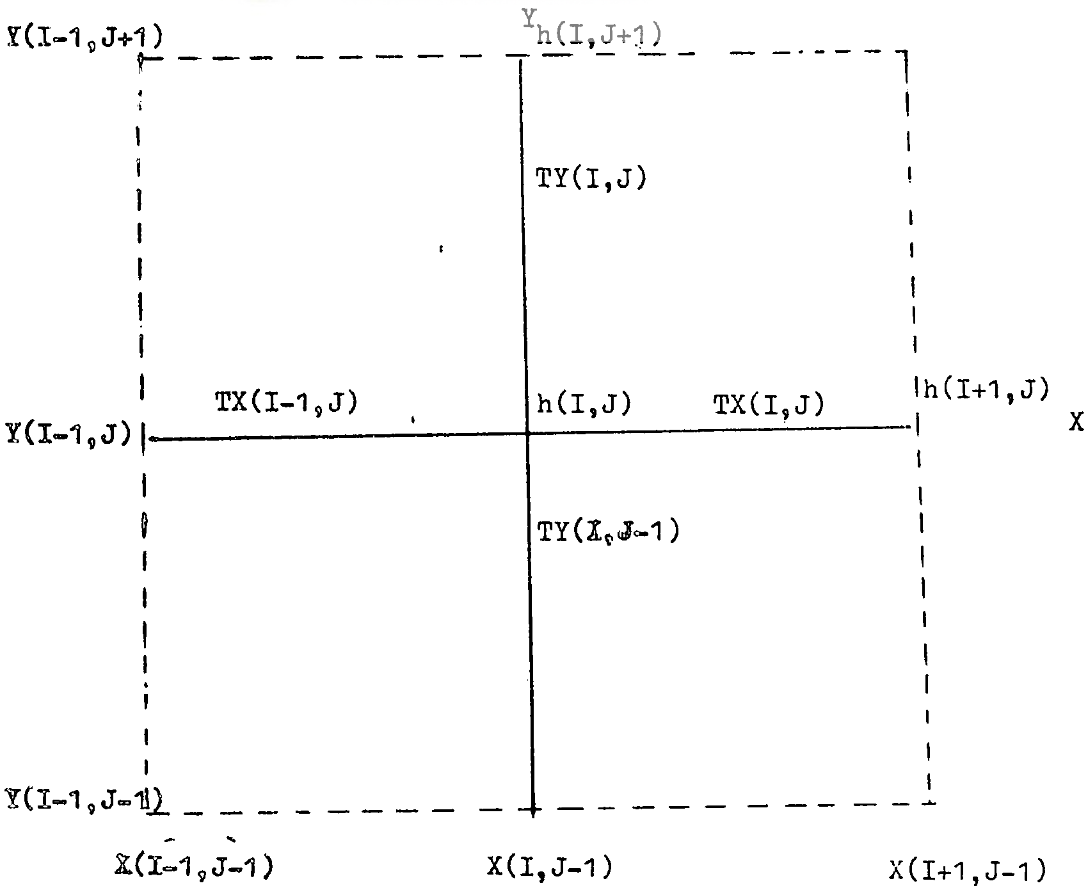


Fig. 3.2 Finite difference mesh

CHAPTER FOUR: THE GROUND WATER MODEL

4.1 Basic features.

The basic equations and the solution procedure described in Chapter three can be used to formulate a model to analyse the ground water flow in a region. A computer program developed by Rushton (1979) has been used for this purpose. In order to represent the physical relationship between water levels and flows in a Mathematical model aquifer parameters, inflows and outflows and boundaries of the area have to be schematised. This was done in the model as explained in following paragraphs.

4.2 Rectangular grid.

The solution of equations by finite differences as outlined in Chapter three is realised on a discrete number of points in a regular grid. The area under study has to be represented by such a grid. The grid lines were chosen parallel to a set of Cartesian coordinate axes. The mesh size will be selected depending on the required accuracy of the computations and also to be compatible with the available data. In general it is possible to change the mesh size in different regions but the selected program uses a constant mesh size. In the program the number of mesh intervals in the X and Y directions are specified as integers M and N. The Y axis is taken alomg the left boundary vertically downwards. The X axis is taken from left to right. The upper left hand corner of the rectangle is numbered I= 2 and J = 2; the integers I and J stand for X and Y coordinates of a particular node. The program works out the X and Y coordinates of all other points depending upon the mesh size.

4.3 Boundary conditions.

The boundary conditions to be used in the model can be classified as follows:

1. Impermeable boundary where no flow across takes place.

2. Fixed head boundary where water level remaims constant or varies in a known manner.

3. Boundary where flow across is known.

The area modelled has to be specified with any of these types depending upon the physical problem. This is certainly the most difficult part in formulating a ground water model as usually sufficient geological data are not available to make a firm conclusion about the nature of the boundaries. The solution of the governing differential equations is strongly dependent upon the boundary conditions supplied and therefore an accurate judgement has to be made regarding the boundary conditions. The model used is flexible to change boundary conditions so that different alternatives could be tried.

4.4 Programming of boundary conditions.

An aquifer modelled is not always of rectangular shape. One of the draw backs in finite difference procedures is that usually the governing equations are discretised using a rectangular grid so that irregular boundaries could not be accommodated very easily. Even though it is possible to change the size of the grid in different regions the grid still remains rectangular. This difficulty is overcome by assigning multiplying factors to coefficients A,B,C,D,S and Q in difference equation 3.9 at the boundary nodes depending upon the shape of the boundary.

The use of these coefficients can be justified as follows. The coefficient A in equation 3.9 represents the flow towards the node (I,J) from positive X direction and if the boundary of the region is parallel to X direction and passes through the node only half the flow will reach the node so that A is taken as 0.5. Same applies to other coefficients. The use of these coefficients is illustrated in fig 4.1 and table 4.1.

Table 4.1 Coefficients for boundary nodes.

Coefficient	Multiplying factor			
	(b)	(c)	(d)	(e)
A	0.5	1.0	1.0	0.0
B	1.0	1.0	1.0	0.5
C	0.5	0.5	0.0	0.5
D	0.0	0.0	0.0	0.0
S	0.5	0.625	0.5	0.25
Q	0.5	0.625	0.5	0.25

The computations at the boundary nodes are carried out in the same way as for internal nodes but with changed coefficients These coefficients are also used for identification of the boundary and the nodes outside the boundary.The nodes just outside the boundary are assigned with a value - 99.9 and the computer ignores the calculation for the nodes where the head is - 99.9.

This procedure for identifying boundaries is only possible for linearised ground water flow equation where transmissivity is assumed to be independent of time. For alternating direction methods employing double sweep procedures to eliminate coefficients in the system of linear equations only rectangular boundaries are possible whereas for the method adopted boundaries at 45 degrees to the grid are also possible.

The boundary nodes where the water level is maintained constant are specified seperately by giving the coordinates of the nodes and the value of the fixed head. In the computation these heads are over written after each computation so that internal fixed head boundaries also can be specified in a similar manner.

Similarly if there is flow across the boundary the quantity of flow at the node can be specified as a fraction of total flow. All the other boundaries where neither a flow nor a head is specified will be treated as an impermeable boundary.

4.5 Aquifer parameters.

The transmissivity and storage coefficient of all the nodes in the aquifer have to be specified. These parameters at each node are interpolated from available maps. Overall transmissivity and storage coefficient are read as TRANX, TRANY and STOR. These values are stored in the arrays TX(I,J),TY(I,J) and S(I,J). The local values of these parameters are then over written in the same arrays.

4.6 Inflows and outflows.

The abstraction from the aquifer usually takes place from a number of wells situated at different points in the aquifer. These abstraction may have different time distributions. But the model is designed to take the total abstraction distributed proportionately among different abstraction points thus all of them will have the same time distribution of abstraction rates. The same applies to recharge. The flows are specified as different types, the type one being reserved for recharge. Type two and three for example can be abstraction and outflow as leakage.

The number of types of flows is indicated by the variable NFCS. Each type of flow is distributed among different nodes by a factor QFLOW representing a particular node. The magnitudes of these flows are given as yearly blocks of data. If time distribution of any of the type differs significantly at some of the points, these flows can be accommodated as separate types of flows.

Although abstraction takes place from discrete points for the purpose of computation of Q in equation 3.7 the net flow at the point is distributed uniformly over the area represented by the node. This produces a kind of local smoothening of piezometric levels; but since the interest is on regional ground water flow pattern this dees not really affect the final results. If one is interested on local piezometric level changes a detail model such as a radial flow model representing the unsteady flow during a pump test can be used. On the other hand it is possible to incorporate a correction for water levels at the pumping well so that the regional model can be used to compute the drawdowns at a pumping well (Rushton 1979). However this was not attempted as sufficient information was not available to compare draw downs at pumping wells.

4.7 Computations

The flow chart for the computation part of the program is shown in Fig 6.1.

Program first computes all the coefficients in equation 3.7. Initially the steady state heads are computed taking the average flows as given in block one. In this process the successive over-relaxation calculation (S.O.R.) is carried out for 300 iterations irrespective of the convergence criteria. The computer identifies the steady state calculation by the dummy variable IFIRST. When the staedy state calculations are over, the program reads the relevent external flows appropriate to the time level of computation. In this stage the S.O.R. iteration process is carried out until the convergence criteria is satisfied. The dummy variable IND makes sure that the calculation is carried out for all the internal nodes until convergence.

4.8 Definition of important variables.

Following is a list of important variables in the computer program.

X, Y	Coordinate at grid points in X and Y directions (A)
M, N	Number of mesh intervals in X and Y directions.
TRANX	Overall transmissivity in X direction.
TRANY	Overall transmissivity in Y direction.
STOR	Overall storage coefficient.

HSTART	Overall initial head.
RECH	Overall recharge coefficient.
OFAC	Over relaxation factor.
ERROR	Permissible error for convergence criteria.
MIN	Extreme right internal node in X direction.
MBOND	Extreme right boundary node in X direction.
MFICT	Extreme right fictitious node in X direction.
TX	Local value of transmissivity in X direction(A).
S	Local storage coefficient (A).
HOLD	Initial head (A).
RCHG	Local recharge coefficient(A).
HFIXA	Fixed heads.
HFIX	Dummy variable to identify the status of variable H (A).
NFCS	Number of types of flows.(A)
NF	Counter for flow types.
NFLOW	Number of nodes where flow NF id distributed.
IW	Location of flow type NF with fraction QFLOW (A).
QFLOW	Fraction of flow type NF with location IW,JW (A)
A,B,C,D	Coefficients of finite difference equation.(A)
AA,BB,CC,DD	Multiplying factors for boundary nodes.(A)
NBLOK	Number of yearly blocks of data.
QAV	Monthly average flow at a node (A).
IFIRST	Counter for identifying the steady state calculation.
KDAY	Number of time steps per month.
IBLOCK	Counter for yearly blocks of data.
IYEAR	Year counter
IMONTH	M0nth counter.
RS	Total flow at the node.(A)
DELT	Time step.
DAYT	Number of days in the month to end of time step.

Note: (A) refer to arrays.

4.9 Input description

The data input for the model is outlined below with the aid of data for a typical computer run.

Number of mesh intervals in X and Y directions.

M	N
10	13

Overall aquifer parameters.

TRANX	TRANY	STOR	HSTART	RECH
100.0	100.0	0.0001	4.0	0.0

Computational parameters.

OFAC	ERROR
1.500	0.0000001

Mesh positions.

X(I), I = 1, MFICT

- 1000.0
0.0
1000.0
2000.0
3000.0
...
...
11000.0

Y(J), J=1, NFICT

- 1000.0
0.0
1000.0
2000.0
3000.0
...
...
14000.0

Aquifer parameters which are non standard. This set of data terminates when I = 1 and J = 1.

I	J	TX	TY	S	HOLD	RCHG
7	2	100.0	100.0	.0005	4.0	0.0
8	2	100.0	100.0	.0005	4.0	0.0
9	2	100.0	100.0	.0005	4.0	0.0
6	3	300.0	300.0	.0005	4.0	0.0
7	3	1200.0	1200.0	.0005	4.0	0.0
8	3	2000.0	2000.0	.0005	4.0	0.0
..	..	..	..	..	..	..
..	..	..	..	..	..	..
6	7	500.0	500.0	.0005	4.0	0.0625
7	7	1000.0	1000.0	.0005	4.0	0.0625
1	1	100.0	100.0	.0005	4.0	0.0

Fixed heads

I	J	HFIXA
5	2	4.0
6	2	4.0
7	2	4.0
8	2	4.0
9	2	4.0
10	2	4.0
11	2	4.0
12	2	4.0
1	1	0.0

Number of inflows and out flows. Flow type one is reserved for recharge.

NFCS

3

Number of nodes where flow type N is distributed.

NFLOW(N)

10

Distribution of flow type N (Abstraction)

IW	JW	QFLOW
7	11	0.0350
3	8	0.0097
8	7	0.0850
9	4	0.3251
8	3	0.0774
8	10	0.0573
7	11	0.0357
8	9	0.1072
9	5	0.1821

Distribution of flow type N = 3 (Out flow as leakage)

10

7	3	0.100
8	3	0.100
9	3	0.100
7	4	0.100
8	4	0.100
9	4	0.100

8	5	0.100
9	5	0.100
8	6	0.100
9	6	0.100

Multiplying factors for boundary nodes.

I	J	AA	BB	CC	DD	SS
5	2	0.5	0.0	0.0	0.5	0.25
5	3	1.0	0.5	0.0	0.5	0.25
5	4	1.0	0.5	0.0	1 .0	0.625
4	5	1.0	0.0	0.0	1.0	0.5
..	..	..	..	..	..	..
..	..	..	..	..	..	..
1	1	0.0	0.0	0.0	0.0	0.0

Number of yearly blocks of data.

NBLOK

2

Time distribution of yearly blocks of data.

Block 1

NDAY	QFC(NF = 1, NFCS)		
31	11.0	-2.0	-5.0
28	10.0	-2.0	-5.0
31	10.0	-2.0	-5.0
30	10.0	-2.0	-5.0
31	9.5	-2.0	-5.0
30	9.0	-2.0	-5.0
31	9.0	-2.0	-5.0
31	9.0	-2.0	-5.0
30	10.0	-2.0	-5.0
31	10.5	-2.0	-5.0
30	11.0	-2.0	-5.0
31	11.0	-2.0	-5.0

Time distribution for block 2

..	..	..	..
..	..	..	..

Number of time steps per month except the last step.

KDAY

2

Days at which these time steps occur .

KDAY

10.0

20.0

Yearly blocks of data used for computations.

IBLOCK = -1 computation stops.

1

1

1

2

2

2

-1

End of data file.

4.10 Out put description.

Once all input data are read by the computer, it organises them into table form and are printed by activating the subroutine PRINT. After computations it prints initial steady state heads for all nodes. Then after each time step heads at selected number of nodes are printed. At the end of each year heads at all nodes are printed in a tabular form. The output is written in an output file OUTRGF. The printing is activated by calling the subroutine PRINT.

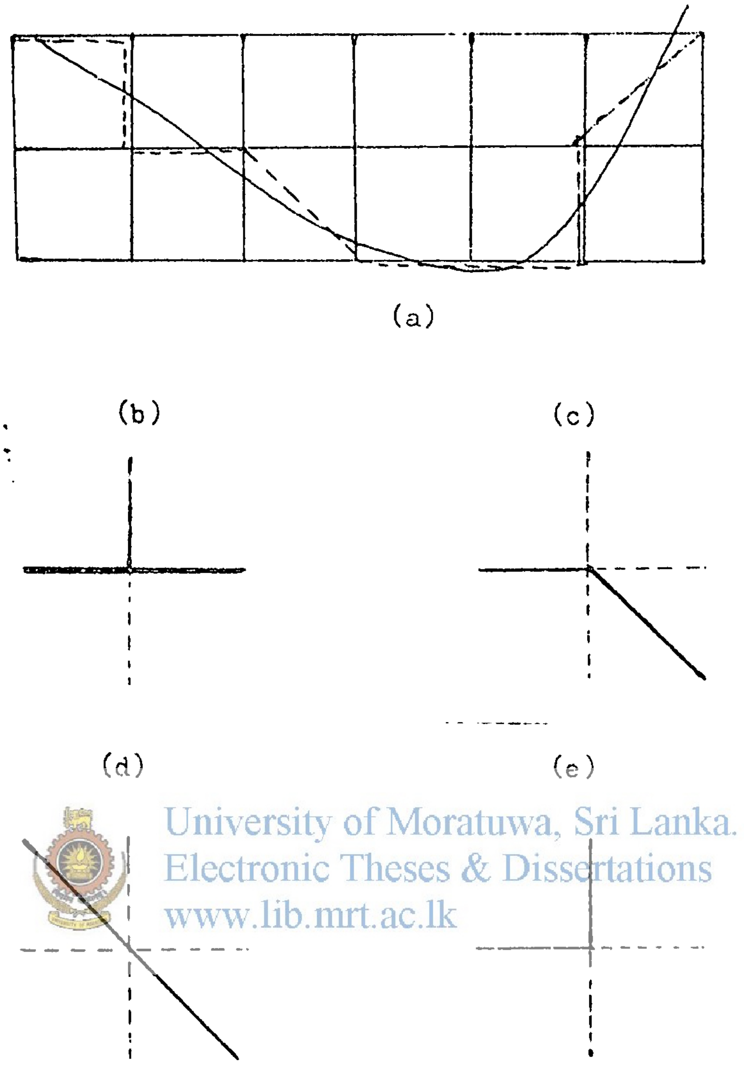


Fig. 4.1 Coefficients for irregular boundaries.

(Refer table 4.1 for values of coefficients)

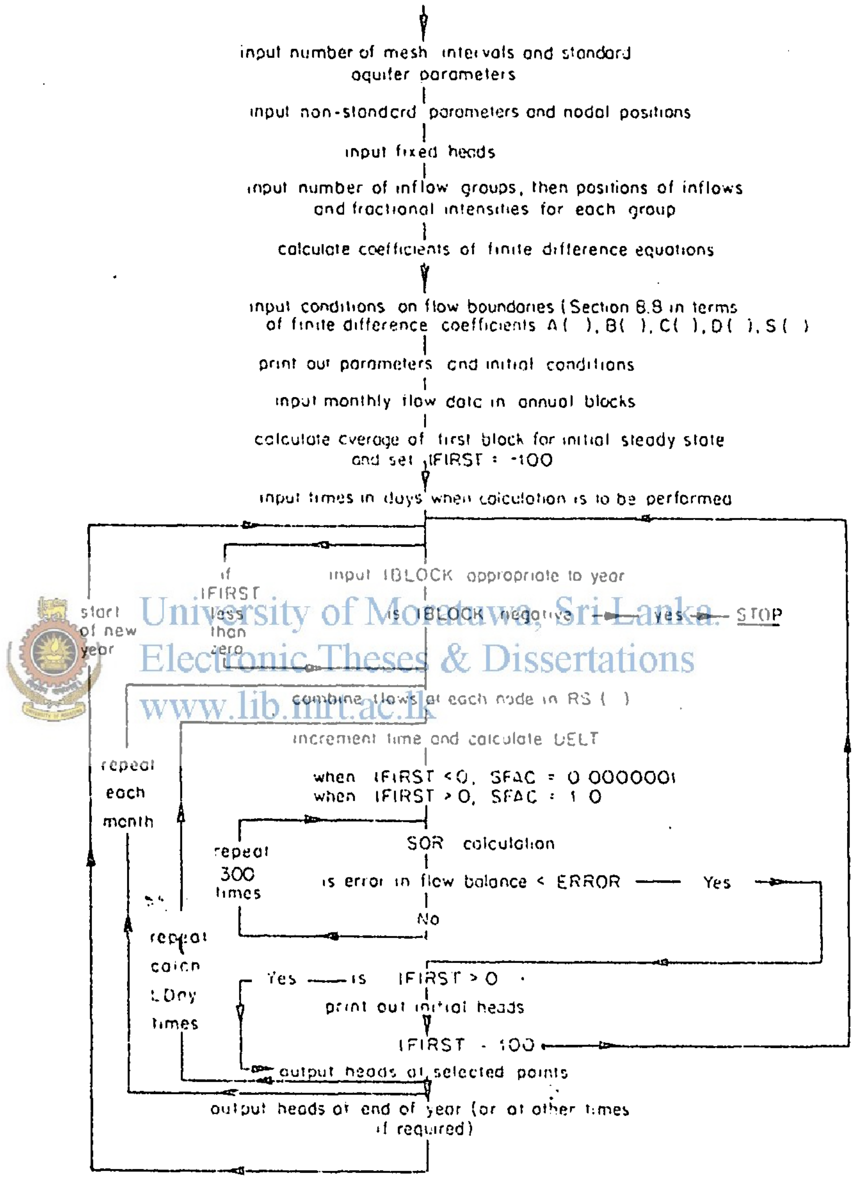


Fig. 4.2. Flow Chart for computer program.

CHAPTER FIVE: HYDROGEOLOGICAL DESCRIPTION OF THE AREA

5.1 Boundaries.

As described in chapter one the boundaries of the aquifer are not yet certain due to inadequacy of the geologic data. The eastern boundary of the aquifer is fairly well defined which can be taken as the crystalline basement rock outcrop. Along the coast line in the west a geological fault line runs for a cosiderable distance and can be regarded as a no flow boundary. This assumption is consistent with the fact that in the southern part the flow direction is predominantly in the north south direction thus no flow takes place in the western direction.

The uncertainties are in the north western, southern and northern boundaries. Since there is no flow in the southern direction the southern boundary should be either a no flow boundary or a boundary with some inflow. However there is no evidence to show that there is any inflow entering from south. As one of the objectives of the modelling exercise is to ascertain the actual recharge taking place into the aquifer the southern boundary was assumed as no flow to investigate the worse case as regards the piezometric levels. The location of the boundary in the south was arbitrarily taken at Moongil Aru, an intermittent stream which is also the boundary for the investigation carried out by the Water Resources Board.

The northern and the north western boundaries pose the greatest difficulty of all. Some observations in the shallow hand dug wells in the north shows that the water levels remain constant throughout the year irrespective of the rainfall whereas wells in other areas show significant drawdowns during the dry season. The vegetation in the area near the flowing well P 14 appears to be green throughout the year indicating that the soil remains moist throughout the year. These observations suggest that there is some upward seepage from limestone aquifer into the upper soil layer. However this upward seepage has not been quantified by any measurements. In the model the northern boundary was assumed to coincide with KalaOya estuary as a fixed head boundary with level coinciding with the mean water level in the river. The effect of changing the boundary conditions in the north has been investigated in the model.

5.2 Recharge.

The piezometric levels in the lower limestone aquifer is more than 12 m lower than the water levels in the unconfined aquifer in all parts of the region except at the northern boundary. This large difference indicates the fact that the two water bearing formations are seperated by a clay layer acting as an aquitard, and the flow from upper layer to lower layer has to be very low. However there is a small leakage from upper to lower layer as can be seen from the increase of piezometric levels after the monsoonal rains. The amount of leakage difficult to estimate from leakage factors obtained from pump tests as the response of water table elevation to pump tests in the lower aquifer is very low. Different pump tests give leakage factors varying from 700 m to about 6000 m.

5.3 Radial flow model to analyse pump test data.

To estimate the leakage factor an attempt was made to analyse the pump test data on well V 3 using a radial flow model in finite differences. Only the leakage factor was varied in the model to get a close simulation of the water levels in the observation wells. The parameters S and T were taken from the results of the graphical analysis. The water levels recorded in the observation wells showed a steady state condition which could not be simulated with any leakage factor when a free head boundary was assumed at the radius of influence due to instabilities in the computations. Instability originated when more recharge took place while pumping than the rate of abstraction. The best simulation was obtained when a leakage factor of 6000 m with a fixed head boundary at a radius of influence 300 m. The results of the runs executed are shown in fig. 5.1 and 5.2. A leakage factor of 6000 m corresponds to a vertical permeability of 0.003 m/d taking the thickness of the aquitard as 50 m. The details of the radial flow calculation are given in appendix B.

5.4 Water balance for the water table aquifer.

In order to estimate the amount of leakage into the lower limestone aquifer a water balance study was carried out based on available data for the period September 1979 to August 1980. Due to inadequacy of most of the relevent data the computation cannot be regarded as very reliable.

The water balance equation for a one year period can be written as follows:

$$I - O - L - A - E = S \qquad 5.1$$

where,

I = Percolation from the soil moisture zone

O = Outflow into sea and KalaOya.

L = Leakage into limestone aquifer

A = Amount of abstraction from the wells.

E = Evaporation from open water bodies and places where the ground water table is close to the surface.

S = Change in storage in the water table aquifer.

5.4.1 Percolation from the soil moisture zone.

A water balance study carried out by Lawrence and Dharmagunawardena(1981) has been used to estimate the percolation. In this study weekly data on pan evaporation and rainfall in one place in the area has been used to calculate the soil moisture deficiency (S.M.D.) assuming different root constants for different types of crops as given in table 5.1. It has been assumed that at the end of dry season before rain started the soil moisture deficiency to be at its maximum value, that is equal to root constant. When the S.M.D. is equal to the root constant the evaporation was taken as zero and when S.M.D. is zero the evaporation was assumed to be at the potential rate. When S.M.D. is zero any excess rain over the potential evapotranspiration is assumed to percolate to the water table.The summary of the calculation is reproduced in table 5.1.

The calculation assumes that the surface runoff is zero which is reasonably correct as the very high permeable latesol absorbs all the rain falling on it.

According to these calculations the total percolation is 15.9 million cubic metres(MCM). The least percolation was from the forest areas and this may give an explanation for the rise in piezometric levels in the limestone aquifer over a period of 12 years by about 6m, which can now be considered as the effect of increase of recharge due to removal of forest cover.

Table 5.1 Water balance for soil moisture zone.

Type of vegetation	Root constant mm	Area sq. km.	Percolation mm	Recharge MCM
Annual crop grass land	50	6.4	550	3.5
Cocónut mangoes	150	24.8	339	8.4
Forest cover	300	48.8	83	4.0

5.4.2 Outflow into sea and Kala Oya.

The water levels in 12 shallow hand dug wells are available throughout the area. This number of observation points is inadequate to complete a water table contour map for the area, as most of the wells are confined near the centre of the area. However a contour map was drawn approximately in order to estimate the hydraulic gradient at the discharging area. The outflow was calculated assuming a constant transmissivity of 20 m^2/day. The length of the flow path was measured as 15km. (fig. 5.3)

Hydraulic gradient = 10.0/1250

Outflow = (10.0/1250) * 20.0 * 15000 m^3/day

= 2.63 MCM/year

5.4.3 Abstractions

The abstraction from water table aquifer was only for domestic purposes by a population of about 10,000. Assuming a consumption of 50 lpcd the total abstraction amounts to about 0.2 MCM/year.

5.4.4 Evaporation from open water surface.

There are a number of low lying areas in the region which are thought to have formed by collapsing of the limestone caverns below the surface locally known as 'villus'. These villus intersects the water table. The total area of water surface in the villus is 1.3 sq.km. and assuming that the twice as much area is saturated under the influence of water table in the villus the total area from which evaporation take place at the potential rate is 3.9 sq.km. The measured pan evaporation for the year was 1300mm and hence the total evaporation is 3.9 * 1.3 = 6.6 MCM/year.

5.4.5 Change in storage.

The period considered was one water year so that the change in storage was fairly small except in a few locations. In the low lying areas there was an average increase in water level by about 0.5 m. Assuming a specific yield of 5% and an area of 40 sq.km. the increase in storage will be around 1.0 MCM. The water table variation for the areas is given in fig.5.4.

5.4.6 Leakage into the limestone aquifer.

All the variables except leakage L in equation 5.1 have been evaluated independantly so that the only remaining unknown is L. The solution gives the leakage as 5.4 MCM per year.

5.5 Water balance for limestone aquifer.

The piezometric levels in the limestone aquifer are available for about 12 boreholes since August 1979. All the bore-holes are concentrated on the middle strip of the region where transmissivity is somewhat higher. No data are available in the north western and southern parts of the region. Therefore it is difficult to draw a piezometric map of the area. Such a map drawn using the available information regarding the boundary conditions is shown in fig.5.5. Details of a flow net analysis is shown in table 5.2. The transmissivities have been obtain by averaging across the flow path.

The total abstraction prior to the line considered for flow calculation is 1640m m^3/day. The flow across the line considered was 10700 m^3/day. Hence the total recharge into the aquifer is 12300m^3/day. This is equal to an annual recharge of 4.5MCM. The recharge obtained in the water balance study for the water table aquifer was 5.4 MCM/year, and the difference between two estimates is about 20% . Considering the nature of the data available the difference is reasonable.

Table 5.2 Through flow calculation for limestone aquifer.

Flow path number	T m^2/day	Length m	hydraulic gradient	Flow m^3/day
1	100	4500	.001 6	720
2	500	1000	.0024	1170
3	750	1600	.0050	6000
4	1000	1000	.0027	2660
5	100	1000	.0016	160

5.6 Leakage from the limestone aquifer.

The leakage believed to take place in the north from the limestone aquifer near Kala Oya cannot be estimated accurately as there is no information about the vertical permeability and the extent of the leakage area. Considering the topography and the vegetation in the area, the extent of the seepage area was estimated as 12 sq.km. The average ground water gradient of 0.15 and a vertical permeability of 0.003 m/day were assumed to estimate the upward seepage. This amounts to about 5 Ml/d which was taken as an initial estimate for the numerical computatiom.

5.7 Transmissivity and storage coefficient.

The results of seven pump tests carried out in the area has been used to draw a transmissivity contour map of the area. Some of the tests are single well tests while three tests are of long duration using one or more observation wells. The results reported appears reasonable (Lawrence and Dharmagunawardena 1981) as can be seen from the analysis using a radial flow model. The transmissivity varies from about 100m^2/d in the east and south to about 2000 m^2/d in the north central parts. The map drawn on the basis of these results have been used in the model with minor modifications. The storage coefficient reported varies from about 0.008 to 0.0001 and a consatnt value of 0.0005 was used in the model. The results of the pump tests are reproduced in table 5.3.

5.8 Abstraction from the limestone aquifer.

No records of abstraction rates are available before 1979. The abstraction mainly take place for irrigating small farms having diversified crops, and hence the abstraction is uniform throughout the year. The cement corporation is the largest single user of this water abstracting nearly half a million cubic metres a year which is approximately one third of the total abstraction. The total abstraction in the year 1979 was 1.22MCM. (Table 5.4) In the model the ratio of the abstraction from each well to, total abstraction was assumed to remain constant.

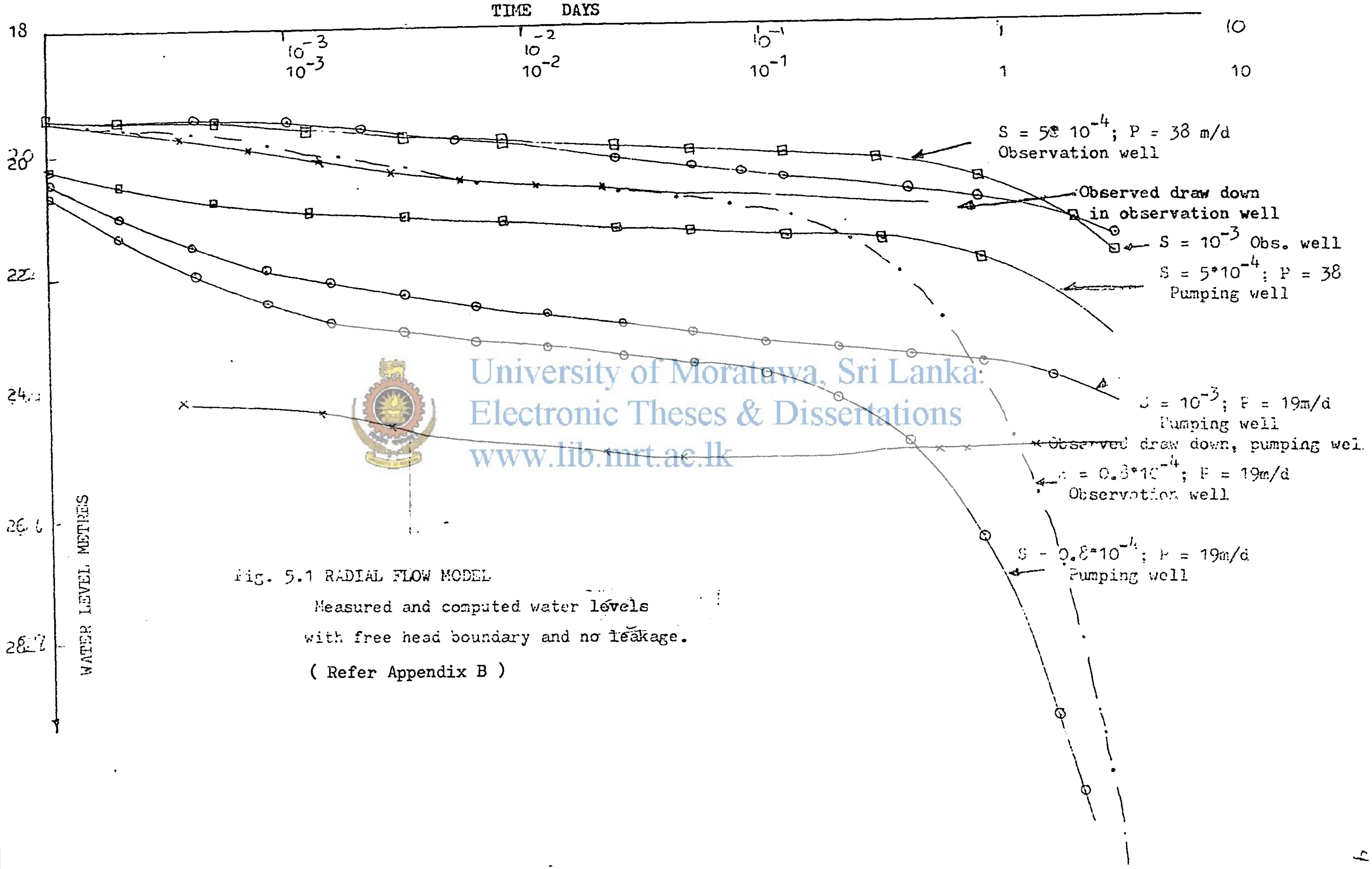


Fig. 5.1 RADIAL FLOW MODEL

Measured and computed water levels
with free head boundary and no leakage.

(Refer Appendix B)

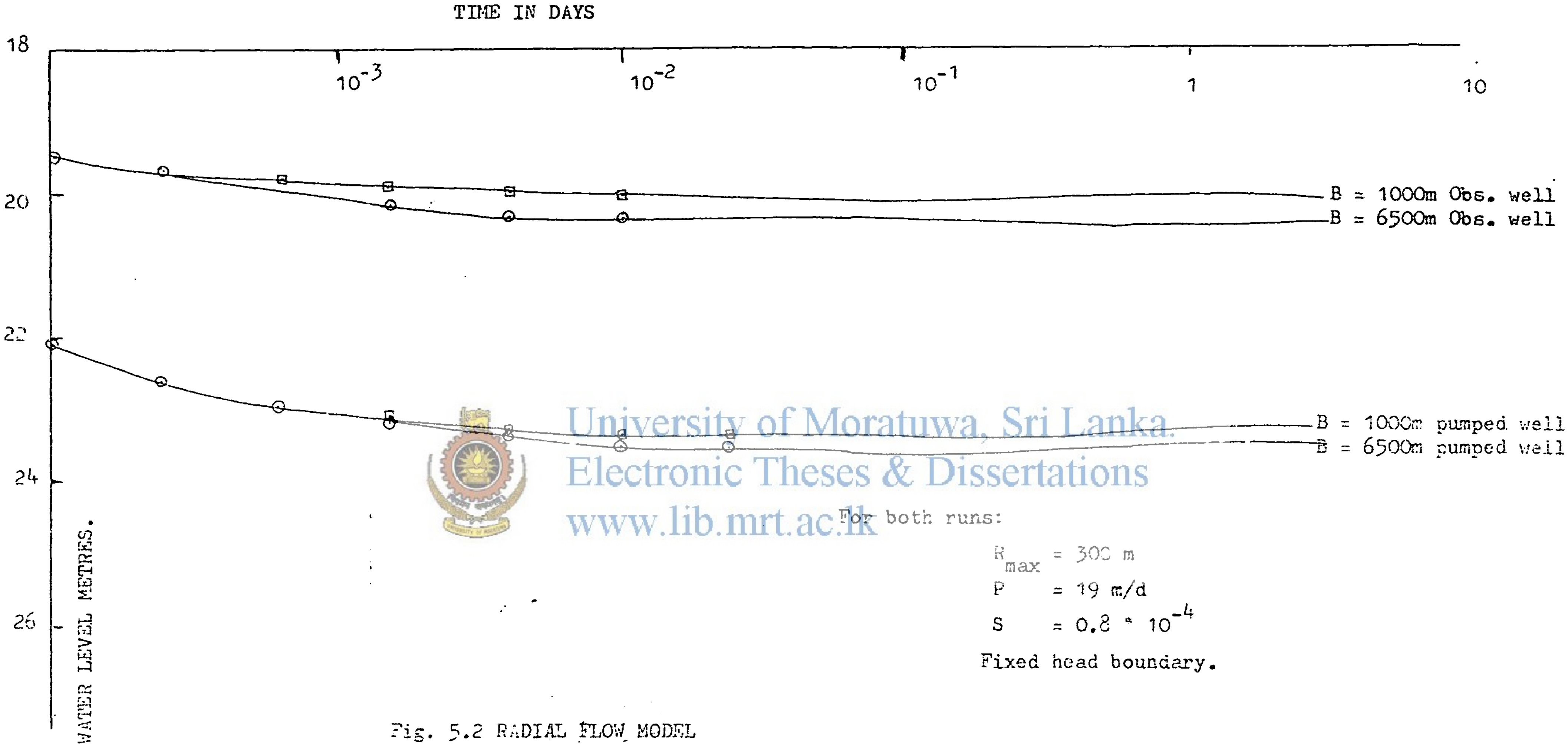


Fig. 5.2 RADIAL FLOW MODEL
Model runs with leakage and
fixed head boundary.
(Refer Appendix B)

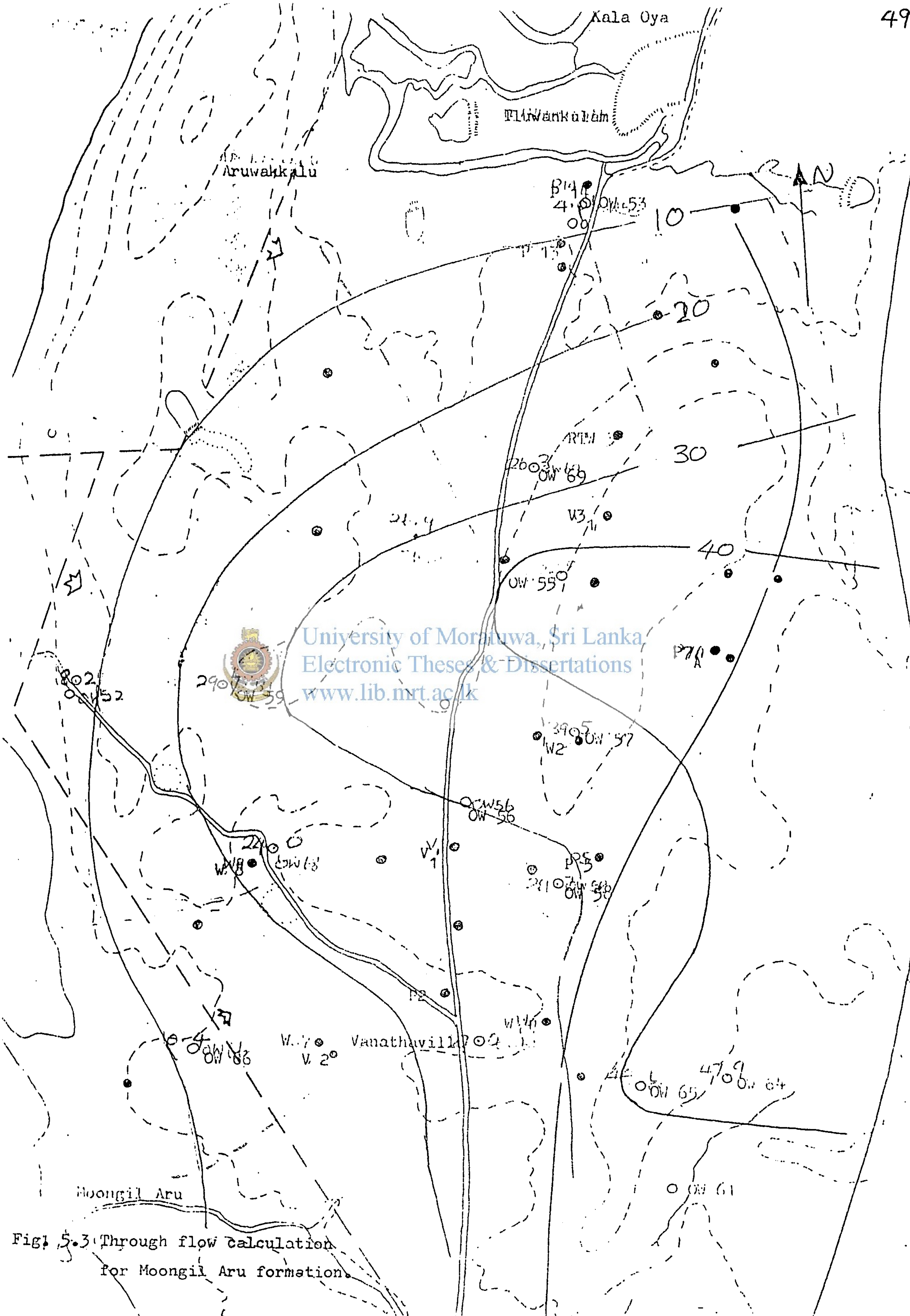


Fig. 5.3 Through flow calculation
for Moongil Aru formation.

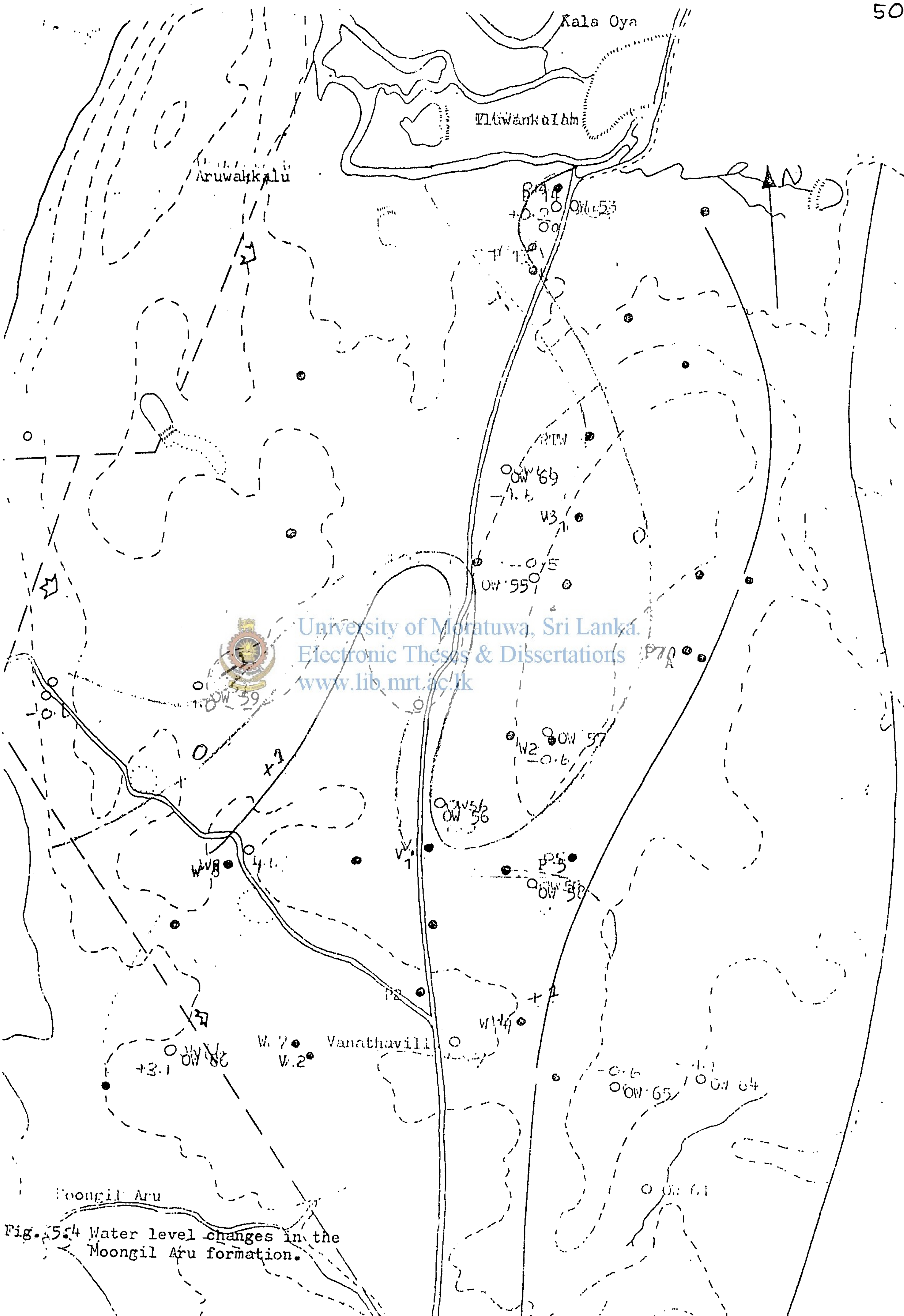


Fig. 5.4 Water level changes in the Moongil Aru formation.

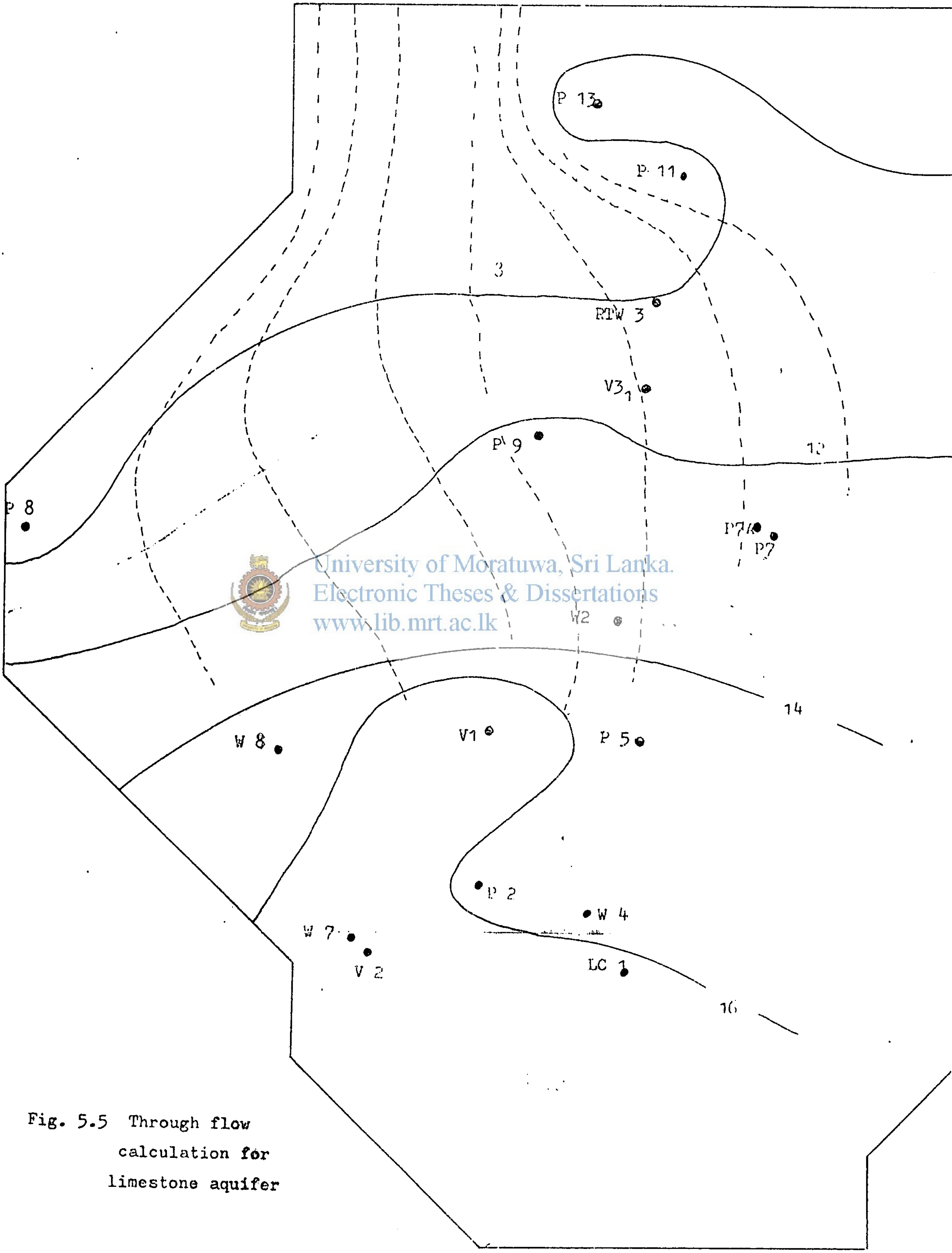


Fig. 5.5 Through flow calculation for limestone aquifer

RESULTS OF MIOCENE AQUIFER TESTS, INCLUDING OBSERVATION WELL DATA

SITE	TYPE OF TEST	DISCHARGE (l/s)	METHOD OF ANALYSIS	TRANSMISSIVITY (m^2/d)	STORAGE COEFFICIENT	LEAKAGE FACTOR(m)	VERTICAL PERMEABILITY OF OVERBURDEN (m/d)	REMARKS
V1 1 obs.well at V1-3	72 hours constant discharge	27	Hantush	727	6×10^{-5}	1065	.04	
			Walton	737	9×10^{-5}	784	.07	Specific capacity = 4.53 l/s/m
V3 (3 obs.wells V3-1, RTW3, ADW3)	72 hours constant discharge		V3-1 Hantush	1735	3×10^{-4}	1592	.027	" = 10.0 l/s/m
			V3-1 Walton	1639	3.5×10^{-4}	1114	.053	
			RTW3 Hantush	2407	9×10^{-5}	4323	.004	
			RTW3 Walton	2857	9×10^{-5}	5620	.003	
			ADW3 Hantush	2565	1.3×10^{-4}	3194	.0075	
			ADW3 Walton	2786	1.35×10^{-3}	3833	.006	
P2 (1 obs.well P2A)	420 minutes constant discharge	22.5	Theis	483	5.7×10^{-5}	-	-	" = 2.14 l/s/m
P9 (1 obs.well P9A)	24 hours constant discharge	25.5	Theis	734	3.7×10^{-3}	-	-	" = 3.42 l/s/m
P13 (1 obs.well P13A)	350 minutes constant discharge	18	Walton	2590	2.6×10^{-4}	-	.066	-

Table 5.3 Results of the pump tests for the limestone aquifer.

RESULTS OF SINGLE WELL TESTS OF THE MIOCENE AQUIFER

WELL NO.	DISCHARGE (l/s)	DRAWDOWN (m)	SPECIFIC CAPACITY (l/s/m)	TRANSMISSIVITY (m^2/d) From specific capacity graph	Logans Method
W7	2.8	20	0.14	30 (approx)	16
W8	19.94	8.42	2.37	410	250
W9	10.5	6.7	1.57	280	165
P5	0.680	5.6	0.12	25 (approx)	12.7
P7A	0.27	1.64	0.17	30 (approx)	16.5
V5	1.5	3	0.5	90	-

Table 5.3 (contd.) Results of the pump tests for the limestone aquifer.

TABLE 5.4

~~APPENDIX 3~~

ESTIMATES OF THE VOLUME OF ANNUAL ABSTRACTION OF GROUNDWATER AT VANATHAVILLU FOR DOMESTIC, INDUSTRIAL AND IRRIGATION PURPOSES, 1980

Well No.	Estimated Volume of abstraction 1980 (m^3/year)	Pumping Rate (m^3/hour)	ABSTRACTION FOR DOMESTIC AND INDUSTRIAL USES			ABSTRACTION FOR IRRIGATION USE				Remarks
			Days pumped	Average Hrs. Pumping/day	Volume pumped	Days pumped	Av. Hrs. pumping. per day	Volume pumped	Irrigated acreage	
P 2	443 150	59	365	2	43 150	–	–	0	0	–Well operated by Irrig. Dept. for Vanathavillu Township
P 8	11 800	1.35	365	2.4	11 800	–	–	0	0	–Artesian for Karaitivu Township
P 9	103 950	59	365	2	43 150	300	4	70 800	25	–Well operated by Irrig. Dept. for latosol crops on settlement
P11	398 000	90	365	12	398 000	–	–	0	0	–Well operated by Cement Corporation for industrial use
P14	94 600	10.8	–	–		*365	24	94 600	25	–Artesian, irrigates adjacent paddy
W 3	70 000	63	–	–		*300	5	70 000	25	–Well operated by a private company for latosol crops
W 4	104 500	63	365	1.5	34 500	300	3.5	66 000	30	–Well operated by Irrig. Dept. for latosol crops on settlement
W 5	43 590	27.3	365	1.5	14 950	0	3.5	28 640	25	– ditto –
W 6	131 000	82	365	1.5	44 900	300	3.5	86 100	20	– ditto –
ADW 1	81 900	68	365	2	44 000	300	2	40 900	5	–Well Operated by Dept. of Agric. Res. Sta. for latosol crops
ADW 3	122 700	68	–	–	0	300	6	1222700	15	– ditto –
ADW 4	18 800	62.7	–	–	0	300	11	18 800	14	– ditto –
Total: abstraction	1223 990									

* = a small volume of abstraction is used for domestic purposes, included in the irrigation use category.

Table 5.4 Abstractions from the Limestone aquifer.

CHAPTER SIX: RESULTS OF THE MODEL RUNS

6.1 General remarks.

As outlined in chapter one and five, the nature of the data available does not permit computations with a definte set of data. The computations carried out are mainly designed for obtaining a reliable set of data which are consistent with field observations using a somewhat trial and error approach. A large number of computer runs were made using different boundary conditions and inflows and out flows.

6.2 Geometry of the aquifer boundary.

Initially the aquifer was assumed to be of rectangular shape with uniform grid as showm in fig.6.1. The purpose of this run was to identify the effect of varying the recharge and transmissivity on piezometric levels. Later the geometry was changed as shown in fig.6.2., so that the boundaries coincide with assumed geological boundaries. The effect of changing the shape of the boundaries were not significant as can be seen from the plot of piezometric levels in a north south section through the centre of the area for both cases. The type of boundaries and the abstraction rates for both computations were kept the same.

6.3 Sensitivity analysis.

A sensitivity analysis was carried out for the model with rectangular boundary to investigate the effect of varying transmissivity, storage coefficient and inflows. Six computer runs were carried out for this purpose by varying the parameters as shown in table 6.1.

Table 6.1 Summary of data for sensitivity analysis.

Run no.	Recharge Ml/d	Abstraction Ml/d	Tx m^2/d	Ty m^2/d	S
	4.5	0	400	400	
1	6.5	0	500	500	0.0001
	6.5	3.3			
	6.5	2.0	400	400	
2	8.5	3.3	500	500	0.0005
	10.0	2.0	400	400	
3	10.0	3.3	500	500	0.0005
	5.0	2.0	400	400	
4	5.0	3.3	250	250	0.0005
	5.0	2.0	400	400	
5	5.0	3.3	500	500	0.0005
	10.0	2.0	200	200	
6	10.0	3.3	250	250	0.0005
	10.0	2.0	400	400	
7	10.0	3.3	500	500	0.0001

Example: The run 3 was made with the following data.

Total recharge (1st block)	=	10.0Ml/d
Total recharge (2nd block)	=	10.0 Ml/d
Abstraction (1st block)	=	2.0 Ml/d
Abstraction (2nd block)	=	3.3 Ml/d
Storage coefficient	=	0.0005
Overall transmissivity	=	400 m^2/d
Maximum transmissivity	=	500 m^2/d

For all computations the fixed head at the northern boundary was taken as 4.0 m above mean sea level.

The recharge is distributed in an area of 20 sq. km. as shown in fig. 6.2. The distribution of abstraction from the wells is shown in table 6.2.

Table 6.2 The distribution of abstraction.

Location I	J	Fraction of total abstraction.
7	11	0.0350
3	8	0.0097
8	7	0.0850
9	4	0.3251
8	3	0.0774
8	10	0.0573
8	12	0.0855
7	11	0.0357
8	9	0.1072
9	5	00.1821

An overall transmissivity of 400 m^2/d and a maximum transmissivity of 500 m^2/d was arbitrarily assumed in order to find the sensitivity of the aquifer. The location of the high transmissivity points were selected in accordance with the transmissivity contour map.

The piezometric surface for a north south line passing through the centre of the area is shown in fig. 6.3. The piezometric surface obtained was reasonably close to the observed piezometric levels. except at the nothern boundary. A second run with all data same as the first one but transmissivity reduced by 50% was made. The resulting piezometric levels were significantly higher than observed levels showing high sensitivity of piezometric levels to variation in transmissivity.

The run 4 was made with both recharge and transmissivity reduced by a factor of two . (curve 2) The resulting piezometric surface was only slightly different from that of curve 1 indicating that there may exist some other combination of transmissivity and recharge different from those used for run 3 which produce piezometric levels similar to observed data, However in such a case the piezometric level variation with time will be different from those observed unless the recharge values are realistic.

Computer runs 2 and 5 were made with data similar to those of run 3 but recharge reduced by 15% and 50% respectively. When the recharge was reduced by 15% the resulting piezometric levels were slightly low but the time variation at one point showed a greater fluctuation of levels. The piezometric levels when recharge was reduced by 50% were very low. compared with observations.

The computer run 1 was made with all data similar to those of run 3 but the storage coefficient changed to 0.0001. The ~~eff~~ effect of changing the storage coefficient is not very significant when compared with changes in transmissivity and recharge.

From the above sensitivity analysis it is clear that transmissivity and recharge values effect significantly the piezometric levels. If a mathematical model is to be used effectively in predicting the behaviour of the aquifer both recharge and transmissivity values have to be known accurately. If at least one of these is not known accurately it is futile to attempt to develope a mathematical model. Fortunately in the Vanathavillu aquifer the transmissivity values appear to be reliable so that a mathematical model could be attemted.

6.4 Model for Vanathavillu aquifer.

6.4.1 Model parameters.

The aquifer boundary was taken as shown in fig. 6.2. The basis for this schematisation was outlined in chapter 5.1. The area was represented in a rectangular mesh as shown in the figure. A mesh size of 1000 m was taken in both X and Y directions. The X direction was taken from west to east along the northen boundary. The Y axis was taken vertically downward. The choice of the mesh size was based on the following reasoning:

1. The computational grid consists of 13 X16 grid points which is a reasonable number to work with the available micro computer. The computer time required for one computational step was approximately 30 s and assuming atleast 36 time steps are required for each year, to compute for three years of data the time required is about 40 minutes.
2. Not more than 15 observation points are available to get the information about the aquifer behaviour and the distance between these points are always more than 1 km. Therefore no accuracy would be gained by reducing the grid size further.
3. The area modeled is too small to take a mesh size larger than 1 km.

All the data used for computations were obtained from maps drawn to a scale of 1: 50,000.

As the finite difference scheme used is unconditionally stable there are no restrictions about the time step to be used other than the accuracy. For problems where large local draw downs occur such as pumping wells Rushton (1973) reported that parasitic oscillations occur when the non dimensional parameter $\Delta t T/b^2 S$ exceeds a certain value. Hrere b is the shortest distance betwwen aquifer boundaries. There are different values reported for this limit but all agree that it should be less than 0.5. In the aquifer under study this parameter is 0.2 for a time step of 10 days. In this calculation,

$$\Delta t = 10 \text{ days.}$$
$$T = 100 \ m^2/d$$
$$b = 10,000 \ m$$
$$S = 0.0005$$

6.4.2 Transmissivity and storage coefficient

The transmissivity at each point was interpolated from the transmissivity contour map (fig, 6.5) and are shown in fig 6.2. The overall transmissivity was taken as 100 m^2/d. The aquifer was assumed to be isotropic. The storage coefficient was taken as 0.0005 throughout.

6.4.3 Recharge.

The recharge into limestone aquifer takes place as leakage from the upper water table aquifer. The location of this leakage area has been indicated by Lawrence and Dharmagunawardena(1981) on the basis of a chemical analysis of waters found in the two aquifers. In the model this area was represented by 16 grid points as indicated in fig. 6.2. The total recharge was distributed pniformly among these points.

6.5 Results of the model runs for Vanathavillu aquifer.

A large number of computer runs were made with data outlined in 6.4. These runs can be roughly grouped into three ctagories. The first set of computations named as group A were made with changing boundary conditions, while flows and aquifer parameters were not changed. The group B computations were made by modifying inflows and outflows, while transmissivities and boundaries were kept unchanged. In the same computations the effect of having an out flow as leakage in the north also was investigated. In group C the aquifer was modelled as a two layer system consisting of water table and limestone aquifers. Th. recharge and outflow from the limestone were modelled as vertical flow from and to the upper Moongil Aru formation. However the water table was assumed to remain constant. The summary of data used for these computations are given in tables 6.3, 6.4 and 6.5.

6.6 Model runs with different boundary conditions.. (group A)

In these computations only the limestone aquifer was considered. The southern and eastern boundaries were assumed impermeable while the effect of changing the other boundaries were investigated.

Run no. A1

The northern boundary was assumed to be fixed with a piezometric level of 4.5 m. Recharge of 16.4 Ml/d and a zero abstraction rate was used. The resulting piezometric levels along a central north south line is shown in fig 6.8(line A1). This was compared with the observed mean piezometric levels for the year 1980. The computed piezometric levels are significantly higher than observed levels. This was due to an over estimate of the recharge as could be seen from later runs.

Run no. A2

This was made with all the data kept the same as for A1 except that the fixed head boundary was extended to the north western corner of the area. (line A2)

Run no. A3

Here the recharge was reduced to one tenth of that for A1. All the other data remained same.

Run no. A4

The fixed head boundary was extended to the north west and the western coast line. The head in the west was taken as zero while along Kala Oya a head of 4.5 m was used. Three computations were made with recharge and abstraction as follows.

Recharge Ml/d	Abstraction Ml/d
16.4	0.0
21.9	0.0
21.9	3.25

Run no. A5

This was made with all data the same as those for A 1 but transmissivity increased by a factor of 10. The results of this was exactly similar to those of A3.

The results of these computations clearly show the effect of changing boundary conditions. When the fixed head boundary was extended to north and north west piezometric levels in the centre of the area became highest causing a flow towards south west. This was contrary to the observations. The piezometric levels were best simulated with a fixed head boundary at the north.

Further it is observed that recharge assumed was an over estimate. The recharge of 16.4 Ml/d (or 6.0 MCM/year) produced much higher piezometric levels. This lead to re- estimation of recharge as outlined in Chapter five. The next series of computations were made with modified inflows and outflows.

6.7 Results of the model runs with different inflows and out flows.

From the sensitivity analysis made in section 6.2 and 6.3 it is clear that modification of inflows and transmissivities in the aquifer is extremely dangerous as both parameters effect the piezometric levels with a high sensitivity. But the analysis of pump test data reveals that transmissivities reported are fairly reliable and could be used in the model without any major change. This gives an opportunity to modify inflows and outflows so as to get an agreement of computed piezometric levels with observed ones.

The two estimate obtained in chapter five for recharge into the limestone aquifer were 5.4 and 4.5 MCM/year giving a mean value of 13.0 Ml/d. The inflow was varied around this figure until an agreement was reached between computed and observed piezometric levels.

<u>Run no. B1</u>

In this a recharge of 8.5 Ml/d and and abstraction of 3.3Ml/d were used. The resulting piezometric levels were lower than observed ones for most parts of the aquifer (see curve B1 fig. 6.9)

<u>Run no. B2</u>

A recharge of 13.0 Ml/d and an abstraction of 3.3 Ml/d were used. The resulting piezometric levels were much higher than observed levels. Further the computed piezometric gradient near the northen boundary was steeper than observed gradient. This may be due to an upward leakage in that area. This phenomenon was investigated in the subsequent set of computations.

<u>Run no. B3</u>

This computation was made with a recharge of 10.0 Ml/d and an abstraction of 3.3 Ml/d. In addition an outflow as leakage 5.0 Ml/d was introduced in an area of 10 sq.km. near the northern boundary. The coordinates of the leakage points were as follows.

I	J
6	3
7	3
8	3
9	3
6	4
7	4
8	4
9	4
5	5
6	5

The results obtained (curve B3) appear to agree with observed piezometric level at all points except at the northern boundary. This discrepancy may be due to following:

1. Incorrect assumption of fixed head at the northern boundary.
2. Incorrect location of leakage area.
3. Differences in local transmissivity.

These have been investigated in the next computer runs.

Run no. B5

This was made with increased recharge and outflow as leakage. The piezometric levels were higher.

Run no. B6

Even though the inflows and outflows used in run no. B3 appear to be consistent with observations the computed piezometric levels near the ouflow boundary differ from observed levels. This difference may be due to the assumption of fixed head at the northern boundary. The discharge of the limestone aquifer is assumed to take place into the Kala Oya estuary under a constant head. This estuary is spread over a wide area and this fact may be incorporated in the model by assuming a low transmissivity at the outflow boundary. This will avoid unneccessary enlargement of the model to represent the leakage area of which no additional information is available. Run B6 was made with transmissivity re-adjusted at the northern boundary. The computed piezometric levels are now in better agreement with observed levels (curve B6)

Run no. B7

This was made with leakage area slightly moved to the south. The new location of the leakage area are as follows:

I	J
7	3
8	3
9	3
7	4
8	4
9	4
8	5
9	5
8	6
9	6

The results appear to give a better agreement than B6. With the same data the abstraction was increased by 50 % to 5.0 Ml/d. The resulting piezometric levels are shown in fig.6.13. With increased abstraction the piezometric levels dropped by about 4 m throughout the aquifer.

6.8 Results of the two layer model.

For these computations water table elevation in the unconfined aquifer was treated as constant. The recharge and outflow were computed depending on the difference in heads between the two aquifers. A vertical permeability of 0.001 to 0.004 as reported in the pump tests were used in the calculation.

The results of these runs are plotted in fig. 6.12. When the vertical permeability of 0.001 m/d was used the piezometric levels were very low. A reasonable agreement with observed levels were obtained when vertical permeability of 0.003 m/d for recharge area and 0.004 m/d for outflow area were used. (curve C3 in fig. 6.12)

From these results it is clear that two aquifers are inter dependant. The recharge into the limestone aquifer depends on the difference of heads in the two aquifers and the vertical permeability. Thus when more water is abstracted from the limestone aquifer, there will be more recharge into it, so that water table in the unconfined aquifer will drop. More over the water table in the Moongil Aru formation is subjected to seasonal fluctuations depending on the rainfall. Any induced leakage or abstraction from the Moongil Aru formation will effect the water table elevation and therefore the recharge into the limestone aquifer. For this reason both layers of the aquifer should be modelled simultaneously as one system.

When the abstraction from the limestone aquifer was increased by 50 % to 5 Ml/d in the one layer model the piezometric levels lowered by more than 4 m. But in the two layer model lowering of the piezometric levels for an abstraction of 7.0 Ml/d was only about 2 m. This shows clearly why both layers have to be modelled simultaneously. (fig. 6.13)

However in the two layer model used the water table elevation was assumed to be constant. Therefore the drawdown computed with this model will be an under estimation, as water table elevation also drops as a result of an increased abstraction. Therefore the validity of the model presented is limited to abstraction rates not very different from the values used for calibration. For predictive purposes a detailed two layer model has to be used.

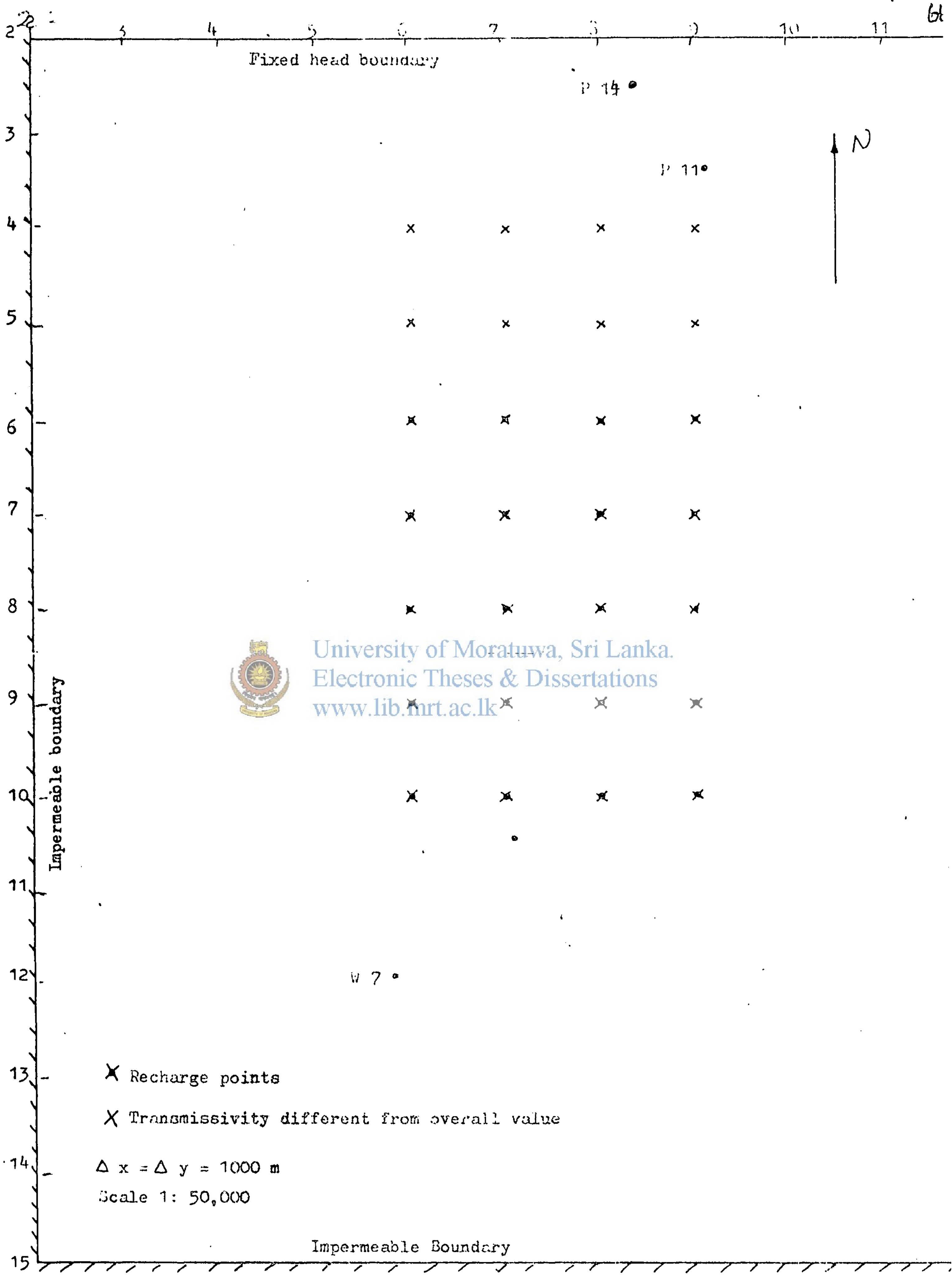


Fig. 6.1 Rectangular area modelled for sensitivity analysis.

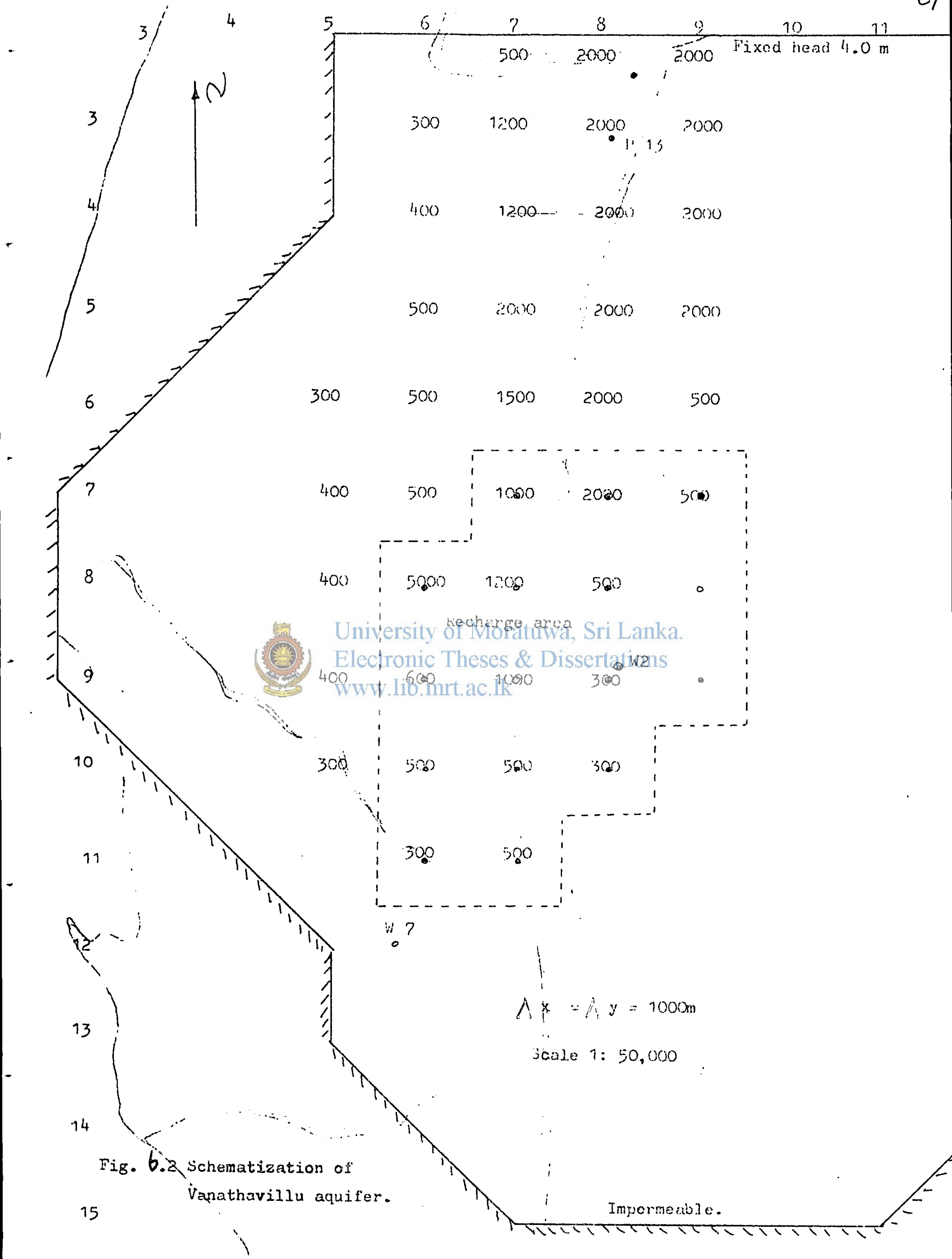


Fig. 6.2 Schematization of Vanathavillu aquifer.

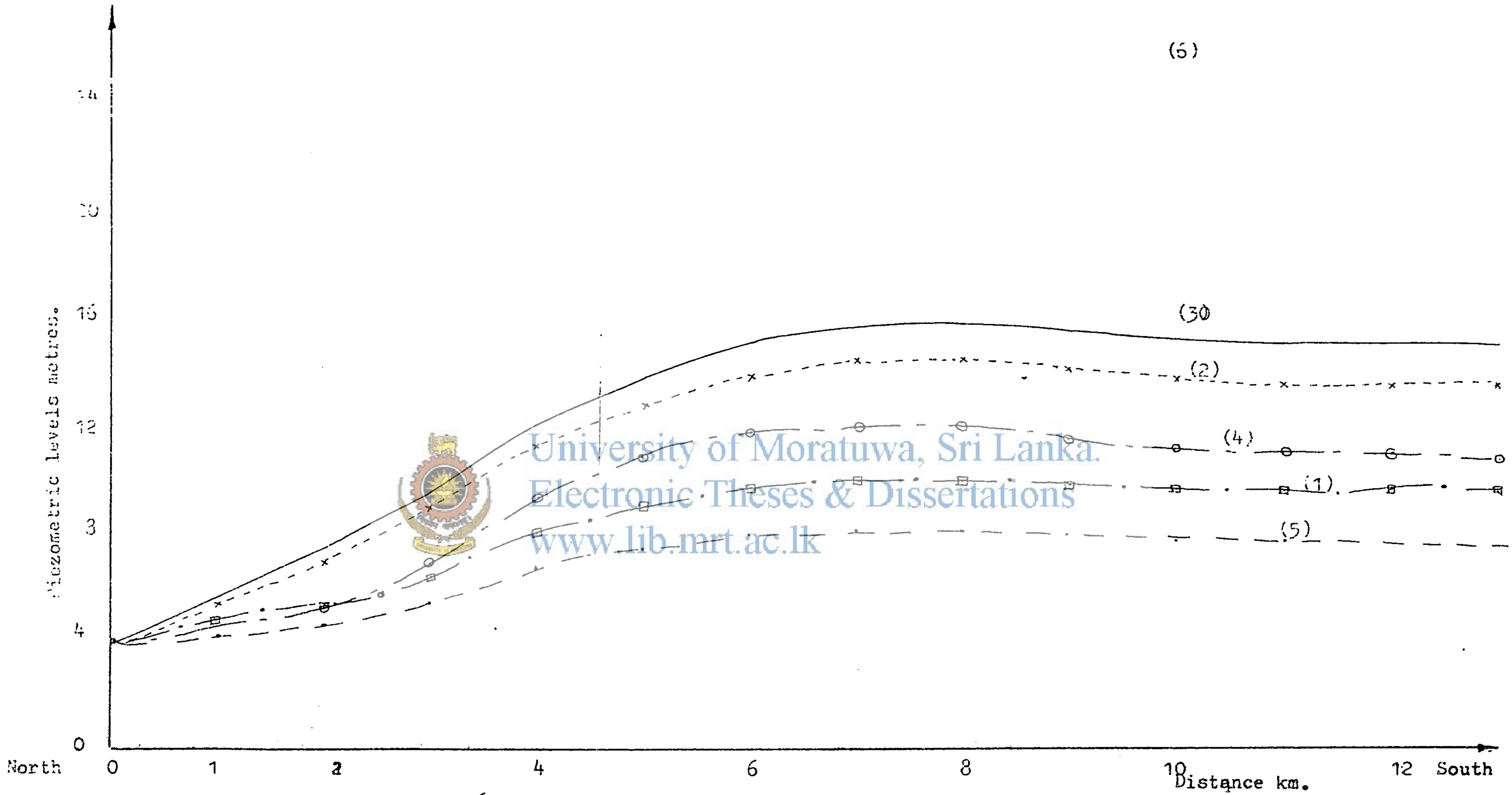


Fig. 6.3 Sensitivity analysis: Piezometric levels along a north south line.
(Refer table 6.1)

(1)
(2)
(3)
(4)
(5)

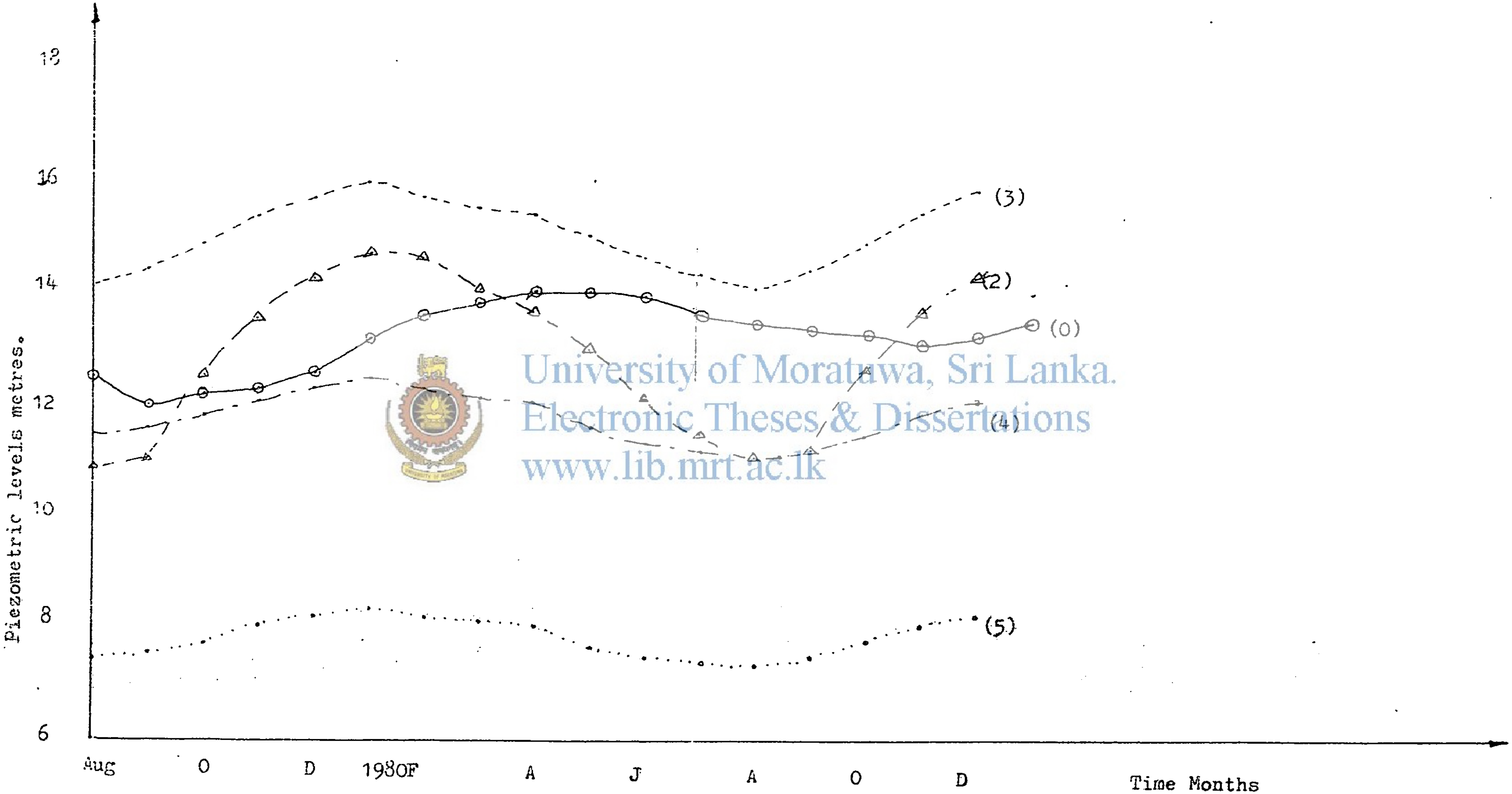


Fig 6.4 Sensitivity analysis: Time variation of piezometric levels near W2
(Refer table 6.1)

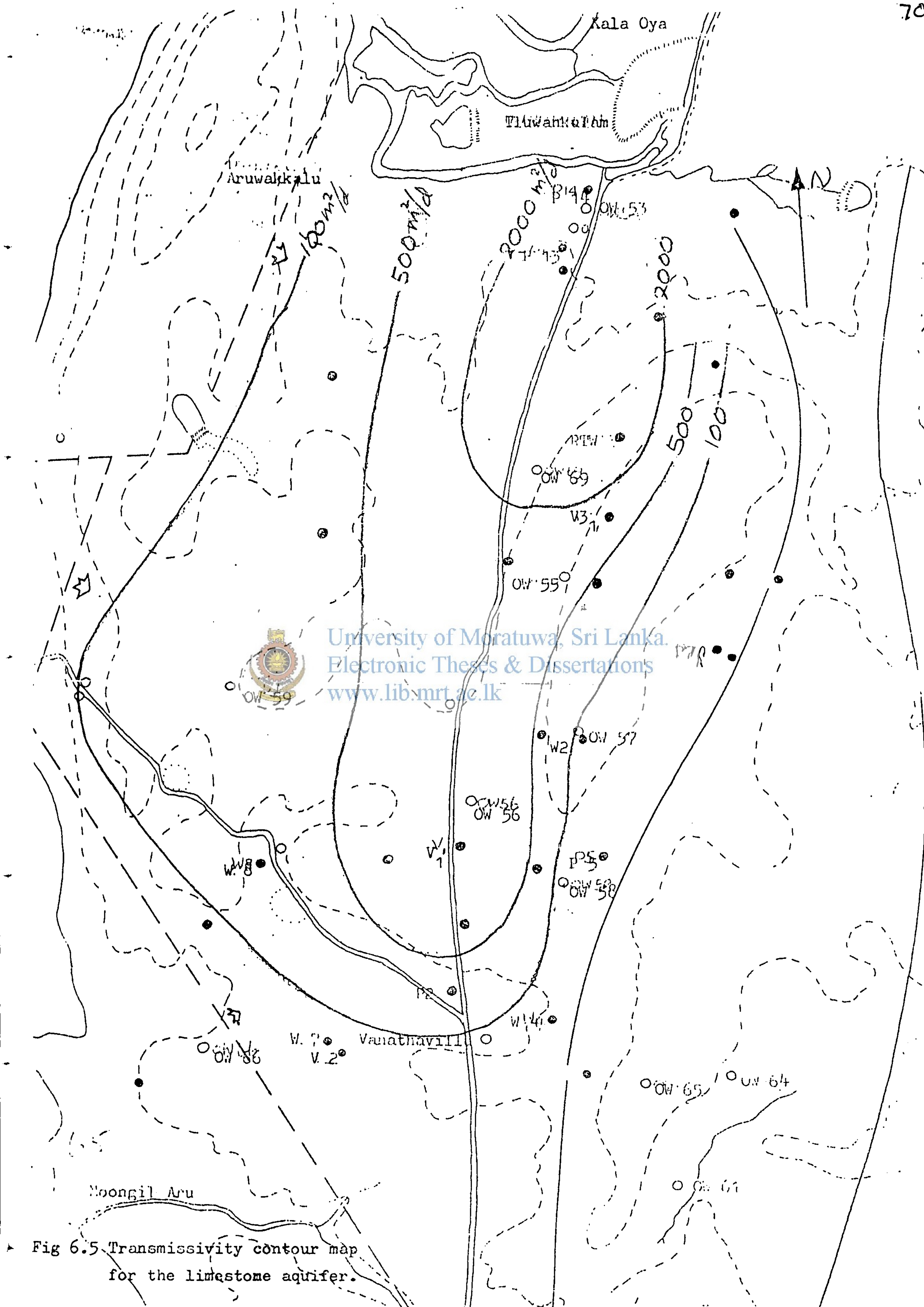


Fig 6.5 Transmissivity contour map for the limestone aquifer.

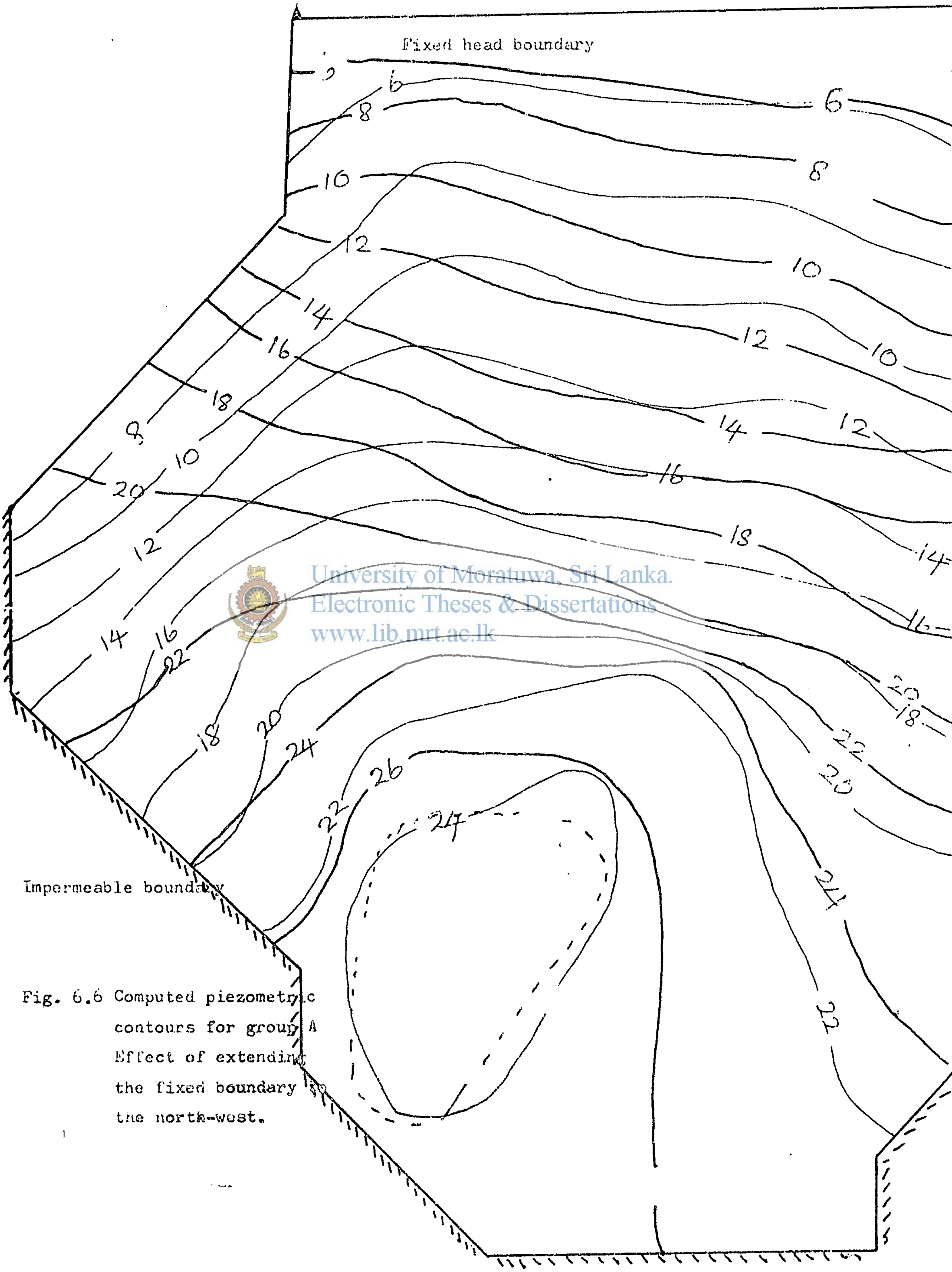


Fig. 6.6 Computed piezometric contours for group A Effect of extending the fixed boundary to the north-west.

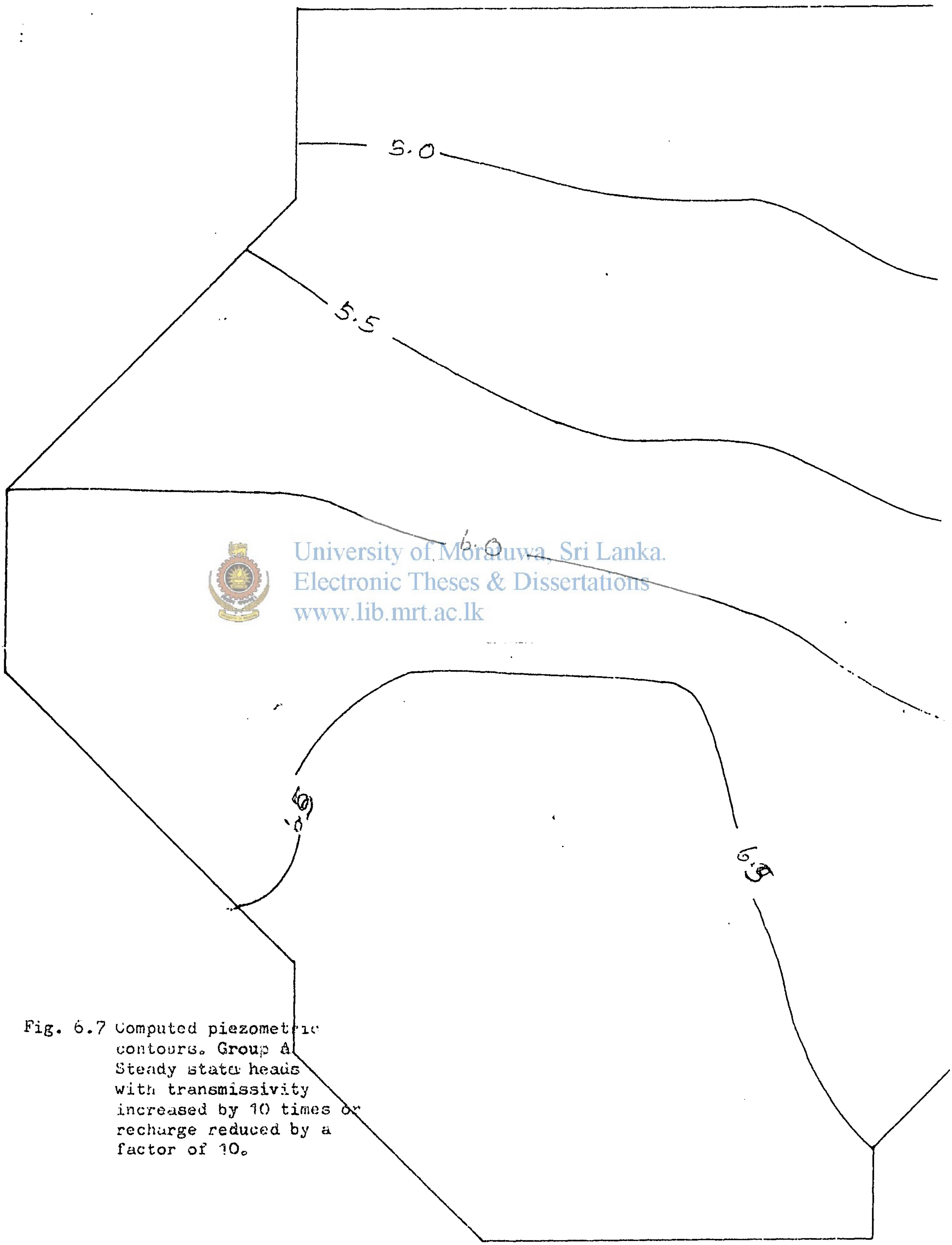


Fig. 6.7 Computed piezometric contours. Group A Steady state heads with transmissivity increased by 10 times or recharge reduced by a factor of 10.

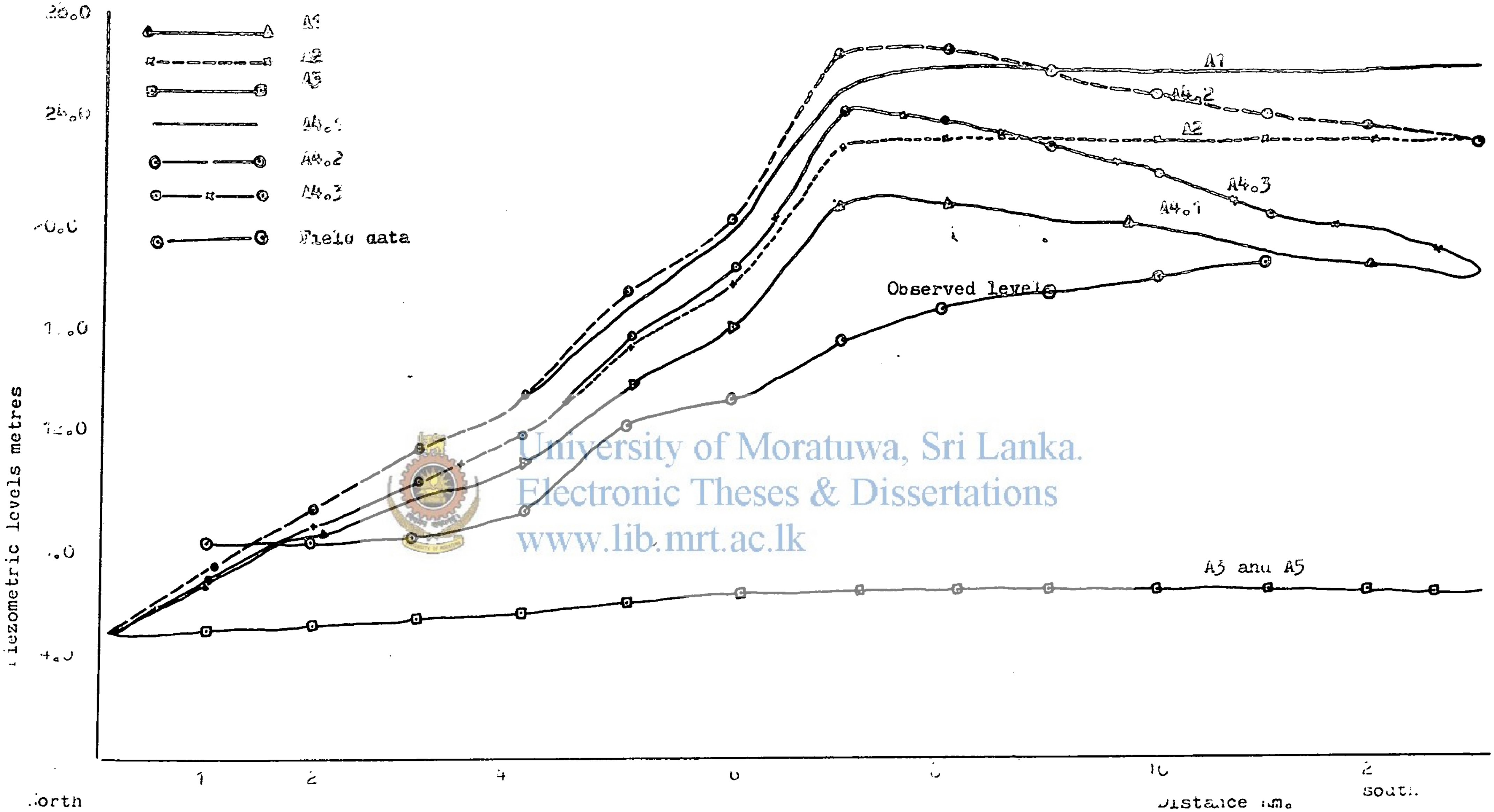


Fig. 5.[illegible] Computed piezometric levels along a north south line
for Group A. (Refer Table [illegible].5)

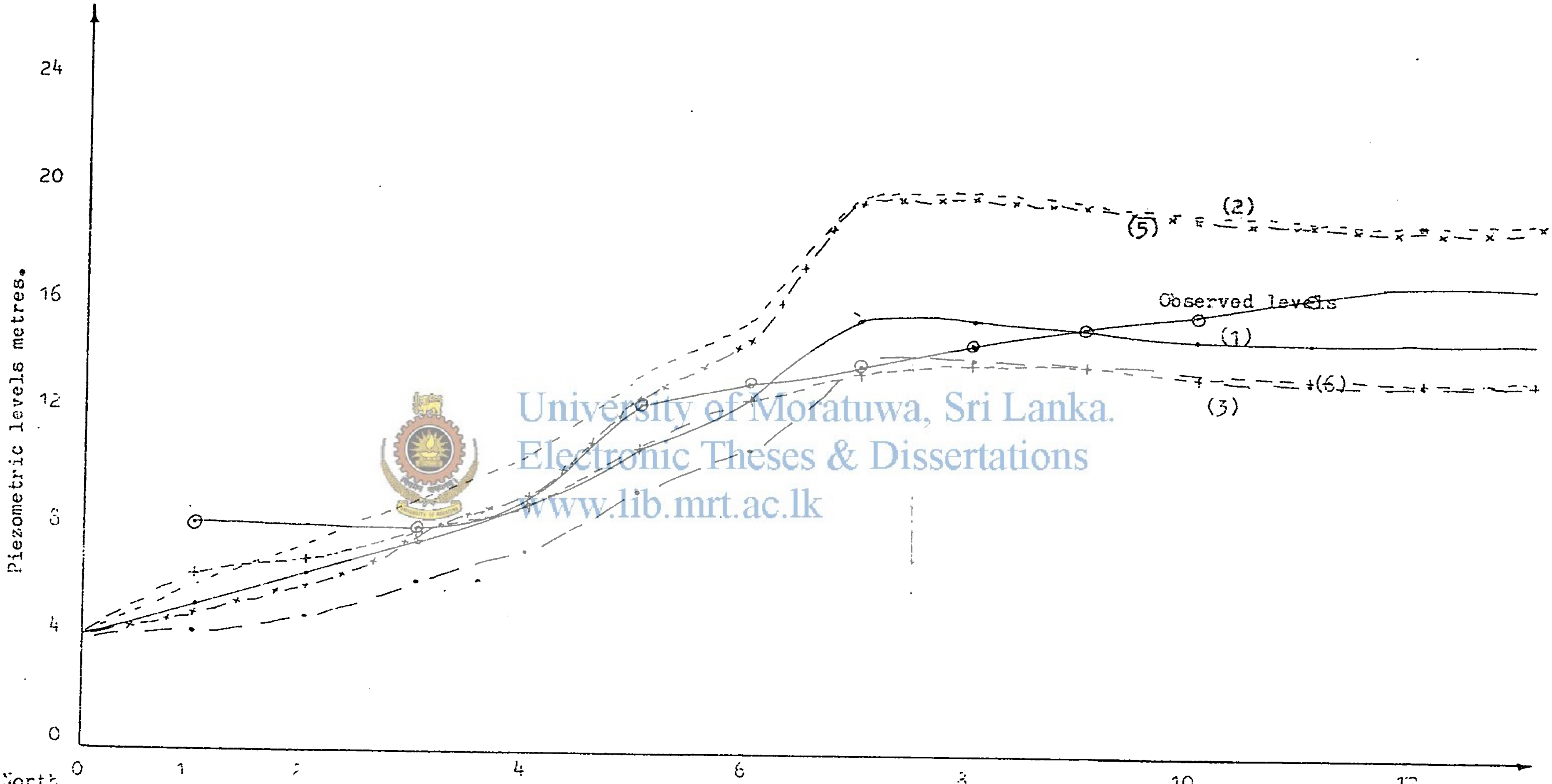


Fig. 6.9. Computed piezometric levels along a nort south line.
Group B (Refer table 6.4)

B1
B2
B3
B5
Observed levels

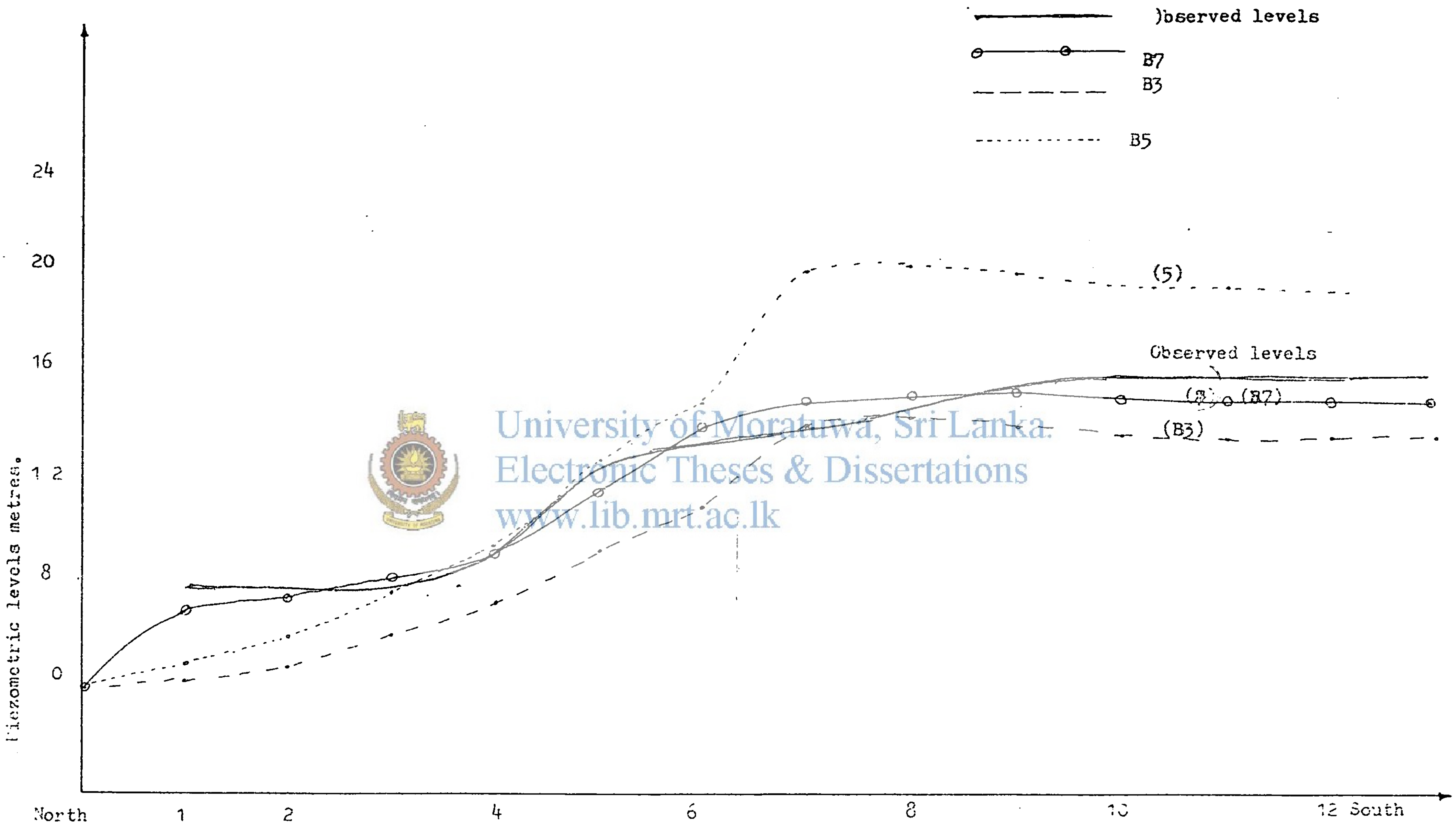


Fig. 6.10 Computed piezometric levels along a north south line.

Group B (Refer table 6.4)

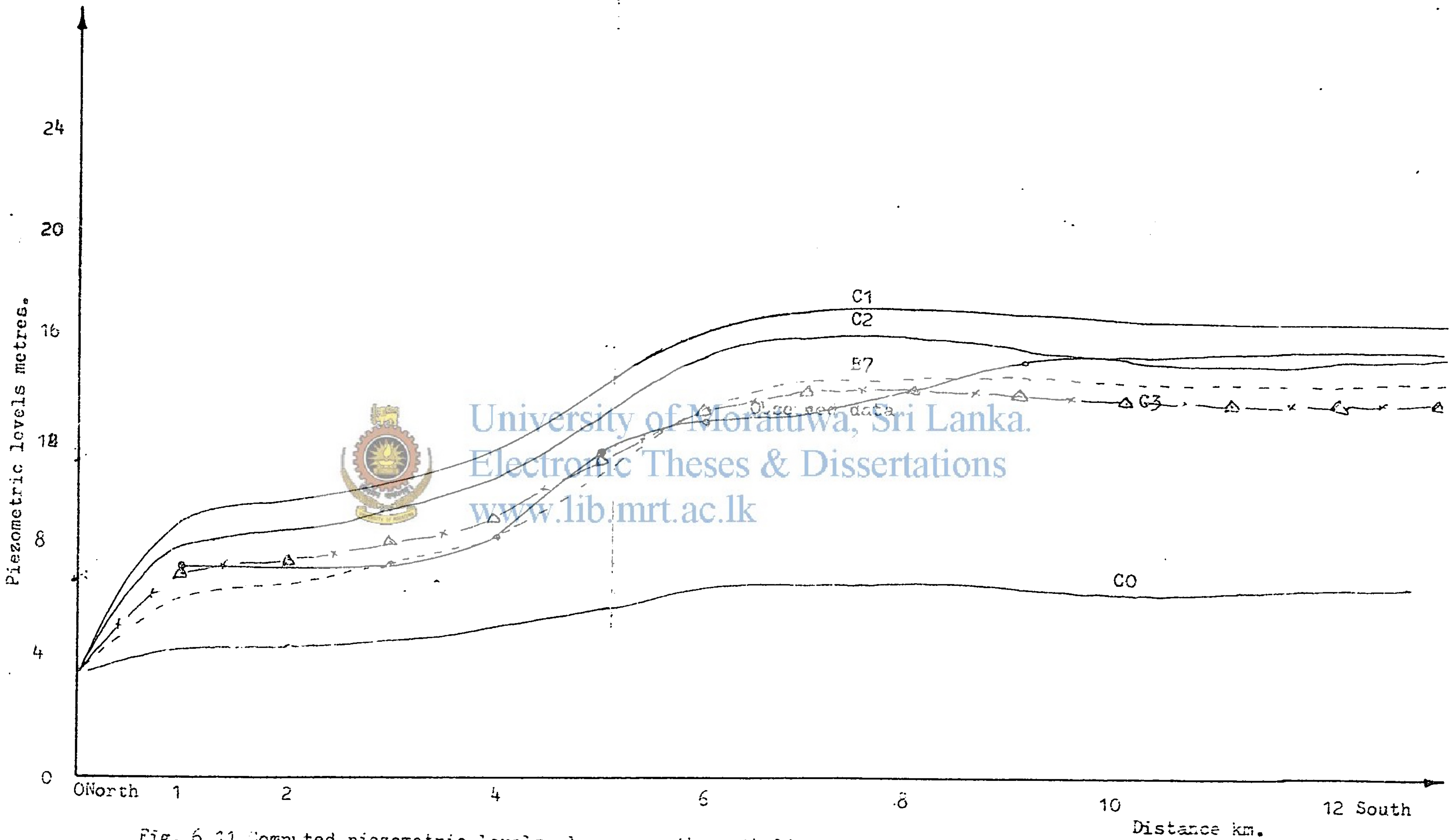


Fig. 6.11 Computed piezometric levels along a north south line.

(Refer table 6.5)

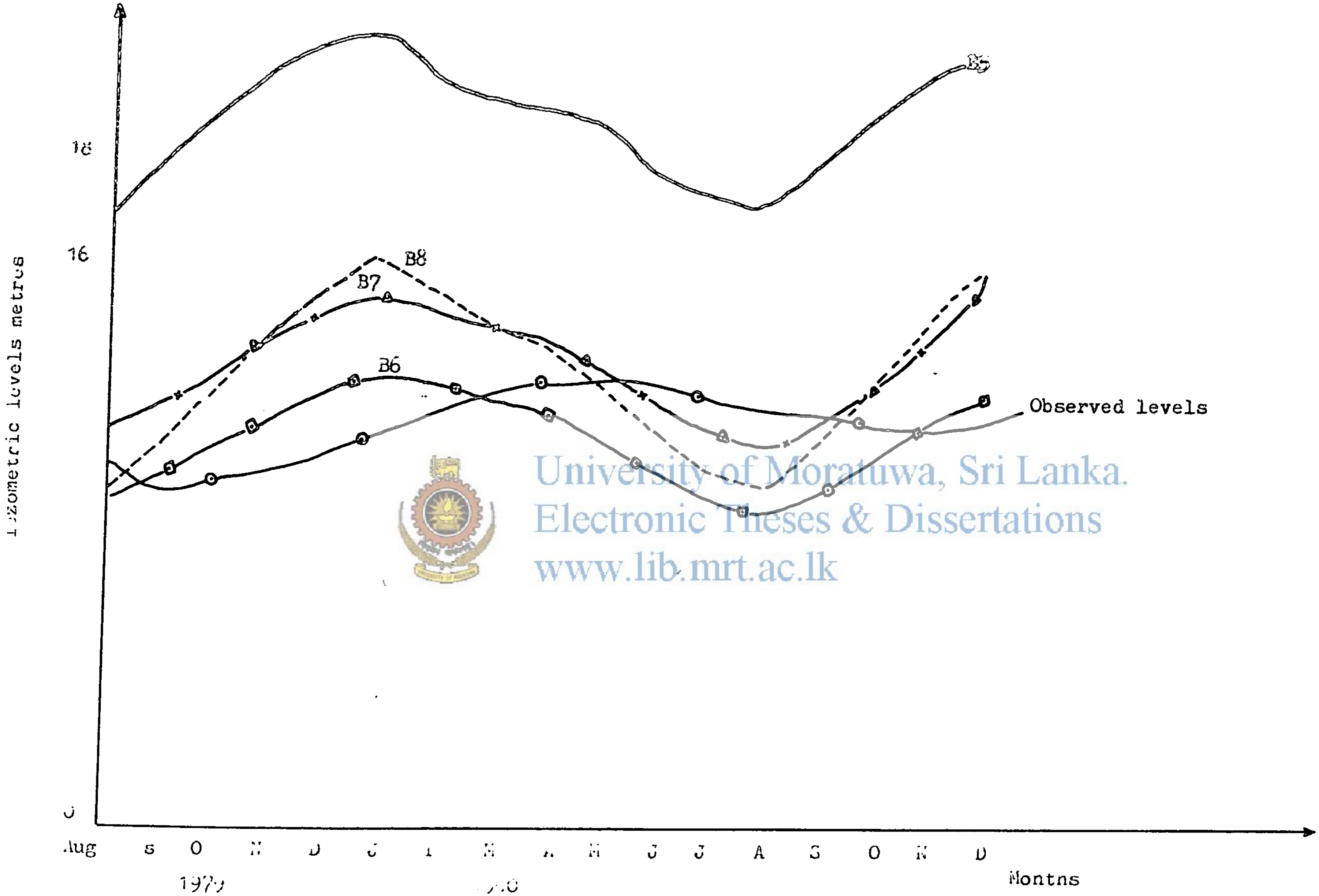


Fig. 6.12 Computed variation of piezometric levels near W2.

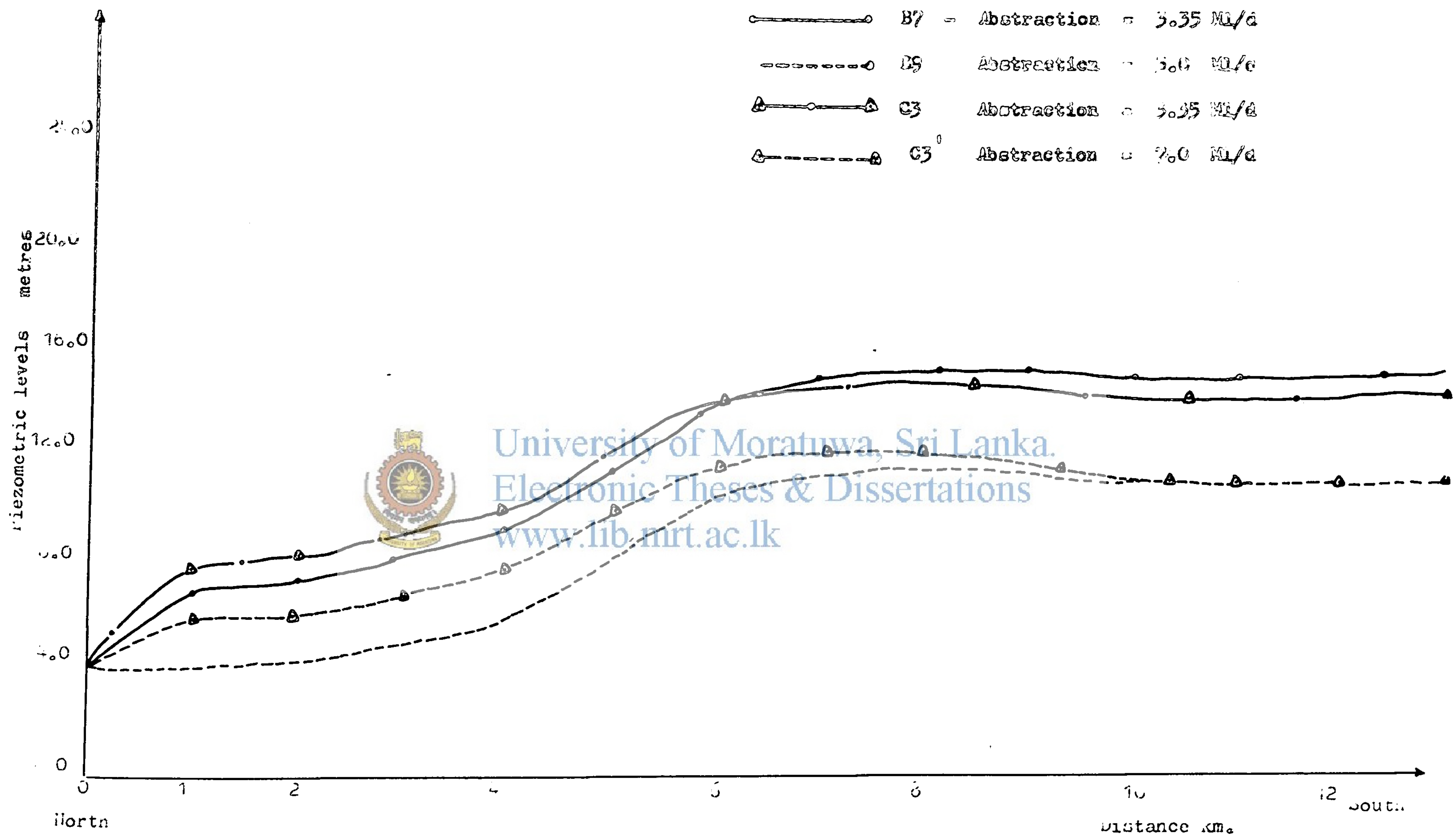


Fig. 5.15 Response of the aquifer to changes in abstraction rates

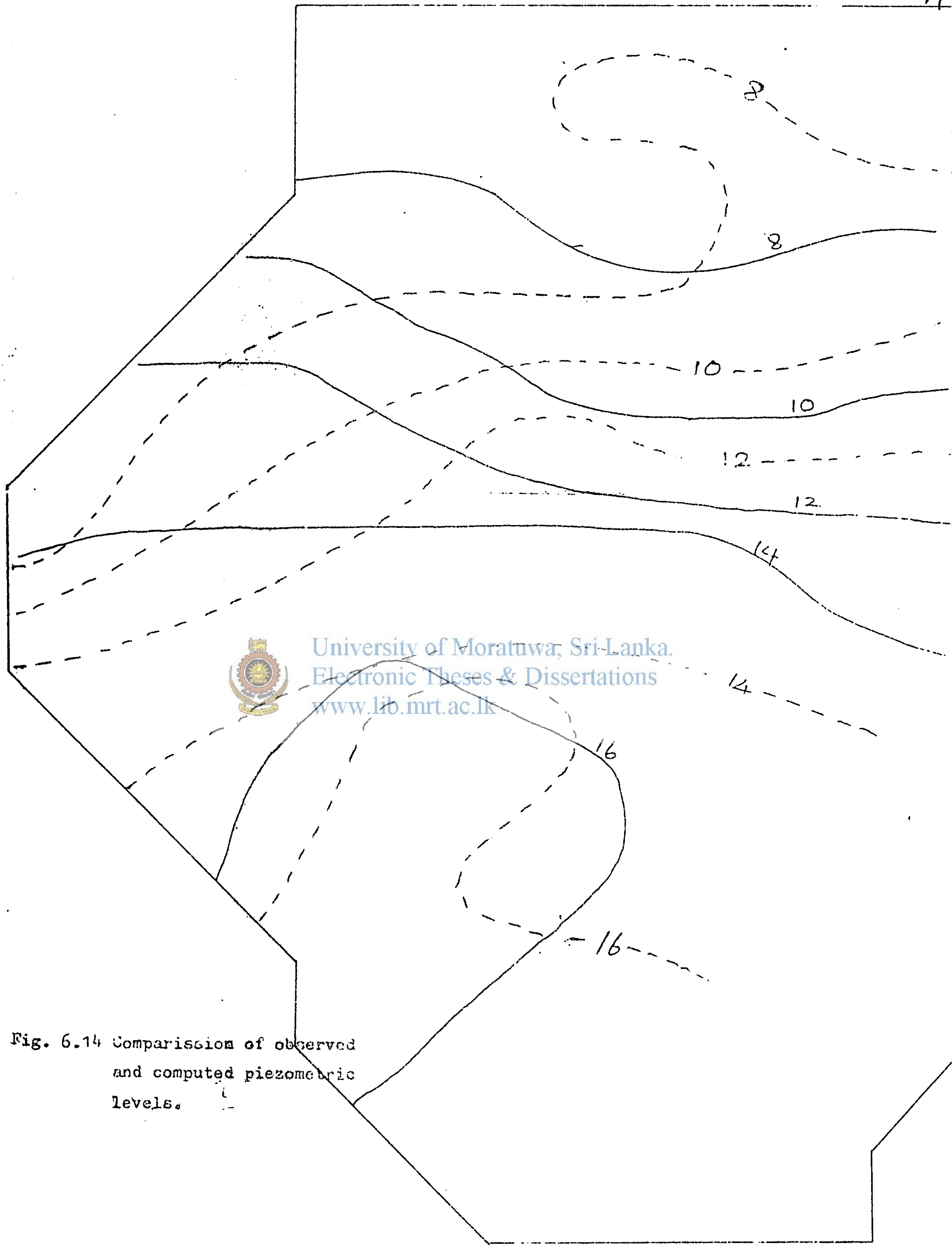


Fig. 6.14 Comparission of observed and computed piezometric levels.

Table 6.3 Data for computer runs group A

Run no.	Transmis-sivity m^2/d	Storage coefficient 10^{-4}	Recharge Ml/d	Leakage Ml/d	Abstraction Ml/d	Northern boundary
1	No change	0.5 to	16.4	0.0.0	0.0 0.0	4.5 m
2	no changr	3.0				
2	no change	0.5 to 3.0	16.4	0.0	0.0	4.5 m extended fixed head
3	no change	0.5 to 3.0	1.64	0.0	0.0	4.5 m
4	no change	0.5 to 3.0	16.4	0.0	0.0	0.0 along west
			21.0	0.0	0.0	and north west.
			21.0	0.0	3.25	4.5 m in the north
5	increased	0.5 to	16.4	0.0	0.0	4.5 m in the north.
	by 10 times	3.0	16.4	0.0	2.6	

(Refer fig. 6.8)

Table 6.4 Data for computer runs group B

Run no.	Transmissivity	Storage coefficient $.10^{-4}$	Recharge Ml/d	Leakage Ml/d	Abstraction Ml/d	Northern boundary	
1	no change	5.0	8.5	0.0	3.3	4.0 m	
2	no change	5.0	13.6	0.0	2.0 3.3	4.0 m	
3	no change	5.0	10.0	5.0	2.0 3.3	4.0 m	
4	no change	5.0	15.0	7.5	2.0 3.3	4.0 m	
6	Locally changed near Kala Oya	5.0	10.0	5.0	2.0 3.3	4.0	
7	same as 6	5.0	10.0	5.0	2.0 3.3	4.0 m	Leakage area moved to south
8	same as 6	1.0	10.0	5.0	2.0 3.3	4.0 m	
9	same as 6	5.0	10.0	5.0	2.0 3.3 5.0	4.0 m	

(Refer fig. 6.9 and 6.10)

Table 6.5 Data for computer runs group C

Run no. Ru	Transmis-sivity	Storage c coefficient 10^{-4}	Recharge Ml/d	leakage Ml/d	Abstraction Ml/d	Northern boundary	Leakage factor m
0	same as B7	5.0	dependent	dependent	3.35	4.5 m	50000
11	same as B7	5.0	dependant	dependant	3.35 5.00	4.5 m	12500
2	no change	5.0	dependant	dependant	3.35	4.5 m	16650
3	no change	5.0	dependant	dependant	2.0 3.35 7.00	4.5 m	16650 12500

(Refer fig. 6.11)

CHAPTER SEVEN: CONCLUSIONS AND RECOMENDATIONS

7.1 The aquifer system

The study showed that the aquifer system consisting of upper Moongil Aru formation and the lower miocene limestone deposits are highly inter-dependent. Leakage from upper layer to lower layer takes place in the south central parts and upward leakage takes place in the north. The time resistance factors for leakage to and from the lower limestone aquifer have been estimated at 16650 days and 12500 days respectively. The location of leakage areas are shown in fig.7.1.

As outlined in chapter six the model presented with constant water table elevations in the upper Moongil Aru formation is only of limited utility. To predict the behaviour of the system with considerably increased abstractions, a detailed two layer model has to be used. Further field investigations required in such a study are outlined under recommendations.

The aquifer boundaries as established in the study are:

1. Fixed head boundary in the north. This is the only outflow boundary for the limestone aquifer.
2. Impermeable boundaries in all other directions.

The north - west boundary of the area is not yet very definite, although in the model this was assumed as no flow. Some monitoring points should be established in this area in order to ascertain the nature of the north - west boundary.

7.2 Aquifer parameters.

The transmissivity of the limestone aquifer varies from about 100 m^2/d to about 2000 m^2/d. The higher transmissivities are obtained in the north central parts of the region. The values used in the model are shown in fig. 7.1. The storage coefficient was taken as 0.0005 throughout.

7.3 Inflows and outflows

Under the level of abstraction used for calibration of the model there is a recharge of 10.0 Ml/d and an outflow of 5.0 Ml/d as leakage. The recharge into the aquifer amounts to 3.65 MCM/year. This figure is about half of the estimate arrived in the earlier study using conventional methods. The outflow amounts to 1.8 MCM/year which occurs as upward leakage. The abstraction was about 1.2 MCM/year.

7.4 Potential for developments.

In the year 1980 the total abstraction amounted to 1.2 MCM/year. This was about one third the inflow at the time. When the abstraction was increased by 100 % to 2.5 MCM/year in the model a drawdown of about 3 m was estimated in the two layer model. With a drawdown of 3 m in the limestone aquifer a significant change in the water table elevations in the unconfined aquifer is not expected. Therefore an increase of abstraction by 100 % will not pose any significant problem as regards to the quantity of water available. When the abstraction is increased beyond 100 % the drawdown in the water table aquifer will become significant so that the model presented will no longer be valid. A detail two layer model has to be developed to investigate such situations.

7.5 Recommendations

The investigation revealed a serious lack of data which are essential in a detailed analysis of the combined aquifer system. Some of them are listed in the following with suggestions for improvements.

1. More monitoring points have to be established in the water table aquifer. These are essential close to the outflow boundary along the coast and Kala Oya.
2. A few pump tests have to be carried out to estimate flow parameters for the water table aquifer.
3. As the water balance study of the upper unconfined region is very important hydrological parameters [illegible]

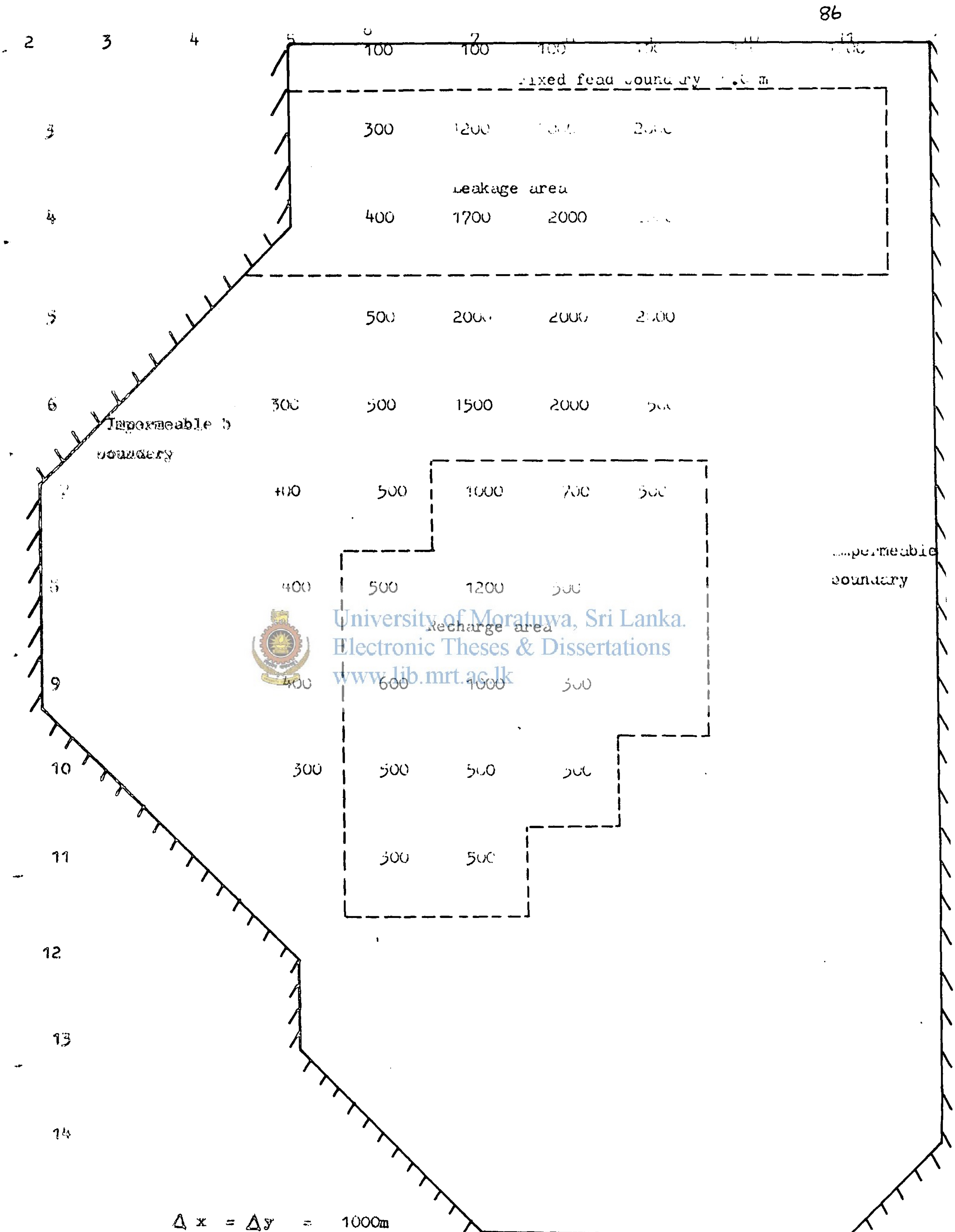
such as rainfall, evapotranspiration and surface runnoff have to be measured accurately. Monitoring of the water levels in the surface water bodies may be helpful. It may also be important to measure the soil moisture deffi- ciency in a few selected sites in the north in order to estimate the upward seepage.

4. A number of boreholes reaching the limestone aquifer have to be established near Kala Oya and along the coast in north west. These are required to ascertain the boundary conditions.

5. High discharge long term puming tests have to be carried out in the recharge area of the limestone aquifer in order to estimate the leakage factor.

With additional data concerning the behaviour of the aquifer system it may be possible to formulate a more comprehensive model treating both layers of the system together. Such a model would help to formulate a long term development plan to get the maximum benefit from the available water resources.

****



$\Delta x = \Delta y = 1000m$

Scale: 1 : 50,000

Fig. 7.1 Calibrated model for Vanathavillu aquifer.

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APPENDIX 'A' THE COMPUTER PROGRAM

```
C  REGIONAL GROUND WATER FLOW
      DIMENSION X(21),Y(16),TX(21,16),TY(21,16),H(21,16),
     &              HFIX(21,16)
     &             RCHG(21,16),HOLD(21,16),S(21,16),
     &             A(21,16),B(21,16),
     &             C(21,16),D(21,16),RS(21,16),NFLOW(4),
     &             IW(8,11),
     &             JW(8,11),QFLOW(8,11),NDAY(20,12),
     &             QFC(5,20,12),QAV(5),
     &             TDAY(15)
      OPEN(2,FILE ='DATARGF',STATUS='OLD')
      OPEN(4,FILE ='OUTRGF',STATUS='NEW')
c   TOP LEFT HAND CORNER NUMBERED (2,2)
C   N IS NO.OF MESH INTERVALS IN THE VERTICAL DIRECTION
      READ(2,*)M,N

C  INPUT  OVERALL AQUIFER PARAMETERS
      READ(2,*)TRANX,TRANY,STOR,HSTART,RECH
      READ(2,*)OFAC,ERROR
      WRITE(4,130)
130   FORMAT(4X,'XMESH',2X,'YMESH',2X,'TRANS.X',2X,'TRANS.Y',
     &    'STORAGE INITIAL H RECHARGE',3X,'FACTOR',4X,'ERROR')
      WRITE(4,140)M,N,TRANX,TRANY,STOR,HSTAT,RECH,OFAC,ERROR
140   FORMAT(1X,I7,2X,I5,6F9.4,F16.10)
C   NUMBERING OF THE BOUNDARY CONDITIONS
      MIN   =M+1
      NIN   = N+1
      MBOND  = M+2
      NBOND  = N+2
      MFICT  = M+3
      NFICT  = N+3
C  SET OVERALL VALUES IN ARRAYS

      DO 122 I = 1,MFICT
      DO 122 J = 1,NFICT
          TX(I,J)   = TRANX
          TY(I,J)   = TRANY
          S(I,J)    = STOR
          HOLD(I,J)= HSTAT
          RCHG(I,J)= RECH
          HFIX(I,J)=  999999.9
          H(I,J)    = 0.0
          RS(I,J)   = 0.0
          A(I,J)    = 0.0
          B(I,J)    = 0.0
```

```
              C(I,J)    = 0.0
              D(I,J)    = 0.0
122       CONTINUE
C   INPUT MESH POSITIONS
          READ(2,*)(X(I),I=1,MFICT)
          READ(2,*)(Y(J),J=1,NFICT)
          WRITE(4,180)(X(I),I=1,MFICT)
          WRITE(4,180)(Y(I),I=1,NFICT)
180       FORMAT(1X,12F10.2)
C   INPUT PARAMETERS TAHT ARE NON STANDARD I=1,J=1 FOR LAST LINE
200       READ(2,*)I,J,TX(I,J),TY(I,J),S(I,J),HOLD(I,J),RCHG(I,J)
          IF(I.EQ.1.AND.J.EQ.1) GO TO 220
          GO TO 200
220       CONTINUE
C
C      INPUT FIXED HEADS
230       READ(2,*)I,J,HFIXA
          IF(I.EQ.1.AND.J.EQ.1)GO TO 250
          H(I,J)    = HFIXA
          HFIX(I,J)= HFIXA
          HOLD(I,J)= HFIXA
          GO TO 230
250       CONTINUE
C
C    FACTORS FOR RIVERS WELLS ETC.  NFCS=NO.OF INPUTS AND OUTPUT'
          READ(2,*)NFCS
          WRITE(4,*)'WELL RIVERS ETC'
          WRITE(4,*)'   I    J   FRACTION'
          DO 280 NF = 2,NFCS
          WRITE(4,285)NF
285       FORMAT('INPUT  GROUP',I4)
C   NFLOW(N)=NO.OF NODES WHERE FLOW IS DISTRIBUTED
          READ(2,*)NFLOW(NF)
          NN   = NFLOW(NF)
          DO 280 L = 1,NN
C    IW() JW() ARE LOCATIONS,QFLOW IS FRACTION OF FLOW
          READ(2,*)IW(NF,L),JW(NF,L),QFLOW(NF,L)
          WRITE(4,290)IW(NF,L),JW(NF,L),QFLOW(NF,L)
290       FORMAT(2I4,F8.5)
280       CONTINUE
C
C     COEFFICIENT OF FINITE DIFERENCE EQUATIONS
          DO 500 I = 2,MBOND
          DO 500 J = 2,NBOND
          A(I,J) = 2.0*TX(I,J)/((X(I+1) X(I 1))*(X(I+1) X(I)))
          C(I,J) = 2.0*TX(I 1,J)/((X(I+1) X(I 1))*(X(I+1) X(I)))
          B(I,J) = 2.0*TY(I,J 1)/((Y(J+1) Y(J 1))*(Y(J) Y(J 1)))
          D(I,J) = 2.0*TY(I,J)/((Y(J+1) Y(J 1))*(Y(J+1) Y(J)))
500       CONTINUE
C
C   INPUT FLOW BOUNDARIES
          DO 510 INODE = 1,1000
          READ(2,*)I,J,AA,BB,CC,DD,SS
          IF ((I.EQ.1).AND.J.EQ.1)GO TO 540
          A(I,J)    = AA*A(I,J)
          B(I,J)    = BB*B(I,J)
```

```
         C(I,J)   = CC*C(I,J)
         D(I,J)   = DD*D(I,J)
C     99.9 OUTSIDE NO FLOW BOUNDARY
         IF(AA.LE.0.000001)HFIX(I+1,J) =  99.9
         IF(BB.LE.0.000001)HFIX(I,J-1) =  99.9
         IF(CC.LE.0.000001)HFIX(I-1,J) =  99.9
         IF(DD.LE.0.000001)HFIX(I,J+1) =  99.9
         IF(AA.LE.0.000001)C(I+1,J) = 0.0
         IF(BB.LE.0.000001)D(I,J-1) = 0.0
         IF(CC.LE.0.000001)A(I-1,J) = 0.0
         IF(DD.LE.0.000001)B(I,J+1) = 0.0
         S(I,J)   = SS*S(I,J)
510      CONTINUE
C
C    SET INITIAL HEADS
540      DO 530 I = 1,MFICT
         DO 530 I = 1,NFICT
         H(I,J)    = HOLD(I,J)
530      IF(HFIX(I,J).EQ. 99.9)H(I,J) = 99.9
C
C   PRINT OUT INITIAL CONDITIONS
         CALL PRIN(TX,1,2,MBOND,2,NBOND,TIME)
         CALL PRIN(TY,2,2,MBOND,2,NBOND,TIME)
         CALL PRIN(RCHG,5,2,MBOND,2,NBOND,TIME)
         CALL PRIN(S,3,2,MBOND,2,NBOND,TIME)
         CALL PRIN(HFIX,6,1,MFICT,1,NFICT,TIME)
C
C    NBLOCK IS NO. OF YEARLY  BLOCK OF DATA
         READ(2,*)NBLOCK
         DO 580 IBLOCK = 1,NBLOCK
         WRITE(4,560)IBLOCK
560      FORMAT(10X,'BLOCK NO. = ',I3)
         DO 580 IMONTH = 1,12
         DO 765 NF = 1,NF
765      QFC(NF,IBLOCK, IMONTH) = 0.0
C   NDAY() = NO OF DAYS IN MONTH,QFC()= FLOWS IN ML/D
         READ(2,*)NDAY(IBLOCK,IMONTH),
      &           (QFC(NF,IBLOCK,IMONTH),NF=1,NFCS)
         WRITE(4,570)NDAY(IBLOCK,IMONTH),
      &          (QFC(NF,IBLOCK,IMONTH),NF=1,NFCS)
570      FORMAT(I5,10F7.1)
C
C    CONVERT INPUT VALUES OF ML/D INTO M**3/D
         DO 580   NF = 1,NFCS
580      QFC(NF,IBLOCK ,IMONTH) = QFC(NF,IBLOCK,IMONTH)*1000.0
C
C    CALCULATE AVE. OF FIRST BLOCK FOR STEADY STATE
         DO 590 NF = 1,NFCS
590      QAV(NF) = 0.0
         DO 600 NF = 1,NFCS
         DO 600 IMONTH = 1,12
600      QAV(NF) = QFC(NF,1,IMONTH) + QAV(NF)
         DO 610 NF = 1,NFCS
610      QAV(NF) = QAV(NF)/12.0
C
C   SET IFIRST NEGATIVE FOR INITIAL STEADY STATE
```

```
      IFIRST =  100
C
C  INPUT OF TIMES IN DAYS WHEN CALCULATION IS PERFORMED
      READ(2,*)KDAY
      DO 620 K = 1,KDAY
620   READ(2,*)TDAY(K)
9000  WRITE(4,*)
C
C  TIME INCREASED , CHANGE IN YEAR
      DO 700 IYEAR = 1,100
      IF(IFIRST.LT.0)GO TO 755
      READ(2,*)IBLOCK
C  IF IBLOCK NEGATIVE CALCULATION STOPS
      IF(IBLOCK.LT.0)GO TO 8000
      WRITE(4,720)IBLOCK,IYEAR
720   FORMAT(10X,'BLOCK NO. = ',I3,'YEAR NO.= ',I3)
C
C  CHANGE IN MONTH
755   DO 730 IMONTH = 1,12
      IF(IFIRST.LT.0)GO TO 750
C
C  COMBINE ALL FLOWS PER NODE INRS(I,J),UNITS M**3/D
C  SPECIAL CALCULATION FOR INITIAL HEADS
      WRITE(4,740)IMONTH,NDAY(IBLOCK,IMONTH),
     *(QFC(NF,IBLOCK,IMONTH),NF = 1,NFCS)
740   FORMAT(1X,'MONTH= ',I4,'NO.OF DAYS= ',I4,
     *       'FLOWS=  ',5F17.1)
750   CONTINUE
      DO 800 I = 2,MBOND
      DO 800 J = 2,NBOND
      IF(IFIRST.LT.0)GO TO 810
      RS(I,J) = RCHG(I,J)*QFC(1,IBLOCK,IMONTH)
      GO TO 800
810   RS(I,J) = RCHG(I,J)*QAV(1)
800   CONTINUE
      DO 820 N = 2,NFCS
      NN = NFLOW(N)
      DO 820 I1 = 1,NN
      I = IW(N,I1)
      J = JW(N,I1)
      IF(IFIRST.LT.0)GO TO 830
      RS(I,J) = RS(I,J) + (QFC(N,IBLOCK,IMONTH)*QFLOW(N,I1))
      GO TO 820
830   RS(I,J) = RS(I,J) + QAV(N)*QFLOW(N,I1)
820   CONTINUE
C
C   DIVIDE NODAL FLOW BY AREA TO GIVE  M/D
      DO 840 I = 2,MBOND
      DO 840 J = 2,NBOND
840   RS(I,J) = 4.0*RS(I,J)/((X(I+1) X(I 1))*(Y(J+1) Y(J 1)))
C
C   INCREASE TIME ; CALCULATE DELT
      LDAY = KDAY+1
      DO 900 IDAY=1,LDAY
      IF(IDAY.NE.1)GO TO 910
      DELT = TDAY(1)
```

```
         DAYT=  TDAY(1)
         GO TO 930
910      IF(IDAY.EQ.LDAY)GO TO 920
         DELT = TDAY(IDAY) TDAY(IDAY 1)
         DAYT = TDAY(IDAY)
         GO TO 930
920      DAYT = FLOAT(NDAY(IBLOCK,IMONTH))
         DELT = DAYT   TDAY(KDAY)
930      IF(DELT.LE.0.001)GO TO 900
         SFAC = 1.0
C
C  START OF S.O.R. CACULATION
C  MULTIPLYING FACTOR FOR STORAGE
         IF(IFIRST.LT.0)SFAC =0.00000001
C
C MULTIPLIER AND PREVIOUS TIME STEP FACTORS;USE ARRAYS TX TY
         RDELT =1.0/DELT
         DO 940 I = 2,MBOND
         DO 940 J = 2,NBOND
         HOLD(I,J) = H(I,J)
         TX(I,J)= (SFAC*S(I,J)*RDELT+A(I,J)+B(I,J)+C(I,J)+D(I,J))
940      TY(I,J)= SFAC*S(I,J)*H(I,J)*RDELT
C
C  ITERATION LOOP ; MAX.NO.OF ITERATION 300
         DO 950 ICYCLE=1,300
         IND = 0
         DO 960 I = 2,MBOND
         DO 960 J = 2,NBOND
         HOLD(I,J) = H(I,J)
         IF(HFIX(I,J).GE.10000.0) GOTO 970
         AB=A(I,J)*H(I+1,J)+B(I,J)*H(I,J 1)+C(I,J)*H(I 1,J)
      *    +D(I,J)*H(I,J+1)+TY(I,J)+RS(I,J)
         IF(ABS(AB TX(I,J)*HOLD(I,J)).LT.ERROR)GO TO 980
         IND = 100
980      H(I,J) = (1.0 OFAC)*HOLD(I,J)+OFAC*AB/TX(I,J)
         GO TO 960
970      H(I,J) = HFIX(I,J)
960      CONTINUE
         IF(IFIRST.LT.0) GO TO 950
         IF(ICYCLE.LT.2)GO TO  950
         IF(IND.EQ.0) GO TO 990
950      CONTINUE
         IF(IFIRST.GT.0) GO TO 1000
C
C  OUTPUT SECTION FOR INITIAL STEADY HEADS
         IFIRST = 100
         WRITE(4,1010)
1010     FORMAT(1X,'INITIAL STEADY STATE HEADS')
         CALL PRIN(H,7,2,MBOND,2,NBOND,TIME)
         GO TO 9000

1000     WRITE(4,*)'CONVERGENCE NOT ACHIEVED IN 300 ITERATIONS'
C     END OF SOR ROUTINE
C
990      CONTINUE
C  SECTION FOR CALCULATING FLOW INSERTED HERE
```

```
          FLOW = 0.0
C
          WRITE(4,1040)ICYCLE,DAYT,H(9,3),H(9,4),H(9,5),H(9,7),
     *                 H(9,9),H(9,11),H(9,13),H(9,15),H(2,7),H(5,7)
     *                 H(7,7),H(11,7)
1040      FORMAT(1X,I5,F10.2,12F9.3)
900       CONTINUE
730       WRITE(4,*)
C
C  FULL PRINT OUT AT END OF EACH YEAR
          CALL PRIN(H,7,2,MBOND,2,NBOND,TIME)

700       WRITE(4,*)
8000      STOP
          END


          SUBROUTINE PRIN(FUNC,NO,IBEG,IEND,JBEG,JEND,TIME)
          DIMENSION FUNC(21,16)
100       FORMAT(10X,'TRANSMISSIVITY IN X DIRECTION')
101       FORMAT(10X,'TRANSMISSIVITY IN Y DIRECTION')
102       FORMAT(10X,'STORAGE FACTORS')
103       FORMAT(10X,'INITIAL VALUES OF HEADS')
104       FORMAT(10X,'RECHARGE VALUES')
105       FORMAT(10X,'FIXED HEADS')
106       FORMAT(10X,'VALUES OF HEADS AT',F6.2,' DAYS')
107       FORMAT(1X,1P14E9.2)
108       FORMAT(5(/))
110       FORMAT(1X,14(I4,4X))
111       FORMAT(1X,I3,1X,19F6.2)
112       FORMAT(3X,19I6)
115       FORMAT(1X,' 1.00E+06 SIGNIFIES FREE HEAD',5X,' 9.99E+1
     *                IS NODE OUTSIDE BOUNDARY')
C
C
          IF(NO.EQ.7)GO TO 6
          IF(NO.NE.1)GO TO 1
          WRITE(4,100)
          GO TO 7
1         IF(NO.NE.2)GO TO 2
          WRITE(4,101)
          GO TO 7
2         IF(NO.NE.3)GO TO 3
          WRITE(4,102)
          GO TO 7
3         IF(NO.NE.4)GO TO 4
          WRITE(4,103)
          GO TO 7
4         IF(NO.NE.5) GO TO 5
          WRITE(4,104)
          GO TO 7
5         IF(NO.NE.6)GO TO 6
          WRITE(4,105)
          WRITE(4,115)
          GO TO 7
6         WRITE(4,106)TIME
```

```
        WRITE(4,112)(I,I=IBEG,IEND)
        DO 11 J = JBEG,JEND
11      WRITE(4,111)J,(FUNC(I,J),I=IBEG,IEND)
        GO TO 10
7       WRITE(4,110)(I,I=IBEG,IEND)
        DO 8 J=JBEG,JEND
8       WRITE(4,107)(FUNC(I,J),I=IBEG,IEND)
10      WRITE(4,108)
        RETURN
        END
```

Appendix B: Radial flow model to analyse pump test at V1 site.

A computer model based on finite difference approximation of following equation was used to analuse the pump test results for the borehole V1.

$$\frac{\partial}{\partial r}\left[m\, k_r \frac{\partial s}{\partial r}\right] + \frac{m}{r}\left[k_r \frac{\partial s}{\partial r}\right] = S\frac{\partial s}{\partial t} + q \qquad B.1$$

where,

- s = drawdown in the aquifer at r radius from the well.
- m = saturated thickness of the aquifer
- k_r = radial permeability
- S = storage coefficient
- q = vertical flow

Following data were used in computations:

Radius of the well = 100 mm

Pumping rate = 2304 m^3/d

Number of computations were made with following sets of data.

Run no.	Radial permeability m/d	Storage coefficient	Radius of influence m	Boundary condition	Leakage factor m
1	19.0	0.001	1000	free	No leakage
2	19.0	0.00008	1000	free	no leakage
3	38.0	0.00008	1000	free	no leakage
4	19.0	0.00008	300	fixed head	1000
5	19.0	0.0008	300	fixed head	6500

... WELL MONITORING NETWORK AT VANATHAVILLU : WATER LEVELS, ... IVITY AND TEMPERATURE DATA, 1979 - 1981. PART 1 REDUCED WATER LEVELS

Well No.	Elevation m.s.l. (m)		1979 Aug.	Sept.	Oct.	Nov.	Dec.	1980 Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	1981 Jan.	Feb.	March	April	May	June
P2A	30.028	TWP	–	–	–	–	–	–	–	–	15.27	15.12	14.94	13.73	14.[illegible]5	14.25	14.[illegible]7	14.23	14.00	14.72	14.67	–	–	14.27	14.22
P7	28.650	TWP	–	–	–	–	–	–	–	–	16.03	15.[illegible]3	15.65	–	15.31	14.58	14.5[illegible]	14.49	14.56	14.9[illegible]	14.88	–	–	14.53	14.52
P5	41.775	TWP	–	–	–	–	–	–	–	–	14.90	14.94	14.94	14.85	14.75	14.64	14.62	14.52	14.76	14.87	14.92	–	–	14.63	14.73
P8	3.492	TWP	–	–	–	–	0.41	–0.07	–0.10	–0.09	0.01	0.07	0.19	0.24	–	0.49	0.44	–	0.49	0.44	–	–	–	–	–
P9A	29.148	TWP	–	–	–	–	–	–	–	–	12.56	12.59	12.55	12.55	11.64	11.76	11.74	12.04	12.27	12.31	11.75	–	–	11.65	11.42
P13	19.53	TWP	–	–	–	–	–	8.58	8.40	8.47	8.49	8.55	8.55	8.45	–	–	–	–	–	–	7.53	–	–	pump fitted	
W2	46.0	TCL	12.32	12.25	12.13	12.27	12.66	13.20	13.56	13.73	13.78	13.78	13.70	13.46	13.30	13.22	13.17	13.08	13.29	13.32	13.33	13.40	13.36	13.28	13.11
W4A	34.975	TWP	–	–	–	–	–	–	–	–	15.15	14.66	14.80	14.60	14.07	Well	blocked								
W4B	35.665	TWP	–	–	–	–	–	–	–	–	15.15	14.77	14.81	15.61	14.41	14.16	14.13	14.16	14.46	14.58	14.58	–	–	14.26	14.14
W3-1	44.881	TCL	–	–	–	–	–	–	–	–	–	–	–	–	–	–	8.90	8.90	8.87	8.94	8.83	–	–	8.70	8.59
W3-2	42.567	TWP	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	8.62	–	–	–	–
LC1	58.01	TCL	–	–	–	–	15.65	16.33	16.60	16.60	16.48	16.28	16.11	15.92	15.72	–	15.43	15.41	15.25	15.88	–	–	–	–	–
W8	28.141	TWP	–	–	–	–	–	15.59	15.33	15.35	15.23	15.07	14.90	14.68	14.47	14.26	14.22	14.22	14.49	14.66	14.64	–	–	14.33	14.20
W1-1	36.416	TWP	–	–	–	–	–	–	–	–	–	–	17.14	16.90	–	16.50	16.44	–	16.82	–	–	–	–	–	–
W1-3	36.581	TWP	–	–	–	–	–	–	–	–	–	–	17.33	17.10	16.95	16.69	16.63	16.66	17.01	17.13	17.13	–	–	16.76	16.62
P7A	34.706	TCL	12.56	12.51	12.[illegible]9	12.31	–	–	13.49	13.75	13.83	13.92	13.86	13.81	13.77	13.50	13.49	13.49	13.60	13.67	13.68	–	–	–	–
P7	34.512	TWP	–	–	–	–	–	–	13.40	13.58	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
W2-1	28.288	TWP	–	–	–	–	–	–	–	–	–	–	–	–	8.33	7.94	7.88	7.82	7.34	7.84	7.75	–	–	–	–
RTW3	35.556	TCL	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	8.25	9.02	8.76	–	–	9.14	9.06
ROWG	31.348	TWP	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	9.10	–	–	–	–	–
W3-3	42.907		–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	38.35	–	–	–	–
W1-2	36.373	TCL	–	–	–	–	–	–	–	–	–	–	33.81	33.00	31.85	30.23	31.17	32.47	33.87	33.43	33.03	–	–	30.95	30.84
OW51	29.413	TPW	24.24	27.39	26.21	28.61	28.90	28.47	28.26	27.90	27.98	28.48	27.47	26.96	27.36	27.31	27.51	27.36	27.68	27.50	26.52	–	–	dry	25.15
OW52	20.741	TPW	7.90	7.44	7.24	7.33	9.79	10.60	11.34	10.81	10.06	9.09	8.70	8.35	8.05	7.50	7.33	7.42	7.69	7.76	7.60	–	–	6.95	6.78
OW53	5.446	TPW	4.44	4.48	4.60	4.74	4.70	4.73	4.62	4.39	4.68	4.69	4.60	4.61	4.52	4.68	4.76	4.76	4.76	4.77	4.54	–	–	4.71	4.60
OW54	2.855	TPW	- .03	0.31	0.71	1.45	1.60	1.49	1.17	0.91	0.63	0.53	0.38	0.28	0.08	0.29	0.91	0.86	1.16	0.93	0.72	–	–	0.56	0.58
OW55	51.844	TPW	42.59	42.46	42.24	42.69	44.48	44.82	44.46	44.08	43.78	43.45	43.21	42.76	42.50	41.94	42.10	41.99	42.27	42.38	42.31	–	–	41.60	41.49
OW56	36.903	TPW	29.45	36.54	30.55	33.42	33.27	33.31	32.84	31.84	31.24	30.58	30.25	30.23	dry	dry	31.13	31.06	31.04	31.05	30.13	–	–	dry	29.62
OW57	50.556	TPW	–	39.38	38.99	39.73	40.22	41.01	41.15	40.89	40.53	40.14	40.87	39.49	39.06	37.80	37.71	37.71	38.85	38.54	38.47	–	–	dry	37.79
OW58	37.762	TPW	–	28.91	28.94	34.49	34.22	33.38	32.65	33.04	31.52	30.75	30.28	29.65	29.41	28.84	31.02	31.54	32.70	31.97	31.06	–	–	28.88	29.01
OW59	38.941	TPW	–	29.23	28.62	29.04	30.54	31.94	31.97	31.55	31.29	30.91	30.40	29.69	28.54	27.44	28.07	28.35	28.67	29.05	28.96	–	–	dry	dry
OW61	37.504	TPW	–	–	–	–	–	–	–	–	32.04	34.27	34.03	33.80	33.83	34.14	34.79	34.72	34.52	34.26	33.84	–	–	34.08	34.10
OW64	59.227	TPW	–	–	–	–	–	–	–	–	–	–	–	47.89	47.44	–	46.61	46.79	46.82	46.73	46.64	–	–	45.33	46.41
OW65	65.759	TPW	–	–	–	–	–	–	–	–	–	–	–	44.61	44.56	–	–	43.96	44.03	43.83	43.86	–	–	43.62	43.54
OW66	15.718	TPW	–	–	–	–	–	–	–	–	–	–	–	10.38	8.99	–	–	12.20	13.11	12.60	12.51	–	–	12.22	12.57
OW67	32.216	TPW	–	–	–	–	–	–	–	–	–	–	–	26.92	26.42	dry	26.84	27.17	28.42	28.12	27.67	–	–	26.52	26.57
OW68	31.091	TPW	–	–	–	–	–	–	–	–	–	–	–	24.09	dry	dry	25.13	24.65	25.81	25.90	25.48	–	–	24.39	24.39
OW69	30.036	TPW	–	–	–	–	–	–	–	–	–	–	–	26.30	–	–	24.66	25.18	24.72	26.56	26.81	–	–	dry	dry

TWP - Top of Well Protector
TCL - Top of Casing Lip
TPW - Top of Parapet Wall

NOTE: OW Wells are hand-dug open wells, all other wells are tubewells.