OPTIMUM PERFORMANCE OF A BATTERY OF WELLS USING A CCMPUTER SIMULATION MODEL

BY

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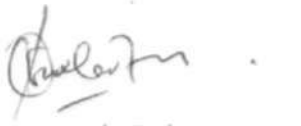
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DECLARATION

I hereby declare that this thesis is a record of the research work carried out by me under the Supervision of Dr. L.I. Odigie. To the best of my knowledge, it has not been previously published or accepted anywhere for the award of any degree.

All works, opinions and ideas, other than mine have been fully acknowledged.



Oladapo Kemi Isiolaotan

August, 1986.

DEDICATION

To my Parents.

CERTIFICATION

This thesis entitled "Optimum Performance of A Battery of Wells Using a Computer Simulation Model" by Isiolaotan, Oladapo Kemi meets with the regulations governing the award of the degree of Master of Science (Water Resources and Environmental Engineering) of Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

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ABSTRACT

This study investigates the optimum use to which a number of wells, sunk for water supply to the Ipaja Housing Estate in Lagos State could be put. The study is in two parts. The first part involves graphical analysis of the available pumping test data from sixteen wells located within the estate to determine the hydraulic characteristics of the underlying aquifer being tapped.

In the second part, the regional groundwater reservoir is represented by a radial flow numerical model. The model was used to verify the aquifer properties. A sensitivity analysis was carried out to investigate the effect of varying model input parameters on model suits.

A study of the model results show that a minimum drawdown value optimum performance of the well field is attained when all 16 are put into operation in meetinq the estate's daily requirenr of 12000m^3 of water.

TABLE OF CONTENTS

Chapter		Page
	Title Page	i
	Declaration.....	ii
	Dedication	iii
	Certification	iv
	Acknowledgement	v
	Abstract	vi
	Table of Contents	vii
	List of Tables	xi
	List of Figures	xii
	List of Appendices	xiii
1	INTRODUCTION	1
1.1	Brief History	1
1.2	Objective	1
1.3	Approach	3
1.4	Report Outline	3
2	DEFINITION OF HYDROGEOLOGICAL TERMS AND LITERATURE REVIEW	6
2.1	Definitions	6
2.1.1	Aquifers	6
	Unconfined Aquifers	6
	Confined Aquifers	6
	Semi-Confined or Leaky Aquifers	7
2.1.2	Hydraulic Properties of Aquifers	8
	Hydraulic Conductivity	8
	Transmissivity	8

Chapter		Page
	Storage Coefficient and Specific Yield....	9
	Hydraulic Resistance	9
	Leakage Factor	9
2.1.3	Aquifer Boundaries	10
2.1.4	States of Groundwater Flow	10
	Steady State Flow	11
	Unsteady State Flow	11
2.2	Literature Review	11
2.2.1	Equations of Groundwater Flow to a Pumping Well..	12
	Steady State or Equilibrium Equation	12
	Unsteady State or Non-Equilibrium Equation...	14
3	DESCRIPTION OF MODEL	18
3.1	Definition	18
3.2	Aquifer System Analysis-Hydrogeological Models..	18
3.2.1	Mathematical Models	19
3.2.2	Analog Models	20
3.2.3	Numerical/Digital Models	21
3.3	The Radial Groundwater Flow Model	22
3.3.1	Derivation of Discrete Model	23
3.3.2	Derivation from Differential Equation	23
3.3.3	Numerical Solution	27
	Numbering of nodes and Mesh Spacing	28
	Position of Observation Wells	28
	Time Step	29
	Confined to Unconfined	30
	Variable Saturated Depth	30
	Excessive Drawdown	31

Chapter		Page
3.3.4	Program Computations	31
	Solution to Equations	35
4	THE STUDY AREA	36
4.1	Location	3
4.2	Topography	36
4.3	Geology	36
4.4	Climate	38
4.5	Hydrogeology.....	38
5	ANALYSIS OF PUMPING TEST DATA, ESTIMATION OF MODEL INPUT PARAMETERS AND RESULTS	41
5.1	Analysis of Pumping Test Data	41
5.1.1	Borehol Construction Information	41
5.1.2	Pumping Test Measurements and Analysis	46
5.1.3	Drawdown Tests (Step Drawdown/Constant Discharge Test)	49
5.1.4	Recovery Tests	51
5.1.5	Observation Wells	53
5.1.6	Calculations	54
5.2	Model Input Parameters	83
5.2.1	First Trial (Initial Parameters)	83
5.2.2	Varying Parameters	85
	Transmissivity, T-Varied	85
	Storage Coefficient, S-Varied	85
	Pumping Duration, TSTOP-Varied	86
	Recharge, RECH- Varied	86
	Abstraction Rate, QPUMP-Varied	86

Chapter		Page
5.3	Model Results	88
5.3.1	Drawdown Estimates	88
5.3.2	Proposed Pilot Scheme	102
5.3.3	Alternative Scheme	103
6	DISCUSSION OF RESULTS	105
6.1	Hydraulic Properties of the Aquifer, T and S	105
6.2	Hydraulic Properties of Tested Wells	106
6.3	Drawdown Estimates	107
6.4	Limitations	109
7	CONCLUSIONS AND RECOMMENDATIONS	111
7.1	Conclusions	111
7.2	Recommendations	112
	REFERENCES	114
	APPENDICES	117

LIST OF TABLES

TABLE		PAGE
5.1	Summary of Borehole Construction and Pumping Test Information	47
5.2	Summary of T and S values for Abstraction and Observation Wells	76
5.3	Model Input Parameters for the 1st trial run	85
5.4	Summary of Model runs with varying input Parameters	87
5.5	Field and Numerical Drawdown values at Observation Well No. 4 (Simulations 1 - 10)	89
5.6a	Numerical Drawdown values at Observation Well nodes 18 and 19 (Simulations 1 - 10)	91
5.6b	Field Drawdown at Observation Well No. 3....	94
5.7	Drawdown at the end of one year at Different Observation Well nodes (Simulations 11 - 29)	101
5.8	Maximum Drawdown Estimates at the Abstraction Well for Varying Abstraction rates (Simulations 17 - 29)	104

LIST OF FIGURES

Figure		Page
1	Location Map of Ipaja Housing Estate	2
2.1	Different Types of Aquifers	7
2.2	Radial Flow to a Well fully Penetrating an Extensive Confined Aquifer	12
3.1	Radial Finite Difference Mesh with Logarithmic Increase in Mesh Interval	25
3.2	Equivalent Hydraulic and Time Resistance.....	26
3.3	Representation of Discrete model with allowance for Water in Well	28
3.4	Numbering Observation Well nodes	32
4.1	Geological Map of part of Lagos State	39
5.1	Borehole Location within the Ipaja Housing Estate	42
5.2	Borehole Location Sketch	43
5.3	Lithological Cross-Section of the Test Site ..	44
5.4	Typical Borehole Sketch and Strata Log ...	45
5.5a	Specific Drawdown, s/Q Versus Weighted time, $\bar{t}$ for Abstraction Well no. TW - Drawdown Test	55
5.5b	Residual Drawdown,s" Vs Time Ratio t/t" for Abstraction Well no. TW- Recovery Test	57
5.6a	Specific Drawdown s/Q Vs Weighted time, $\bar{t}$ for Abstraction Well no. 3 - Drawdown Test	58
5.6b	Residual Drawdown s" Vs Time ratio t/t" for Abstration Well n. 3 - Recovery Test	60
5.7a	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Abstraction Well no. 5 - Drawdown Test	61
5.7b	Residual Drawdown s" Vs Time ratio t/t" for Abstraction Well no. 5 - Recovery Test	62

LIST OF FIGURES

Figure		Page
5.7c	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Observation Well no. 3 - Drawdown Test	64
5.7d	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Observation Well no. 4 - Drawdown Test	66
5.8a	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Abstraction Well no. 6 - Drawdown Test	68
5.8b	Residual Drawdown, s" Vs Time ratio t/t" for Abstraction Well no. 6 - Drawdown Test	69
5.9a	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Abstraction Well no. 11 - Drawdown Test	71
5.9b	Residual Drawdown, s" Vs Time ratio, t/t" for Abstraction Well no. 11 - Recovery Test	72
5.10a	Specific Drawdown, s/Q Vs Weighted time, $\bar{t}$ for Abstraction Well no. 12 - Drawdown Test	73
5.10b	Residual Drawdown, s" Vs Time ratio, t/t" for Abstraction Well no.12 - Recovery Test	75
5.11	Formation and Well losses for Abstraction Well no. TW	77
5.12	Formation and Well losses for Abstraction Well no. 3	78
5.13	Formation and Well losses for Abstraction Well no. 5	79
5.14	Formation and Well losses for Abstraction Well no. 6	80
5.15	Formation and Well losses for Abstraction Well no. 11	81
5.16	Formation and Well losses for Abstraction Well no. 12	82
5.17	Semi-log Plot of Field Drawdown against time for Observation Well, OW4	95

LIST OF FIGURE

Figure		Page
5.18	Semi-log plot of Field Drawdown against time for Observation Well OW3	96
5.19a	Semi-log plot of Numerical Drawdown against time for Observation Well, OW4 (Simulation nos. 1 - 5)	97
5.19b	Semi-log plot of Numerical drawdown against time for Observation Well, OW 4 (Simulation nos. 6 - 10)	98
5.20a	Semi-log plot of Numerical Drawdown against time for Observation Well nodes 18 and 19 (Simulations 1-5)	99
5.20b	Semi-log plot of Numerical Drawdown against time for Observation Well nodes 18 and 19 (Simulations 6-10)	100

LIST OF APPENDICES

Appendix		Page
I	Flow Chart for Pumping Test Program	117
II	Pumping Test Program Listing	118
III	Tables for formation and Well losses	120
IV	Pumping Test Data	124

Chapter 1

INTRODUCTION

1.1 Brief History

In this project the water supply situation at the Ipaja Housing Estate situated between Alimosho and Baruwa village, along Ipaja road and off the new Abeokuta expressway in Lagos State is being studied. The new estate is being developed by the Federal Housing Authority to alleviate the hardships suffered by the middle income group of people living in this part of metropolitan Lagos, due to shortage of houses, and is expected to provide accomodation for an ultimate population of about 40,000 people.

The source of water supply to the estate is through groundwater, and an estimated water supply of 12,000 m^3/day is expected to meet the estate's water requirement. This amount is based on an ultimate per capita consumption of 300 litres per day, including provision for the requirement of schools and other light industries that might develop within the estate.

Between October 1982 and July 1983, a total of 16 boreholes were sunk and developed within the estate, and pumping tests carried out to determine the yield of each borehole and also evaluate the hydraulic characteristics of the aquifer being tapped. Figure 1 shows the location map of the Ipaja Housing Estate.

1.2 Objective

The objective of this project is two fold:

Firstly, to determine whether or not the aquifer under

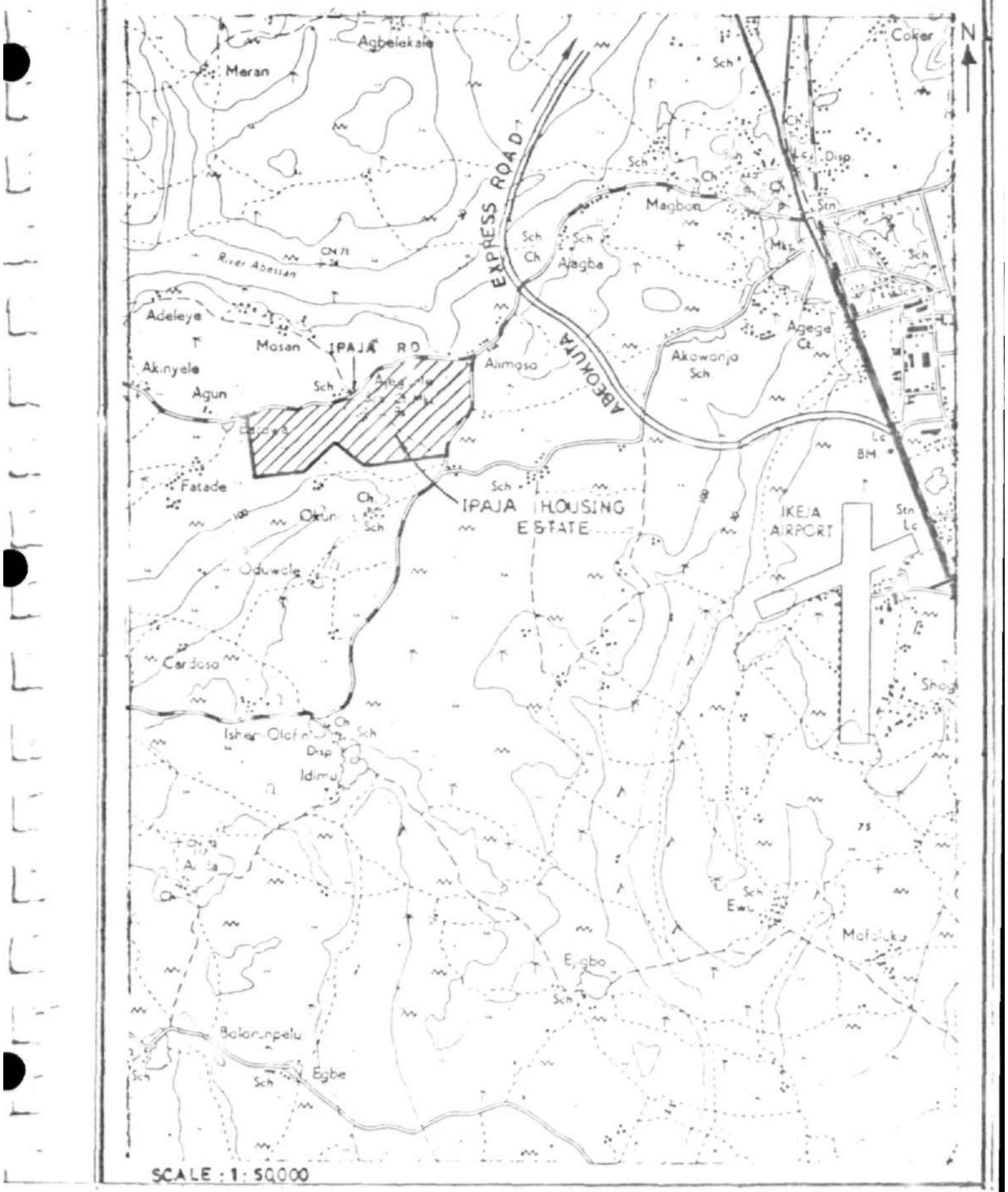


Fig. 1: Location Map of Ipaja Housing Estate (Adeyemi, Ogundipe and Partners.)

exploitation can supply the estimated requirement of 12000m^3 of water per day, and secondly, to determine and evaluate, for any one day, the optimum number of wells to be operated, rate of pumping and arrangement or sequence of operating such wells in order to meet the ultimate water demand of the housing estate.

1.3 Approach

The following approach was adopted towards the achievement of the stated objectives:

(i) Analysing the available pumping test data by conventional graphical methods to estimate the hydraulic properties of transmissivity, T, and storage coefficient, S, of the aquifer.

(ii) Using a numerical groundwater flow model to verify and where necessary modify the values obtained in (i) above, and then proceed to investigate the effect of varying model parameters on the model results. In this exercise, the numerical radial flow model for analysing pumping tests which was developed by Rushton and Redshaw (1979) was used.

(iii) A study of the results obtained in (ii) would then form the basis for drawing conclusions and making recommendations on how the stated objectives could be achieved.

1.4 Report Outline

In order to give a comprehensive treatment, and at the same time allow for easy understanding of the work carried out,

this report is being presented in seven chapters as follows:

The first chapter, gives a general introduction and brief history of the scheme being investigated. The objectives of, approach to, as well as a report outline of the work are also presented.

In the second chapter, some important hydrogeological terms commonly encountered in groundwater studies are defined. A review is given of existing literature of works related to the study, which were carried out by other researchers.

The third chapter is devoted to describing three main hydrogeological models normally used for groundwater resource evaluation. Their relative advantages and limitations are highlighted. Finally, the numerical radial groundwater flow model for analysing pumping tests developed by Rushton and Redshaw (1979), and which was used in this work is discussed in more details.

Next is the fourth chapter, in which the study area where the housing estate is located is described in detail and for which mention is made of its location, topography, geology and climate. The hydrogeology of the underlying formation is also discussed.

In chapter five, the available pumping test data are analysed by conventional graphical methods to estimate the hydraulic properties of the aquifer. These values were checked and where necessary modified, using the numerical radial pumping test model. The model was then run to investigate the response of the aquifer to varying input parameters like duration of pumping,

recharge, and abstraction rate. These parameters were seperately varied at first to determine their appropriate values which were later incorporated in the final simulations. The model results are also presented.

The results and other information from the analysis and model runs carried out in the fifth chapter are fully discussed in chapter six. Limitations of the results are mentioned.

The final chapter contains conclusions which are drawn from the results of the work carried out. Suggestions and recommendations which are considered vital to achieving the best performance of the wells, and also ensure the optimum exploitation and utilisation of the available groundwater resources of the area are discussed.

Chapter 2

DEFINITIONS OF HYDROGEOLOGICAL TERMS AND LITERATURE REVIEW

This chapter contains definitions of some of the most important hydrogeological terms commonly encountered in groundwater studies. A brief review of past works related to the study, which were carried out by other researchers is also given.

2.1 DEFINITIONS

2.1.1 Aquifers

An aquifer is a geological formation that can store as well as transmit or yield significant quantities of water. A formation which is porous and contains water, but not capable of transmitting water in significant quantities is called an aquiclude. In the extreme case where a formation contains no interconnecting pores and can therefore, neither store nor transmit water, it is termed an aquifuge. Aquifers are generally classified as unconfined, confined and semi-confined.

Unconfined Aquifers: An unconfined aquifer is a permeable bed partly filled with water and overlying a relatively impervious layer. It's upper boundary is formed by a free water table or phreatic level which is under atmospheric pressure. Such an aquifer is also called a water-table aquifer.

Confined Aquifers: A confined aquifer is a completely saturated bed whose upper and lower boundaries are impervious layers. In confined aquifers, the pressure of the water is usually greater than that of the atmosphere, and water in piezometers

or wells penetrating the aquifer, will stand above the top confining layer of the aquifer.

Semi-confined or Leaky Aquifers: A semi-confined or leaky aquifer is a completely saturated aquifer that is bounded above by a semi-pervious layer and below by a layer that is either impervious or semi-pervious. Such aquifers will lose or gain water through adjacent semi-pervious layers when the piezometric head is lowered, say by pumping.



Fig. 2.1. Different Types of Aquifers (Hantush, 1964)

2.1.2 Hydraulic Properties of Aquifers

The most important hydraulic properties of aquifers that influence the flow of groundwater are: hydraulic condutivity or permeability, transmissivity, storage coefficient or specific yield, hydraulic resistance and leakage factor.

Hydraulic Conductivity, K: The hydraulic conductivity or permeability is a measure of the ease with which water flows through the aquifer. It depends, not only on the permeability of the medium, but also on the unit weight and viscosity of the water being transmitted.

The hydraulic conductivity may be defined as the volume of water per unit time passing through a medium of unit cross-sectional area, if the magnitude of the hydraulic gradient prevailing at that section is unity. It has the dimension of velocity (L/T).

Transmissivity, T: Transmissivity is the product of the average hydraulic conductivity (or permeability) and the thickness of the aquifer. Thus, transmissivity can be defined as the rate of flow, under unit hydraulic gradient, through a cross-section of unit width over the entire thickness of the aquifer. It is generally designated by the symbol T or KD and has the dimension of length^2/Time (L^2/T) i.e. m^2/day or m^2/sec.

i.e. $T = KD$ (L^2T^{-1})

where,

K = Hydraulic conductivity (LT^{-1})

D = Saturated thickness of the aquifer (L).

The aquifer transmissivity is a decisive factor of how much water can be obtained from a well.

Storage Coefficient, S, and Specific Yield: The storage coefficient of an aquifer is the volume of water yielded per unit horizontal area per unit drop of water table (for unconfined aquifer) or piezometric surface (confined aquifers). In confined aquifers, it depends on the elasticity of the aquifer material. In unconfined aquifers, the storage coefficient is also referred to as the specific yield, which is the volume of water released from a unit volume of saturated aquifer material drained by a falling water table. Thus if sufficient time is allowed for drainage, the specific yield of an unconfined aquifer is the same as its storage coefficient. Both terms are dimensionless.

Hydraulic Resistance, C: The hydraulic resistance is the ratio of the saturated thickness of the semi-pervious layer, D', to the hydraulic conductivity of the semi-pervious layer for vertical flow, K'. It is perculiar to semi-confined aquifers, and indicates the resistance of the semi-pervious layer to upward or downward leakage. It has the dimension of time (T)

$$\text{i.e} \qquad C = D'/K'$$

where

D' = Saturated thickness of semi-pervious layer (L)

K' = Hydraulic conductivity of semi-pervious layer (LT^{-1})

Leakage Factor, L: The leakage factor determines the distribution of leakage into the semi-confined aquifer. Thus it indicates the origin of the water withdrawn from a well tapping the aquifer, and is expressed as:

$$L = \sqrt{KDC}$$

K = Hydraulic conductivity of the confined aquifer (LT^{-1})

D = Saturated thickness of the confined aquifer (L)

C = Hydraulic resistance of the semi-pervious layer (T)

A high value of L indicates a great resistance of the semi-pervious strata to flow, as compared with the resistance of the aquifer itself. In which case, the influence of leakage will be small. Leakage factor has the dimension of length (L).

2.1.3 Aquifer Boundaries

All aquifers must ultimately, have boundaries. Two distinct types of boundaries are recognised in groundwater study:- (i) Free-head boundary and (ii) Fixed/Constant-head boundary.

A Free-head boundary is one, across which no flow can occur. The drawdown within the area of influence will increase during pumping.

A constant head-boundary is one at which no drawdown occurs. This type of boundary will eventually result in steady state conditions. It is also referred to as fixed-head boundary, and is typified by a boundary that opens into the sea or a large surface of water

2.1.4 Statesof Groundwater Flow

There are two states of groundwater flow; the steady state flow, and the unsteady state flow.

Stead-State Flow: Steady-state or equilibrium flow occurs when there is equilibrium between the discharge from a pumped well and the recharge into the aquifer by an outside source. Steady-state conditions rarely occur in nature,and in practice, a steady state is said to be reached if changes in drawdown with time become negligible, or the hydraulic gradient becomes constant.

Unsteady-State Flow: Unsteady-state or non-equilibrium flow occurs from the moment pumping starts, until the steady-state condition is reached. In practice, well flow is considered to be in unsteady state as long as there is significant variation in drawdown with time; or when hydraulic gradient changes in a measurable way.

2.2 Literature Review

In evaluating the groundwater resource of an area, values of the various parameters, earlier defined, and which determine the characteristics of the aquifer must be ascertained. Methods of estimating these parameters as reported in literature (Health and Trainer, 1968) include:

(a) Laboratory tests on undisturbed soil samples to determine particle size, porosity and permeability.

(b) In the field, the use of tracer techniques and electrolytes to determine flow velocity under known hydraulic gradient. Permeability can thus be determined.

(c) Carrying out a pumping test, which is in effect a controlled field experiment to investigate the aquifer response. It involves a withdrawal of known quantity of water from an abstraction bore-hole, and monitoring changes in groundwater potential at both the abstraction and other observation wells.

(d) The use of modelling technique.

(e) Regional analysis of water-level map.

2.2.1 Equations of Groundwater Flow to a Pumping Well

A. Steady State or Equilibrium Equation

The steady state or equilibrium condition depicts the long-term behaviour of groundwater flow in an aquifer. It is a state where drawdown changes with time is negligible, $(\frac{dh}{dT}=0)$ and the hydraulic gradient has become constant.

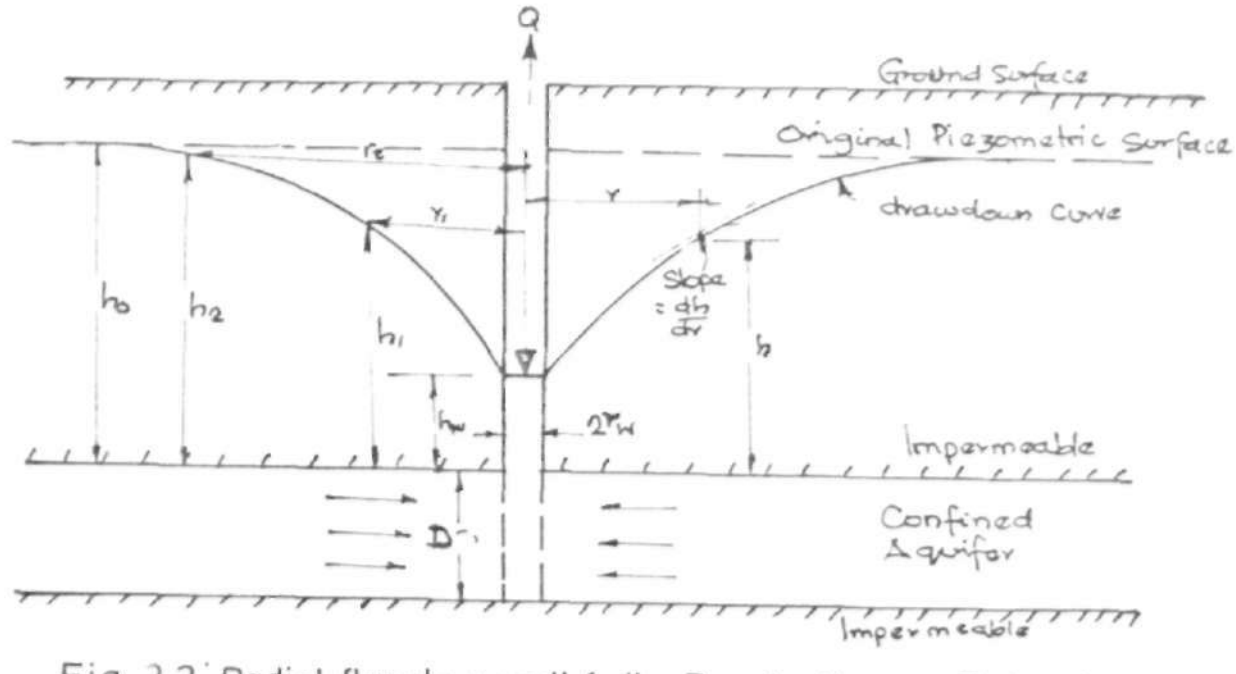


Fig. 2.2: Radial flow to a well fully Penetrating an Extensive Confined Aquifer (Todd, 1959)

For a well fully penetrating an extensive confined aquifer (Figure 2.2) the steady state radial flow equation to the well is

$$Q = 2\pi rDK\frac{dh}{dr} \quad \text{..... 2.1}$$

where

Q = The well discharge (L^3/T)

r = Radial distance from the well (L)

D = Thickness of aquifer (L)

K = Hydraulic conductivity of the confined equifer (L/T)

$\frac{dh}{dr}$ = Charge in head with distance ir the radial direction .

Thiem (1906) showed that for a pumped well in which groundwater elevations are measured in at least two piezometers. equation 2.1, when rearranged and integrated becomes

$$Q = \frac{2\pi KD\ (h_2 - h_1)}{\log_e(r_2/r_1)} \quad \text{..... 2.2a}$$

or

$$K = \frac{Q}{2\pi D\ (h_2 - h_1)} \log_e\ (r_2/r_1) \quad \text{..... 2.2b}$$

where

r_1, r_2 are respective distances of piezometers from the pumped well (m).

h_1, h_2 are respective groundwater elevations in the piezometers (m).

Thus, the hydraulic conductivity, K, and hence transmissivity, KD, of the confined aquifer can be determined.

For an unconfined aquifer, there is a modification to the above equations since the saturated depth or thickness of the aquifer varies along the radius of influence. The applicable expressions at steady state are;

$$Q = 2\pi rKh\frac{dh}{dr} \qquad \ldots\ldots\ldots 2.3$$

and

$$Q = \pi K\frac{(h_2^2 - h_1^2)}{\log_e(r_2/r_1)} \qquad \ldots\ldots\ldots 2.4a$$

or

$$K = \frac{Q}{\pi K(h_2^2-h_1^2)}\log_e(r_2/r_1) \qquad \ldots\ldots\ldots 2.4b$$

It should be noted that all the above equations apply to homogeneous, isotropic aquifers of infinite areal extent in which horizontal, laminar flow at steady state is assumed.

B. Unsteady State or Non-Equilibrium Equations:

As stated earlier, the flow of groundwater towards an abstraction well is normally assumed radial, and the non-equilibrium or unsteady state partial differential equation governing flow in a confined aquifer when the vertical components of flow are sufficiently small to be neglected is (Jacob, 1950).

$$T\left[\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r}\right] = S\frac{\partial h}{\partial t} + q \qquad \ldots\ldots 2.5.$$

where

r is the radial coordinate direction (L)

h is the groundwater potential (L)

T is the transmissivity in the radial direction (L^2/T)

S is the storage coefficient.

t is the time (T)

q is the inflow into the aquifer per unit area (L/T).

Theis Solution: Theis (1935) in a pioneering work derived an expression for the variation of drawdown with time due to abstraction from a well in an infinite confined aquifer, taking the effect of storage into consideration. He noted that when a well penetrating an extensive confined aquifer, is pumped at a constant rate, the influence of the discharge extends outwards with time. The rate of decline of head, multiplied by the storage coefficient and summed over the area of influence, equals the discharge. The head will continue to decline as long as the aquifer is infinite and theoritically therefore, no steady-state flow can exist. However, the rate of decline in head decreases continuously as the area of influence enlarges, and it will eventually become so small and negligible. In practice, it is this state that is considered steady-state.

The solution to the non steady-state or Theis (1935) equation, which was derived from an analogy between the flow of groundwater and the conduction of heat may be written as:

$$s = \frac{Q}{4\pi KD} \int_u^\infty \frac{e^{-u}du}{u} = \frac{Q}{4\pi KD} W(u) \quad \ldots\ldots\ldots 2.6$$

where

$$u = \frac{r^2 S}{4KDt} \quad \text{or} \quad S = \frac{4KDtu}{r^2} \qquad \ldots\ldots\ldots 2.7$$

s is the drawdown in metres, measured in a piezometer at a distance, r in metres from the pumped well.

Q is the constant well discharge (m^3/day)

S is a dimensionless coefficient of storage

KD or T is the transmissivity of the aquifer (m^2/day)

t is the time in days since pumping started

W(u) is the "Well Function of u" or "Theis Well Function" and is expressed as

$$W(u) = -0.5772 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \ldots 2.8$$

Since the publication by Theis, many additional analytical solutions and curve-fitting techniques have been developed by other researchers for a wide variety of natural conditions, including cases of leaky aquifers, partially penetrating wells, boundary effects, water contained within the well, aquifers which are partially confined and partially unconfined, anisotropic aquifers, delayed yield from storage, vertical components of flow, and stepped abstraction rates. Treatment of these cases have been described in details by Walton (1970), and Kruseman and DeRidder (1970).

Very versatile alternatives to the analytical and curve-fitting techniques, have also been developed for solving the groundwater flow differential equation by other researchers. These include the discrete space-discrete time numerical model

and the electrical analog model.

Stallman (1956) derived a finite difference form of equation 2.5 for determination of permeability of an aquifer by numerical analysis of the free surface, while Trelease and Bittinger (1963) used a digital computer for numerical evaluation of the series form of the analytical solution to equation 2.5 as obtained by Theis (1935)in equation 2.8. These methods have the advantage that as many conditions as necessary can be included in a single numerical solution.

In the area of analog modelling, Shibitzke (1961) employed an electrical analog to solve the finite difference equation in simulation of aquifers, and Walton and Prickett (1963) have also described a comprehensive analog model to solve the groundwater flow differential equation.

Chapter 3

DESCRIPTION OF MODEL

3.1 Definition

A model of a groundwater reservoir can be defined as any system which can duplicate the response of the reservoir. The operation of the model and manipulation of the results is called simulation (Prickett, 1970).

3.2 Aquifer System Analysis - Hydrogeological Models

In planning for the optimum exploitation and management of the groundwater resource of an area, it is necessary to understand the working system of the aquifer. Hydrogeological models provide a powerful tool for analysing aquifer system and examining the distribution and flow process of water within the aquifer. Extensive information and data from wide variety of sources have to be obtained before a model investigation of groundwater flow in an aquifer can be carried out. Pilot scheme investigations provide a useful source of such data.

When all the necessary information are incorporated in the model, it serves as a check on the consistency of the data, and where deficiencies become apparent, further investigation to obtain the required data can be initiated. In cases where certain vital information are not available, a probable range of values within which the parameter should lie can be used. A number of solutions can then be obtained for the range, and the sensitivity of the results to the varying values will

indicate the overall accuracy of the model. Even when a satisfactory model has been devised and tested, a continuous monitoring of the variation of the groundwater potential is still necessary, and additional information obtained are used to update the aquifer model.

Once the groundwater model has been tested and proved against historical data, it can then be used to simulate and predict the effects of various possible abstraction schemes. Hydrogeological models which have been used to study and evaluate groudwater flow can be classified into three broad categories (Kampsax Krügger and Sshwed, 1977):-

(1) Mathematical Models

(2) Analog Models

(3) Numerical /Digital Models

The choice of the modelling technique will largely depend on the basic data available. In this work, a numerical model is used, but both mathematical and analog techniques will be briefly described. The advantages and disadvantages of each of the three models will also be highlighted.

3.2.1 Mathematical Models

The Mathematical formulation of a model consists of working with the appropriate differential equations and their associated initial and boundary conditions. Where hydraulic parameters are constant throughout the whole aquifer and the boundary configurations are simplified, a simple mathematical

model can be applied to determine the aquifers' response, i.e. drawdown in a pumping well and aquifer for a given withdrawal.

If the aquifer is bounded by recharge or discharge boundaries, such are simplified to form simple geometrical figures to make the mathematical model operational. If no boundaries are known to exist, the modelled aquifer is extended to infinity. While generally simple and inexpensive to operate, mathematical models are known to cause considerable errors in drawdown calculation, particularly when the boundary condition is not properly represented. They are also not very useful when the aquifer parameters vary widely from one location to another.

3.2.2 Analog Models

The behaviour of an aquifer can be described by differential equations which are derived from basic principles such as the laws of continuity and energy conservation. Differential equations governing response in other systems in different physical categories, such as heat flow through a slab of material or electrical current flow through a resistive medium have also been derived, based upon the same laws of continuity and energy conservation.

When a one-to-one correspondence occur between all terms of the same differential orders, then the similarity of the two differential equations is established, and the two systems are said to be analogs of one another. A physical model of one can then be used to evaluate response in the other.

Analog models that have been successfully used in studying groundwater problems include (Prickett, 1970); (1) the viscous fluid models, (2) Electrical continuous systems - models, and (3) Electrical discrete system models.

Analog models are useful where the hydraulic properties of an aquifer vary in space, and more exact evaluation of aquifer's response to withdrawal is desirable. They however have the disadvantage of been time-consuming, and requiring different instruments and laboratory facilities to build physical models.

3.2.3 Numerical/Digital Models

A third, and perhaps more powerful method of evaluating groundwater resources of an aquifer is through the use of numerical models. Numerical methods take into account any real variation of the hydraulic parameters, and existing boundaries can be modelled with any desired accuracy. Where the available geologic and hydrologic data are not sufficient to permit a rigorous analysis of an aquifer, numerical methods can still be used to approximate the performance of the well and aquifer using the so called "idealised aquifer". With this assumption, complex hydrogeologic boundaries of the aquifer can be idealized into elementary geometric forms made up of straight-line demarcations, such as wedges and infinite or semi-infinite rectilinear strips. Average hydrogeological properties of the aquifer and confining layers are then selected on the basis of any known field data, and used in the model.

All numerical methods involve 3 steps:

(1) Re-writing the appropriate differential equation in its discrete space-time form, in which continuous deri.atives are replaced by ratios of changes in variables over a small finite interval.

(2) With know. boundary or initial values, a set of algebraic aquations can be written.

(3) Solving the set of algebric equations, numerically with the aid of a digital computer.

Both finite difference and finite element methods of analysis have been used in formulating numerical models. (Stallman 1956, Prickett and Lonnguist, 1977). The main advantage of these methods is that as many conditions as necessary can be included in a single numerical solution. These conditions include leakage; wells of finite radius, variable abstraction rates, boundary effects, change between confined and unconfined conditions, well losses, delayed yield conditions, etc. A disadvantage of the digital and other models is that the model's boundary conditions have to be well known if the boundary will be reached by the cone of depression before a state of equilibrium between pumpage and recharge is attained otherwise drawdown results will be in error.

3.3 The Radial Groundwater Flow Model

In this section, the derivation of a discrete model of horizontal radial flow through an aquifer to an abstraction well

is presented. A method of obtaining numerical solution for the model using a digital computer program developed by Rushton and Redshaw (1979) is discussed. It should be noted that the model is restricted to cases where the vertical components of flow are assumed to be very small and negligible.

3.3.1 Derivation of Discrete Model

There are two method of derivation: (1) the derivation from the governing differential equation, and (2) derivation using the "lumping concept", - an approach which was introduced by Rushton and Booth in 1976. Only the first method will be treated, since the two methods lead to identical solution.

3.3.2 Derivation from Differential Equation

As earlier stated, the non-equilibrium differential equation describing horizontal radial flow in a homogeneous aquifer, in which vertical flow components are negligible, can be written as:

$$\frac{\partial}{\partial r}\left[mK_r \cdot \frac{\partial h}{\partial r}\right] + \frac{mK_r}{r} \cdot \frac{\partial h}{\partial r} = S \frac{\partial h}{\partial t} + q$$

or

$$T\left[\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \cdot \frac{\partial h}{\partial r}\right] = S \cdot \frac{\partial h}{\partial t} + q \qquad \ldots\ldots 3.1$$

where

$T = mK_r$ = Transmissivity of aquifer in radial direction (L^2/T)

h = Water level piezometric level above an arbitrary datum (L)

r = radial coordinate (L)

m = aquifer thickness/saturated depth (L)

K_r = radial permeability (L/T)

t = time (T)

S = storage coefficient (confined or unconfined)

q = inflow per unit area which may be a function of radius (L/T).

Introducing a radial variable 'a' such that

$$a = \log_e r, \text{ or } \frac{\partial a}{\partial r} = \frac{1}{r} \qquad \ldots\ldots\ldots 3.2$$

since

$$\frac{\partial h}{\partial r} = \frac{\partial h}{\partial a} \cdot \frac{\partial a}{\partial r} = \frac{1}{r} \cdot \frac{\partial h}{\partial a} \qquad \ldots\ldots\ldots 3.3$$

and

$$\frac{\partial^2 h}{\partial r^2} = \frac{\partial}{\partial r}\left[\frac{1}{r} \cdot \frac{\partial h}{\partial a}\right] = -\frac{1}{r^2} \cdot \frac{\partial h}{\partial a} + \frac{1}{r^2} \cdot \frac{\partial^2 h}{\partial a^2} \quad \ldots\ldots 3.4$$

Substituting into equation 3.1 gives

$$T\left[\left(-\frac{1}{r^2} \cdot \frac{\partial h}{\partial a} + \frac{1}{r^2} \cdot \frac{\partial^2 h}{\partial a^2}\right) + \frac{1}{r} \cdot \frac{1}{r} \frac{\partial h}{\partial a}\right] = S \frac{\partial h}{\partial t} + q$$

or

$$\frac{T}{r^2} \frac{\partial^2 h}{\partial a^2} = S \cdot \frac{\partial h}{\partial t} + q \qquad \ldots\ldots 3.5$$

Multiplying through by r^2 gives

$$T\frac{\partial^2 h}{\partial a^2} = Sr^2 \cdot \frac{\partial h}{\partial t} + qr^2$$

or

$$mK_r\frac{\partial^2 h}{\partial a^2} = Sr^2 \frac{\partial h}{\partial t} + qr^2 \qquad \ldots\ldots\ldots\ldots 3.6$$

A finite-difference form of equation 3.6 is expressed by dividing the radial ordinate into a discrete mesh which increases logarithmically by an amount Δa as shown below:

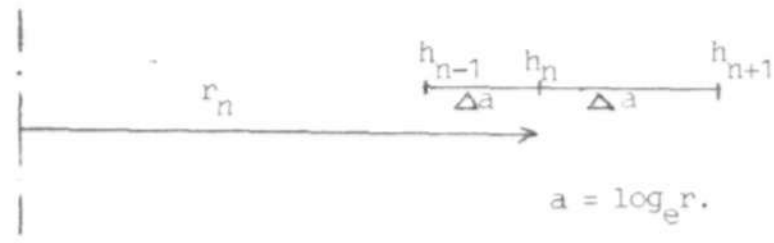


Fig. 3.1: Radial finite difference mesh with logarithmic increase in mesh interval.

The finite difference form, with centre at node n; then becomes;

$$\frac{mK_r}{\Delta a^2}(h_{n-1} - 2h_n + h_{n+1})_{t+\Delta t} = \frac{Sr_n^2}{\Delta t}(h_{n,t+\Delta t} - h_{n,t}) + q_{t+\frac{1}{2}\Delta t} r_n^2 \qquad \ldots\ldots\ldots\ldots 3.7$$

With appropriate initial and boundary conditions, above equation 3.7 can be solved by numerical methods. It is to be noted that with the logarithmic mesh interval adopted, nodal points are closely spaced within the vicinity of the abstraction well, but become more widely spaced at greater distances away. This provides a suitable arrangement for modelling pumping tests

since rapid changes in drawdown occur in the vicinity of the pumped well. Logarithmic time increment is also adopted to cater for very long pumping duration.

Equation 3.7 can be viewed in terms of equivalent hydraulic resistance as shown in fig. 3.2 below. Such that :

$$H_n = \frac{\Delta a^2}{mK_r} \qquad \ldots\ldots\ldots\ldots 3.8$$

$$T_n = \frac{\Delta t}{Sr_n^2} \qquad \ldots\ldots\ldots\ldots 3.9$$

where,

H_n = Hydraulic resistance to horizontal flow at node n.

T_n = Time resistance of node n.

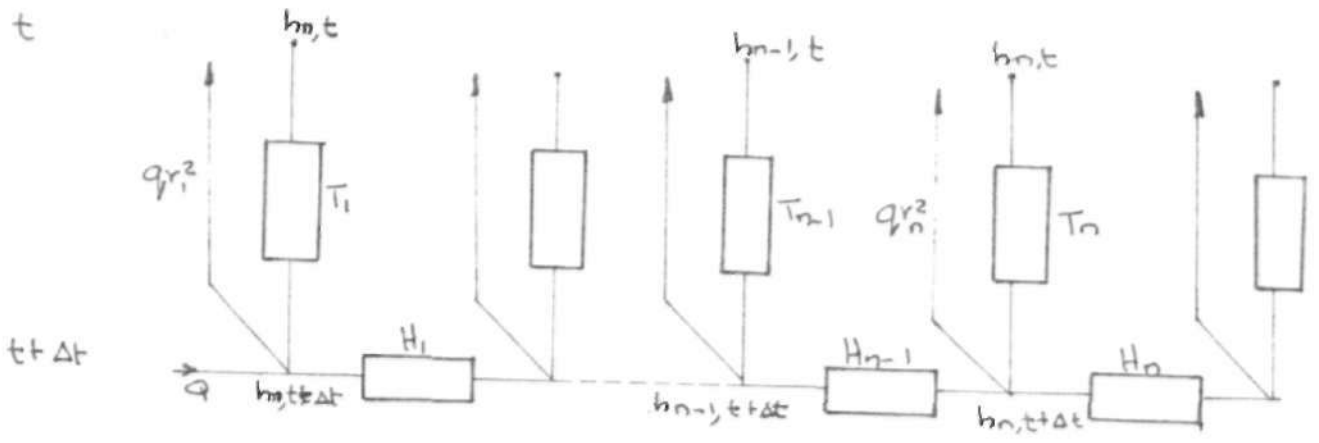


Fig. 3.2: Representation of equivalent hydraulic and Time resistance (Rushton and Redshaw, 1979)

Assuming a positive flow from each node equal to qr_n^2, then the equation for node, n, becomes

$$\frac{(h_{n-1} - 2h_n + h_{n+1})_{t+\Delta t}}{H_n} = \frac{h_{n,\, t+\Delta t} - h_{n,t}}{T_n} + qr_n^2 \quad \ldots\ldots 3.10$$

The equivalent hydraulic resistance concept will be found to be particularly useful when considering the respresentation of boundary conditions. The abstraction from the pumped well, Q (L^3/T) is represented in the model as a negative recharge at node 1, and it is converted to an equivalent recharge, q (L/T), at any nodal point by:

$$q = \frac{-Q}{A_n} \quad \ldots\ldots 3.11$$

where A_n is the area represented by a node, and is given as,

$$A_n = 2\pi r \Delta r \quad \ldots\ldots 3.12$$

Since $a = \log_e r$, or $\frac{\partial a}{\partial r} = \frac{1}{r}$, and $\Delta r = r\Delta a$

$$\therefore \quad A_n = 2\pi r^2 \Delta a \quad \ldots\ldots 3.13$$

and

$$q = \frac{-Q}{2\pi r^2 \Delta a} \quad \ldots\ldots 3.14$$

3.3.3 Numerical Solution

In obtaining a numerical solution for the horizontal radial flow model described above, the following factors are

considered in the digital computer program used. (see Fig. 3.4).

Numbering of nodes and mesh spacing: The nodes were numbered from 1 to NMAX, with node 1 representing the region within the well, the well radius is R (2) and the outer boundary R(NMAX). An increment Δa, equal to 0.38376 was adopted, so that there are six mesh intervals for a tenfold increase in radius. The final interval ends at R(NMAX).

Position of Observation Wells (Node number of Obs. Wells): Since a logarithmic mesh spacing was adopted in deriving the finite-difference form of the model equation, it becomes convenient to position observation wells at nodal points in the computer program.

Thus, we have:

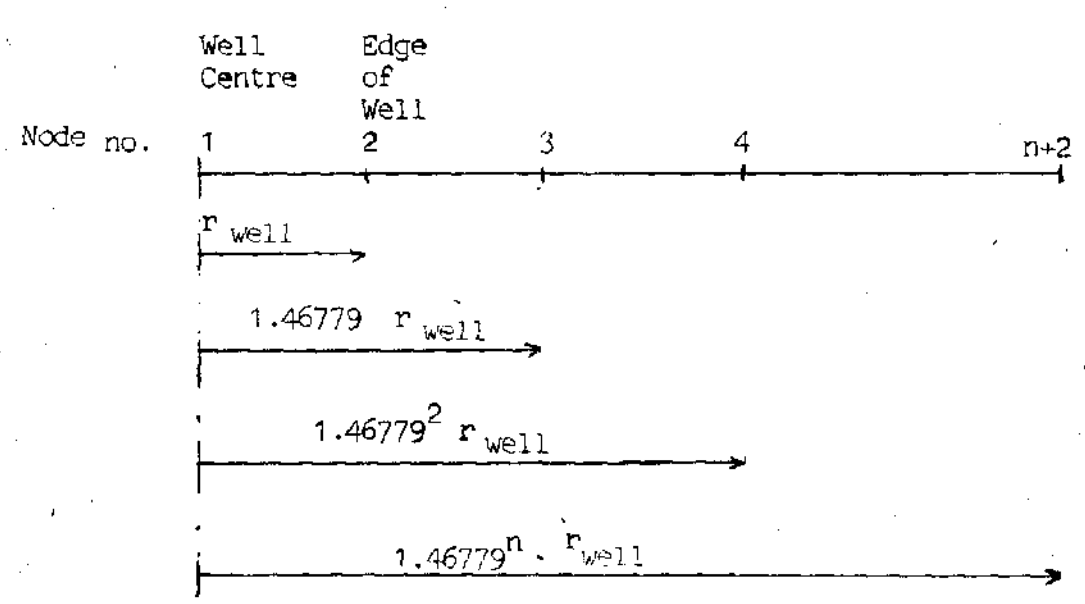


Fig. 3.3 Numbering of observation wells nodes.

For $$\Delta a = \log_e \left(\frac{r_{n+1}}{r_n}\right) = 0.38376$$

$$r_{n+1} = e^{\Delta a} = e^{0.38376} = 1.46779$$

Since node 1 is at the centre of the well, and node 2 at the radius of the well, i.e . $r_2 = r_{well}$

$\therefore \quad r_3 = 1.46779\, r_{well}$

$r_4 = 1.46779\, r_3 = 1.46779^2\, r_{well}$

and so on.

Thus, we generally have;

$$r_{n+2} = 1.46779^n\, r_{well}.$$

For a well of 200mm radius, an observation well at 200m away will have a node no. of 20 as shown below:-

$$1.46779^n = \frac{r_{n+2}}{r_{well}} = \frac{200}{0.200} = 10^3$$

$$n \log (1.46779) = \log (10^3) = 3$$

$$n = 3/\log (1.46779) = 18.003$$

$$n + 2 = 18 + 2 = \underline{20}$$

This arrangement has the added advantage of being able to monitor rapid changes in drawdown which occur within the vicinity of the pumped well since the nodes become more closely spaced towards the well.

Time Step: In order to be able to accurately monitor the response at the smallest radius, the initial time step is set to a very small value of;

$$\Delta t = \frac{0.025 r_w^2\, S_c}{T} \qquad \ldots\ldots\ldots 3.18$$

where,

S_c = confined storage coefficient

r_w = radius of abstraction well

T = Transmissivity.

The time is thereafter, increased by a factor $10^{0.1}$ for each calculation, thus providing ten time steps for a ten-fold increase in time. Each time the abstraction rate is changed, the time step is returned to the initial small value given in equation 3.18.

Confined to Unconfined: At any node, the level of drawdown relative to the top of the aquifer determines whether the aquifer is confined or unconfined at that node. If a node has drawdown less than TOP, which is the position of the top of the aquifer below a datum level, then the aquifer is confined and the confined storage coefficient is applied at that node. If on the other hand, the drawdown is greater than TOP, the aquifer becomes unconfined, and the unconfined storage coefficient is used.

Variable Saturated Depth (SD): In a confined aquifer, the saturated depth SD, is equal to the aquifer thickness, and is usually assumed constant.

i.e. SD = BASE - TOP.

However, in an unconfined aquifer, the saturated depth is a function of the drawdown, and is calculated as

SD = BASE - D(N).

Since SD depends on an unknown drawdown D(N), an iterative loop for each time step is incorporated in the program after

statement no.50, in which calculation is repeated four times with successively improved approximations to the saturated depth. This rigorous approach has been adopted because the horizontal hydraulic resistance, H(N) which is an important parameter in the model equation is also a function of the saturated depth, SD.

Excessive drawdown: To avoid having a situation where the groundwater potentials calculated will fall below the base of the aquifer, a test is incorporated in the program to check excessive drawdown.

Rapid changes in drawdowns are likely to occur when the saturated depth becomes less than half of the original saturated thickness. The calculation is therefore stopped once the depth of water in the abstraction well becomes less than 10 percent of the original saturated thickness. (See comment statement after statement no 110 in the program listing).

3.3.4 Program Computations

(a) Any node N.

For any node N. the hydraulic resistors H(N) and H(N-1) are calculated from equation 3.8, and the corresponding time resistance T(N) from equation 3.9 .

With the unknown drawdown, and the drawdown at the previous time step represented by D(N) and OLDD(N) respectively, the model equation for node, N, can then be written (Rushton and Redshaw, 1979)-see figure 3.4.

$$\frac{-D(N-1)}{H(N-1)} + D(N)\left[\frac{1}{H(N-1)} + \frac{1}{H(N)} + \frac{1}{T(N)}\right] - \frac{D(N+1)}{H(N)} = \frac{OLDD(N)}{T(N)} - RR(N)\ RECH(N) \qquad \ldots\ldots 3.15$$

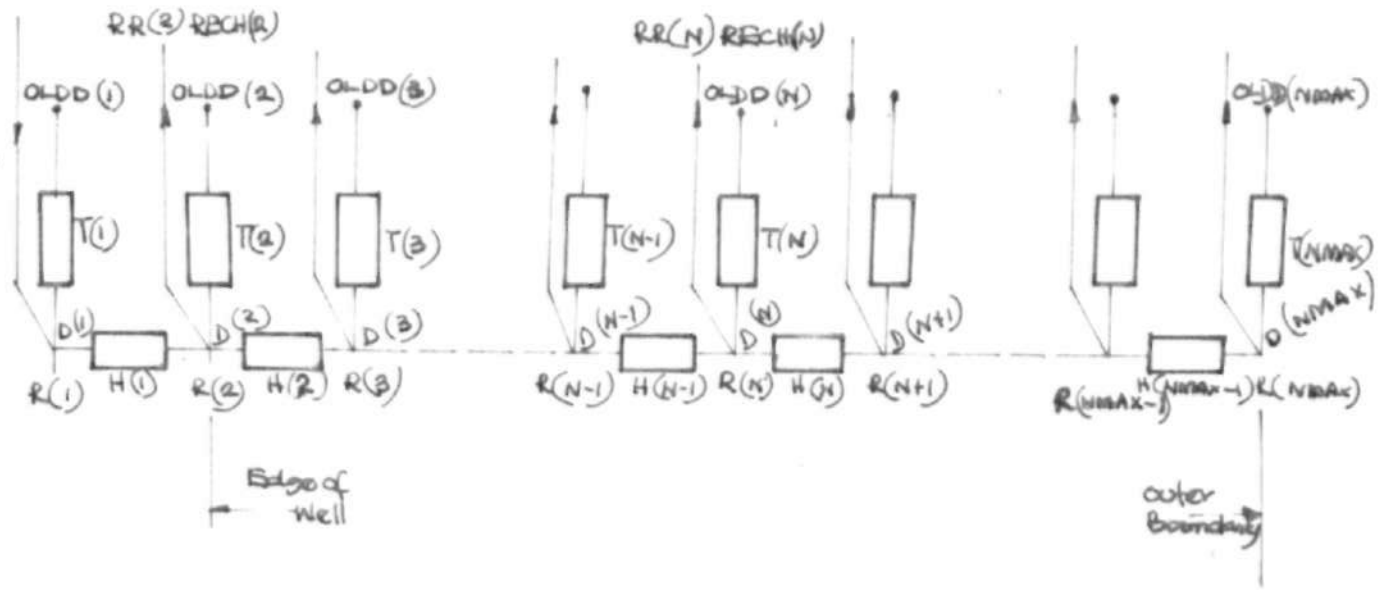


Fig. 3.4: Representation of discrete model with allowance for water in Well (Rushton and Redshaw, 1979)

where,

R(N) is the radius at node N, and RR(N) = R(N) x R(N).

(b) Well nodes (Nodes 1 and 2)

The well nodes are considered separately from other nodes because, while a significant quantity of water is initially present within the well before pumping started, the discrete model derivation only considered flow in the aquifer. To simulate this initial free water within the well, the aquifer is assumed to extend into the well but with different properties. Aquifer properties within the well are assumed such that the transmissivity is very high and the storage coefficient is unity, which correspond to free water within the well.

Thus, horizontal hydraulic resistance H(1) is set to a low value, and 'time resistance' T(1), is expressed as

$$T(1) = \frac{\Delta t}{S r_1^2}$$

From equation 3.13 the area represented by node 1 is

$$A_1 = 2\pi r_1^2 \cdot \Delta a$$

But node 1 also represents the plan area of the well of radius r_2, thus

$$A_{well} = \pi r_2^2$$

This gives the ratio of the unmodified area of node 1, to the area of the well as

$$\frac{A_1}{A_{well}} = \frac{2\pi r_1^2 \Delta a}{\pi r_2^2}$$

The modified 'time resistance' for free water within the well with a storage coefficient of unity then becomes

$$T(1) = \frac{\Delta t}{1 \times r_1^2} \cdot \frac{2\pi r_1^2 \Delta a}{\pi r_2^2}$$

$$= \frac{2\Delta t \Delta a}{r_2^2}$$

i.e $$T(1) = \frac{2\Delta t \Delta a}{RR(2)}$$

The amount of water pumped from the well per unit time, QPUMP, is modelled by expressing it as a negative recharge at node 1. This amount, which at the initial pumping stage is drawn partly from the free water within the well and also partly from the aquifer is expressed in equation 3.14 as

$$q = \frac{-QPUMP}{2\pi RR(1)\Delta a}$$

The flow-balance equation for nodel 1 (see fig. 3.4) becomes

$$D(1)\left[\frac{1}{H(1)} + \frac{1}{T(1)}\right] - \frac{D(2)}{H(1)} = \frac{QLDD(1)}{T(1)} - \left[\frac{-QPUMP}{2\pi RR(1)\Delta a}\right] RR(1)$$

or

$$D(1)\left[\frac{1}{H(1)} + \frac{1}{T(1)}\right] - \frac{D(2)}{H(1)} = \frac{QLDD(1)}{T(1)} + \frac{QPUMP}{2\pi\Delta a} \quad \ldots\ldots 3.16$$

(c) Outer Boundary (Node, NMAX)

At the outer boundary, where radius is R(NMAX), the model is terminated. H(NMAX) is made to have a very large value, and

the equation for node NMAX (Rushton and Radshaw, 1979) becomes

$$-\frac{D(NMAX-1)}{H(NMAX-1)} + D(NMAX)\left[\frac{1}{H(NMAX-1)} + \frac{1}{H(NMAX)}\ (\text{negligible}) + \frac{1}{T(NMAX)}\right]$$

$$= \frac{OLDD(NMAX)}{T(NMAX)} - 0.5\ RR(NMAX)\ RECH\ (NMAX)$$

or

$$-\frac{D(NMAX-1)}{H(NMAX-1)} + D(NMAX)\left[\frac{1}{H(NMAX-1)} + \frac{1}{T(NMAX)}\right]$$

$$= \frac{OLDD(NMAX)}{T(NMAX)} - 0.5RR(NMAX)\ RECH\ (NMAX) \quad \ldots\ldots\ldots\ 3.17$$

Equation 3.17 can be modified slightly for different outer boundary conditions. For example if the outer boundary drawdown remains at constant value, the time resistance is set to a small value which implies very large storage coefficient. With such a large storage, the drawdown at outer boundary will virtually remain unchanged, thus satisfying the constant drawdown condition.

Solution to Equations: In matrix form, equations 3.15, 3.16 and 3.17 form a tridiagonal matrix, whose solution is obtained in the program by Gaussian elimination.

The flow chart for the pumping test program and the full program listing are presented in the appendix I and II respectively.

Chapter 4

THE STUDY AREA

This chapter is used for describing the project study area. The description would include the location, topography, climate, geology and hydrogeology.

4.1 Location

The project site is within the Ipaja Housing Estate, which is located in Lagos State of Nigeria at approximately latitude 6° 36' North and longitude 3° 17' East. The housing estate is accessible through Ipaja Road which originates from Agege, crosses the Abeokuta Express Road and terminates at Ipaja Village. The location map is as earlier shown in Figure 1.

4.2 Topography

The Housing Estate lies between 32 and 43 metres above sea level and covers an area of 160 hectares. The shape of the estate is somewhat rectangular, with a maximum distance of about 1.0 kilometres in the north-south direction, and 2.2 kilometres in the east-west direction. The terrain slopes gently towards north eastern direction.

4.3 Geology

The geology of the area which lie within the south-western part of Nigeria has been described in details by De Swardt (1953),

Jones and Hockey (1964), and Reyment (1964).

The area lies within the coastal basin of the Niger, wherein the oldest sediments were deposited during a transgression of the sea from the south in late Senonian times. Gradual recession of the sea has been accompanied by an extensive down warping of the crystalline basement floor, which helped accumulate more than 1 km of Senonian and Paleocene sediments at the present day coastline. After the Paleocene period, deltaic and estuarine conditions prevailed and sedimentation continued from the marine to the partly continental coastal plain sand.

The present day coastal sediments, overlying the coastal plains sand extend over a long belt, stretching from the Benin border up to the Niger Delta and further east.

To the north of the belt of coastal sediments, the coastal plains sand outcrops in a belt of variable width around a latitude through Abeokuta along Ogun River. A vertical profile shows the succession of deposits beneath this belt to be as follows:-

At the base of the succession, at a depth of about 750 metres lies the Senonian Abeokuta formation to a thickness of about 300m. This is overlaid by the Ewekoro formation deposited during Paleocene times to a thickness of 250m. The Ilaro formation comprising Eocene sediments and the coastal plains sands lie unconformably upon each other over the Ewekoro formation to a total thickness of about 200m. Ilaro formation and coastal plains sands are indistinguishable in the

field due to lack of clear lithological features. Both however consist of inter bedded sand and clay layers in an ill-sorted fashion. Figure 4.1 shows the geological map of the area.

4.4 Climate

The climate of the area is typical of coastal tropical climate and is characterised by well marked wet and dry seasons. The dry season lasts from November to March, while the wet season lasts from April to October (Asseez, 1971).

The average daily temperature varies from 23°C to 31°C and the annual average daily temperature is 27°C within the last 25 years, Ikeja which is about 8 km away from the Housing Estate, has recorded tempertures as high as 34°C and as low as 17°C.

The average annual rainfall in Ikeja is 1,565mm. During the wet season, rainfall is heaviest in the month of June, while August is the driest month.

The average daily humidity over a year is 83 percent in the morning and 76 percent in the evening.

4.5 Hydrogeology

From hydrogeological point of view, the Abeckuta formation and the Ilaro-Coastal plains sands formations contain the most important aquifers for abstraction of groundwater. The Ewekoro formation consists mainly of shales with silt partings and fine sands, and is therefore of little interest as a water

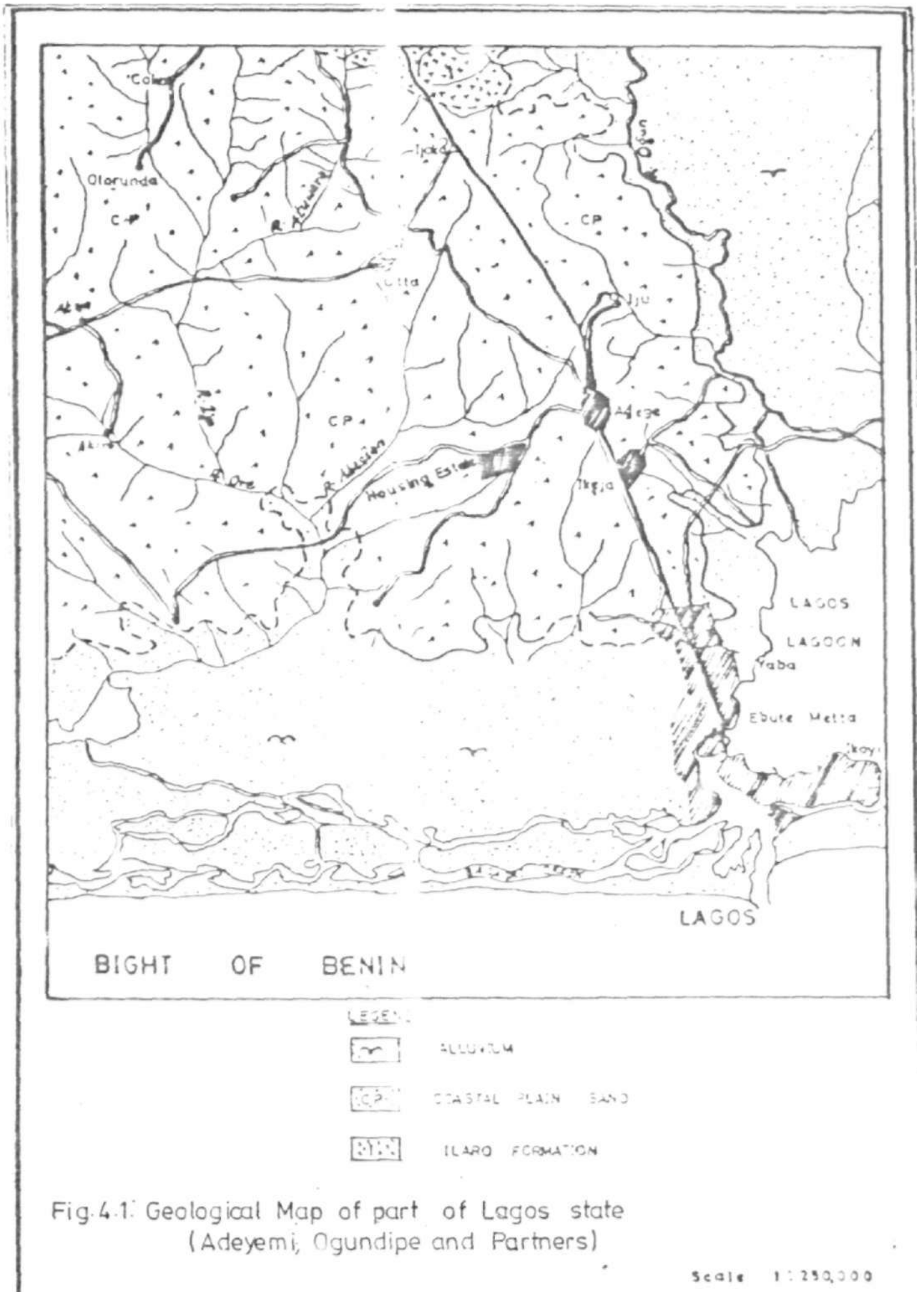


Fig. 4.1: Geological Map of part of Lagos state
(Adeyemi, Ogundipe and Partners)

Scale 1:250,000

bearing stratum (NWRD, 1965, and Asseez, 1971).

There are four major aquifer horizons in the area. The three upper aquifers are seperated from one other by alternating sequences of clay and sand layers of varying thickness (Kampsax Krügger and Sshwed, 1977).

The first or uppermost aquifer lies at the top of the coastal plains sands at approximately sea level. It is of minor importance for any major water supply because of its poor yield. There is also a potential risk of groundwater pollution because of its nearness to the surface.

The second aquifer occurs in the basal beds of the coastal plains sands at elevation ranging from 20 - 70 metres below sea level (m.b.s.l) in the northern part, and 40 - 100 m.b.s.l near the coast.

The base of the Ilaro formation holds the next aquifer at elevations ranging from 130m to 160m.b.s.l. in central part of Lagos, and dipping towards the coast to 170m - 210m.b.s.l.

The fourth aquifer lies in the Abeokuta formation above the basal beds of carbonaceous clay mudstone, and calcareous silt. The aquifer is approximately 60 m thick and occurs at elevation greater than 400m, b.s.l. A thick layer of shale seperates this aquifer from the third aquifer and it is certainly confined. Hot water of temperature greater than 60^{o}C was reported from a borehold drilled to this aquifer at Guiness Nigeria Limited in Ikeja (Kampsax Krugger and Sshwed, 1977).

Chapter 5

ANALYSIS OF PUMPING TEST DATA, ESTIMATION OF MODEL INPUT PARAMETERS AND RESULTS.

In this chapter, a detailed report is given of the analysis of the Field pumping test data and how the model input parameters were estimated. The computer progam is then run with input parameters being varied, one at a time and results compiled.

5.1 Analysis of Pumping Test Data

5.1.1 Borehole Construction Information

Between October 1982 and July 1983, a total of 16 boreholes we drilled and located within the Ipaja Housing Estate as shown in Figure 5.1. An enlarged sketch of the borehole locations with their distances apart is given in Figure 5.2. The boreholes are numbered TW for the central well, and 1,2,3..... up to 15.

All the boreholes were drilled by rotary method, to depths varying from 120 meters to 136 meters below ground level, with soil samples being taken at every 2m depth. A Lithological cross-section of the pumping test site, based on data obtained from the boreholes is shown in Figure 5.3 A typical borehole sketch and strata log is presented in Figure 5.4.

The drilled diameter was 500mm, while steel casing of 200mm internal radius and 16mm thickness was installed. Johnson screen of same radius as the steel casing was positioned at the appropriate water bearing, granular zones. The overall lengths

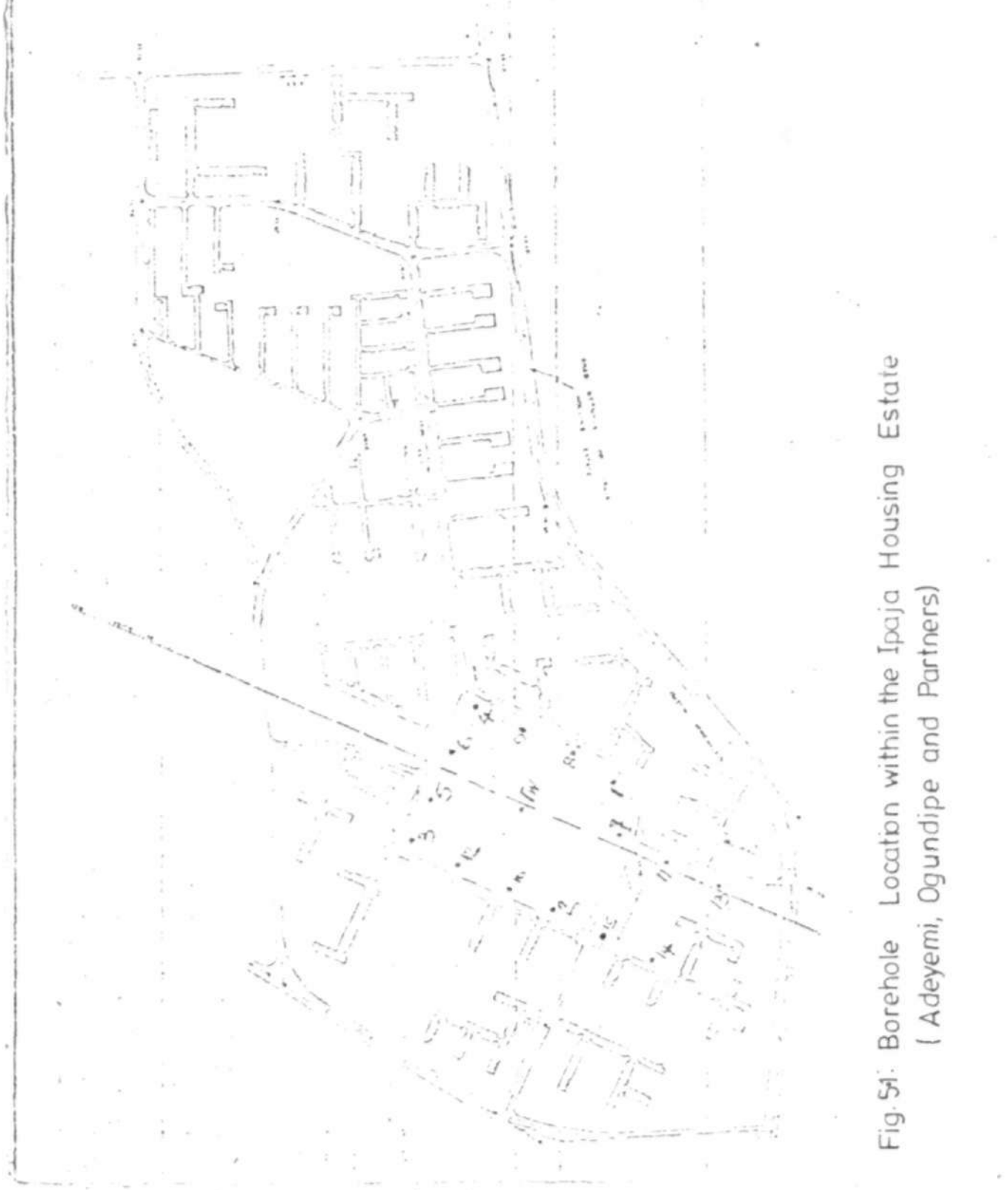


Fig. 5f: Borehole Location within the Ipaja Housing Estate
(Adeyemi, Ogundipe and Partners)

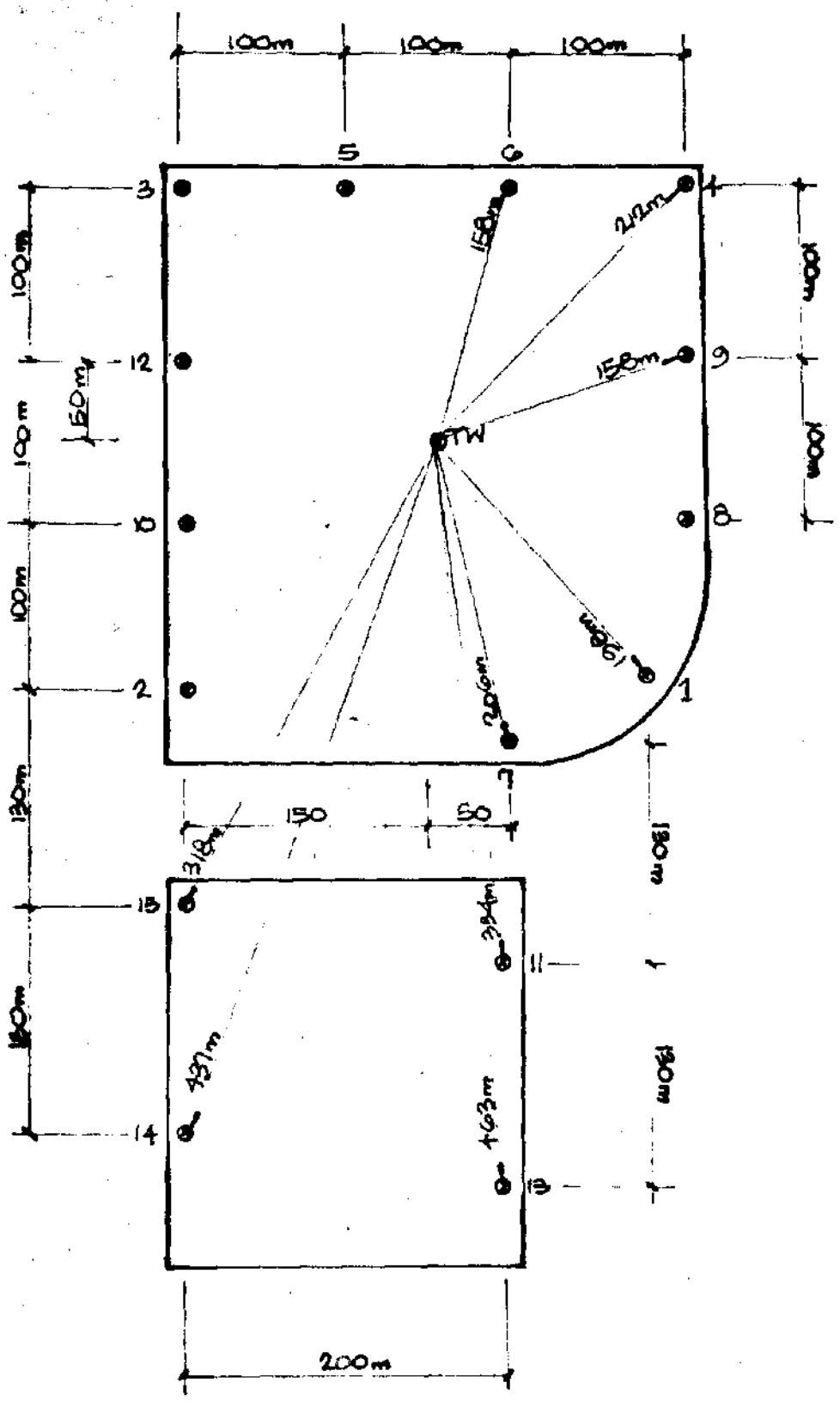


Fig. 5.2: Borehole Location Sketch (16 Nº Boreholes)

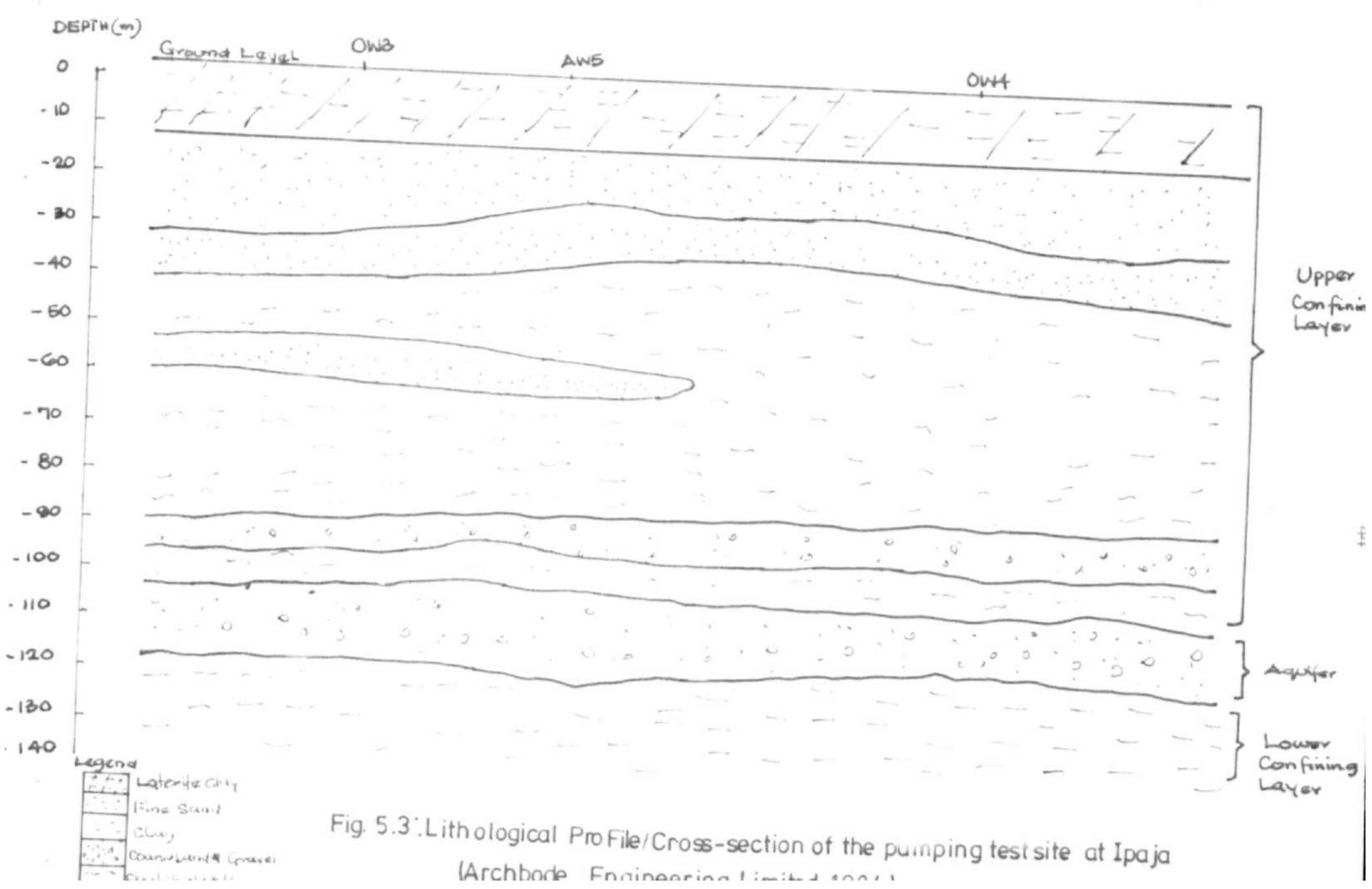


Fig. 5.3: Lithological Profile/Cross-section of the pumping test site at Ipaja (Archbode Engineering Limited 1994)

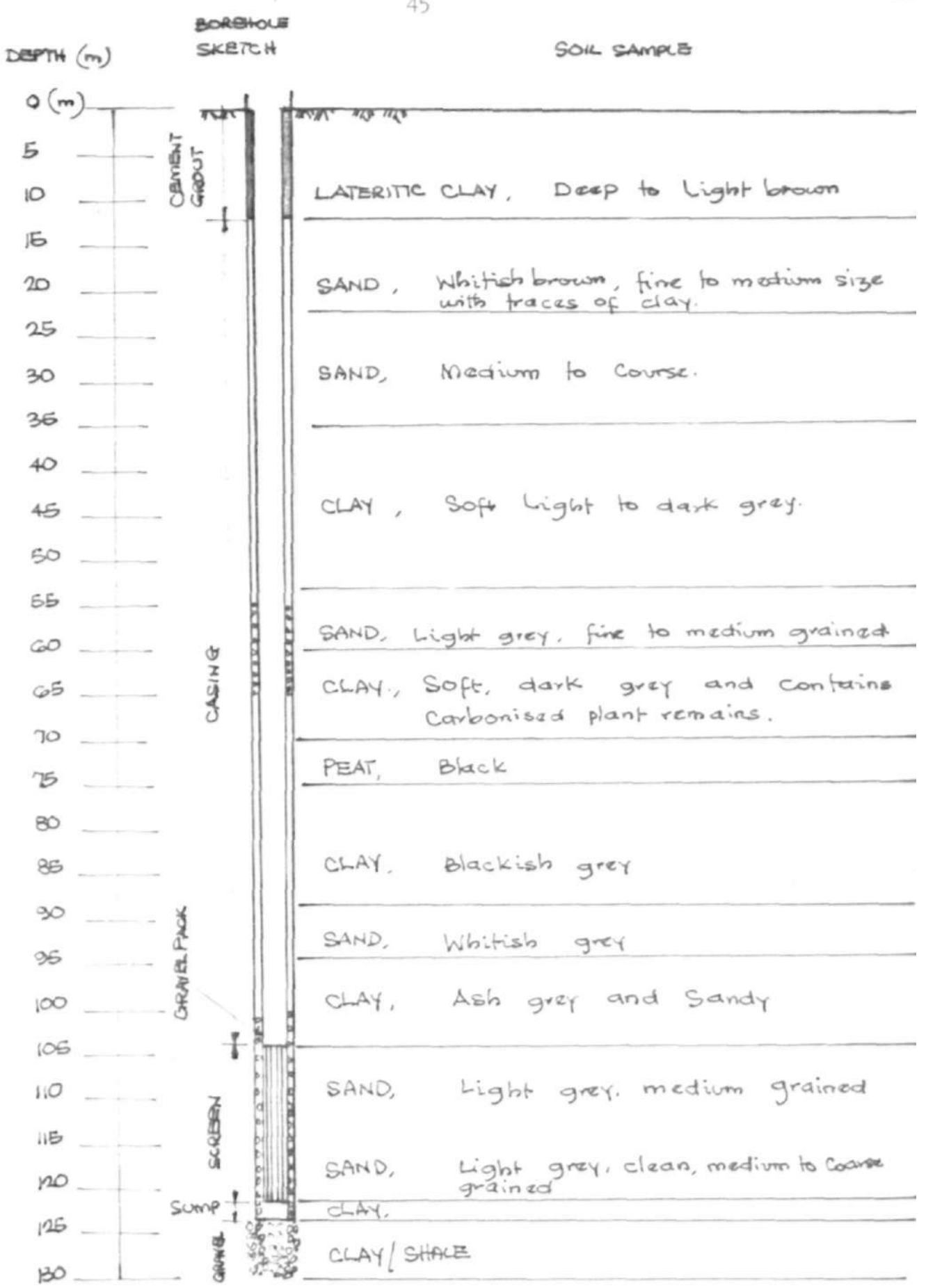


Fig. 5.4 Typical strata log and Borehole sketch (AW5)
(Archbode Engineering Limited, 1984)

of screens and casings used vary slightly from one borehole to another, but the screen locations are generally within the same zone, indicating that all boreholes tap water from the same aquifer. All borehole casings project some 600mm above the ground level, and the annular space between the casings and the wall of the well filled with graded gravel.

Each well was developed using powerful compressors connected to the borehole through pipes which terminate at the screen location. The compressor allows water to surge back and forth through the screen and gravel pack at high velocities removing fine sand and other particles which might hinder the smooth flow of water into the well. This process was continued until clean, sand-free water was obtained. Apart from ensuring sand-free pumping, development operations help to stabilized the aquifer structure near the well and remove mud-cake which might have formed on the face of the hole during drilling.

After development, a submerssible pump was installed and the well is now ready for testing. The level of installation of pump in all wells was 84m below ground level, and the static water level before pumping began ranged from 40.22 to 48.66m below the top of casing. A summary of construction and pumping Test information for the 16 boreholes is given in Table 5.1 .

5.1.2 Pumping Test Measurements and Analysis

Pumping tests were carried out on the boreholes for two main reasons: (1) To determine the hydraulic characteristics

Table 5.1: Summary of Borehole Construction and Pumping Test Information

Borehole No.	Depth below ground (m)	Total Casing Length (m)	Total Screened Length (m)	Pump Installation level (m) below ground	Static Water level (m) below ground	Final Water level (m.b.g)	Total Drawdown (m)	Maximum Discharge (m^3/hr)	Duration of Pumping (hrs)
TW	130	111	19	84.0m	45.41	56.48	11.07	75	71 hrs
1	130	120	10	"	45.40	75.62	30.22	60	
2	120	105	15	"	46.47	65.89	19.42	84	
3	120	102	18	"	48.50	67.87	19.37	84	49
4	130	118	12	"	43.70	59.10	15.40	80	
5	124	106	18	"	43.62	62.41	18.79	70	$65\frac{1}{2}$
6	130	112	18	"	45.92	63.96	18.04	64	54
7	130	112	18	"	44.42	63.32	21.40	78	
8	136	118	18	"	46.37	64.68	18.31	87	
9	130	112	18	"	47.25	64.56	17.31	80	
10	120	102	18	"	47.50	67.66	20.16	86	
11	130	112	18	"	40.22	62.37	22.15	84	$49\frac{1}{2}$
12	130	112	18	"	48.66	62.15	13.49	79	48
13	130	112	18	"	41.30	65.51	23.71	80	
14	128	110	18	"	43.45	73.88	30.43	78	50
15	130	112	18	"	42.66	62.23	19.57	60	

m.b.g. = Metres below ground level.

Source: (Archbode Engineering Limited, 1984).

of the aquifer being tapped, and (2) To obtain information about the yield of each well.

Each well was tested as it was being completed, with the static water level before the start of pumping, measured and recorded. During pumping, measurements of the water level were taken with time. Initially, readings were taken at very short time interval, but the time interval was gradually increased as pumping progressed. This was in recognition of the fact that the water level in the pumped well changes rapidly during the first one or two hours of the test, after which they become gradual. Water level measurements were taken using a hand - operated water level indicator, which basically consists of a water-sensitive probe to which a tape is attached. The probe is lowered into the borehole casing, and once the water level is reached, an indicator light comes on, with a continuous hissing sound. The tape is then read. Time measurements were noted using a stop watch. Abstraction rate was measured using a flow-meter coupled to the discharge pipe. This was sometimes verified by measuring the time required to fill a drum or container of known capacity.

5.1.3 Drawdown Tests: (Step Drawdown/Constant Discharge Test)

The fall in water level within the well was measured with time as pumping proceeded. In all cases, the abstraction rate was increased in three steps, with the first two steps being of relatively short durations of about 30 to 45 minutes each, and the third step lasting for as long as 48 hours to 72 hours after which the pump was shut down.

The nature of measurements made it difficult to directly use the Theis' "type curve" method of analysis. Rather, the Cooper-Jacob's method which is based on Theis solution was used. Recalling Theis forumla,

$$s = \frac{Q}{4\pi kD} W(u) = \frac{Q}{4\pi kD}\left(-0.5772 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \ldots\right) \ldots 5.1$$

where $u = \frac{r^2 S}{4kDt}$ 5.2

From equation 5.2, it can be seen that u decreases as the time of pumping increases. Thus, for large values of, t and small values of r, the terms after $\log_e u$ in the series of equation 5.1 become negligible. Therefore, for small values of u ($u < 0.01$) the drawdown can be expressed as

$$s = \frac{Q}{4\pi kD}\left[-0.5772 - \log_e \frac{r^2 S}{4kDt}\right] \quad \ldots\ldots 5.3$$

Rewriting equation 5.3 and changing into decimal logarithmic gives

$$s = \frac{2.30Q}{4\pi kD} \log \frac{2.25\, kDt}{r^2 S} \quad \ldots\ldots\ldots 5.4$$

A plot of drawdown s versus the logarithm of time t, will give a straight line. At the intercept with the time-axis, s=0 and $t = t_o$.

$$\therefore \quad 0 = \frac{2.30Q}{4\pi KD} \log \frac{2.25KD}{r^2S} t_o$$

Since $\frac{2.30Q}{4\pi KD} \neq 0$, it follows that $\log \frac{2.25KDt_o}{r^2S} = 0$

i.e $\frac{2.25KD}{r^2S} t_o = 1$

or

$$S = \frac{2.25KD}{r^2} t_o \qquad \ldots\ldots 5.5$$

Substituting S, into equation 5.4, gives

$$s = \frac{2.30Q}{4\pi KD} \log t/t_o \qquad \ldots\ldots 5.6$$

For $t/t_o = 10$, $\log t/t_o = 1$.

Thus, s in equation 5.6 can be replaced by Δs, i.e. the drawdown difference per log cycle of time, and it follows that

$$\Delta s = 2.30Q/4\pi KD$$

or $$KD = \frac{2.30Q}{4\pi\Delta s} \qquad \ldots\ldots 5.7$$

In applying this method to step drawdown test, where discharge is not constant, drawdown measurements are converted to specific drawdown s/Q, i.e drawdown per discharge, and time measurements t, are converted to the weighted mean time $\bar{t}$, which represents the time at which an observed drawdown would have occured if pumping had been at a contant discharge rate equal to the actual discharge at time t.

Specific drawdown s/Q is then plotted against weighted meantime $\bar{t}$ on a semi-log graph paper (with $\bar{t}$ on the log-axis), the best straight line through the points is drawn. Transmissivity is then calculated from the equation (Cooper and Jacob, 1946)

$$T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)} \qquad \ldots\ldots 5.8$$

where,

$\Delta(s/Q)$ is the change in specific drawdown per log-cycle of $\bar{t}$ and is read from the graph.

Calculations illustrating the application of this method are given in section 5.1.6.

5.1.4 Recovery Tests

After the pump has been shut down, the rise in water level is continually measured with time until the original position is reached, or nearly reached, thus forming a recovery test. The recovery test data obtained are used to calculate transmissivity using Theis recovery method, thus giving a check on the results from analysis of the drawdown pumping test data.

The Theis recovery method can be used to calculate the hydraulic properties of an aquifer if the assumptions and conditions of the Jacob method are satistied.

According to Theis (1935), the residual drawdown, s'' , during the recovery period is

$$s'' = \frac{Q}{4\pi KD}\left[\log_e \frac{4KDt}{r^2S} - \log_e \frac{4KDt''}{r^2S''}\right] \qquad \ldots\ldots\ldots 5.9$$

where

s" = Residual drawdown i.e the difference between the original water level prior to pumping and the actual water level measured at a certain time t" since pumping stopped (m).

r = Distance in metres from pumped well to observation well

S" = Coefficient of storage during recovery (dimensionless)

S = Coefficient of storage during pumping (")

t = Time in minutes (or days) since pumping started

t" = " " " (or days) " " stopped

Q = Rate of recharge = Rate of discharge (m^3/day)

When S and S" are constant and equal, and $u = \frac{r^2 S}{4\,KDt"}$ is sufficiently small, equation 5.9 can be written as

$$s" = \frac{2.30\,Q}{4\,\pi\,KD} \log t/t" \qquad \ldots\ldots 5.10$$

A plot of s" versus t/t" on a semi-log graph paper (t/t" on the logarithmic scale) gives a straight line with a slope equal to 2.30Q/4 π KD. The value of the residual drawdown difference, $\Delta s"$, per log cycle of t/t" can be read from the graph as

$$\Delta s" = \frac{2.30Q}{4\,\pi\,KD}$$

or

$$KD = \frac{2.30\,Q}{4\pi\,\Delta s"} \qquad \ldots\ldots\ldots 5.11$$

No value of S can be obtained by this method. To apply the Theis recovery method to step drawdown test, the constant discharge rate Q, is replaced by the weighted mean discharge rate Q" given by,

$$Q" = \frac{\sum Q_i \cdot \Delta t_i}{\sum \Delta t_i} \qquad \ldots\ldots\ldots 5.12$$

where,

Q'' = Weighted mean of discharge rate (m^3/d)

Q_i = Discharge rate for i^{th} step (m^3/d)

t_i = Duration of pumping for i^{th} step (d)

Equation 5.11 then becomes

$$T = KD \frac{=2.30\ Q''}{4\pi\ \Delta s''} \qquad \ldots\ldots\ldots\ 5.13$$

$\Delta s''$ is the residual drawdown difference per log cycle of t/t''

Calculations illustrating the application of this method are given in section 5.1.6.

5.1.5 Observation Wells

In the two tests described above, water level measurements were taken in the pumped well only. Such data, while useful in providing information about the performance and yield of each well, they do not allow for accurate determination of the hydraulic characteristic of the equifer. In particular, the storage coefficient of the aquifer cannot be determined from abstraction well measurements. Observation wells are therfore necessary.

A test was carried out in which one of the wells, borehole no 5, was pumped, while water level changes in two observation boreholes, BH nos. 3 and 4, at 100m and 200m away respectively were monitored.

Data from the two observation wells were analysed using the Cooper-Jacob method, and the transmissivity and

storage coefficient of the aquifer were calculated using the equations below:

Transmissivity: As derived earlier in equation 5.8 transmissivity T, is given by

$$T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$$

Storage Coefficient S: The storage coefficient is calculated from equation 5.5

$$S = \frac{2.25\ KD\ t_o}{r^2}$$

where,

t_o = intercept of the straight line plot with the time-axis

r = distance of observation well from the pumped well.

Illustrative calculations for observation wells nos 3 and 4, using this method are given in the next section.

5.1.6 Calculations

(1) Test Borehole TW:

a) Cooper-Jacobs' step Drawdown Method:

From semi-log graph of specific drawdown, s/Q versus weighted time $\bar{t}$ (with $\bar{t}$ on the log-axis), Figure 5.5a

$\Delta(s/Q)$ over 1 log-cycle = 1.602 min/m^2

$\therefore$ Transmissivity, $T = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$

$= \frac{2.30}{4\pi} \times \frac{1}{1.602} = 0.1143\ m^2/min \times 60 \times 24$

$= 164.6\ m^2/day.$

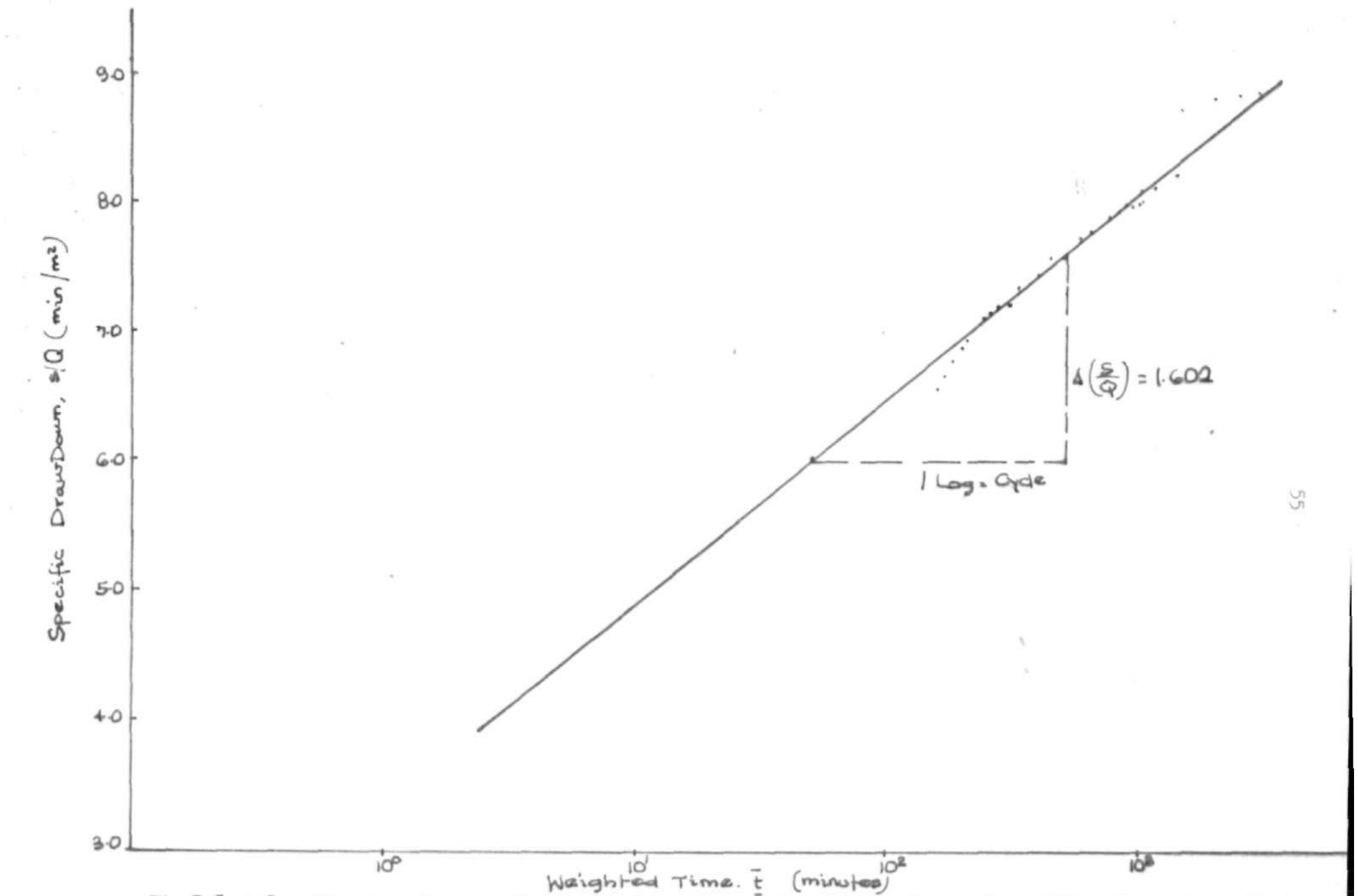


Fig. 5.5 a: Specific drawdown, s/Q Vs weighted time $\bar{t}$ for Abstraction well no TW - Drawdown Test

b) Theis Recovery Method:

Transmissivity, $T = KD = \frac{2.30\ Q''}{4\pi \Delta s''}$

Q'' = Weighted mean discharge rate = $\frac{\sum Q_i \Delta t_i}{\sum \Delta t_i}$

$= \frac{(53.0x40+70.6x40+91.1x40+75x4115)/60}{(4235)/60}$

$= 74.9\ m^3/hr$

From Figure 5.5b

The residual drawdown difference $\Delta s''$, per log cycle of t/t'' is 1.75

$\therefore T = KD = \frac{2.30 \times 74.9}{4\pi \times 1.75} \times 24\ m^2/day$

$= 188.0\ m^2/day.$

(2) Borehole No 3:

a) Cooper-Jacobs' step Drawdown Method:

Transmissivity, $T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$

From Figure 5.6a of specific drawdown s/Q, Vs Weighted time $\bar{t}$ graph, we have;

$\Delta(s/Q)$ over 1 - log cycle of $\bar{t}$, $= 1.64\ min/m^2$

$\therefore T = KD = \frac{2.30}{4\pi} \times \frac{1}{1.64} \times 60 \times 24\ m^2/day$

$= 160.71\ m^2/day.$

b) Theis Recovery Method:

Transmissivity, $T = KD = \frac{2.30\ Q''}{4\pi \Delta s''}$

Q'' = Weighted mean discharge rate = $\frac{\sum Q_i \Delta t_i}{\sum \Delta t_i}$

$= \frac{(24.5 \times 30 \times 68 \times 30 + 84.6 \times 2880)/60}{(2940)/60}$

$= 83.8\ m^3/hr.$

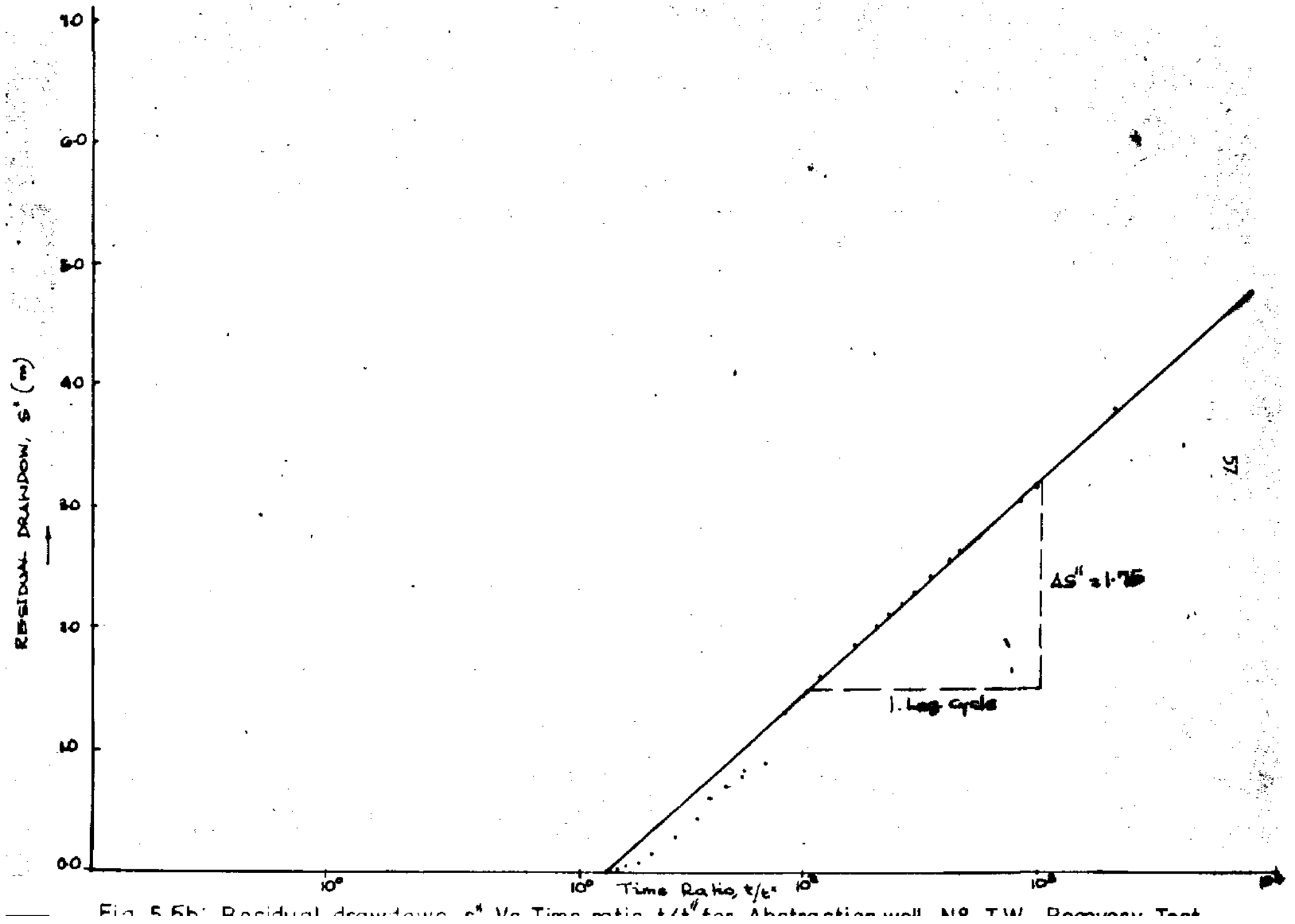


Fig. 5.5b: Residual drawdown, s' Vs Time ratio, t/t' for Abstraction well N8, T.W. Recovery Test

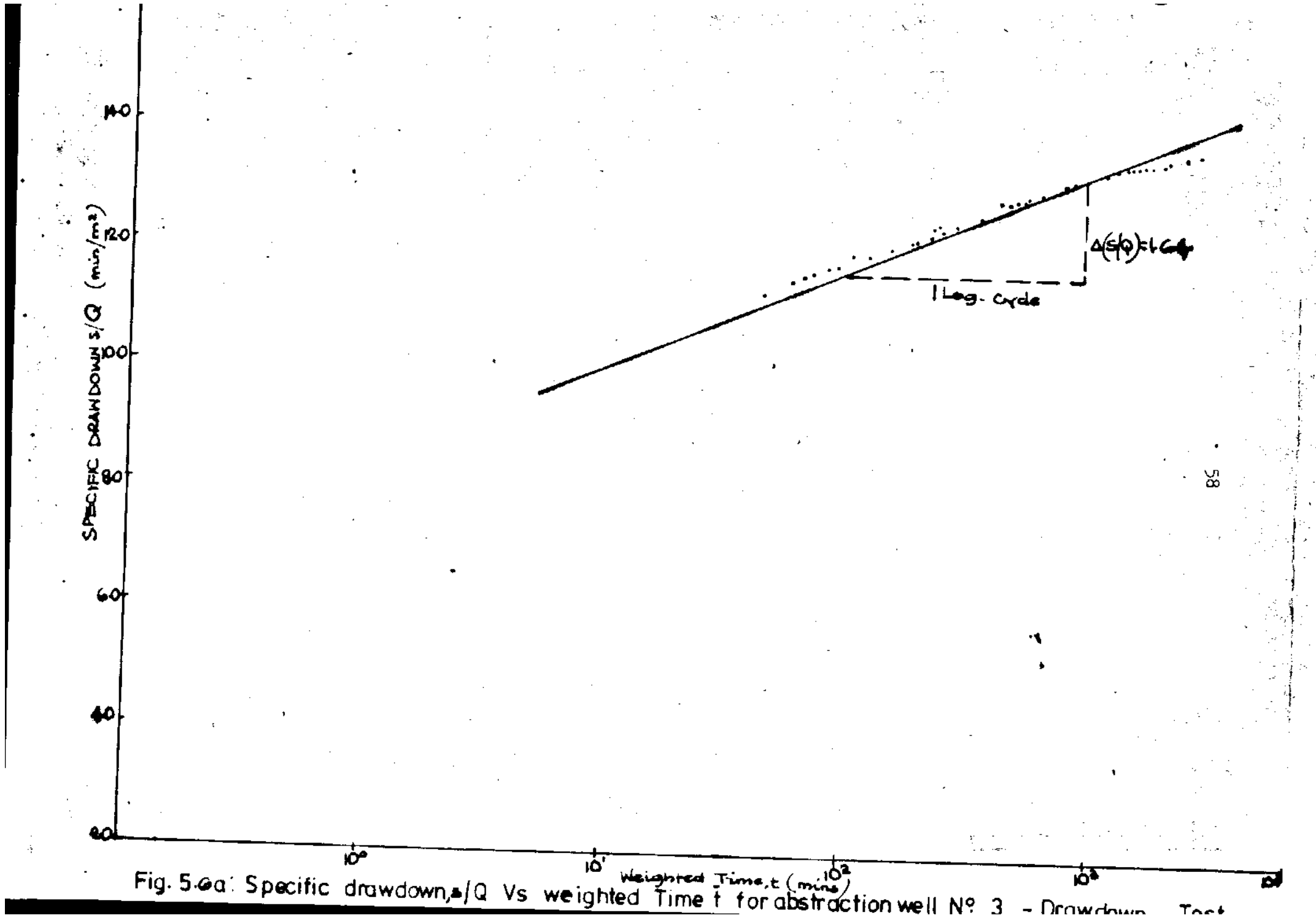


Fig. 5.6a: Specific drawdown, s/Q Vs weighted Time t for abstraction well No. 3 - Drawdown Test

From Figure 5.6b, $\Delta s''$ over 1 log-cycle of t/t'' = 2.36m

$$\therefore T = KD = \frac{2.30 \times 83.8}{4\pi \times 2.36} \times 24$$

$$= 156.01 \ m^2/day$$

Borehole No 4:

Pumping test data for borehole no. 4, were not available, however, the drilling contractor's record showed a transmissivity T value of 249.7 m^2/d, for the step drawdown test. There is no T value for recovery test.

(3) Borehole No. 5:

(i) Abstraction well no AW 5:

a) Cooper-Jacob's step Drawdown Method:

$$\text{Transmissivity } T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$$

From Figure 5.7a of, s/Q Vs $\bar{t}$ graph, we have

$\Delta(s/Q)$ over 1 log cycle of $\bar{t}$, = 0.8 mm/m^2

$$\therefore T = KD = \frac{2.30}{4\pi} \times \frac{1}{0.8} \times 60 \times 24$$

$$= 329.5 \ m^2/day.$$

b) Theis Recovery Method:

$$\text{Transmissivity, } T = KD = \frac{2.30\ Q''}{4\pi \Delta s''}$$

Q'' = Weighted mean discharge rate

$$= \frac{(61 \times 30 + 65 \times 30 + 69.9 \times 3870)/60}{(3930)/60}$$

$$= 69.8 \ m^3/hr$$

From Figure 5.7b, $\Delta s''$ over 1 log cycle of t/t'', = 1.2m

$$\therefore T = KD = \frac{2.30 \times 69.8}{4\pi \times 1.2} \times 24$$

$$= 255.5 \ m^2/day$$

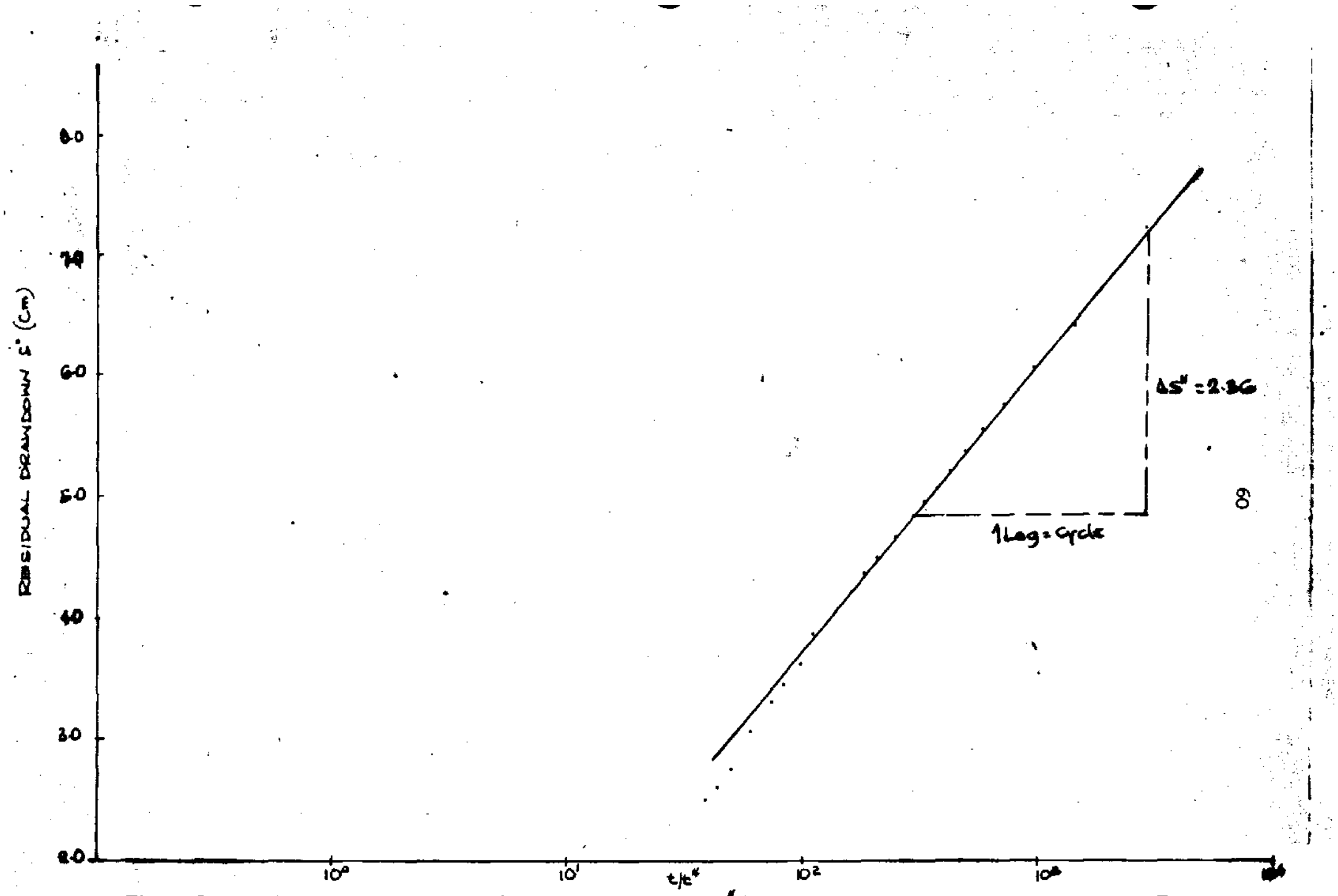


Fig. 5·6b Residual drawn down s' Vs Time ratio, t/t' for Abstraction well Nº 3 - Recovery Test

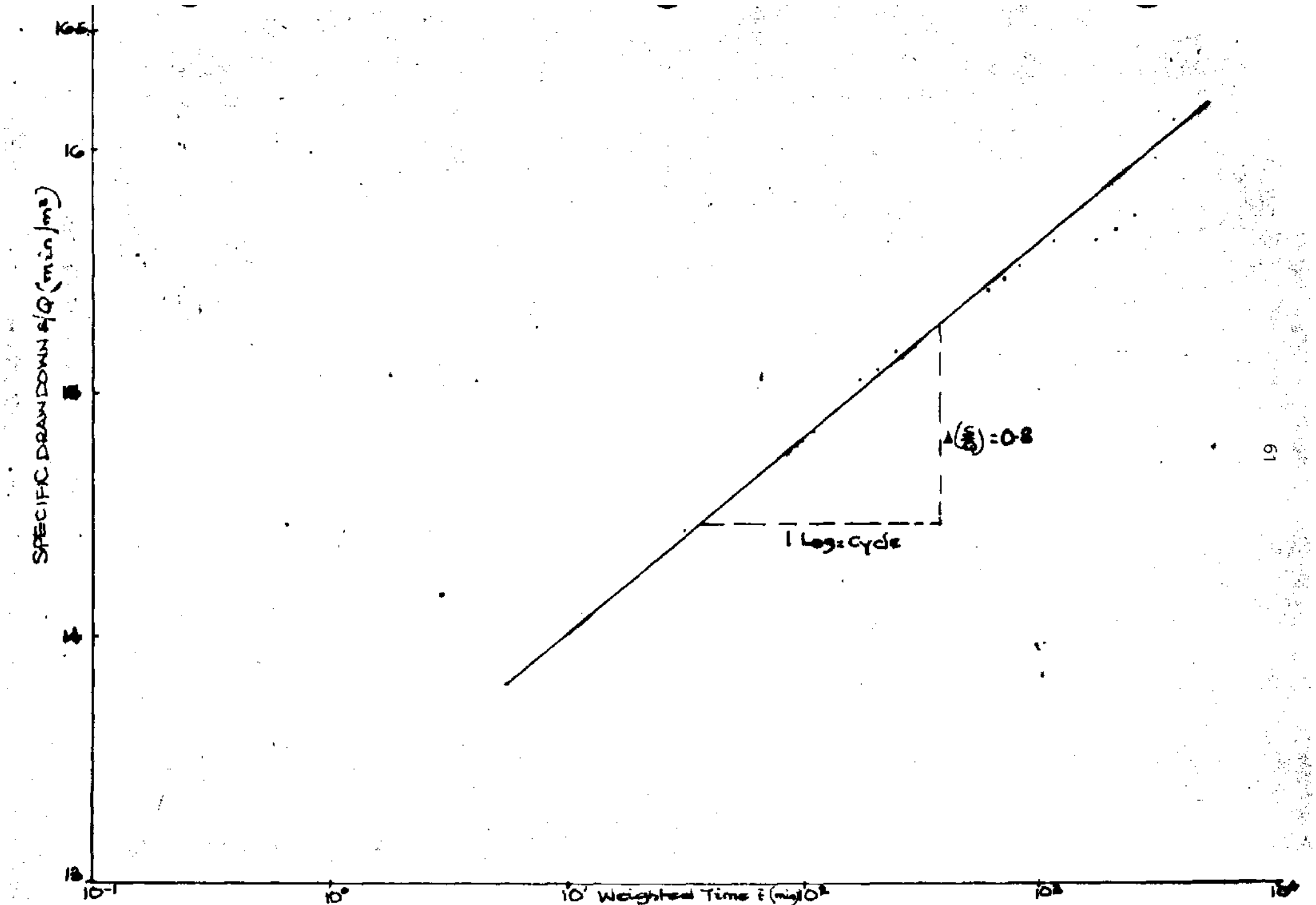


Fig. 5.7a: Specific drawdown s/Q Vs weighted time t̄ for Abstraction well No 5 - ...drawn Test

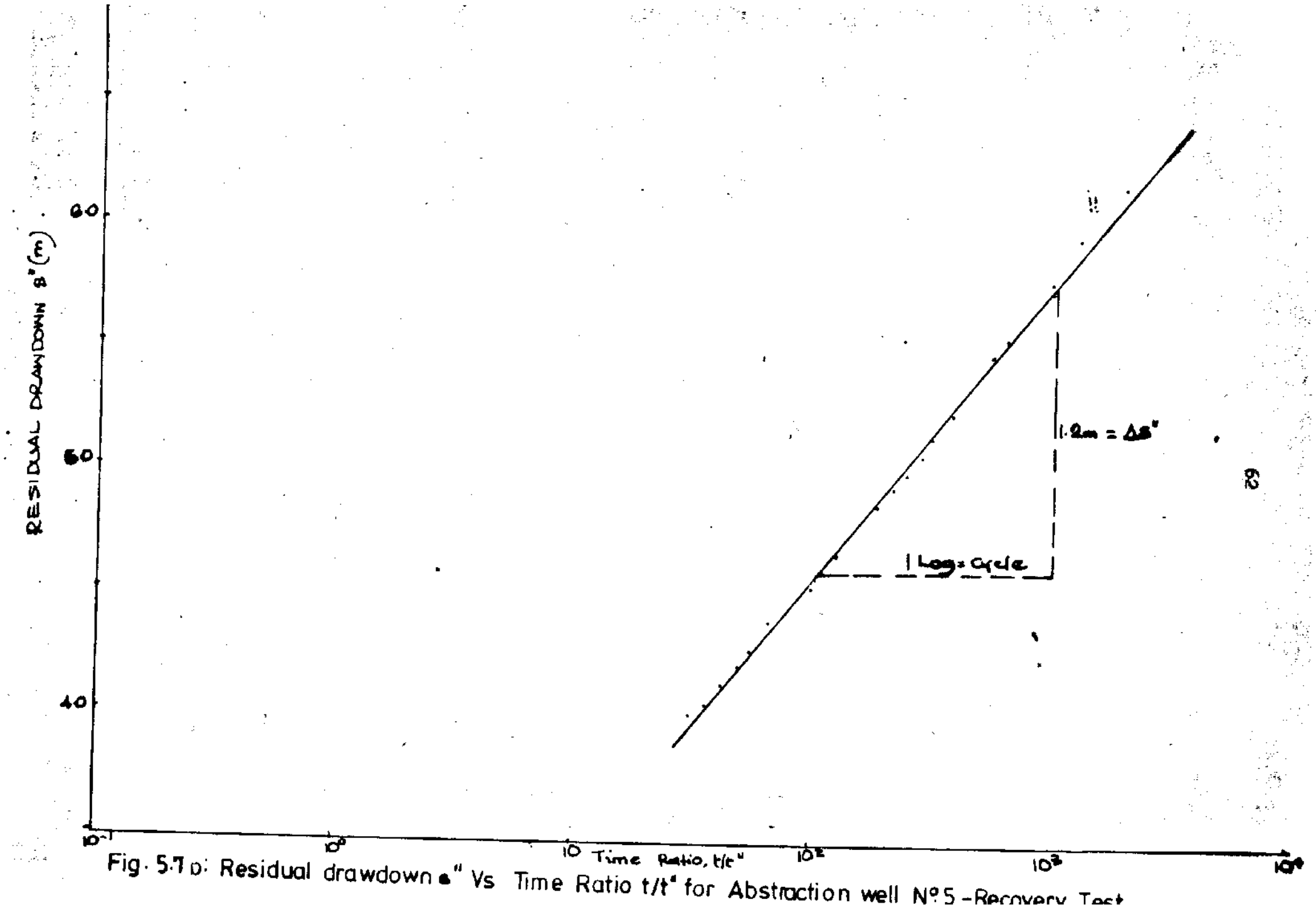


Fig. 5.7 b: Residual drawdown s" Vs Time Ratio t/t" for Abstraction well N°5 - Recovery Test

(3) (ii) Observation Well no OW3: - 100m away from AW5

Cooper - Jacobs' step Drawdown Method:

a) Transmissivity, $T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$

From Figure 5.7c of, s/Q Vs $\bar{t}$ graph, we have,

$\Delta(s/Q)$ over 1 log cycle of $\bar{t}$, = 1.075 min/m^2

$\therefore T = KD = \frac{2.30}{4\pi} \times \frac{1}{1.075} \times 60 \times 24$

$= 245.2\ m^2/day$

b) Storage Coefficient S:

$$S = \frac{2.25\ T\ t_o}{r^2}$$

From Figure 5.7c, t_o = intercept in time axis of s/Q Vs $\bar{t}$ plot = 5.0min.

Also, r = distance of observation well, OW3, from abstraction well AW5 = 100m.

$\therefore S = \frac{2.25 \times 245.2 \times 5.0}{100^2} \times \frac{1}{60 \times 24}$

$= 0.00019$

Check: $u = \frac{r^2 S}{4Tt} < 0.01$

$= \frac{100^2 \times 0.00019}{4 \times 245.2 \times 3917} \times 60 \times 24 = 0.00072$

0.00072 < 0.01,

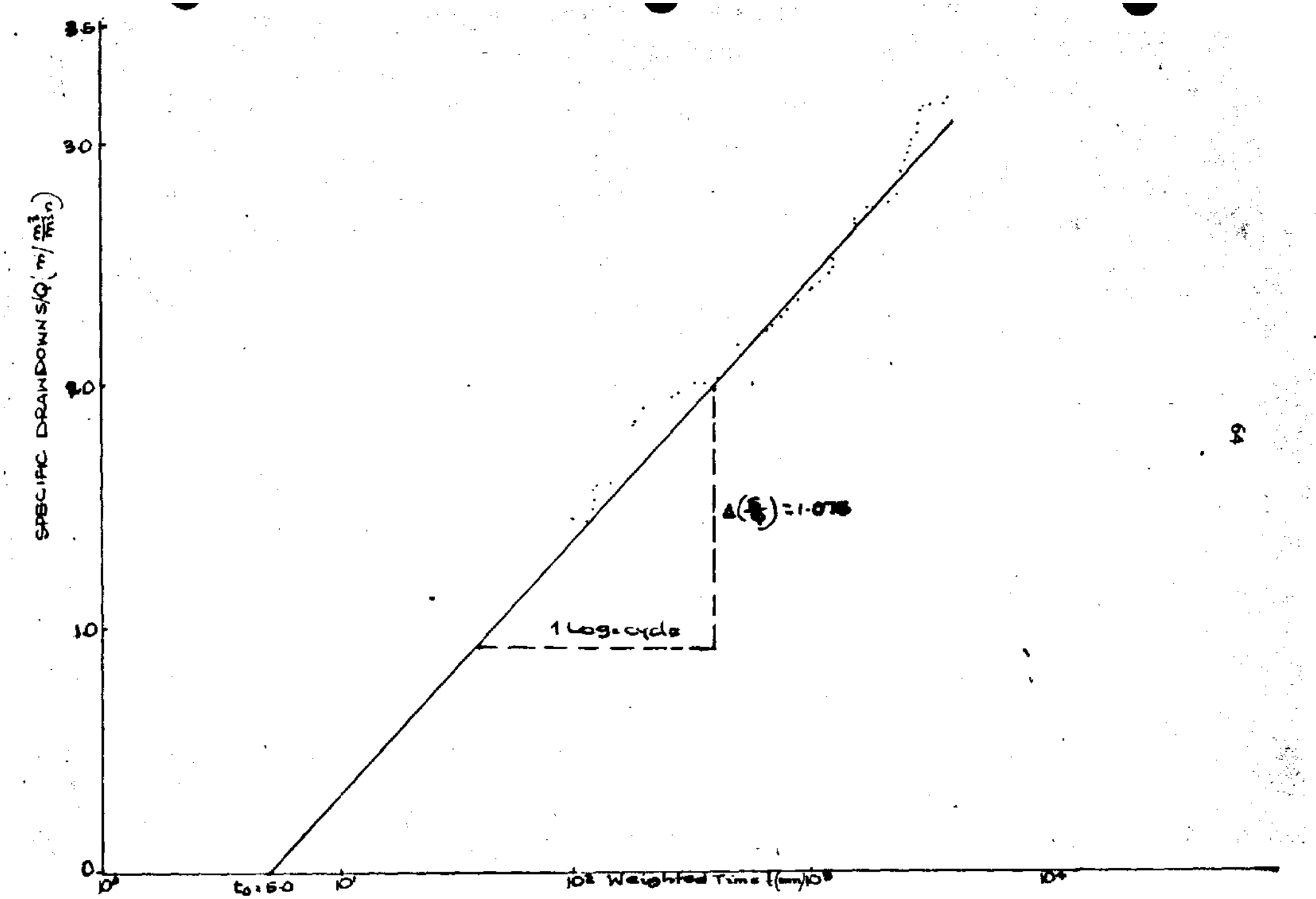


Fig. 5.7c: Specific drawdown, s/Q Vs weighted Time $\bar{t}$ for observation well N° 3 - Drawdown Test

(3) (iii) Observation Well no OW4 - 200m away from AW no 5

Cooper - Jacob's step Drawdown Method:

a) Transmissivity, $T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$

From Figure 5.7d of s/Q Vs $\bar{t}$ graph, we have

$\Delta(s/Q)$ over 1 log-cycle of t, = 1.077

$$\therefore \quad T = KD = \frac{2.30}{4\pi} \times \frac{1}{1.077} \times 60 \times 24$$

$$= 244.7 \ m^2/day$$

b) Storage Coefficient S:

$$S = \frac{2.25 \ T \ t_o}{r^2}$$

From Figure 5.7d; t_o = intercept on time axis of s/Q Vs $\bar{t}$ plot = 16.6 min.

Also, r = distance of observation well, OW4, from abstraction well AW5 = 200m

$$\therefore \quad S = \frac{2.25 \times 244.7 \times 16.6}{200^2} \times \frac{1}{60 \times 24}$$

$$= 0.00016$$

Check : $u = \frac{r^2 S}{4Tt} < 0.01$

$$= \frac{200^2 \times 0.00016}{4 \times 244.7 \times 3924} \times 60 \times 24 = 0.0024$$

$$= 0.0024 < 0.01$$

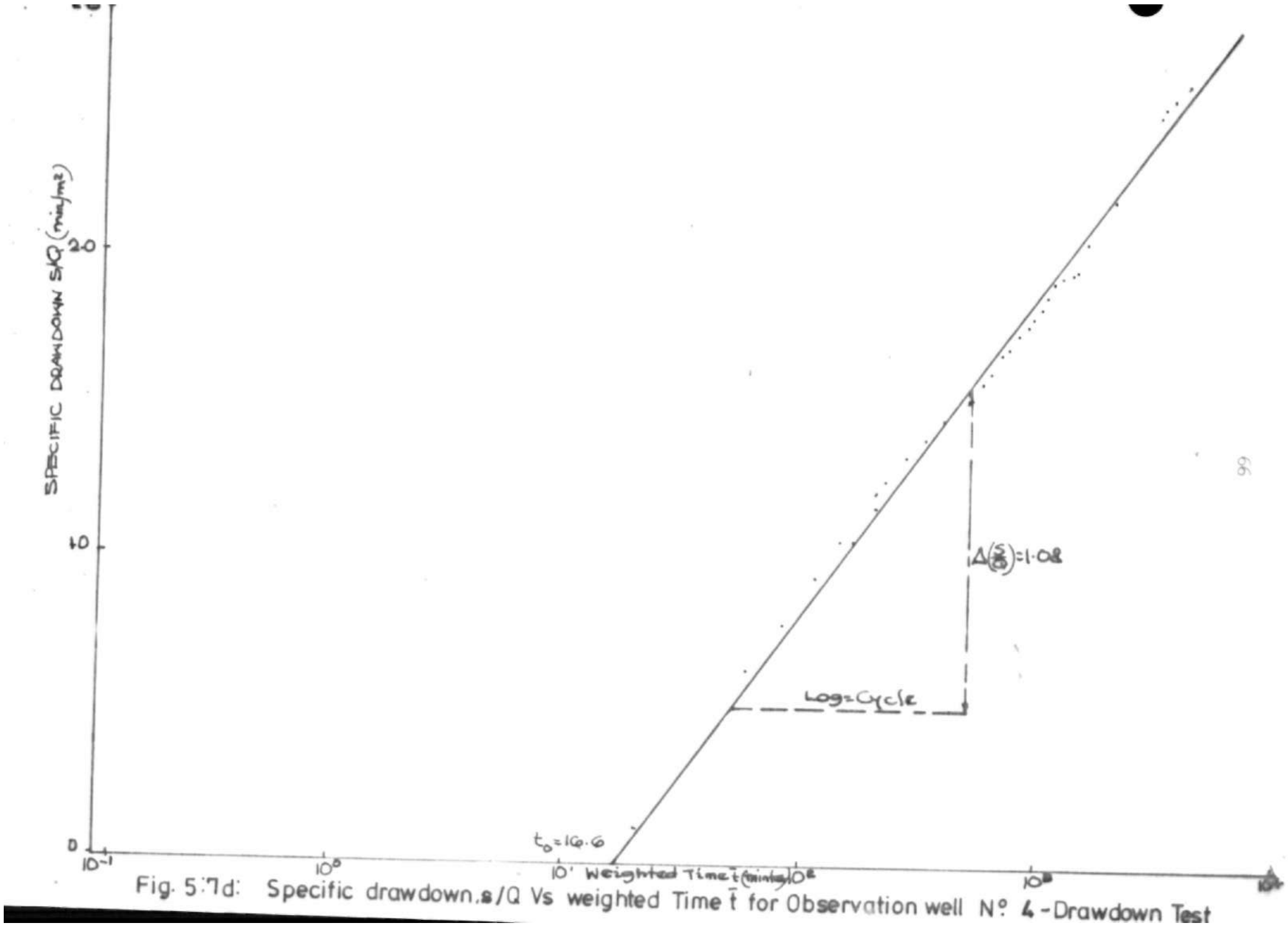


Fig. 5:7d: Specific drawdown, s/Q Vs weighted Time $\bar{t}$ for Observation well N° 4 - Drawdown Test

(4) Borehole No 6:

(a) Cooper-Jacobs' Step Drawdown Method:

$$\text{Transmissivity, } T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$$

From Figure 5.8a of, s/Q Vs $\bar{t}$ graph we have:

$\Delta(s/Q)$over 1 log-cycle of $\bar{t}$, = 2.0 mm/m^2

$$\therefore \quad T = KD = \frac{2.30}{4\pi} \times \frac{1}{2.0} \times 60 \times 24 \text{ m}^2/\text{day}$$

$$= 131.8 \text{ m}^2/\text{day}.$$

(b) Theis Recover Method:

$$\text{Transmissivity, } T = KD = \frac{2.30\ Q''}{4\pi\ \Delta s''}$$

Q" = Weighted mean discharge rate

$$= \frac{(61 \times 45 + 69.6 \times 45 + 83.9 \times 3140)}{3230}$$

$$= 83.4 \text{ m}^3/\text{hr}$$

From graph 5.8b, $\Delta s''$ over 1 log cycle of t/t", = 2.28m

$$\therefore \quad T = KD = \frac{2.30 \times 83.4}{4\pi \times 2.28} \times 24$$

$$= 160.7 \text{ m}^2/\text{day}.$$

(5) Borehole No 11:

(a) Cooper-Jacobs' Step Drawdown Method:

$$\text{Transmissivity, } T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$$

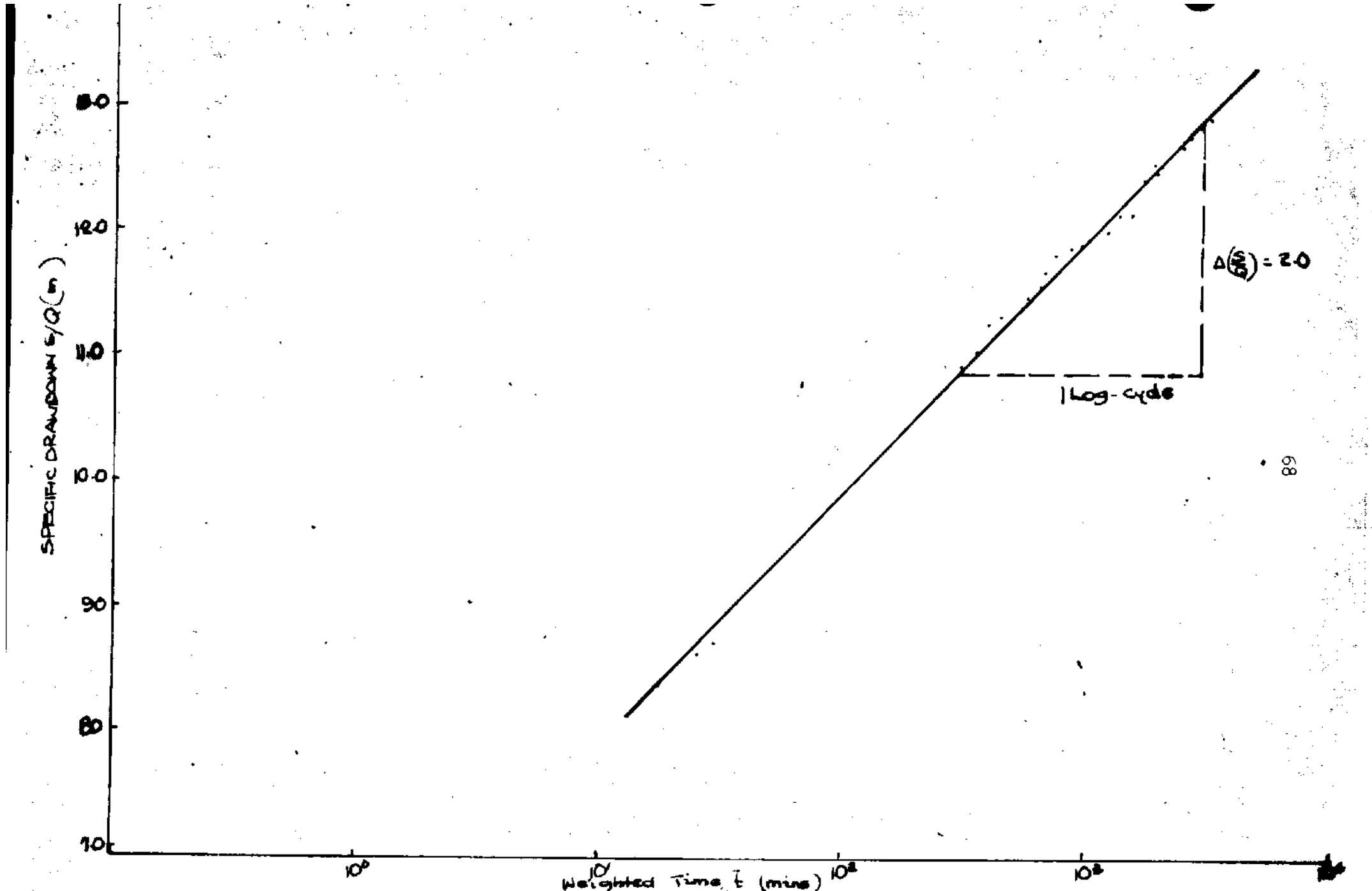


Fig. 5.8a Specific drawdown s/Q Vs weighted Time $\bar{t}$ for Abstraction well Nº 6 - Drawdown Test

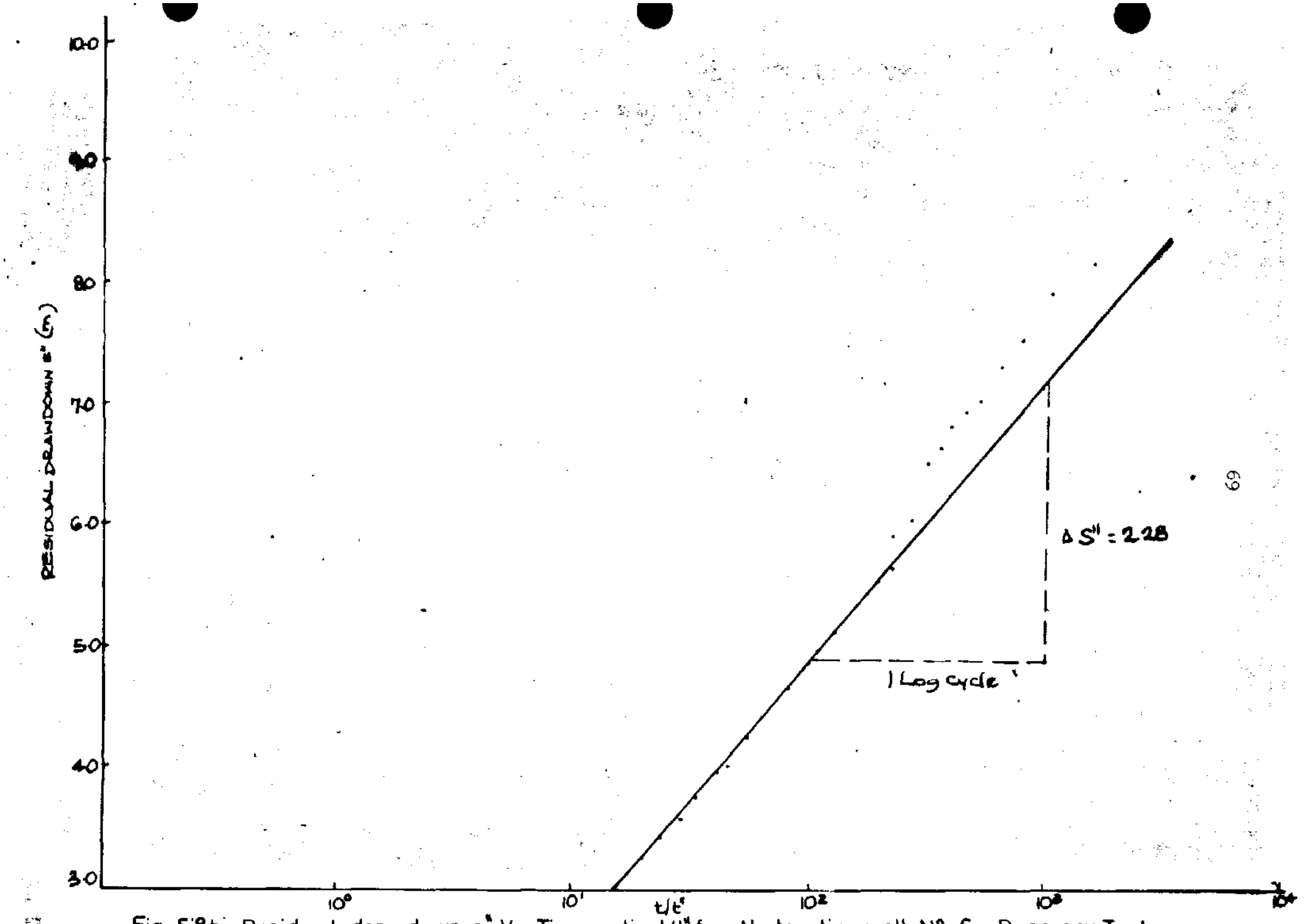


Fig. 5:8b: Residual drawdown, s" Vs Time ratio t/t' for Abstraction well Nº 6 – Recovery Test

From Figure 5.9a of, s/Q Vs $\bar{t}$ graph, we have

Δ(s/Q) over 1 log cycle of t = 3.5 min/m^2

$$\therefore \quad T = KD = \frac{2.30}{4\pi} \times \frac{1}{3.5} \times 60 \times 24$$

$$= 75.3 \ m^2/day.$$

(b) Theis Recovery Method:

$$\text{Transmissivity, } T = KD = \frac{2.30\ Q''}{4\pi\ \Delta s''}$$

$$Q'' = \text{Weighted mean discharge rate} = \frac{\Sigma Q_i\ \Delta t_i}{\Sigma_i\ \Delta t_i}$$

$$= \frac{(54.5 \times 30 + 64.3 \times 30 + 83.7 \times 2910)}{2970}$$

$$= 83.2 \ m^3/hr$$

From graph 5.9b, $\Delta s''$ over 1 log cycle of t/t", = 4.32m

$$\therefore \quad T = KD = \frac{2.30 \times 83.2}{4\pi \times 4.32} \times 24 \ m^2/d$$

$$= 84.6 \ m^2/day$$

(6) Borehole No 12:

(a) Cooper-Jacob's Step Drawdown Method:

$$\text{Transmissivity, } T = KD = \frac{2.30}{4\pi} \times \frac{1}{\Delta(s/Q)}$$

From Figure 5.10a of s/Q Vs $\bar{t}$ graph we have that Δ(s/Q) over 1 log-cycle of $\bar{t}$, = 1.55 min/m^2

$$\therefore \quad T = KD = \frac{2.3}{4\pi} \times \frac{1}{1.55} \times 60 \times 24$$

$$= 170.0 \ m^2/day$$

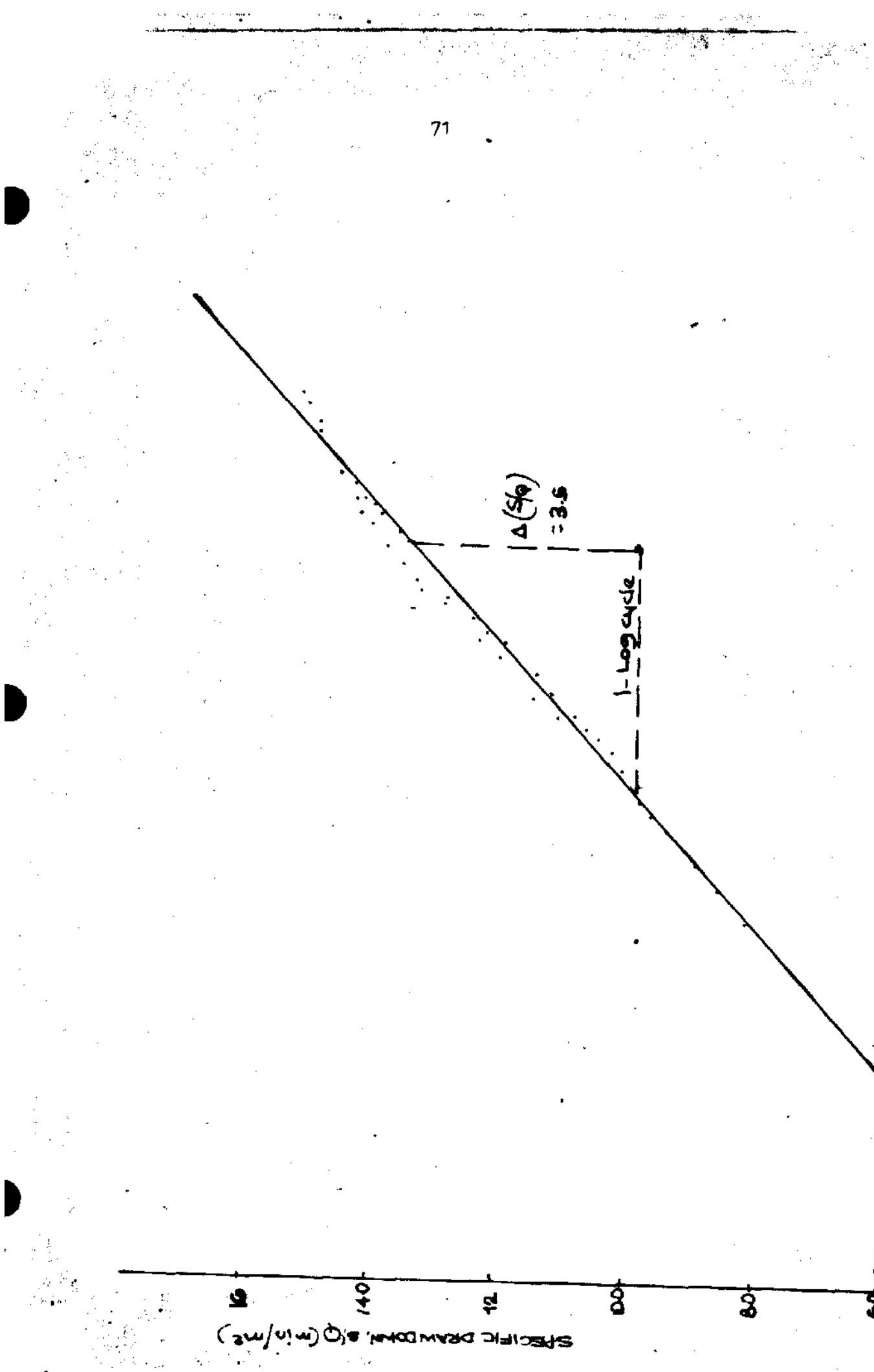


Fig 5.9a: Specific drawdown, s/Q Vs weighted Time t for Abstraction well Nº 11 – Drawdown Test

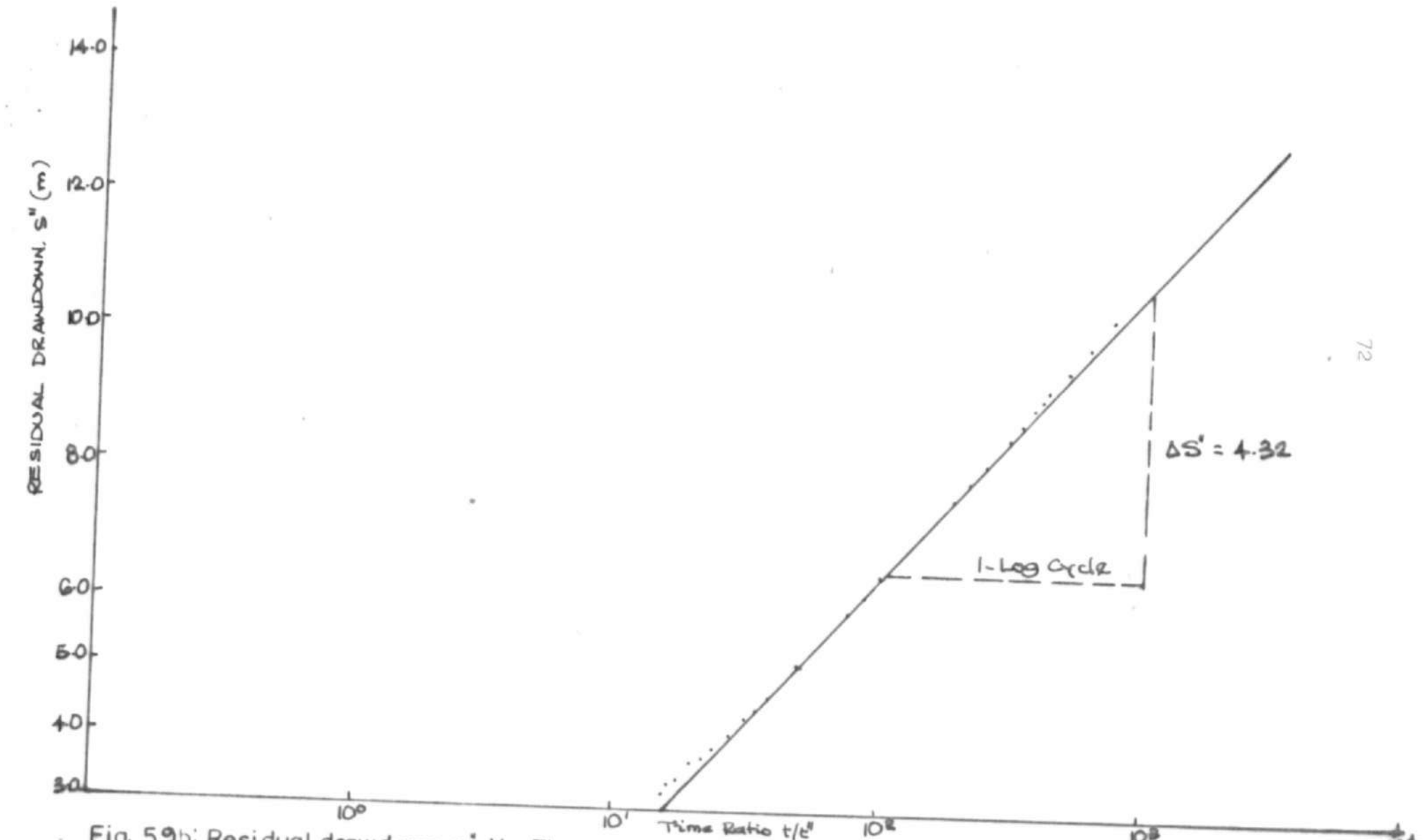


Fig. 5.9b: Residual drawdown s' Vs Time ratio t/t" for Abstraction well Nº 11 - Recovery Test

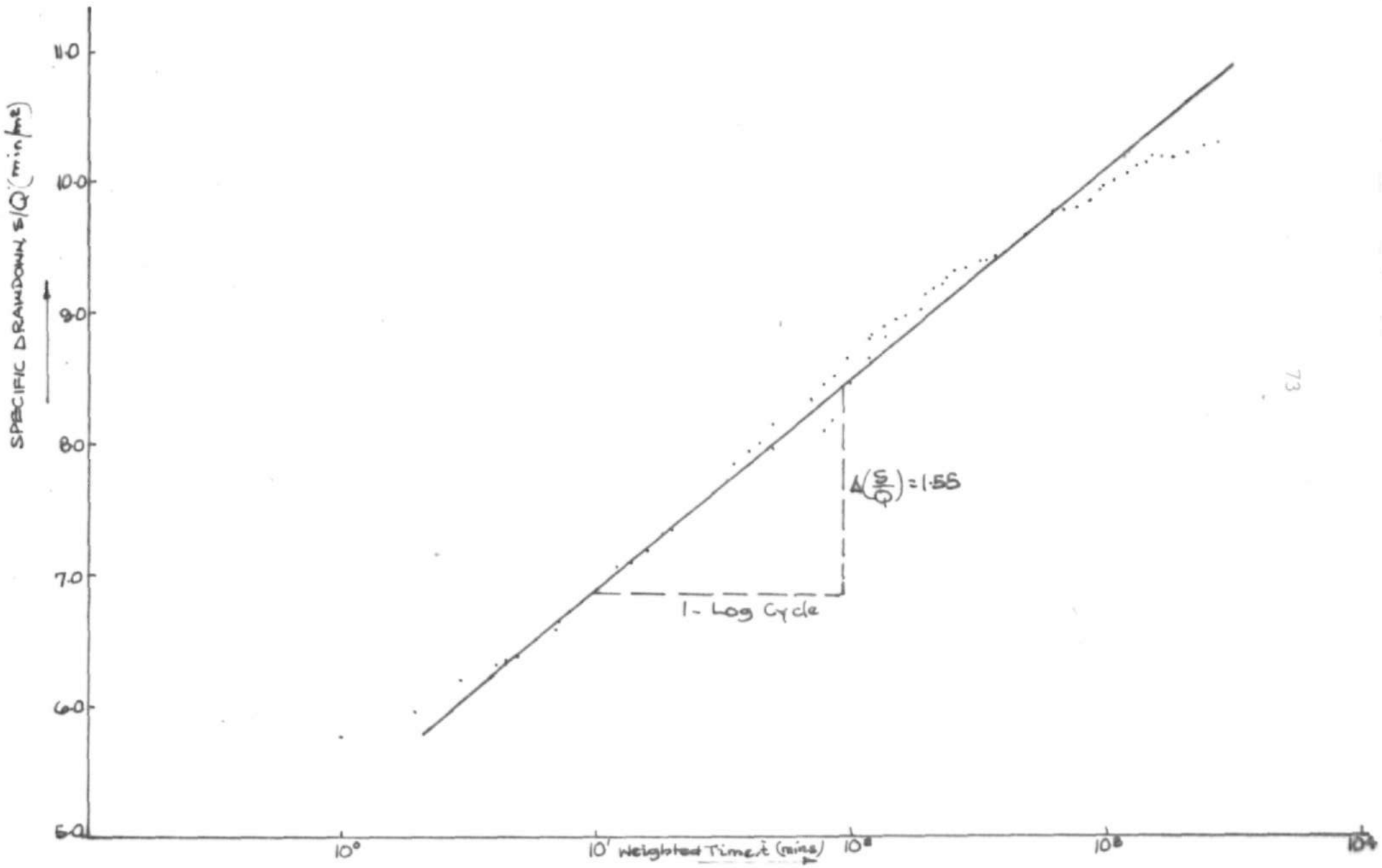


Fig. 5.10g: Specific drawdown, s/Q Vs weighted Time, $\bar{t}$ for Abstraction well N° 12 - Drawdown Test

(b) Theis Recovery Method:

$$\text{Transmissivity, } T = KD = \frac{2.30\ Q''}{4\pi \Delta s''}$$

$$Q'' = (50 \times 30 + 67 \times 30 + 78.7 \times 2820)/2800$$

$$= 78.28\ m^3/hr$$

From Figure 5.10b $\Delta s''$ over 1 log cycle of t/t'', = 1.8.

$$\therefore\ T = KD = \frac{2.30 \times 78.23}{4\pi \times 1.8} \times 24\ m^2/d$$

$$= 190.9\ m2/day.$$

A summary of the T and S values from both abstraction and observation wells are shown in Table 5.2. For boreholes for which pumping test data were not available, the values from the drilling contractor's records were used.

Plots of specific drawdown, s/Q versus weighted time $\bar{t}$; and Residual drawdown, s'', versus Time ratio, t/t'', for boreholes for which pumping test data were available are presented in Figures 5.5a to 5.10b. The pumping test data are given in the appendix IV.

The drawdown in a well due to pumpage consists of the logarthmic drawdown curve at the well face and the well loss caused by flow through the well screen and flow inside the well to the pump intake. Well loss is usually associated with turbulent flow, and is proportional to the n^{th} power of the discharge, Q^n. The Jacob's (1946) method of calculating formation and well losses for the various pumped wells are illustrated in Fig. 5.11 to 5.16. Tables from which the figures were drawn are presented in appendix III.

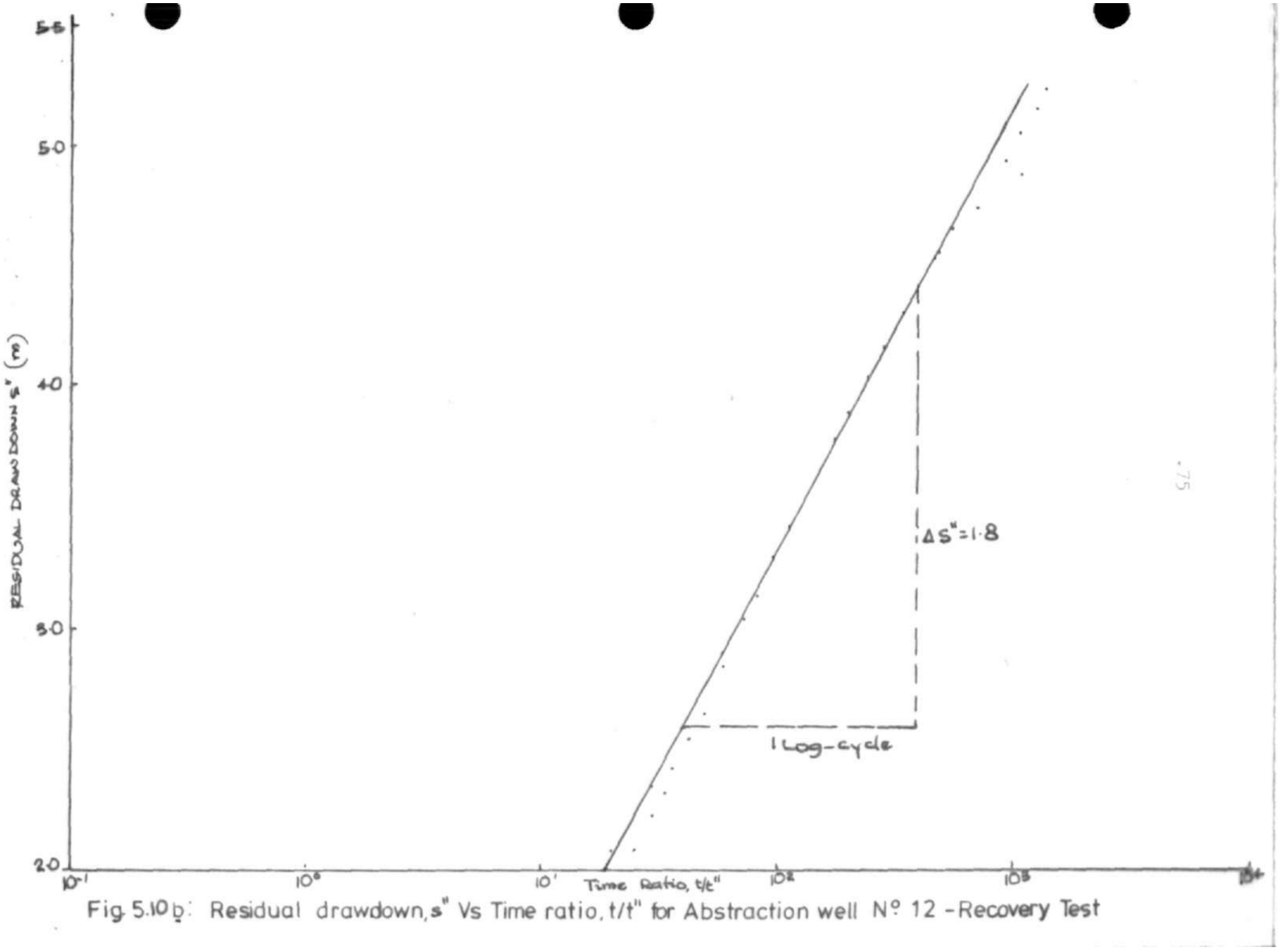


Fig. 5.10b: Residual drawdown, s" Vs Time ratio, t/t" for Abstraction well Nº 12 - Recovery Test

Table 5.2: Summary of T and S values from Abstraction and Observation Wells

Borehole No.	Status		Type of Test		Hydraulic Properties			
	Abstraction Well (AW)	Observation Well (OW)	SD/CD	R	T_1 (m^2/d)	T_2 (m^2/d)	T (m^2/d)	S ($\times 10^{-4}$)
TW	AW		SD/CD	R	164.6	188.0	176.3	
1.	"		"	"	(387.5)	-	(387.5)	
2.	"		"	"	(174.6)	-	(174.6)	
3.	"		"	"	(160.7)	(156.0)	(158.4)	
4.	"		"	"	-	(249.7)	(249.7)	
5.	"		"	R	329.5	255.5	292.5	
3		OW	SD/CD	-	245.2	-	245.2	1.92
4	"	OW	"	-	244.7	-	244.7	1.59
6.	AW		"	R	131.8	160.7	146.3	
7.	"		"	"	(141.0)	-	(141.0)	
8.	"		"	"	(340.)	-	(340.7)	
9.	"		"	"	(234.0)	-	(234.0)	
10.	"		"	"	-	(308.2)	(308.2)	
11.	"		SD/CD	R	75.3	84.6	80.8	
12.	"		"	"	170.0	190.9	180.5	
13.	"		"	"	(155.5)	-	(155.5)	
14.	"		SD/CD	R		(135.5)	(133.5)	
15.	"		"	"	(500.0)	-	(500.0)	
				Average Value			230.6	1.76

N.B:

AW - Abstraction well

OW - Observation Well

SD/CD - Step Drawdown/Constant Discharge Test

R - Recovery Test

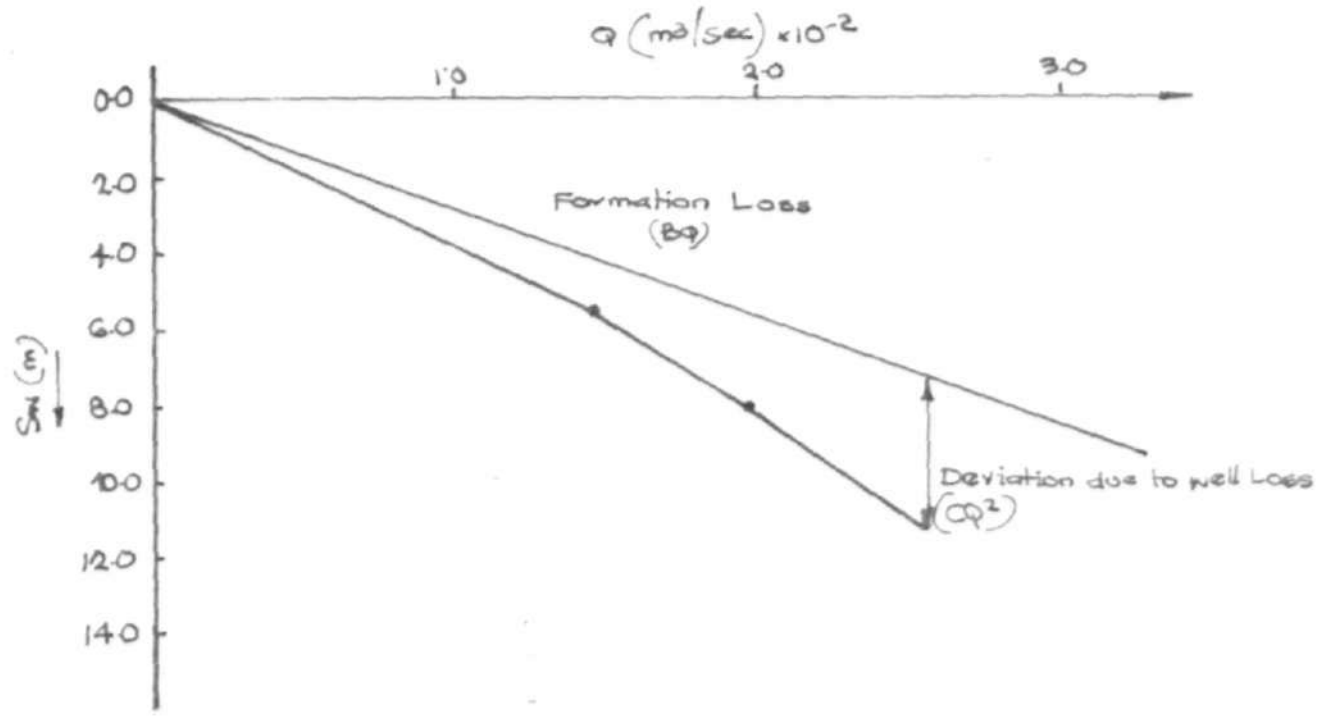
T - Average Transmissivity

S - Storage Coefficient

T_1 - Transmissivity value calculated from Cooper-Jacob method

T_2 - " " " from Theis recovery method

( ) - Value obtained from driller's record.



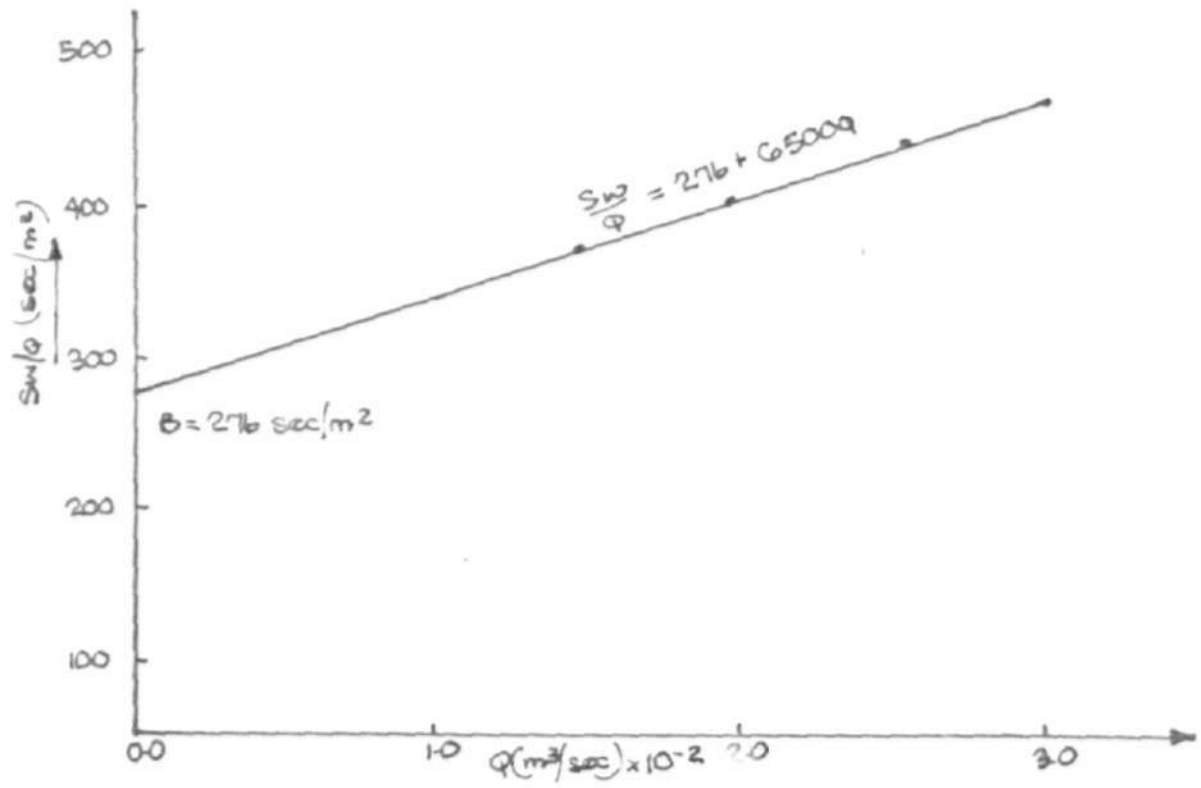


Fig. 5.11: Formation and Well losses for Abstraction well Nº TW

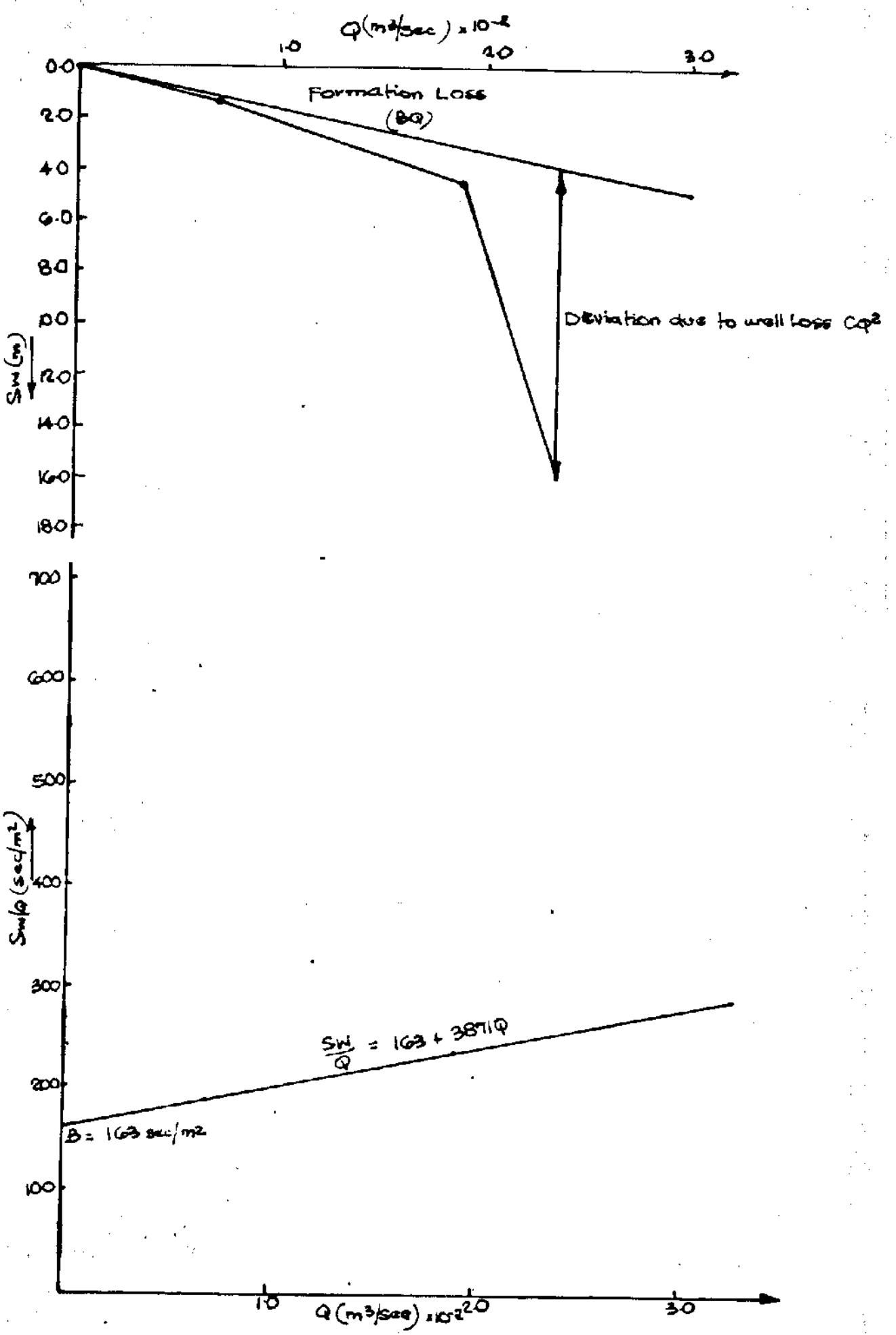


Fig. 5.12: Formation and well-Losses for Abstraction well N° 3

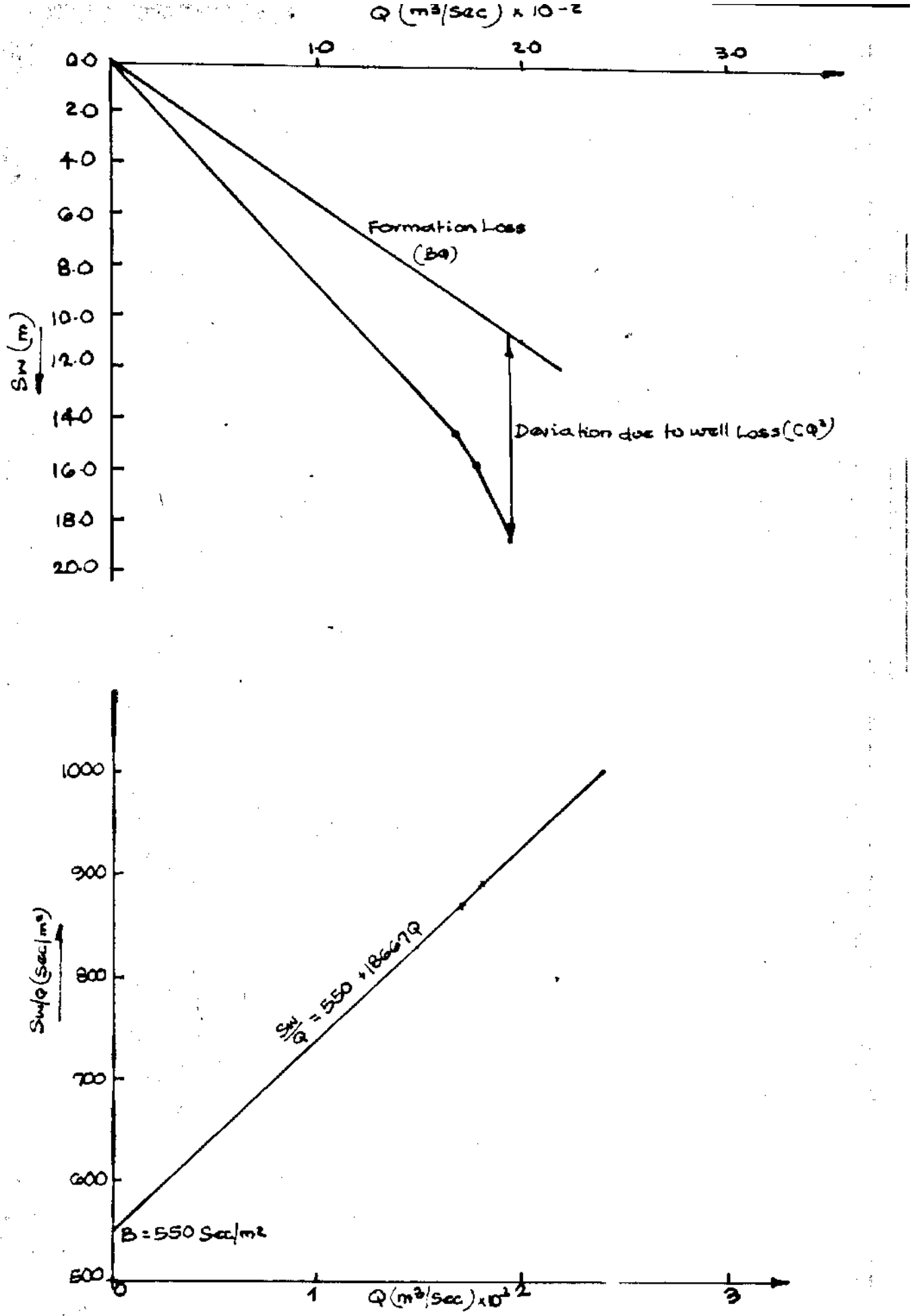


Fig. 5.13: Formation - and well - Losses for Abstraction well N°5

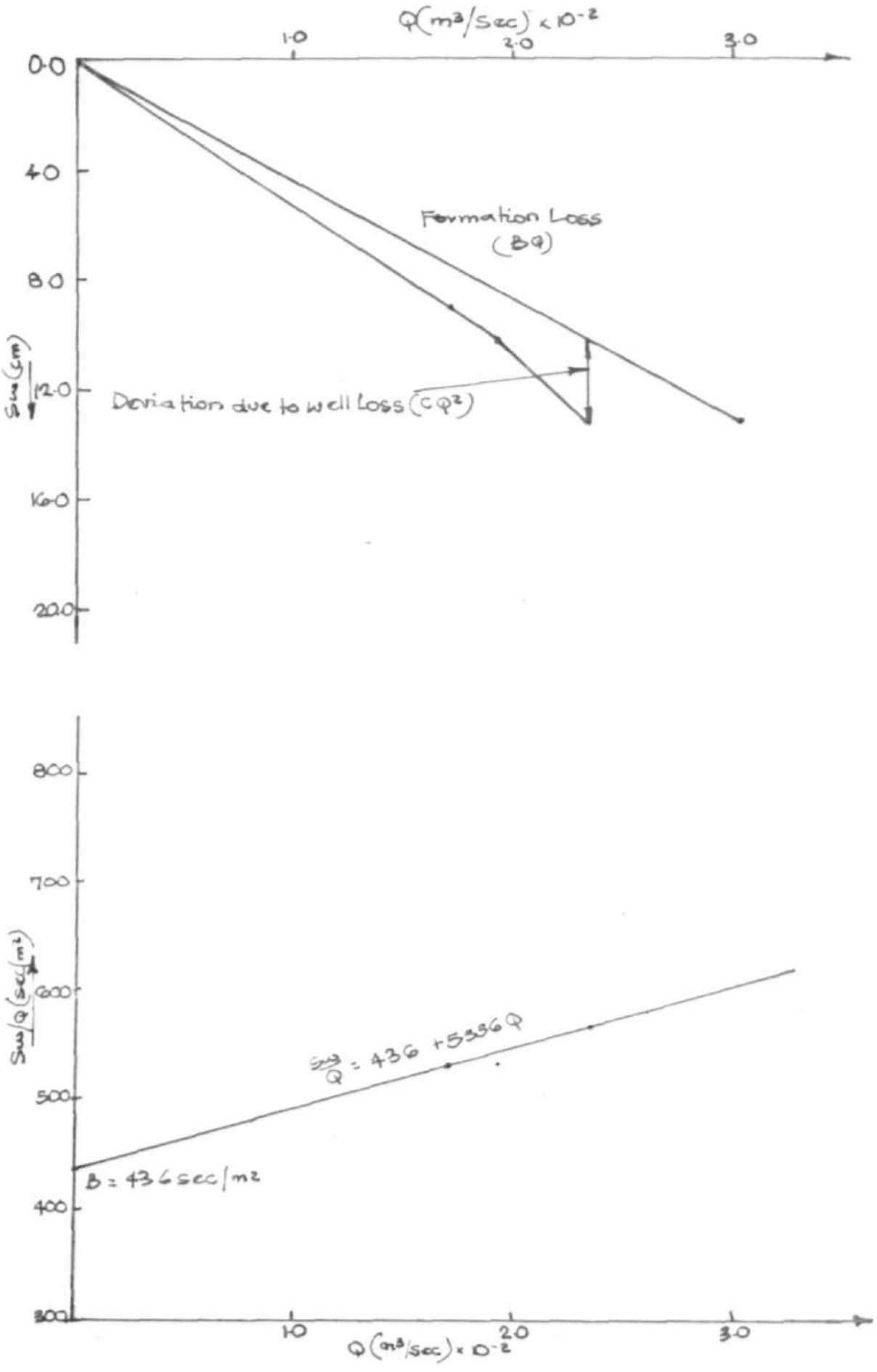


Fig. 5.14: Formation and well-Losses for Abstraction well Nº 6

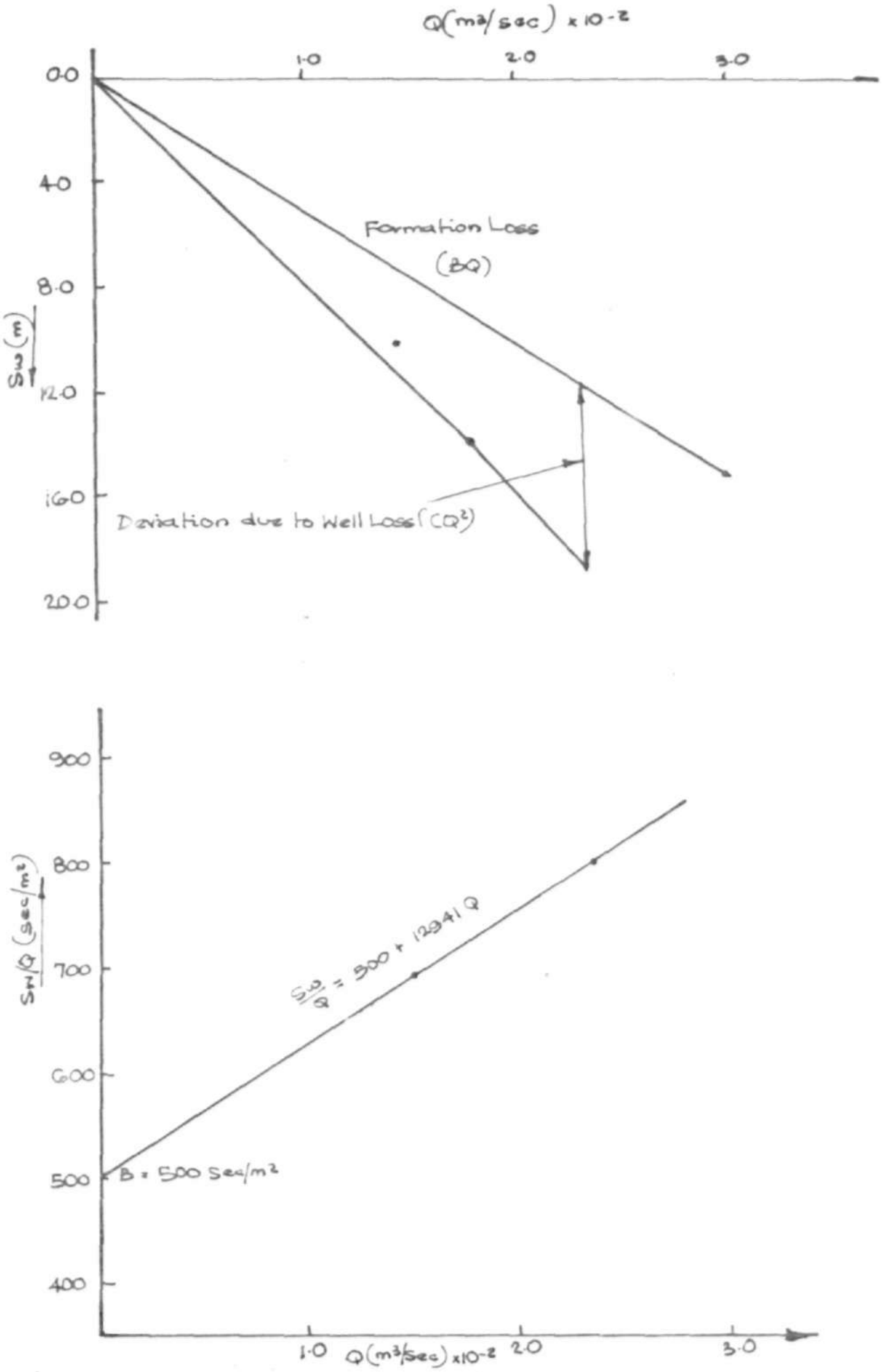


Fig. 5.15: Formation- and well-Losses for Abstraction Well N°11

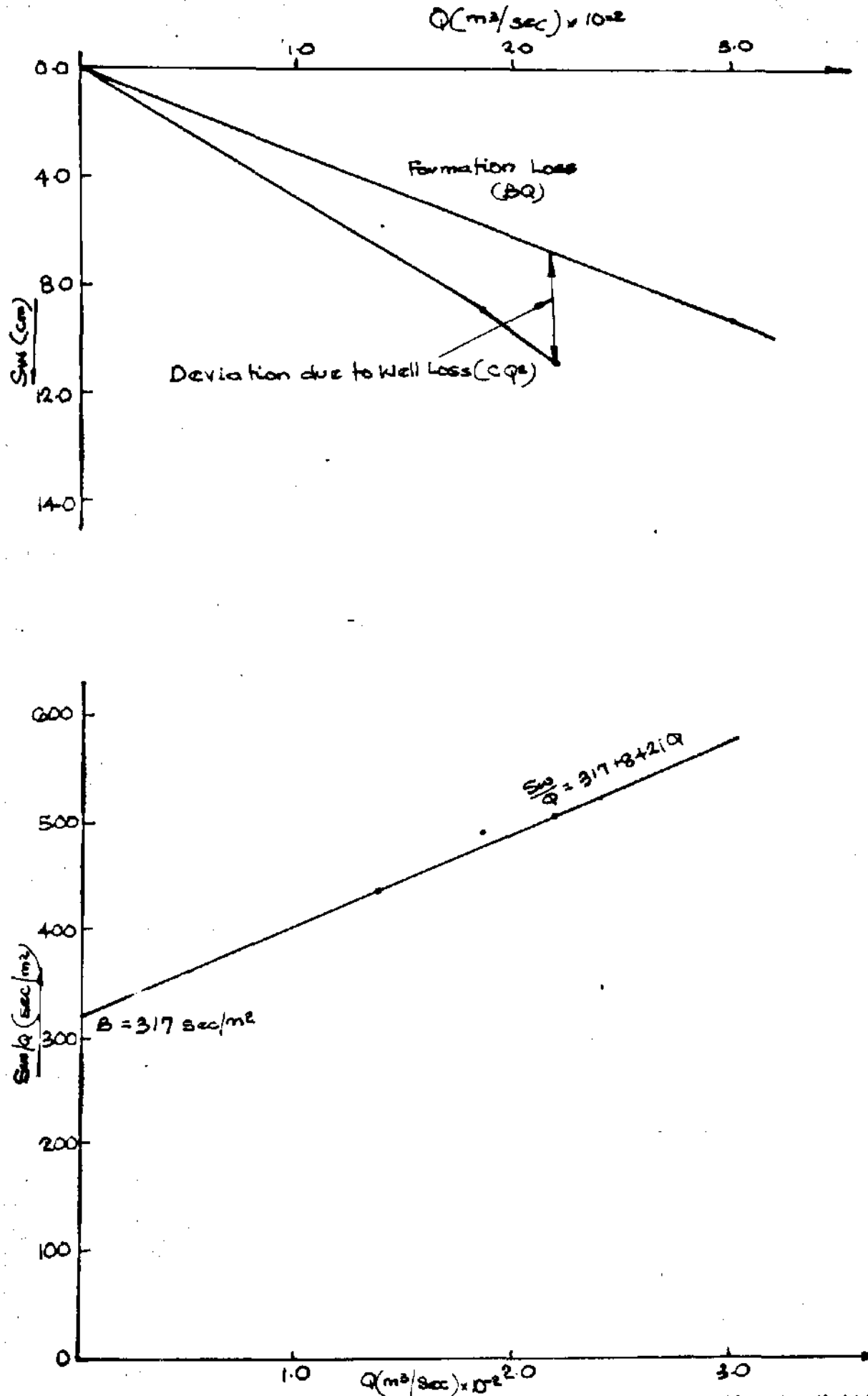


Fig. 5.16: Formation- and Well-Losses for Abstraction Well Nº 12

5.2 Model Input Parameters

5.2.1 First Trial (Initial Parameters)

The values of the various parameters which were read as input into the computer program, for the first trial run of the model are as follows.

A transmissivity value of 230.6m^2/d, which represents the average of all calculated T - values from both abstraction and observation wells was used. With a uniform aquifer thickness D of 30m assumed for the entire area, a permeability, K or PERM value of 7.68667m/d was therefore adopted for the aquifer. The average of the storage coefficient values obtained from the two observation wells was used as the confined storage coefficient SCON. Thus SCON = 0.00018. The unconfined storage coefficient, SUNCON is set to zero, since we are clearly dealing with a confined situation.

The well radius, RWELL is 0.200m, as measured on the site, while the outer boundary represented by RMAX is set at 10.000m. It is expected that the area of influence of pumping would not extend beyond this distance.

The top of the aquifer was set at 90.00m, below ground level while its base was assumed at 120.0m below ground level. The static water level, values of 47.47 and 43.70m were read as initial WLEVEL values for observation wells nos. 3 and 4 at 100m and 200m away respectively. The recharge value RECH, was initially set at zero. This is reasonable because the pumping tests were carried out mainly during the dry season when little or no rainfall is expected to infilltrate into the aquifer from its outcrop as recharge. Also the depth at which the aquifer

is located makes it unlikely to have indirect recharge from river or surface flow. Due to the extensive nature of the aquifer, the water level at the outer boundary is assumed to remain fixed.

Six observation wells are chosen at node nos. 2, 18, 19, 20 , 21, 22 which correspond to distances of 0.200m 92.8, 136.3, 200.0, 293 and 430.9m respectively from the pumped well. It should be noted that these nodes were chosen such that the observation wells for which f eld data are available fall within them.

Since we are presently interested in checking and verifying the hydraulic properties of the aquifer, the field test abstraction rate, QPUMP for borehole no. 5 for which there were observation wells was used. Also, the field test duration was used as duration of pumping TSTOP in the program. Thus, QPUMP = 69.9 m^3/hr or 1678. 0 m^3/day and TSTOP = 3924 min or 2.725 days.

A summary of these initial input parameters values is given in table 5.3 below:

Table 5.3: Model Input Parameters for the 1st Trial Run

Parameter	Symbol	Value
Transmissivity, Permability	T, PERM	230.6m^2/day, 7.68667 m/d
Confined storage Coefficient	SCON	0.00018
Unconfined " "	SUNCON	-
Well radius	RWELL	0.200m
Outer Boundary Radius	RMAX	10,000.0m
Top of Aquifer	TOP	90.0m
Base of Aquifer	BASE	120.0m
Static Water Level	WLEVEL	43.70m (OW4);47.47(OW3)
Recharge	RECH	0.0
Abstraction Rate	QPUMP	1678.0m^3/day
Duration of Pumping	TSTOP	2.725 days

5.2.2 Varying Parameters

A comparison of the field and numerical drawdown values for the first trial run show some differences. It was decided to vary some of the variable input parameters, one at a time until there is close agreement between the field and numerical values.

Transmissivity T - Varied: While maintaining all other parameters contant, the program was run with transmissivity, T -values of 500m^2/d, 200, 375, 245 m^2/d to obtain simulation nos. 2 3, 4 and 5 respectively.

Storage Coefficient, S - Varied: With a T - value of 245m^2/d which so far gave numerical drawdowns values in closest agreement with the field values and maintaining all other parameters constant, the program was run with storage coefficient, S - values

of 0.00180, 0.00002, 0.00016, 0.000019 and 0.000014 to get simulation nos. 6, 7, 8, 9 and 10.

Pumping Duration, TSTOP - Varied: Having established the aquifer hydraulic properties T and S it was then decided to investigate the effect of pumping a well for a whole year, instead of the normal field duration of 24 to 72 hrs. To this end, the program was run with TSTOP = 365 days, QPUMP - 1678 m^3/d. (i.e Field abstraction rate) and T and S at appropriately established values of 245 m^2/d and 0.00014 respectively.

Recharge, RECH - Varied: - So far, a recharge value of zero has been assumed, but in actual fact, there has to be some recharge into the aquifer at its outcrop, otherwise the groundwater resource would eventually be mined dry. The program allows for recharge to be distributed over the entire area of influence of pumping. Thus uniform recharge values of 0.00028, 0.00014, 0.00007, 0.00002 and 0.00001 m/d were tried, while maintaining all other parameters as in the preceeding paragraph.

Abstraction Rate, QPUMP - Varied: Since part of the objectives of this work is to determine the optimum number of wells and rate of pumping such wells in order to meet the housing estate's demand, it became necessary to investigate the effect of varying abstraction rate QPUMP. To this end, while maintaining all other aquifer parameters at their appropriately determined values and allowing a recharge value of 0.00001 m/d as determined in the previous paragraph, the model was run with different abstration rate, QPUMP values of 12000, 6000, 4000, 3000, 2400, 2000, 1714.3,

1500, 1333.3, 1200, 1000, 875.1, 750 m^3/d which correspond to operating 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 14, 16, operating wells respectively to meet the required daily demand of water.

A summary of the various model runs carried out with varying input parameters is given in Table 5.4.

Table 5.4: Summary of Model runs with varying parameters

Simulation/ Trial No	Q(m^3/d)	T(m^2/d)	S	TSTOP (days)	K(m/d)	RECH(m/d)
1.	1677.600	230.6	0.00018	2.725	7.68667	-
2.	"	800	"	"	16.66667	-
3.	"	200	"	"	6.66617	-
4.	"	375	"	"	12.50000	-
5.	"	245	"	"	8.16667	-
6.	1677.600	245	0.00180	"	"	-
7.	"	"	0.00002	"	"	-
8.	"	"	0.00016	"	"	-
9.	"	"	0.00015	"	"	-
10.	"	"	0.00014	"	"	-
11.	1677.600	"	"	365	"	-
12.	"	"	"	"	"	0.00028
13.	"	"	"	"	"	0.00014
14.	"	"	"	"	"	0.00007
15.	"	"	"	"	"	0.00002
16.	"	"	"	"	"	0.00001
17.	12000	"	"	"	"	"
18.	6000	"	"	"	"	"
19.	4000	"	"	"	"	"
20.	3000	"	"	"	"	"
21.	2400	"	"	"	"	"
22.	2000	"	"	"	"	"
23.	1714.3	"	"	"	"	"
24.	1500	"	"	"	"	"
25.	1333.3	"	"	"	"	"
26.	1200	"	"	"	"	"
27.	1000	"	"	"	"	"
28.	875.1	"	"	"	"	"
29.	750	"	"	"	"	"

5.3 Model Results

Extracts of numerical drawdown values from the computer outputs for simulation numbers 1 - 10 are tabulated against the field values for observation well numbers OW4 and OW3 in tables 5.5, and 5.6a and b respectively. Results have also been presented in graphical form to allow for direct visual comparison (Figures 5.17 - 5.20b). Table 5.7 gives the drawdown at the end of one year at different observation well nodes for simulation numbers 11 - 29.

5.3.1 Drawdown Estimates

When groundwater is extracted by a number of wells operating within an area, they are said to constitute a well field. Unless the spacing of wells in a field is so great that their zones of influence do not overlap the performance of individual well will be affected by the neighbouring operating well. Thus, if the locations and discharges of each of the wells in an area are known, the total drawdown at a given well location will be the sum of the drawdown due to the well itself, and that due to all other operating wells within the area.i.e.

$$s_{T1} = s_{1,1} + s_{1,2} + s_{1,3} + \cdots\cdots + s_{1,n}$$

where

s_{T1} = Total drawdown at well location no. 1

$s_{1,1}$ = Drawdown at location no. 1 due to well no.1 itself

Table 5.5: Field and Numerical Drawdown Values at Observation well, OW4-200m away from Abstraction well (Simulations 1 - 10).

Time (minutes)	Drawdown (metres)					
	Field	Numerical Sim. No.1	2	3	4	5
5.5	-	-	0.07		0.05	0.02
11	-		0.16	0.09	0.14	0.11
20	0.12	0.28	0.26	0.25	0.27	0.27
30	0.21	0.40	0.35	0.41	0.38	0.41
57.5	0.70	0.68	0.50	0.71	0.57	0.68
81	0.93	0.86	0.5[illegible]	0.91	0.68	0.84
112	1.12	1.03	0.66	1.09	0.78	1.00
143	1.26	1.15	0.72	1.24	0.87	1.11
203	1.45	1.35	0.81	1.46	0.99	1.30
264	1.58	1.48	0.88	1.62	1.08	1.43
324	1.66	1.61	0.94	1.75	1.15	1.54
444	1.79	1.78	1.02	1.96	1.26	1.70
564	1.88	1.91	1.07	2.11	1.33	1.83
684	1.98	2.02	1.14	2.24	1.39	1.93
864	2.08	2.15	1.20	2.39	1.49	2.06
1104	2.25	[illegible].30	1.27	2.55	1.59	2.19
1404	2.31	2.44	1.33	2.71	1.65	2.32
1704	2.54	2.54	1.38	2.84	1.74	[illegible].43
2184	2.60	2.69	1.44	3.01	1.82	2.56
2784	2.82	2.83	1.51	3.16	1.91	2.70
3444	2.97	2.95	1.57	3.31	1.99	2.81
3924	3.03	3.03	1.60	3.40	2.04	2.88
Transmissivity,T	244.7m^2/d	230.6m^2/d	500	200	375	245m^2/d
S/Coefficient, S	0.00016	0.00018	0.00018	0.00018	0.00018	0.00018
Abstraction rate, Q	1678m^3/d	1678m^3/d	1678	1678	1678	1678

Table 5.5 (Cont'd)

Time (min.)	Field	Numerical 6	7	8	9	10
5.5	-	-	0.53	0.03	0.02	0.05
11	-	-	0.88	0.13	0.10	0.16
20	0.12	0.00	1.20	0.30	0.25	0.35
30	0.21	0.01	1.42	0.44	0.39	0.50
57.5	0.70	0.03	1.78	0.73	0.65	0.79
81	0.93	0.07	1.96	0.89	0.81	1.02
112	1.12	0.13	2.14	1.05	0.97	1.12
143	1.26	0.18	2.27	1.18	1.09	1.25
203	1.45	0.28	2.46	1.33	1.26	1.43
264	1.58	0.37	2.60	1.50	1.41	1.56
324	1.66	0.45	2.72	1.60	1.51	1.67
444	1.79	0.57	2.89	1.77	1.69	1.84
564	1.88	0.67	3.02	1.89	1.80	1.97
684	1.98	0.76	3.13	2.00	1.90	2.07
864	2.08	0.84	3.26	2.13	2.04	2.20
1104	2.25	0.99	3.39	2.26	2.16	2.33
1404	2.31	1.11	3.51	2.39	2.30	2.46
1704	2.54	1.21	3.62	2.50	2.40	2.57
2184	2.60	1.33	3.74	2.63	2.54	2.70
2784	2.82	1.46	3.86	2.77	2.67	2.83
3444	2.97	1.57	3.95	2.88	2.77	2.95
3924	3.03	1.64	4.00	2.95	2.86	3.02
T. (m^2/d)	244.7m^2/d	245m^2/d	245	245	245	245
S	0.00016	0.00180	0.00002	0.00016	0.00019	0.00014
Q. (m^3/d)	1678m^3/d	1678m^3/d	1678	1678	1678	1678

Table 5.6a: Numerical Drawdowns at nodes 18 and 19 (92m and 136m away respectively from Abstraction Well)

Time (minutes)	Drawdown (metres)							
	Numerical Sim. 1		2		3		4	
	Node 18	Node 19	18	19	18	19	18	19
5.5	0.30	0.11	0.30	0.15	0.29	0.09	0.31	0.14
11	0.60	0.30	0.46	0.29	0.61	0.29	0.52	0.30
20	0.91	0.54	0.61	0.44	0.96	0.55	0.72	0.48
30	1.12	0.73	0.72	0.53	1.21	0.77	0.85	0.61
57.5	1.48	1.07	0.89	0.69	1.62	1.15	1.08	0.82
81	1.67	1.25	0.97	0.77	1.84	1.36	1.20	0.94
112	1.86	1.44	1.06	0.86	2.05	1.56	1.31	1.05
143	2.00	1.57	1.13	0.92	2.21	1.72	1.38	1.10
203	2.21	1.77	1.22	1.01	2.45	1.95	1.53	1.26
264	2.36	1.92	1.29	1.08	2.61	2.12	1.62	1.35
324	2.47	2.03	1.35	1.15	2.75	2.25	1.69	1.42
444	2.66	2.22	1.43	1.23	2.97	2.46	1.80	1.53
564	2.79	2.35	1.49	1.29	3.12	2.61	1.89	1.60
684	2.90	2.46	1.55	1.34	3.25	2.74	1.94	1.66
864	3.04	2.59	1.61	1.40	3.41	2.90	2.04	1.77
1104	3.18	2.74	1.68	1.47	3.57	3.06	2.13	1.86
1404	3.32	2.88	1.74	1.53	3.73	3.22	2.19	1.92
1704	3.44	2.99	1.79	1.58	3.86	3.35	2.28	2.01
2184	3.58	3.14	1.85	1.65	4.03	3.52	2.37	2.09
2784	3.72	3.27	1.92	1.72	4.19	3.67	2.46	2.18
3444	3.84	3.40	1.98	1.77	4.38	3.82	2.53	2.26
3924	3.92	3.48	2.01	1.81	4.42	3.91	2.58	2.31
Transmissivity T:	230.6		500		200		375	
S/Coefficient, S:	0.00018		0.00018		0.00018		0.00018	
Discharge, Q:	1678		1678		1678		1678	

Table 5.6a: (Cont'd)

	Numerical 5		6		7		8	
Time (min)	Node 18	Node 19	18	19	18	19	18	19
5.5	0.31	0.11	0.0	0.0	1.23	0.86	0.34	0.14
11	0.59	0.33	0.02	0.0	1.66	1.26	0.64	0.34
20	0.88	0.54	0.09	0.02	2.01	1.60	0.93	0.59
30	1.09	0.72	0.17	0.05	2.24	1.83	1.14	0.77
57.5	1.43	1.04	0.36	0.14	2.61	2.19	1.49	1.09
81	1.61	1.21	0.50	0.23	2.78	2.37	1.67	1.27
112	1.79	1.38	0.64	0.33	2.97	2.55	1.85	1.44
143	1.91	1.51	0.74	0.42	3.10	2.69	1.98	1.57
203	2.11	1.70	0.91	0.56	3.29	2.88	2.14	1.73
264	2.25	1.83	1.04	0.68	3.44	3.02	2.31	1.90
324	2.36	1.95	1.14	0.77	3.55	3.13	2.42	2.01
444	2.53	2.09	1.30	0.92	3.72	3.31	2.60	2.18
564	2.66	2.24	1.43	1.03	3.86	3.44	2.72	2.31
684	2.75	2.35	1.53	1.13	3.97	3.55	2.83	2.43
864	2.89	2.48	1.65	1.25	4.09	3.67	2.96	2.54
1104	3.02	2.61	1.78	1.38	4.23	3.81	3.09	2.67
1404	3.15	2.74	1.93	1.50	4.35	3.93	3.22	2.81
1704	3.26	2.83	2.02	1.61	4.45	4.03	3.32	2.91
2184	3.40	2.98	2.15	1.73	4.58	4.16	3.47	3.05
2784	3.53	3.11	2.28	1.87	4.70	4.28	3.60	3.18
3444	3.65	3.23	2.39	1.98	4.79	4.37	3.72	3.30
3924	3.72	3.30	2.47	2.05	4.84	4.42	3.79	3.37
T (m^2/d)	245		245		245		245	
S	0.00018		0.00180		0.00002		0.00016	
Q (m^3/d)	1678		1678		1678		1678	

Table 5.6a: (Cont'd)

Time (min)	Numerical 9 Node 18	19	10 18	19
5.5	0.29	0.10	0.39	0.16
11	0.57	0.29	0.70	0.39
20	0.84	0.52	1.00	0.65
30	1.06	0.70	1.21	0.83
57.5	1.40	1.01	1.56	1.16
81	1.58	1.18	1.74	1.37
112	1.76	1.35	1.92	1.51
143	1.89	1.48	2.05	1.64
203	2.06	1.66	2.24	1.83
264	2.22	1.81	2.38	1.97
324	2.23	1.91	2.49	2.08
444	2.51	2.10	2.67	2.25
564	2.63	2.21	2.80	2.38
684	2.73	2.32	2.90	2.49
864	2.87	2.45	3.03	2.61
1104	2.99	2.58	3.16	2.75
1404	3.13	2.71	3.29	2.88
1704	3.23	2.82	3.40	2.98
2184	3.37	2.95	3.54	3.12
2784	3.50	3.08	3.67	3.25
3444	3.62	3.20	3.78	3.36
3924	3.69	3.27	3.86	3.44
T (m^2/d)	245		245	
S	0.00019		0.00014	
Q (m^3/d)	1678		1678	

Table 5.6b: Field Drawdown at Observation Well, OW3 - 100m away from Abstraction Well, AW5.

Time (min)	Drawdown.
44	1.0
76	1.47
102	1.69
125	1.83
155	1.95
179	2.14
196	2.22
271	2.27
307	2.29
347	2.35
437	2.46
497	2.52
677	2.58
797	2.67
857	2.71
1037	2.78
1277	2.95
1337	2.98
1637	3.14
2057	3.18
2597	3.40
3257	3.68
3917	3.70
Transmissivity, T	245.2 m^2/d
Storage Coefficient, S	0.00019
Abstraction rate, Q	1678 m^3/d.

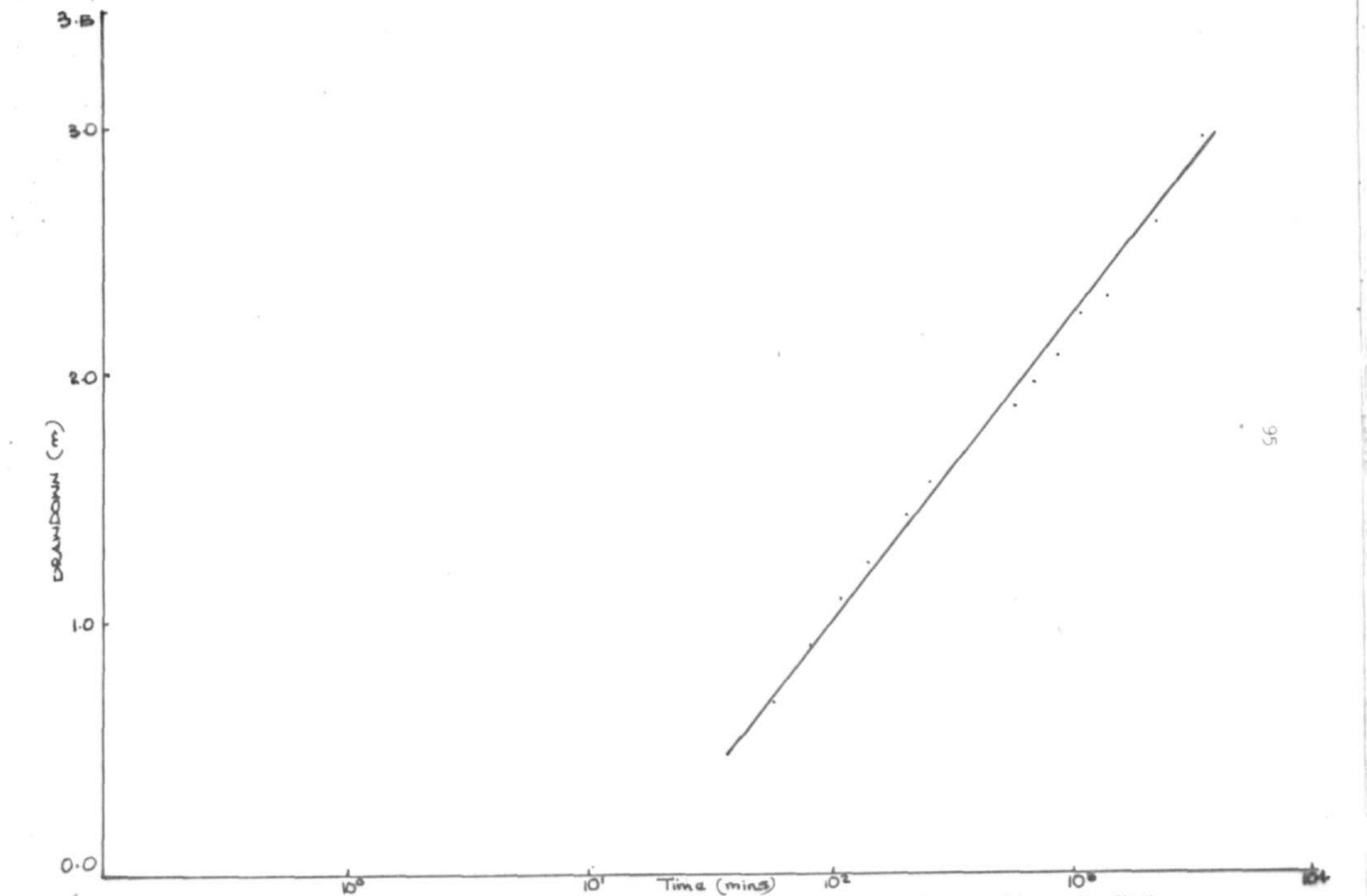


Fig. 5.17 : Semi-Log plot of field Drawdown against Time for Observation well OW4

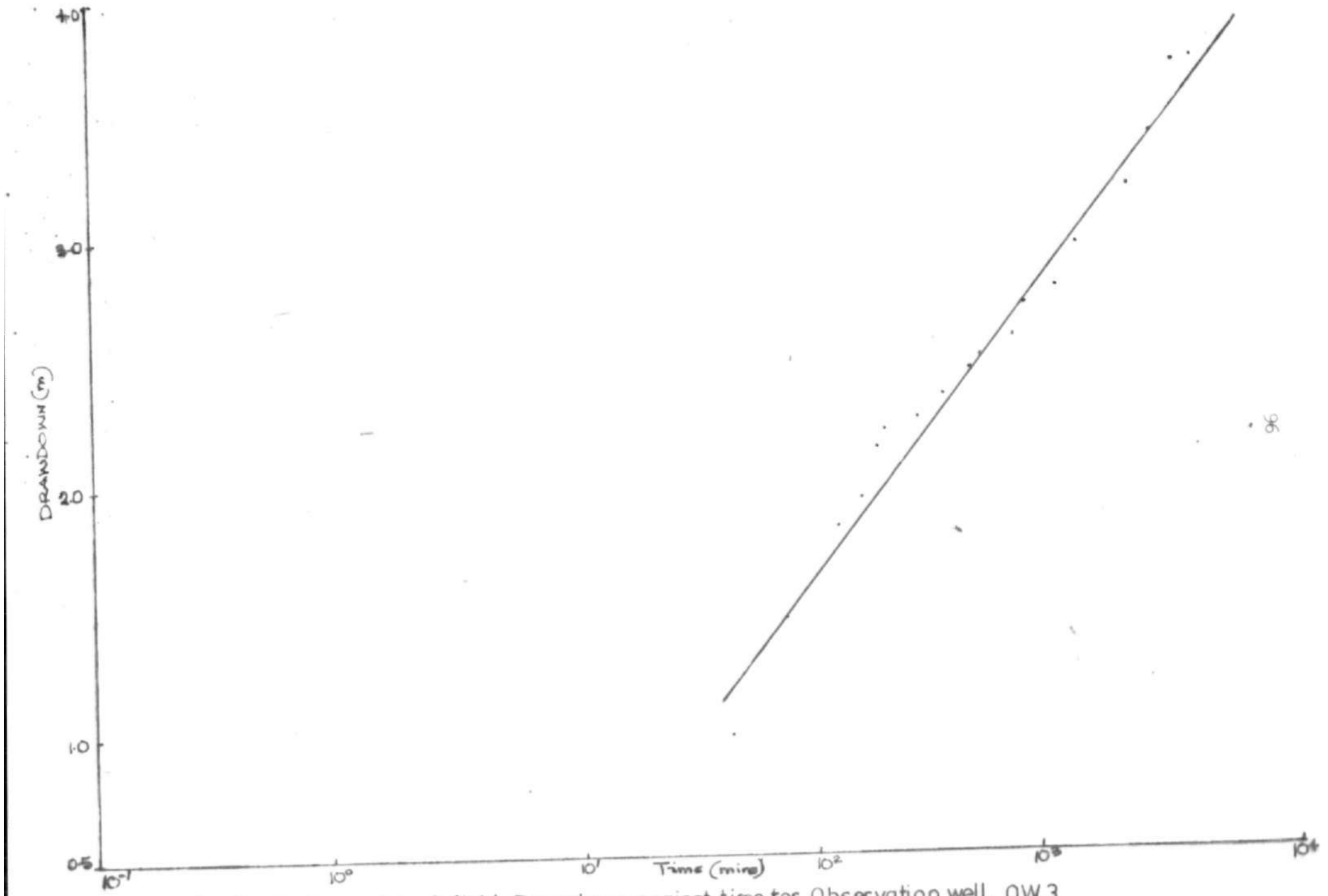


Fig. 5.18 : Semi - Log plot of field Drawdown against time, for Observation well, OW 3

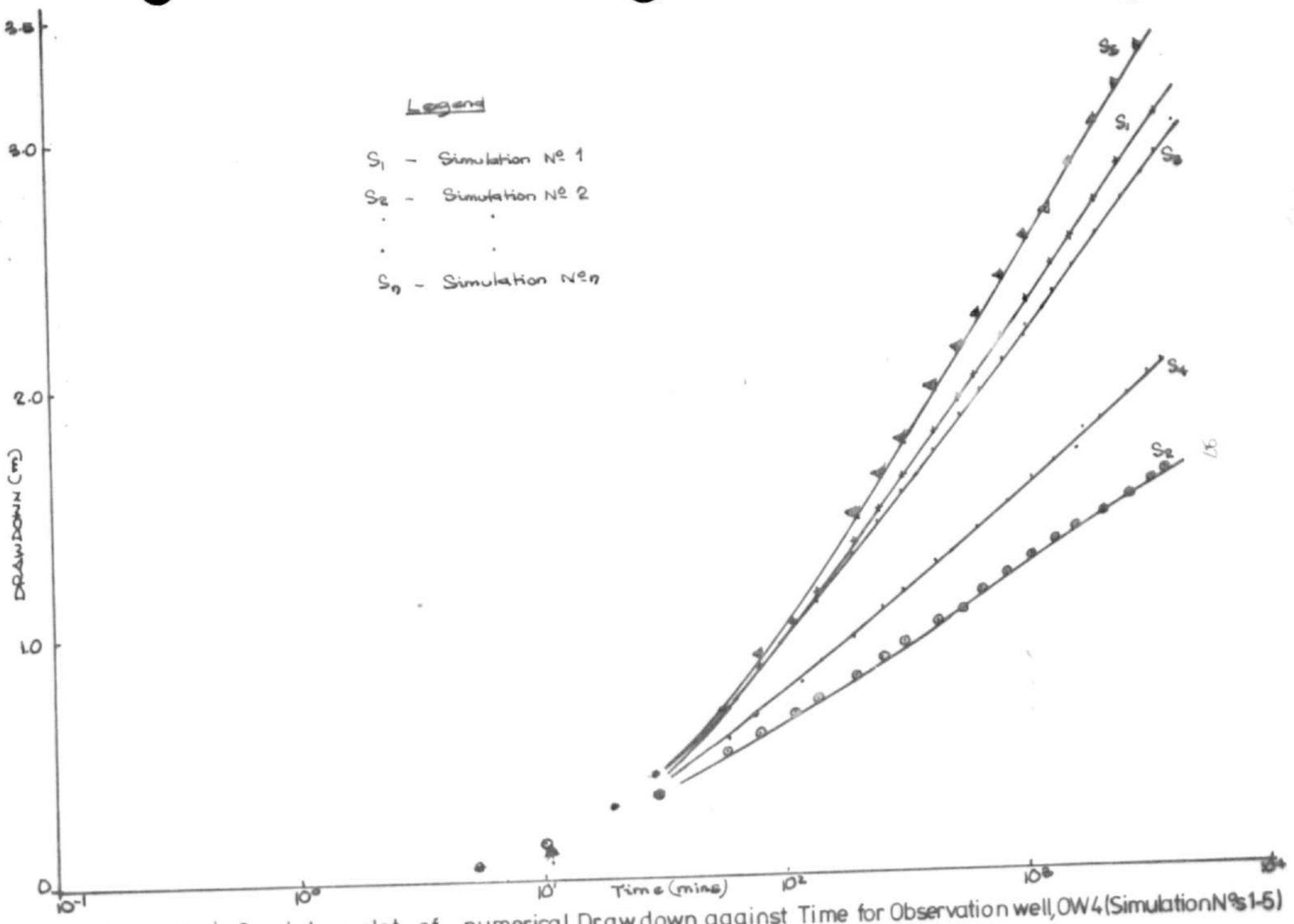


Fig. 5.19g: Semi-Log plot of numerical Drawdown against Time for Observation well, OW4 (Simulation Nºs 1-5)

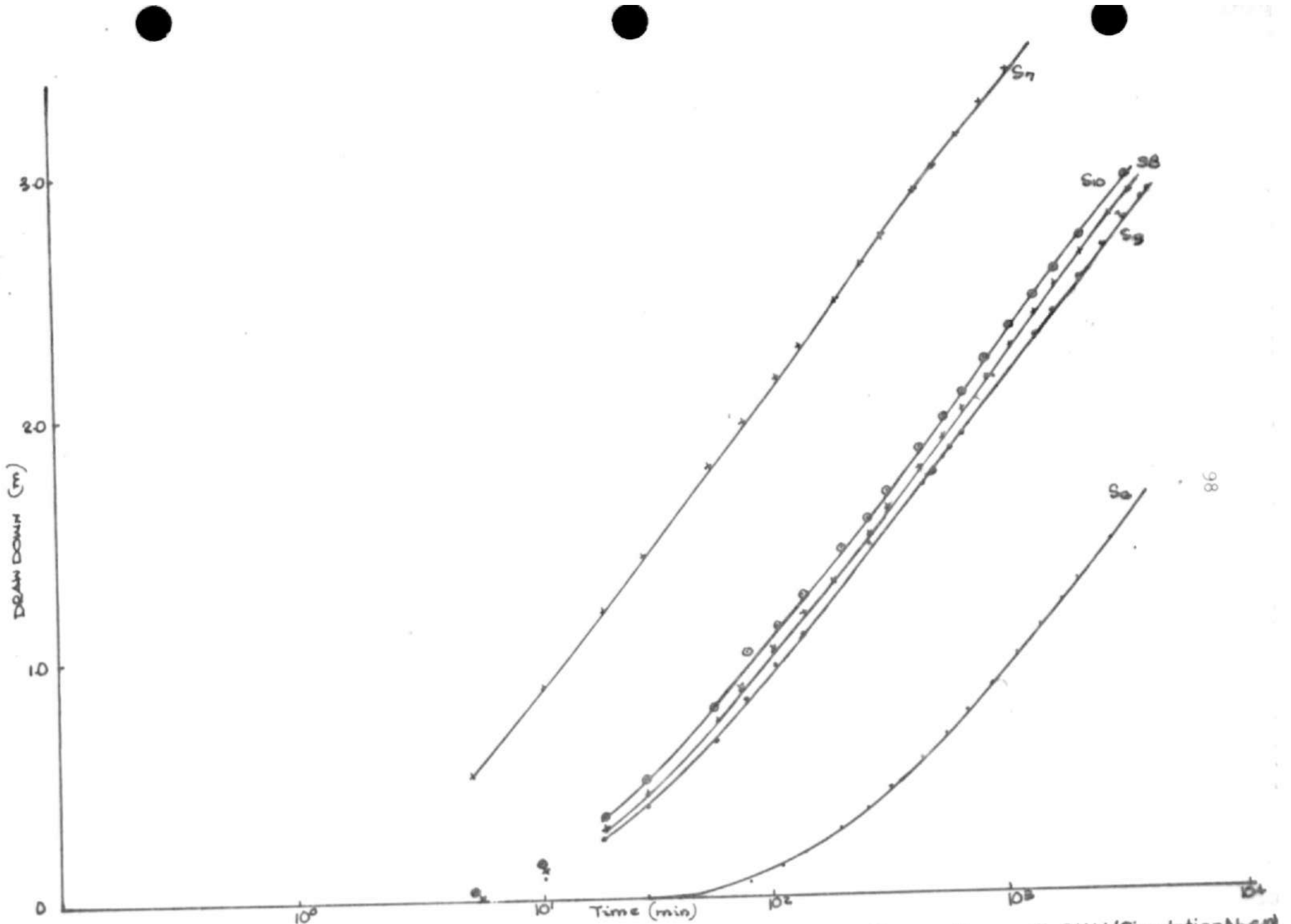


Fig 5.19b: Semi-Log plot of numerical Drawdown against Time for Observation well, OW4 (Simulation No. 8)

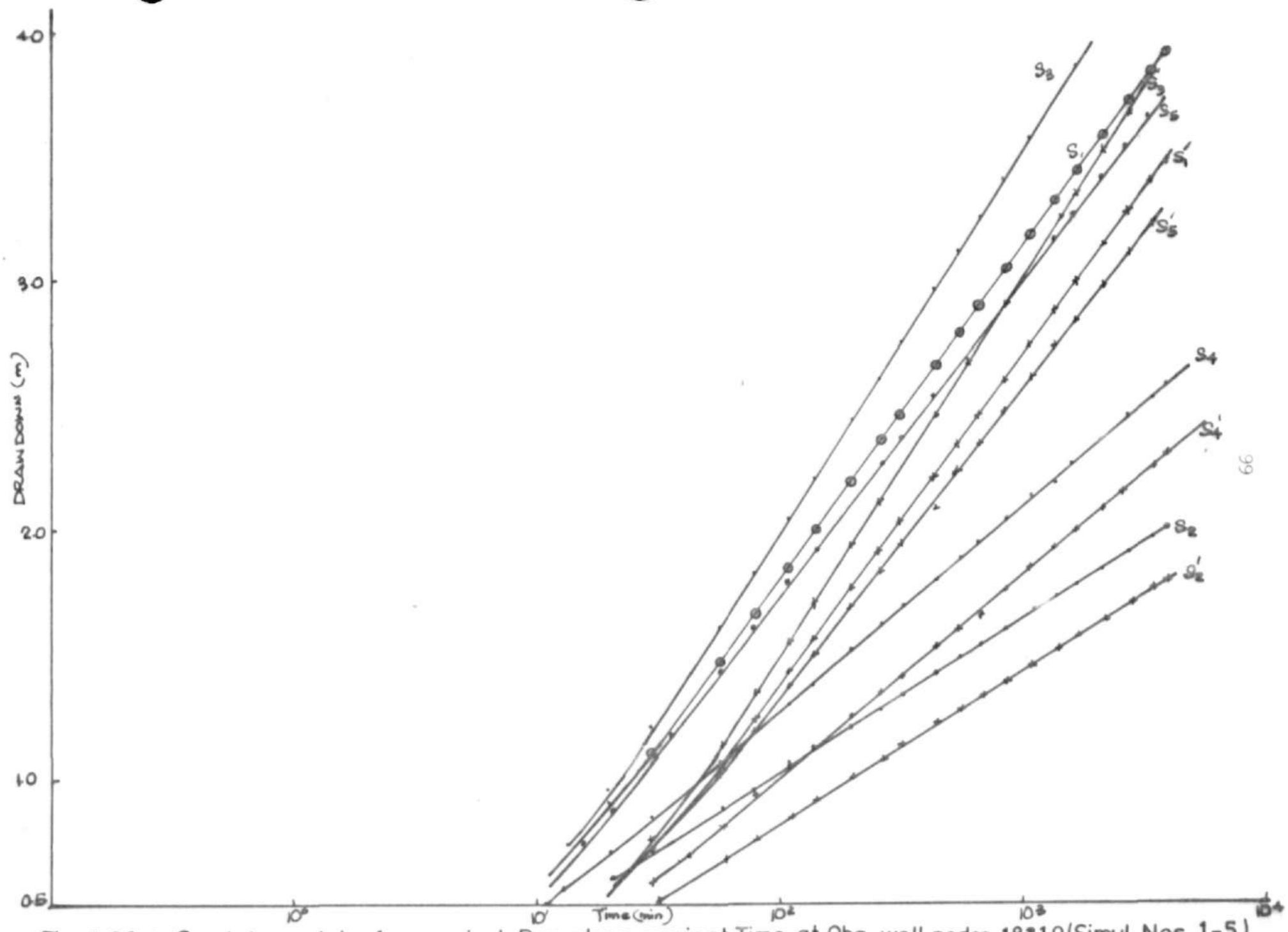


Fig. 5.20g: Semi-Log plot of numerical Drawdown against Time at Obs. well nodes 18 & 19 (Simul. Nos. 1-5)

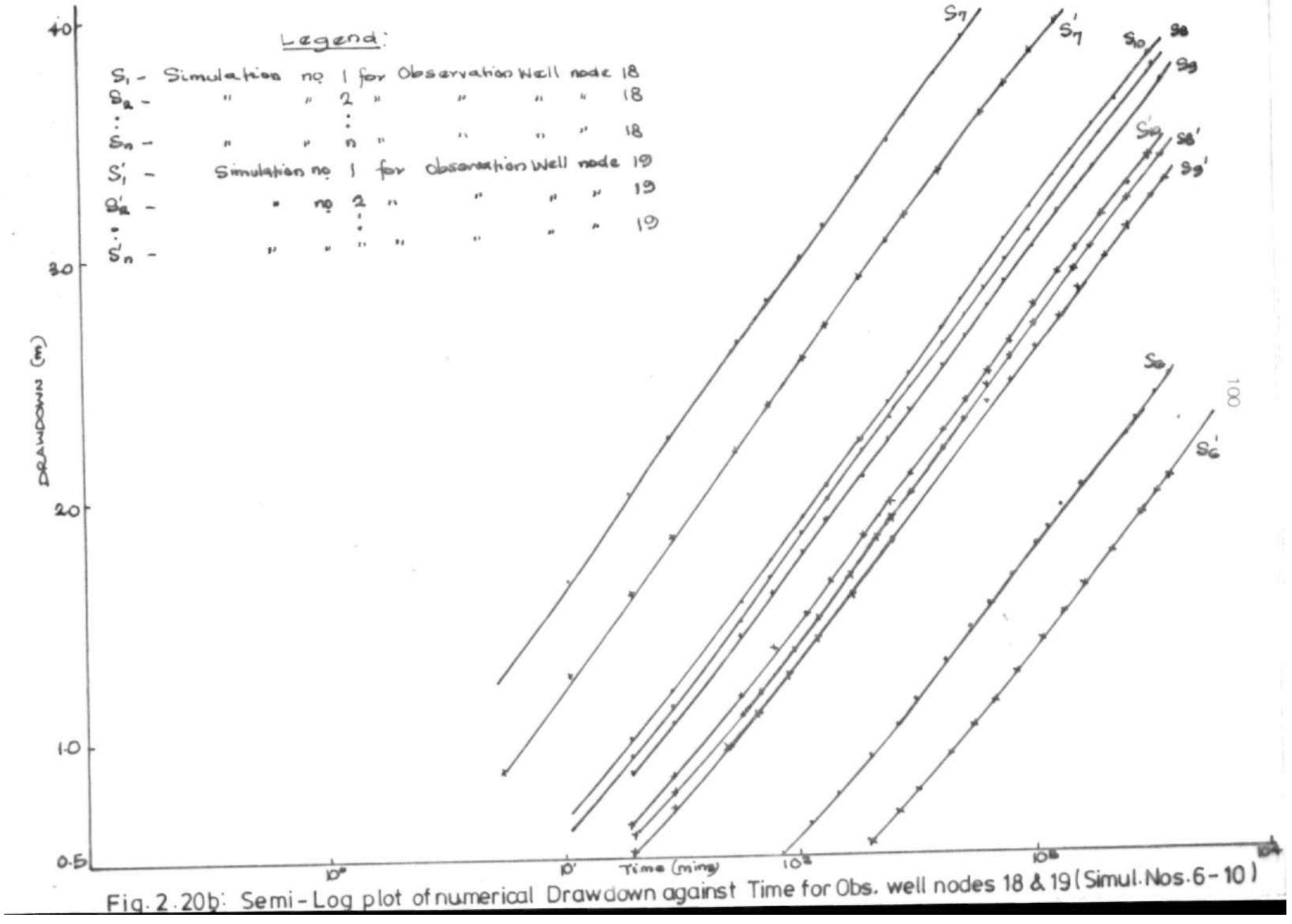


Fig. 2.20b: Semi-Log plot of numerical Drawdown against Time for Obs. well nodes 18 & 19 (Simul. Nos. 6-10)

Table 5.7: Drawdown at the end of one-year at different Observation Well Nodes (Simulations 11 - 29).

Simulation No.	Drawdown at Node Nos.					
	2 (200m)	18 (92.8)	19 (130.2)	20 (200)	21 293	22 (431)
11	11.73	5.03	4.62	4.20	3.78	3.36
12 *	-12.74	-19.43	-19.84	-20.26	-21.66	-21.05
13 *	- 0.51	-7.20	-7.61	-8.03	-8.44	-8.84
14 *	5.61	-1.08	-1.50	-1.92	-2.33	-2.74
15	9.98	3.29	2.87	2.45	2.03	1.62
16	10.85	4.16	3.74	3.32	2.91	2.49
17 **	-	-	-	-	-	-
18	41.06	17.13	15.64	14.14	12.65	11.15
19	26.54	10.91	9.93	8.95	7.98	7.10
20	20.10	8.13	7.38	6.63	5.89	5.14
21	15.90	6.33	5.73	5.13	4.53	3.94
22	13.11	5.13	4.63	4.13	3.63	3.14
23	11.11	4.27	3.84	3.42	2.99	2.56
24	9.61	3.63	3.25	2.88	2.51	2.13
25	8.45	3.13	2.80	2.46	2.13	1.80
26	7.51	2.73	2.43	2.13	1.83	1.53
27	6.12	2.13	1.88	1.63	1.38	1.13
28	5.24	1.75	1.53	1.32	1.10	0.88
29	4.34	1.38	1.19	1.00	0.82	0.63

NB:

* - Negative values indicate a rise in piezometric level above the original piezometric level.

** - Piezometric level fell below the top of the aquifer and the program execution was aborted because of excessive drawdown at Simulation No. 17.

303011

$s_{1,2}$ = Drawdown at location no.1 due to well no.2

$s_{1,n}$ = " " " " " " " " " n.

5.3.2 Proposed Pilot Scheme

The proposed pilot scheme for water supply to the Ipaja Housing Estate was to operate 12 wells for 24 hrs a day with the remaining 4 wells serving as standby. Each of the 12 wells was to be pumped at 1000 m^3/day, giving a total water supply of 12,000 m^3/day. The 12 operating wells are well numbers TW, 1,2,3 .. ... 10 and 12, while the 4 standby wells are, well numbers 11,13, 14 and 15 (see Fig. 5.1).

Recalling simulation number 27 for which Q = 1000 m^3/d, TSTOP = 365 days, T = 245 m^2/d, S = 0.00014, RECH = 0.00001 m/d. The maximum drawdown will occur at the central borehole no. TW, and this is estimated as follows:

Well number	Distance from TW(m)	Drawdown Effect at, TW(m)
TW	-	6.12 = 6.12
2,3,4	212	3 x 1.60 = 4.80
5,6,8,9,10,12	158	6 x 1.79 = 10.74
1,	198	1.64 = 1.64
7	206	1.61 = 1.61
	Total	24.91m

Add well Loss, CQ^2 (see fig. 5.11) 0.87m

Estimated max. drawdown in well, TW. 25.78m

5.3.3 Alternative Scheme

An alternative scheme is to operate all the 16 wells for 24 hrs a day with no standby well. Each well will then be pumped at 750 m^3/day, giving a total water supply of 12000 m^3/day.

Recalling simulation number 29 for which,

Q = 750 m^3/d; TSTOP = 365 days,

T = 245 m^2/d; S = 0.00014; RECH = 0.00001 m/d.

The estimated drawdown occuring at boreholde number T is as follows:

Well number	Distance from TW(m)	Drawdown Effect at TW(m)
TW	-	4.37 = 4.37
5,6,8,9,10,12	158	6 x 1.13 = 6.78
1	198	1.01 = 1.01
7	206	0.99 = 0.99
2,3,4	212	3 x 0.98 = 2.94
14	318	0.79 = 0.79
11	334	0.76 = 0.76
15	437	0.62 = 0.62
13	463	0.57 = 0.57

Total	18.83m
And Well Loss, CQ^2 (see fig. 5.11)	0.49m
Estimated max. drawdown in well TW.	19.32m

The estimated maximum drawdown values at the pumping well for the different abstraction rates tried are given in table 5.8.

The significance and interpretation of the results of the pumping test analysis and model response are discussed in the next chapter.

Table 5.8: Maximum Drawdown Estimates at the Abstraction Well for Varying Abstraction rates (Simulations 17 - 29).

Simulation No.	Abstraction Rate, Q (m^3/d)	Maximum Drawdown Estimates at the Abstraction Well (m)
17	12000.0	-*
18	6000.0	82.06
19	4000.0	54.80
20	3000.0	44.38
21	2400.0	42.16
22	2000.0	37.99
23	1714.0	34.91
24	1500.0	32.50
25	1333.3	30.48
26	1200.0	28.74
27	1000.0	25.78
28	875.1	23.27
29	750.0	19.32

* - Water level fell below the top of the aquifer and the program execution was aborted because of excessive drawdown.

Chapter 6

DISCUSSION OF RESULTS

6.1 Hydraulic Properties of the Aquifer, T and S

A summary of the hydraulic properties of the underlying aquifer being tapped, and which were obtained from the analysis of pumping test data for abstraction and observation wells and driller's record is given in Table 5.2. Transmissivity values ranged from 81 to 500 m^2/day with an average value of 231 m^2/d. This indicates an aquifer of fairly good or medium ability to transmit water. T - values from abstraction wells vary widely, while those from observation wells were rather close. The large variation in the abstraction well T-values may have resulted from the slight variation in thickness and depth of location of the aquifer from one well to another. Differences in the construction and development conditions of each well may also have contributed to such variations.

It was only possible to evaluate the storage coefficient of the aquifer from observation well data. The two values obtained were 1.59×10^{-4} and 1.92×10^{-4}. These clearly indicate a confined aquifer.

With the radial flow numerical model, it was possible to check and verify the aquifer hydraulic properties obtained from the graphical analysis of pumping test data. The field drawdown values for OW3 and OW4, and numerical drawdowns as abstracted from computer outputs for nodes 18, 19 and 20 (at 92.8, 136.3 and 200m respectively) are presented for simulations 1 - 10 in Tables 5.5 and 5.6. Semi-log graphical plots of drawdown in observation wells against time (time on log-scale)

for both field and numerical values have also been presented (5. 17 - 5.20b). In order to make for easy comparison, the field drawdown values were plotted on a transparent paper, while numerical values for the various simulations were plotted on plain semi-log graph paper.

By placing the plain semi-log plot under the transparent plot, aligning the axes and viewing against light, it was possible to see which of the model simulations gave numerical drawdown plot that best agree with the field results for a particular observation well. Simulation no.10, for which T=245 m^2/d and S = 0.00014 gave the best plot for both observation wells, OW4 and OW3. These values were therefore adopted as the true hydraulic properties of the aquifer, and were used in subsequent simulations. It should be noted that in the case of OW3 (located at 100m abstraction well) which did not fall at a node, numerical drawdown plots for the two adjacent observation well nodes at 92.8m and 136.3m were used. (fig 5.20a & b).

The closeness of the T and S values from the analysis of observation well pumping test data, to those verified by the model simulation no. 10, tend to suggest that pumping test measurements from observation wells are more reliable than measurements from abstraction wells for estimating aquifer characteristics.

6.2 Hydraulic Properties of Tested Wells

Since step drawdown/constant discharge and recovery test were carried out on all wells during pumping, it was possible to

determine some hydraulic properties of the tested wells. The available step drawdown data were analysed by Jacob's method, and results are presented in Figs. 5.11 to 5.16.

The calculated formation loss factors, B, vary from 163 see/m^2 to 720 sec/m^2, indicating some dispersion in the aquifer transmissivity. Well loss constant C, ranged from 3871 sec^2/m^5 to 29091 sec^2/m^5. The part of drawdown attributed to well loss i.e CQ^2, for the different steps in the tests vary from 14% to 47% of the the total drawdown in the wells.(see appendix III).

As none of the well losses was higher than the formation loss, it may be concluded that the well development and screening procedures, although not optimal, are however satisfactory.

6.3 Drawdown Estimates

After ascertaining the transmissivity and storage coefficient values for the aquifer, other model parameters like duration of pumping, amount of recharge, and abstration rate were varied, one at a time, to investigate the effect of continuous abstration for a year, determine a reasonable recharge value, and decide the optimum number of wells to be operated to meet the required water demand. In all cases, the one-year drawdown values at different observation well nodes and for each simulation were noted (Table 5.7).

Simulation no. 11, which was carried out to see the effect of 1 year continuous pumpage, showed that the water level in the abstraction well remain constant after about 42 to 60 days when discharging at 70m^3/hr. The maximum drawdown was then 11.73m

at the face of the abstration well.

Simulation nos. 12 to 16 which sought to decide a reasonable recharge value, showed that, with the exception of simulation no. 16 for which a recharge value of 0.00001m/day was assumed, all other trials gave results in which the water level began to rise after dropping to a minimum value. In some cases, negative groundwater potential was obtained, indicating a water level rise above the original piezometric level before the start of pumping.

Simulation nos. 17 to 29 were carried out to investigate the effect of varying abstraction rate QPUMP, while maintaining all other parameters constant at their appropriately determined values. Abstraction rates corresponding to operating 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 12, 14, and 16 wells respectively were tried, and the maximum total drawdown estimate in an operating well estimated for each case. Such total drawdown estimates include the drawdown due to the pumped well itself, that due to the interference effect of other operating wells, and also that due to well losses. Calculations for the original pilot scheme in which 12 wells are operated with 4 serving as standby, and an alternative scheme in which all 16 wells are operated with no standby have been presented (section 5.3). Drawdown estimates for the other cases are summarized in Table 5.8. The results showed that the least total drawdown per well value of 19.32m (well losses inclusive) was obtained when all the 16 wells were operating at a pumping rate of 750m^3/day/well. This represents a 6.46m decrease or 25% reduction when compared with the drawdown estimate for the original pilot scheme.

A general trend of increasing drawdown estimates was noticed as the number of operating wells became fewer. In fact, when only one well was pumped at 12000m^3/day (simulation no. 17), the water level fell below the top of the aquifer and program execution was aborted because of excessive drawdown. The explanation for the above trend is that although, interference effect may become less with fewer wells being operated because of increased spacing, the total drawdown in a well (which is made up of formation losses, well loses and influence of other operating wells) is however increased as a result of higher abstration from the well.

6.4 Limitations

The results presented and discussed above are by no means exact. In fact, they suffer from a few limitations and it is important to highlight some of these at this stage:-

(1) Both the pumping test data analysis and radial flow model assumed a horizontal aquifer with uniform thickness, which is homogeneous, isotropic and of infinite areal extent. Information from the strata log, lithology, casing and screen locations however indicate an aquifer whose thickness vary slightly from one location to another. Homogenity and isotropism are also idealized concepts which hardly apply in nature.

(2) A confined aquifer with no leakage flow was assumed because the confining layers were predominantly clayey, and no water level measurements were recorded from them during pumping.

(3) The lack of sufficient number of observation wells did not enable widespread determination of storage coefficient S, vales.

(4) The model assumed recharge to the aquifer to come from leakage from the upper layers, while in real life, recharge may also occur through the outcrop of the aquifer which may be serveral kilometers away.

It should be noted that some of the assumptions stated may not necessarily apply to the aquifer, but in the absence of adequate hydrogeologic data to permit a more exact description of the aquifer, such assumptions had to be made.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

From the results of the work carried out, the following conclusions can be drawn:

(i) The water supply requirements of the Ipaja Housing Estate of 12,000 m^3/day can be met by the second underlying aquifer that is currently being tapped.

(ii) The hydraulic properties of the aquifer at the well site are such that the transmissivity and storage coefficient are of the order of 250 m^2/day and 0.00015 respectively. These clearly indicate artesian conditions and suggest an aquifer of medium ability to transmit water.

(iii) Individually, the boreholes sunk within the estate are capable of yields ranging from 60 m^3/hr to 84 m^3/hr of water with drawdown of 11m to 30m . . How-ever, when collectively operated to meet the estates demand, drawdowns easily exceed the available drawdown of about 45m due to interference effects at such high pumping rates.

(iv) The analysed step drawdown test data gave formation loss factor, B, ranging from 183 to 720 sec/m^2 indicating some dispersion in the aquifer transmissivity. Also since none of the well losses were higher than the formation losses, it may be concluded that the construc-tion, development and screening procedures for the wells were quite satisfactory, although they may not necessarily

have been optimal.

(v) An optimum use of the well field is attained when all the 16 wells are operated at an abstration rate of 750m^3 per day per well. This arrangement produced a maximum total drawdown of 19.32m in any one well. When compared with the corresponding value of 25.78m for the original pilot scheme of 12 operating and 4 standby wells, a drawdown reduction of 6.4m or 25% is realised.

7.2 Recommendations

Based on the outcome of the study carried out, the following suggestions and recommendations are made:

(i) In meeting the water demand of the Ipaja Housing Estate, all the 16 wells sunk within the estate should be put to use at all times except during maintainance periods when some well, could be shut while others are working. Before starting water supply operations,all wells should be properly cleaned and redeveloped to remove materials that might have clogged the screens as a result of long period of non-use, and to ensure efficient water inflow to the wells.

(ii) It is suggested that further pumping tests should be carried out with more emphasis placed on observation well measurements. This will enable more accurate determination of the hydraulic characteristics of the aquifer. Such tests should be properly planned and carefully conducted so that all necessary information about the sub-surface geological and hydrological conditions in the area are collected e.g.

Lithology, aquifer thickness, aquifer boundaries, location and amount of recharge, piezometric surface elevations etc.

(iii) As additional information become available, the model could be improved upon, with the new hydraulic properties and piezometric surface elevations forming a basis for calibration and thus providing a more exact calculation of the consequences of present and future planned withdrawal

(iv) In this study, a uniform abstration rate has been assumed for all operating wells in order to meet the water requirements of the Ipaja Housing Estate. However, further investigation can be carried out on the model using different abstraction rates for the pumping wells. The transmissivity value can form a basis for choosing the discharge rate at any operating well. The maximum drawdown can then be calculated for different discharge combinations to determine the best performance combination for the estate.

Finally, it should be stated that the suggestions and recommendations above, are considered vital towards achieving the best performance for the wells and ensuring optimum exploitation and utilisation of available groundwater resources in the area .

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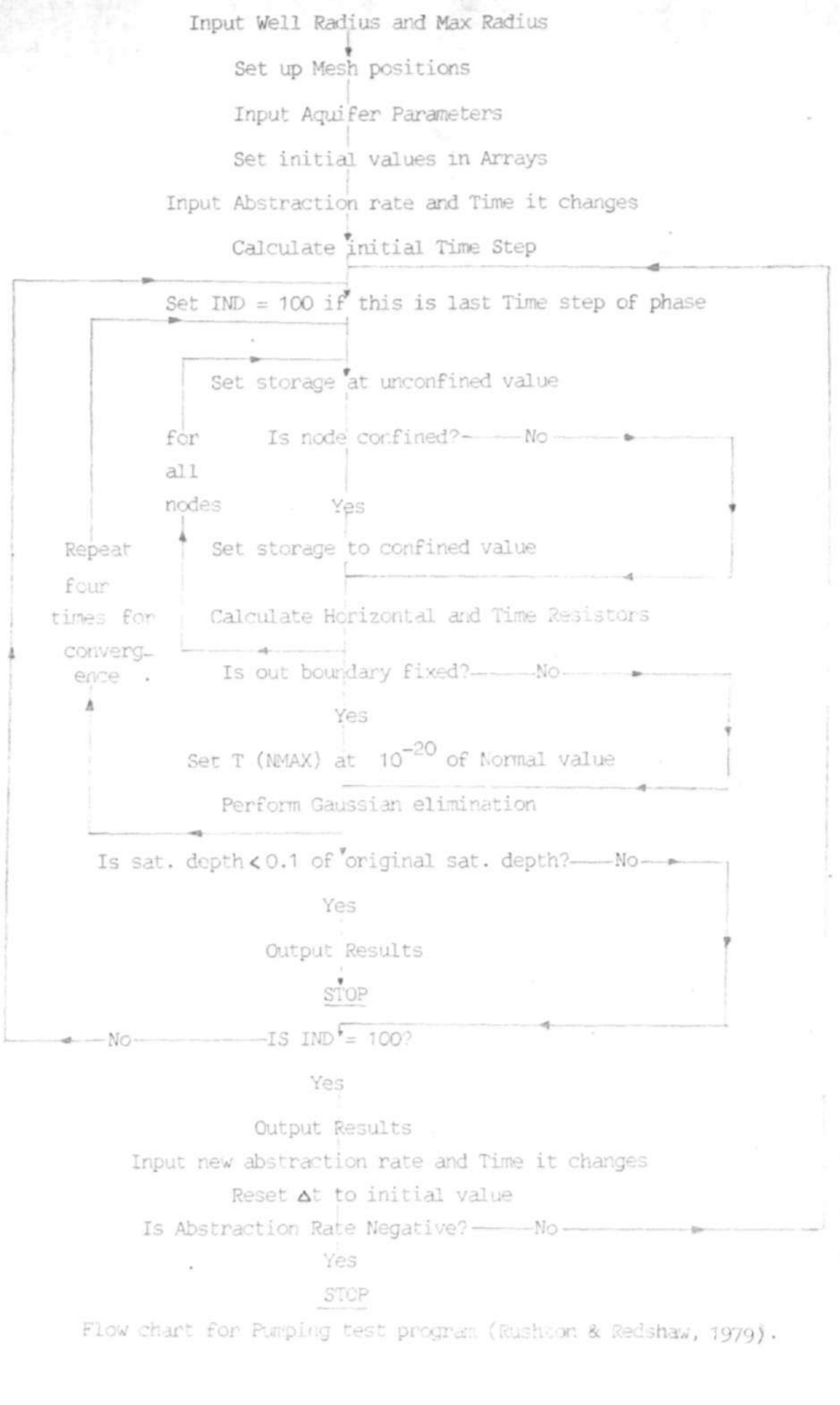
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APPENDIX I



Flow chart for Pumping test program (Rushton & Redshaw, 1979).

APPENDIX II

PUMPING TEST PROGRAM LISTING

```
0011B          PROGRAM P1 (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
          C  NUMERICAL PUMPING TEST, NO VERTICAL FLOW, WATER IN WELL
0021218        DIMENSION R(100),RR(100),D(100),OLDD(100),T(100),H(100),RECH(100
              1 A(100),B(100),C(100),E(100),U(100),V(100),NOB(6),ARRAY(6)
          C  INPUT AQUIFER PARAMETERS
          C
0021218        READ(5,510) PERM,SCON,SUMCON
0045678   511    FORMAT(4F10.5)
0045678        WRITE(6,515) PERM,SCON,SUMCON
0045758   515    FORMAT(1X,24HPERMEABILITY=  ,E12.4,19H CONFINED STORAGE=
              1 E12.4,22H UNCONFINED STORAGE= ,F10.5)
          C
          C  INPUT OF RADII AND SETTING UP OF MESH
0045758        READ(5,500) RWELL,RMAX
0046028   500    FORMAT(2F10.3)
0046028        WRITE(6,505) RWELL,RMAX
0046078   505    FORMAT(1X,24HWELL RADIUS=  ,E12.4,13H MAX RADIUS=  ,E12.4)
          C  SET UP RADIAL MESH
0046078        DO 10 N=1,100
0046078        AN=1.1666666667*FLOAT(N-2)
0046128        R(N)=RWELL*1.0**AN
0046168        IF(R(N).LT.RMAX) GO TO 10
0046218        R(N)=RMAX
0046218        RR(N)=RMAX*RMAX
0046238        NMAX=N
0046243        NMONE=N-1
0046258        GO TO 20
0046268   10   RR(N)=R(N)*R(N)
0046338   20   DELA=0.333782
0046338        DELA2=DELA*DELA
          C
          C  LEVELS MEASURED BELOW DATUM
0046348        READ(5,520) TOP,BASE,HLEVEL,RCH
0046458   520    FORMAT(4F10.0)
0046458        WRITE(6,525) TOP,BASE,HLEVEL,RCH
0046548   525    FORMAT(1X,20HTOP OF AQUIFER  ,E12.4,17H BASE OF AQUIFER ,E12.4,
              1 23H INITIAL WATER LEVEL= ,E12.4,11HRECHARGE= ,E12.4)
          C  SET INITIAL CONDITIONS
0046548        DO 30 N=1,NMAX
0046548        RECH(N)=RCH
0046563        D(N)=HLEVEL
0046578   30   OLDD(N)=HLEVEL
          C
          C  CONDITIONS ON OUTER BOUNDARY
          C  IF JFIX=1 LEVEL AT OUTER BOUNDARY REMAINS AT HLEVEL
0046628        READ(5,530) JFIX
0046678   530    FORMAT(I1)
0046678        IF(JFIX.EQ.1) WRITE(6,531)
0046743        IF(JFIX.NE.1) WRITE(6,532)
0047113   531    FORMAT(22H ** FIXED BOUNDARY **  )
0047118   532    FORMAT(22H ** FREE BOUNDARY **  )
          C
          C  GIVE NODE NUMBER OF SICK OBS WELLS
0047118        READ(5,540) (NOB(J),J=1,6)
0047208   540    FORMAT(6I3)
          C  INPUT ABSTRACTION RATE
0047218        TSTART=0.0
0047288        READ(5,550) QPUMP, TSTOP
0047288
0047288   550    FORMAT(2F10.5)
0047288        WRITE(6,555) QPUMP,TSTOP
0047338   555    FORMAT(1X,19HPUMPING RATE=  ,E12.4,14H TILL TIME OF  ,E12.4,
              1 5H DAYS  )
          C
          C  CONVERTING ABSTRACTION TO RADIUS
0047338        PI=4.0*ATAN(1.0)
0047398        RABST=1.5*QPUMP/(PI*DELA)
0047313        KOUNT=1
          C  INITIALISE TIME AND DELT
0047328        TIME=0.0
0047338        DELT=0.25*RM(1)*SCON/(PERM*(BASE-TOP))
0047418        DELT=DELT
0047418        DO 35 I=1,6
0047433        J=NOB(I)
0047438   35   ARRAY(I)=R(J)
0047478   545    FORMAT(1X,24HTIME  (DAYS)     (PHASE)          ,6F12.4)
```

```
C  CALCULATION FOR A SPECIFIC TIME, IND=1 FOR LAST STEP
0047618  40     TIME=TIME+DELT



0047628          IF(TIME.LT.TSTOP) GO TO 50
0047648          DELT=TSTOP-TIME+DELT
0047678          TIME=TSTOP
0047678          IND=100
C
0047718  50      CONTINUE
C
C  CALCULATION REPEATED FOUR TIMES FOR CONVERGENCE
0047718          DO 60 NUM=1,4
0047723          DO 70 N=1,NMONE
C
C  TAKE AVERAGE SATURATED DEPTH BETWEEN N AND N+1
0047758          SD=BASE-.5*(D(N)+D(N+1))
0050068          STOR=CONST
0050118          IF(SD.LT.(BASE-TOP)) GO TO 5
0050158          SD=BASE-TOP
0050278          STOR=SCON
0050108  60      H(N)=DELA2/(SD*PERM)
0050238  70      T(N)=DELT/(STOR*RR(N))
C
C  TO REPRESENT WATER IN WELL
0050218          H(1)=0.0001*H(1)
0050228          T(1)=2.*DELT*DELA/RR(2)
0050258          T(2)=2.*T(2)
0050278          H(NMAX-1)=(ALOG(R(NMAX))-ALOG(R(NMAX-1)))*(ALOG(R(NMAX))-ALOG
     1  (R(NMAX-1)))/(SD*PERM)
0050448          H(NMAX)=1.0E+5
0050458          T(NMONE)=2.0*DELT*DELA/((R(NMAX)-R(NMONE-1))*STOR*R(NMONE))
0050548          T(NMAX)=2.0*DELT*DELA/((R(NMAX)-R(NMONE))*STOR*R(NMAX))

0050625          IF(JFIX.EQ.1) T(NMAX)=1.0E-10*T(NMAX)
C
C  GAUSSIAN ELIMINATION
C  CALCULATION OF COEFFICIENTS
C  EQUATION -B(N)*D(N-1)+B(N)*D(N)-C(N)*D(N+1)=E(N)
0050648          B(1)=1.0/H(1)+1.0/T(1)
0050708          C(1)=1.0/H(1)
0050738          E(1)=OLD(1)/T(1)+1000T
0050838          DO 80 N=2,NMONE
0050878          A(N)=1.0/H(N-1)
0050968          B(N)=1.0/H(N-1)+1.0/H(N)+1.0/T(N)
0051048          C(N)=1.0/H(N)
0051098  80      E(N)=OLD(N)/T(N)-Q(N)*1000T
0051208          A(NMAX)=1.0/H(NMAX-1)
0051258          B(NMAX)=1.0/H(NMAX-1)+1.5/T(NMAX)
0051338          E(NMAX)=1.5*OLD(NMAX)/T(NMAX)-0.5*R(NMAX)*1000T
C
C  ELIMINATION
0051428          D(1)=B(1)
0051458          Y(1)=E(1)
0051488          DO 100 N=2,NMAX
0051538          D(N)=B(N)-(A(N)*C(N-1))/D(N-1)
0051628  100     Y(N)=E(N)+(A(N)*Y(N-1))/D(N-1)
0051728          D(NMAX)=Y(NMAX)/D(NMAX)
0051778          DO 110 N=1,NMAX
0051828          M=NMONE-N+1
0051878  110     D(M)=(Y(M)+C(M)*D(M+1))/D(M)
C  TEST FOR EXCESSIVE DRAWDOWN
0051958          DRAWMX=1.0*BASE + 1.2*T0
0052028          IF(D(1).LT.DRAW(1)) GO TO 60
0052078          WRITE(6,300)
0052098  90      FORMAT(1X,'EXCESSIVE DRAWDOWN')
0052148          WRITE(6,350)
0052178          DO 120 N=1,NMAX
0052228  120     WRITE(6,370) N,R(N),BE(N),A(N),T(N),D(N)
0052348          STOP
0052368  60      CONTINUE

C  OUTPUT AND CHANGE OF PARAMETERS
0052398          TIMEN=TIME+TSTART
0052428          DO 130 N=1,5
0052468          M=NOB(N)
0052488  130     ARRAY(N)=D(M)
0052548          WRITE(6,500) TIME,TIMEN,J(N),(ARRAY(J),J=1,5),D(NMAX)
0052668  500     FORMAT(1X,2(G12.5,2X))
0052688          DO 135 N=1,NMAX
0052738  135     OLD(N)=D(N)
0052778          DELT=T(1)*1000.0
0052868          IF(IND.EQ.1) GO TO 40
C  END OF CALCULATION FOR A SPECIFIC TIME
```

```
0052508            WRITE(6,565)
0052548   565      FORMAT(1X,8HNODE NO.,4X,5HRADIUS,14X,14HRADIUS SQUARED,6X,
                  1 13HHORIZ HYD RES,7X,15HTIME RESISTANCE,5X,8HDRAWDOWN)



0052548            DO 140 N=1,NMAX
0052568   140      WRITE(6,570) N,R(N),RR(N),H(N),T(N),OLDD(N)
0052778   570      FORMAT(1X,I4,5E20.6)
                   C
                   C  NEW PUMPING PHASE
                   C  RESET PARAMETERS
0052778            DELT=DELI
0053008            IND=0
0053018            TSTART=TIME
0053028            READ(5,525) QPUMP,TSTOP
0053108            WRITE(6,555) QPUMP,TSTOP
0053198            QBSST=1.5*QPUMP/(PI*DELA)
0053218            IF(QPUMP.GE.0.0) GO TO 42
0053268            STOP
0053238            END
```

Pumping Test Program Listing (Rushton and Redshaw 1979)

APPENDIX III

A Formation and Well losses for Abstraction Well no.TW

Step	ΔS_w (m)	S_w (m)	$Q(m^3/sec) \times 10^{-2}$	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	5.45	5.45	1.47	370.2	4.06	1.40	5.46	25.6
2	2.44	7.89	1.96	402.3	5.41	2.50	7.91	31.6
3	3.20	11.09	2.53	438.1	6.98	4.16	11.14	37.3

where,

S_w = Measured drawdown (m)

S_{wc} = Calculated drawdown (m)

Q = Discharge rate (m^3/sec)

S_w/Q = Specific drawdown (sec/m^2)

B = Formation loss constant

C = Well loss constant

BQ = Formation loss (m)

CQ^2 = Well loss (m)

CQ^2/S_{wc} = Fraction of drawdown attributed to well loss.

Illustrative Calculation.

According to Jacob (1947), the total drawdown at a discharging well is given by;

$$S_{wc} = BQ + CQ^2$$

All terms are as defined above.

From Fig. 5.11; B = 276sec/m^2; C = 6500 sec^2/m^5.

For Q = 2.53 x 10^{-2} m^3/sec

$S_{wc} = 276 \times 0.0253 + 6500 \times 0.0253^2$

$= 6.98 + 4.16 = 11.14$ m.

B Formation and Well losses for Abstraction Well no. 3

Step	ΔS_w (m)	S_w (m)	$Q(m^3/sec)$ x 10^{-2}	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	1.29	1.29	0.68	189.6	1.11	0.18	1.29	14
2	3.18	4.47	1.89	236.6	3.08	1.38	4.46	31
3	11.79	16.26	2.35	691.9	4.12	2.14	6.26	34

$B = 163\ sec/m^2$; $C = 3871\ sec^2/m^5$

C Formation - and Well- Losses for Abstraction Well no. 5

Step	ΔS_w (m)	S_w (m)	$Q(m^3/sec)$ x 10^{-2}	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	14.70	14.70	1.69	867.5	9.30	5.33	14.63	36.4
2	1.35	16.05	1.81	888.9	9.96	6.12	16.08	38.1
3	0.20	16.25	1.94	837.6	10.67	7.03	17.70	39.7

$B = 550\ sec/m^2$; $C = 18667\ sec^2/m^5$

D Formation and Well losses for Abstraction Well no. 6

Step	ΔS_w (m)	S_w (m)	$Q(m^3/sec)$ x 10^{-2}	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	9.04	9.04	1.70	531.8	7.41	1.61	9.02	17.8
2	1.22	10.26	1.93	531.6	8.41	2.07	10.48	19.8
3	2.92	13.18	2.33	565.7	10.16	3.02	13.18	22.9

B = 436 sec/m^2; C = 5556 sec^2/m^5

E Formation and Well losses for Abstraction Well no. 12

Step	ΔS_w (m)	S_w (m)	$Q(m^3/sec)$ x 10^{-2}	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	6.05	6.05	1.39	435.6	4.41	1.63	6.04	27
2	3.05	9.10	1.86	489.0	5.90	2.91	8.81	33
3	1.86	10.96	2.18	501.7	6.91	4.00	10.91	37

B = 317 sec/m^2; C = 8421 sec^2/m^5

F Formation and Well losses for Abstraction Well no. 11

Step	S_w (m)	S_w (m)	$Q(m^3/sec)$ x 10^{-2}	S_w/Q (sec/m^2)	BQ (m)	CQ^2 (m)	S_{wc} (m)	CQ^2/S_{wc} %
1	10.47	10.47	1.51	691.6	7.55	2.95	10.50	28.1
2	3.38	13.85	1.79	775.4	8.95	4.15	13.10	31.7
3	4.77	18.62	2.33	800.9	11.65	7.03	18.68	37.6

B = 550 sec/m^2; C = 12941 sec^2/m^5.

APPENDIX IV

Abstraction Well no. TW

A Step Drawdown/Constant Discharge Test: (Cooper-Jacob's Method)

Discharge Q		Time t(min)	Weighted Time (m)	Water Level (m)	Drawdown s (m)	s/Q (min/m^2)
m^3/hr	$m^3/min.$	0	0	45.41	0.0	-
53.0	0.883	1	1	45.61	0.20	0.23
"	"	2	2	48.71	3.30	3.74
"	"	3	3	49.38	3.97	4.49
		4	4	49.71	4.30	4.87
		5	5	49.94	4.53	5.13
		6	6	50.12	4.71	5.33
		7	7	49.71	4.30	4.87
		10	10	49.68	4.27	4.84
		15	15	49.92	4.51	5.11
		20	20	50.23	4.82	5.46
		25	25	50.35	4.94	5.59
		30	30	50.70	5.29	5.99
		40	40	50.86	5.45	6.17
70.6	1.777	41	16.2	51.10	5.69	4.84
"	"	42	19.7	51.29	5.88	5.00
"	"	43	22.1	51.70	6.29	5.35
		44	24.2	51.89	6.48	5.51
		45	26	52.10	6.69	5.69
		46	27.7	52.30	6.89	5.86
		48	30.7	52.41	7.00	5.95
		50	33.5	52.50	7.09	6.03
		55	39.8	52.74	7.33	6.23
		60	45.6	52.91	7.50	6.37
		65	51.2	53.30	7.89	6.71
		74	61	53.22	7.81	6.64
		80	67.3	53.30	7.89	6.71

Q	t	$\bar{t}$	W/Level	s	s/Q
91.14 m^3/hr	81	26.4	53.60	8.19	5.39
(1.519)m^3/min	82	31.2	54.69	9.28	6.11
	83	34.6	54.70	9.29	6.12
	84	37.3	54.91	9.50	6.25
	85	39.7	55.00	9.59	6.31
	86	41.8	55.35	9.94	6.54
	88	45.6	55.33	9.92	6.53
	90	49.0	55.39	9.98	6.57
	95	56.4	55.55	10.14	6.68
	100	63.0	55.67	10.26	6.75
	105	69.3	55.80	10.39	6.84
	110	75.2	55.89	10.48	6.90
	120	86.6	56.50	11.09	7.30
75.0	150	160	53.63	8.22	6.58
(1.25)	155	163.2	53.67	8.26	6.61
	160	166.7	53.71	8.30	6.64
	165	170.5	53.75	8.34	6.67
	170	174.5	53.80	8.39	6.71
	175	178.7	53.84	8.43	6.74
	180	183.0	53.88	8.47	6.78
	185	187.4	53.92	8.51	6.81
	190	191.9	53.96	8.55	6.84
	195	196.4	53.99	8.58	6.86
	200	201	54.01	8.60	6.88
	205	205.6	54.06	8.65	6.92
	210	210.3	54.09	8.68	6.94
	215	215.0	54.12	8.71	6.97
75.0 m^3/hr	230	229.3	54.22	8.81	7.05
(1.25 m^3/min)	245	243.7	54.30	8.89	7.11
	260	258.3	54.36	8.95	7.16
	275	272.9	54.41	9.00	7.20
	305	302.4	54.52	9.11	7.23
	335	332	54.59	9.18	7.34
	395	391.4	54.72	9.31	7.45

Q	t	$\bar{t}$	W/Level	s	s/Q
75.0 m^3/hr	455	451.1	54.88	9.47	7.58
(1.25 m^3/min)	515	510.9	54.95	9.54	7.63
	575	570.7	55.09	9.68	7.74
	635	630.5	55.15	9.74	7.79
	695	690.4	55.25	9.84	7.87
	755	750.3	55.29	9.88	7.90
	815	810.3	55.35	9.94	7.95
	875	870.2	55.37	9.96	7.97
	935	930.2	55.38	9.97	7.98
	955	950.1	55.41	10.00	8.00
	1055	1050.1	55.44	10.03	8.02
	1115	1110.1	55.48	10.07	8.06
	1175	1170	55.56	10.15	8.12
	1235	1230	55.62	10.21	8.17
	1295	1290	55.42	10.01	8.01
	1355	1349.9	55.62	10.21	8.17
	1415	1409.9	56.68	10.27	8.22
	1475	1469.9	5631	10.90	8.72
	1535	1329.9	56.31	10.90	8.72
	1595	1589.9	56.32	10.91	8.73
	1655	1649.9	56.38	10.97	8.78
	1715	1709.8	56.40	10.99	8.79
	1775	1769.8	56.41	11.00	8.80
	1835	1829.8	56.41	11.00	8.80
	1895	1889.8	56.42	11.01	8.81
	1955	1949.8	56.43	11.02	8.82
	2015	2009.8	–	–	–
	2075	2069.8	56.43	11.02	8.82
	2135	2129.8	56.44	11.03	8.82
	2195	2189.8	56.44	11.03	8.82
	2255	2249.8	56.45	11.04	8.83
	2315	2309.8	56.45	11.04	8.83
	2375	2369.8	56.45	11.04	8.83
	2435	2429.7	56.46	11.05	8.84
	2495	2489.7	56.46	11.05	8.84

Q	t	$\bar{t}$	W/Level	s	s/Q
75.0 m^3/hr	2555	2549.7	56.46	11.05	8.84
(1.25 m^3/min)	2615	2609.7	56.47	11.06	8.85
	2675	2669.7	56.47	11.06	8.85
	2735	2729.7	56.47	11.06	8.85
	2795	2789.7	56.47	11.06	8.85
	2855	2849.7	56.48	11.07	8.86
	2915	2909.7	56.47	11.06	8.85
	2975	2969.7	56.48	11.07	8.86
	3035	3029.7	56.48	11.07	8.86
	3275	3269.7	56.48	11.07	8.86
	3515	3509.7	56.49	11.08	8.86
	3755	3749.7	56.48	11.07	8.86
	3995	3989.7	56.48	11.07	8.86
	4235	4229.7	56.48	11.07	8.86

B Recovery Test: Static Water Level Prior to Pumping = 45.41m
Theis Method.

Time t (min)	Time t" (min)	t/t"	Water level s (m)	Residual Drawdown s" (m)
4235	0	∞	56.48	11.07
4235.25	¼	16941	53.74	8.33
4235.50	½	8471	53.18	7.77
4235.75	¾	5647.67	53.07	7.66
4236	1	4236.0	51.28	5.87
4237	2	2118.5	50.20	4.79
4238	3	1412.67	49.85	4.44
4239	4	1059.75	49.64	4.23
4240	5	848	49.45	4.04
4241	6	706.83	49.33	3.92
4242	7	606	49.20	3.79
4243	8	530.38	48.13	2.72
4244	9	471.56	48.05	2.64
4245	10	424	47.97	2.56
4247	12	353.92	47.84	2.43
4249	14	303.50	47.72	2.31
4251	16	265.69	47.62	2.21
4253	18	236.28	47.53	2.12
4255	20	212.75	47.44	2.03
4260	25	170.4	47.27	1.86
4265	30	142.17	47.12	1.71
4270	35	122	47.00	1.59
4275	40	106.88	46.88	1.47
4285	50	85.7	46.72	1.31
4295	60	71.58	46.28	0.87
4310	79	57.47	46.25	0.84
4325	90	48.06	46.11	0.70
4340	105	41.33	46.02	0.61
4355	120	36.29	45.84	0.43

Time t (min)	Time t" (min)	t/t"	Water Level (m)	Residual D. s" (m)
4385	150	29.23	45.70	0.29
4415	180	24.28	45.57	0.16
4445	210	21.17	45.48	0.07
4475	240	18.65	45.45	0.04
4505	270	16.69	45.42	0.01
4535	300	15.12	45.41	0.00
4565	330	13.83	45.41	0.00

NB

t = Time since pumping started.

t" = Time since pumping stopped

s" = Residual drawdown (m) i.e difference between the water level prior to pumping and the actual water level measured at time t" since pumping stopped.

ABSTRACTION WELL No. 3

A Step Drawdown/Constant discharge Test:

Discharge Q		Time t (min)	Weighted Time t (min)	W/Level (m)	Drawdown s (m)	s/Q
(m^3/hr)	(m^3/min)	0	0	48.50	0	0
24.5	0.408	1	1	51.18	2.68	6.56
		2	2	51.20	2.70	6.61
		3	3	50.37	1.87	6.58
		4	4	50.38	1.88	4.60
		5	5	50.38	1.88	4.60
		6	6	50.65	2.15	5.27
		7	7	51.36	2.86	7.00
		8	8	51.75	3.25	7.96
		9	9	51.70	3.20	7.84
		10	10	51.90	3.40	8.33
		12	12	-		
		14	14	-		
		16	16			
		18	18			
		20	20			
		25	25			
		30	30	-	-	-
68.0	1.133	31	3.5	51.79	3.29	2.90
		32	5.4	52.91	4.41	3.89
		33	7.1	52.42	3.92	3.46
		34	8.6	-		
		35	10.1	-		
		36	11.4	52.33	3.83	3.38
		37	12.8	52.37	3.87	3.41
		38	14.0	52.43	3.93	3.47
		39	15.3	52.46	3.96	3.49
		40	16.5	52.47	3.97	3.50
		42	18.8	52.52	4.02	3.55

Q	t	$\bar{t}$	W/level	s	s/Q
68.0 m^3/hr	44	21.5	52.57	4.07	3.19
1.133 m^3/min	46		-		
	48	25.6	52.60	4.10	3.62
	50	27.8	52.60	4.10	3.62
	55	33.2	52.72	4.22	3.72
	60	38.5	52.97	4.47	3.94
84.6 m^3/hr	75	45.6	64.49	15.99	11.34
(1.41 m^3/min)	80	47.9	64.52	16.02	11.36
	85	53.4	64.62	16.12	11.43
	90	58.9	64.76	16.26	11.53
	97	66.4	64.85	16.35	11.60
	100	69.5	64.95	16.45	11.67
	110	80	65.02	16.52	11.72
	120	90.3	65.12	16.62	11.79
	130	100.6	65.25	16.75	11.88
	140	110.8	65.35	16.85	11.95
	150	121	65.35	16.85	11.95
	160	131.1	65.43	16.93	12.01
	180	151.4	65.52	17.02	12.07
	200	171.5	65.63	17.13	12.15
	220	191.7	65.77	17.27	12.25
	250	221.8	65.86	17.36	12.31
	280	251.9	65.99	17.49	12.40
	310	292.0	66.12	17.62	12.50
	340	312.1	66.24	17.74	12.58
	400	372.2	66.35	17.85	12.66
	460	432.3	66.66	18.16	12.88
	520	492.4	66.68	18.18	12.89
	580	552.4	66.76	18.26	12.95
84.6 m^3/hr	640	612.5	66.88	18.38	13.04
(1.41 m^3/min)	700	672.5	66.93	18.43	13.07
	760	732.5	67.02	18.52	13.13
	820	792.5	67.11	18.61	13.20
	880	852.5	67.15	18.65	13.23
	940	912.6	67.25	18.75	13.30

Q	t	$\bar{t}$	W/level	s	s/Q
84.6 m^3/hr	1000	972.6	67.29	18.79	13.33
(1.41 m^3/min)	1060	1032.6	67.34	18.84	13.36
	1120	1092.6	67.39	18.89	13.40
	1180	1152.6	67.42	18.92	13.42
	1240	1212.6	67.44	18.94	13.43
	1300	1272.6	67.49	18.99	13.47
	1360	1332.6	67.52	19.02	13.49
	1420	1393.6	67.54	19.04	13.50
	1480	1452.7	67.57	19.07	13.52
	1540	1512.7	67.58	19.08	13.53
	1600	1572.7	67.60	19.10	13.55
	1660	1632.7	67.62	19.12	13.56
	1720	1692.7	67.64	19.14	13.57
	1780	1752.7	67.63	19.13	13.57
	1840	1812.7	67.64	19.14	13.57
	1900	1872.7	67.66	19.16	13.59
	1960	1932.7	67.67	19.17	13.60
	2020	1992.7	67.68	19.18	13.60
	2080	2052.7	67.70	19.20	13.62
	2140	2112.7	67.71	19.21	13.62
	2200	2172.7	67.73	19.23	13.64
	2260	2232.7	67.75	19.25	13.65
	2320	2292.7	67.77	19.27	13.67
	2380	3252.7	67.81	19.31	13.70
	2440	2412.7	67.81	19.31	13.70
	2500	2472.7	67.82	19.32	13.70
	2560	2532.7	67.81	19.31	13.70
	2620	2592.7	67.83	19.33	13.71
	2680	2652.7	67.84	19.34	13.72
	2740	2712.7	67.86	19.36	13.73
	2800	2772.7	67.87	19.37	13.74
	2860	2832.7	67.89	19.39	13.75
	2940	2912.7	67.87	19.37	13.74

B Recovery Test:

Time, t" (min)	Time t (min)	t/t"	Water Level	Residual Drawdown s " (m)
0	2940		67.87	19.37
1	2941	2941	55.73	7.23
2	2942	1471	54.94	6.44
3	2943	981	54.55	6.05
4	2944	736	54.25	5.75
5	2945	589	54.04	5.54
6	2946	491	53.86	5.36
7	2947	421	53.71	5.21
8	2948	368.5	53.58	5.08
9	2949	327.67	53.46	4.96
10	2950	295	53.35	4.85
12	2952	246	53.16	4.66
14	2954	211	53.00	4.50
16	2956	184.75	52.86	4.36
18	2958	164.33	52.72	4.22
20	2960	148	52.62	4.12
25	2965	118.6	52.36	3.86
30	2970	99	52.12	3.62
35	2975	85	51.96	3.46
40	2980	74.5	51.81	3.31
50	2990	59.8	51.55	3.05
60	3000	50	51.25	2.75
70	3070	43.86	51.10	2.60
80	3080	38.50	51.09	2.59

ABSTRACTION WELL NO. 5

A Step Drawdown/Constant Discharge Test:

Discharge Q(m)	Time t(min)	Weight Time $\bar{t}$ (min)	Water Level (m)	Drawdown s (m)	s/Q
61.0 (m^3/hr)	0	0	43.62	0.0	0.0
(1.017 m^3/min)	1	1	57.98	14.36	14.12
	2	2	58.20	14.58	14.34
	3	3	57.75	14.13	13.89
	4	4	57.78	14.16	13.92
	10	10	58.00	14.38	14.14
	12	12	58.02	14.40	14.15
	14	14	58.06	14.44	14.20
	16	16	58.09	14.47	14.23
	18	18	58.12	14.50	14.26
	20	20	-	-	-
	25	25	58.26	14.64	14.40
	30	30	58.32	14.70	14.45
65 (m^3/hr)	31	25.1	58.92	15.30	14.13
1.083(m^3/min)	32	24.0	59.02	15.40	14.22
	33	28.5	59.05	15.43	14.25
	34	29.8	59.07	15.45	14.26
	35	31.1	59.09	15.47	14.28
	36	32.2	59.11	15.49	14.30
	37	33.4	59.12	15.50	14.31
	39	34.8	59.15	15.53	14.34
	40	36.7	59.16	15.54	14.35
	42	38.9	59.15	15.53	14.34
	50	47.3	59.32	15.70	14.50
	55	52.4	59.49	15.87	14.65
	60	57.5	59.67	16.05	14.82
69.9(m^3/hr(	61	-	-		
1.165(m^3/min)	62	-	-		
	63	-	-		

Q	t(min)	$\bar{t}$(min)	W/level	s (m)	s/Q
69.9 (m^3/hr)	64	50.82	59.52	15.90	13.32
1.165 (m^3/hr)	66	53.9	59.52	15.90	13.32
	67	55.3	59.52	15.90	13.32
	68	56.6	59.51	15.89	13.64
	69	57.9	59.55	15.93	13.67
	70	59.15	59.15	15.93	13.67
	72	61.6	-	-	
	76	66.2	59.69	16.07	13.79
	78	68.5	59.76	16.14	13.85
	80	70.7	59.80	16.18	13.89
	85	76.1	59.81	16.19	13.90
	90	81.4	59.87	16.25	13.95
	100	91.9	60.26	16.64	14.28
	120	112.4	60.90	17.28	14.83
	150	142.9	61.07	17.45	14.98
	180	173.1	61.18	17.56	15.07
	210	203.3	61.22	17.60	15.11
	250	243.4	61.30	17.68	15.18
	290	283.5	61.32	19.70	15.19
	330	323.6	61.32	17.70	15.19
	390	383.7	61.40	17.78	15.26
	450	443.8	61.49	17.87	15.34
	510	503.8	61.53	17.91	15.37
	570	563.8	61.55	17.93	15.39
	630	623.8	61.57	17.25	15.41
	690	683.9	61.60	17.98	15.43
	750	743.9	61.64	18.02	15.47
	810	803.9	61.67	18.05	15.49
	870	863.9	61.67	18.05	15.49
	930	923.9	61.67	18.05	15.49
	990	983.9	61.65	18.03	15.48
	1050	1043.9	61.70	18.08	15.52
	1110	1104	61.78	18.16	15.59
	1170	1164	61.63	18.01	15.46
	1230	1224	61.61	17.99	15.44

Q	t (min)	$\bar{t}$ (min)	W/level	s (m)	s/Q
69.9 (m^3/hr)	1290	1284	61.60	17.98	15.43
1.165 (m^3/min)	1350	1344	61.59	17.97	15.42
	1410	1404	61.60	17.98	15.43
	1470	1464	61.59	17.97	15.42
	1530	1524	61.64	18.02	15.47
	1590	1584	61.75	18.13	15.56
	1650	1644	61.77	18.15	15.58
	1710	1704	61.81	18.19	15.61
	1770	1764	61.84	18.22	15.64
	1830	1824	61.84	18.22	15.64
	1890	1884	61.85	18.23	15.65
	1950	1944	61.85	18.23	15.65
	2010	2004	61.85	18.23	15.65
	2070	2064	61.85	18.23	15.65
	2130	2124	61.86	18.24	15.66
	2190	2184	61.87	18.25	15.67
	2250	2244	61.87	18.25	15.67
	2310	2304	61.90	18.28	15.69
	2370	2364	62.10	18.48	15.86
	2430	2424	62.17	18.55	15.92
	2490	2484	62.22	18.60	15.97
	2550	2544	62.23	18.61	15.97
	2610	2604	62.26	18.64	16.00
	2670	2664	62.26	18.64	16.00
	2730	2724	62.30	18.68	16.03
	2790	2784	62.30	18.68	16.03
	2850	2844	62.28	18.66	16.02
	2910	2904	62.20	18.58	15.95
	2970	2964	62.23	18.61	15.97
	3030	3024	62.31	18.69	16.04
	3090	3084	62.40	18.78	16.12
	3150	3144	62.48	18.86	16.19
	3210	3204	62.48	18.86	16.19
	3270	3264	62.43	18.81	16.15

Q	t (min)	$\bar{t}$ (min)	W/level	s (m)	s/Q
69.9 m^3/hr	3330	3324	62.38	18.76	16.10
1.165 (m^3/min.)	3390	3384	62.38	1876	16.10
	3450	3444	62.38	18.76	16.10
	3510	3504	62.39	18.77	16.11
	3570	3564	62.38	18.76	16.10
	3630	3624	62.38	18.76	16.10
	3690	3684	62.37	18.75	16.09
	3750	3744	62.38	18.76	16.10
	3810	3804	62.38	18.76	16.10
	3870	3864	62.38	18.76	16.10
	3930	3924	62.41	18.79	16.13

B Recovery Test:

Original Water Level Prior to Pumping (i.e Static Water Level) = 43.62m

Time t" (min)	t (min)	t/t"	Water level (m)	Residual Drawdown s_{r}" (m)
0	3930		62.41	18.79
1	3931	3931	50.22	6.60
2	3932	1966	49.78	6.16
3	3933	1311	49.58	5.96
4	3934	983.5	49.40	5.78
5	3935	787	49.28	5.66
6	3936	656	49.15	5.53
7	3937	562.4	49.08	5.46
8	3938	492.3	49.00	5.38
9	3939	437.7	48.90	5.28
10	3940	394	48.85	5.23
12	3942	328.5	48.76	5.14
14	3944	281.7	48.68	5.06
16	3946	246.6	48.60	4.98
18	3948	219.3	48.55	4.93
20	3950	197.5	48.48	4.86
25	3955	158.2	48.40	4.78
30	3960	132	48.28	4.66
40	3970	99.3	48.14	4.52
50	3980	79.6	48.08	4.46
60	3990	66.5	48.00	4.38
70	4000	57.1	47.88	4.26
80	4010	50.1	47.82	4.20
95	4025	42.4	47.75	4.13
110	4040	36.7	47.66	4.04
125	4055	32.4	47.62	4.00

OBSERVATION WELL NO. 3

A Distance from Abstraction Well (AW5) r = 100m : Step Drawdown/ Constant Discharge Test

Discharge, Q	Time t (min)	$\bar{t}$	W/level	s (m)	s/Q
61.0 m^3/hr	0	0	47.47	0.00	0.00
	30	30	-		
69.9 m^3/min	61	44	48.47	1.00	0.923^o
	85	76.1	48.94	1.47	1.262
	110	102.2	49.16	1.69	1.451
	132	124.7	49.30	1.83	1.571
	162	155	49.42	1.95	1.674
	186	179.2	49.61	2.14	1.837
	203	196.3	49.69	2.22	1.906
	242	235.4	49.72	2.25	1.931
	277	270.5	49.74	2.27	1.948
	313	306.6	49.76	2.29	1.966
	353	346.7	49.82	2.35	2.017
	443	436.7	49.93	2.46	2.112
	503	496.8	49.99	2.52	2.163
	563	556.8	50.01	2.54	2.180
	623	616.8	50.03	2.56	2.197
	683	676.9	50.05	2.58	2.215
	743	736.9	50.10	2.63	2.258
	803	796.9	50.14	2.67	2.292
	863	856.9	50.18	2.71	2.326
	923	136.9	50.20	2.73	2.343
	983	976.9	50.25	2.78	2.386
	1043	1036.9	50.25	2.78	2.386
	1103	1097	50.27	2.80	2.403
	1163	1157	50.30	2.83	2.429
	1223	1217	50.37	2.90	2.489
	1283	1277	50.42	2.95	2.532

Discharge, Q	Time t (min)	$\bar{t}$	W/level	$\bar{S}$ (m)	s/Q
61.0 m^3/hr	3383	3377	51.15	3.68	3.159
	3443	3437	51.15	3.68	3.159
69.9 m^3/min	3503	3497	51.13	3.66	3.142
	3563	3557	51.14	3.67	3.150
	3623	3617	51.13	3.66	3.142
	3683	3677	51.13	3.66	3.142
	3743	3737	51.13	3.66	3.142
	3803	3797	51.13	3.66	3.142
	3863	3857	51.15	3.68	3.159
	3923	3917	51.17	3.70	3.176

OBSERVATION WELL NO.4 - 200m away from AW5

A Distance from Test Well, AW5, = 200m
Static Water Level = 43.7m

Step Drawdown/Constant Discharge Test

Discharge, Q	Time t (min)	Weighted Time $\bar{t}$	W/Level	D/Down s	s/Q
61 m^3/hr	0	0	43.70	0.00	0.00
	20	20	43.82	0.12	0.118
	30	30	43.91	0.21	0.207
65	60	57.5			
69.9	90	81.4	44.63	0.93	0.798
	120	112.4	44.82	1.12	0.961
	150	142.9	44.96	1.26	1.082
	210	203	45.15	1.45	1.245
	270	263.5	45.28	1.58	1.356
	330	323.6	45.36	1.66	1.425
	390	383.7	45.43	1.73	1.485
	450	443.8	45.43	1.79	1.536
	510	503.8	45.54	1.84	1.579
	570	563.8	45.58	1.88	1.614
	630	623.8	45.61	1.91	1.639
	690	683.9	45.68	1.98	1.700
	750	743.9	45.72	2.02	1.734
	810	803.9	45.76	2.06	1.768
	870	863.9	45.78	2.08	1.785
	930	923.9	45.82	2.12	1.820
	990	983.9	45.86	2.16	1.854
	1050	1043.9	45.91	2.21	1.897
	1110	1104	45.95	2.25	1.931
	1170	1164	45.97	2.27	1.948
	1230	1224	45.98	2.28	1.957
	1290	1284	45.99	2.29	1.966

Discharge, Q	t	$\bar{t}$	W/Level	D/Down	s/Q
69.9	1350	1344	46.00	2.30	1.974
	1410	1404	46.02	2.31	1.983
	1470	1464	46,04	2.34	2.009
	1530	1524	46.11	2.41	2.069
	1590	1584	46.18	2.48	2.129
	1650	1644	46.22	2.52	2.163
	1710	1704	46.24	2.54	2.180
	1770	1764	46.26	2.56	2.197
	1830	1824	46.27	2.57	2.206
	1890	1884	46.27	2.57	2.206
	1950	1944	46.27	2.57	2.206
	2010	2004	46.29	2.59	2.223
	2070	2064	46.29	2.59	2.223
	2130	2124	46.30	2.60	2.232
	2190	2184	46.30	2.60	2.232
	2250	2244	46.31	2.61	2.240
	2310	2304	46.32	2.62	2.249
	2370	2364	46.34	2.64	2.266
	2430	2424	46.38	2.68	2.300
	2490	2484	46.43	2.73	2.343
	2550	2544	46.46	2.76	2.369
	2610	2604	46.49	2.79	2.395
	2670	2664	46.51	2.81	2.412
	2730	2724	46.51	2.81	2.412
	2790	2784	46.52	2.82	2.421
	2850	2844	46.52	2.82	2.421
	2910	2904	46.52	2.82	2.421
	2970	2964	46.55	2.85	2.446
	3030	3024	46.59	2.89	2.481
	3090	3084	46.61	2.91	2.498
	3150	3144	46.65	2.95	2.532
	3210	3204	46.65	2.95	2.532

Discharge, Q	t	$\bar{t}$	W/Level	D/Down	s/Q
69.9	3270	3264	46.65	2.95	2.532
	3330	3324	46.67	2.97	2.549
	3390	3384	46.67	2.97	2.549
	3450	3444	46.67	2.97	2.549
	3510	3504	46.67	2.97	2.549
	3570	3564	46.67	2.97	2.549
	3630	3624	46.67	2.97	2.549
	3690	3684	46.67	2.97	2.549
	3750	3744	46.68	2.98	2.558
	3810	3804	46.68	2.98	2.558
	3870	3864	46.70	3.00	2.575
	3930	3924	46.73	3.03	2.601

ABSTRACTION WELL NO. 6

A Step Drawdown/Constant Discharge Test

Discharge Q		Time t(min)	Weighted Time t(min)	W/Level (m)	D/Down s (m)	s/Q
m^3/hr	m^3/min	0	0	45.92	0.0	0.0
61	1.017	1	1	55.27	9.35	9.19
		2	2	55.98	10.06	9.89
		3	3	56.30	10.38	10.21
		4	4	56.53	10.61	10.43
		5	5	56.66	10.74	10.56
		6	6	56.38	10.46	10.29
		7	7	56.36	10.44	10.27
		8	8	56.35	10.43	10.26
		9	9	56.13	10.21	10.04
		10	10	55.56	9.64	9.48
		11	11	55.35	9.43	9.27
		12	12	55.18	9.26	9.11
		13	13	54.95	9.03	8.88
		14	14	54.66	8.74	8.59
		15	15	54.50	8.58	8.44
		16	16	54.51	8.59	8.45
		17	17	54.50	8.58	8.44
		18	18	54.52	8.60	8.46
		19	19	54.53	8.61	8.47
		20	20	54.58	8.66	8.51
		21	21	54.57	8.65	8.51
		22	22	54.64	8.72	8.57
		23	23	54.62	8.70	8.55
		24	24	54.63	8.71	8.56
		25	25	54.70	8.78	8.63
		28	28	54.77	8.85	8.70
		30	30	54.80	8.88	8.73

Discharge, Q		t	$\bar{t}$	W/Level	S	s/Q
m^3/hr	m^3/min	35	35	54.83	8.91	8.76
61	1.017	40	40	54.90	8.98	8.83
		45	45	54.96	9.04	8.89
69.6	1.16	46	28.7	55.07	9.15	7.89
		47	31.8	55.09	9.17	7.91
		48	34.1	55.11	9.19	7.92
		49	36.0	55.13	9.21	7.94
		50	37.6	55.14	9.22	7.95
		51	39.1	55.26	9.34	8.05
		52	40.6	55.33	9.41	8.11
		53	42.0	55.45	9.53	8.22
		54	43.3	55.48	9.56	8.24
		55	44.6	55.60	9.68	8.34
		56	45.8	55.64	9.72	8.38
		57	47.0	55.73	9.81	8.46
		58	48.2	55.78	9.86	8.50
		59	49.4	55.81	9.89	8.53
		60	50.6	55.82	9.90	8.53
		61	51.7	55.84	9.92	8.55
		62	52.8	55.88	9.96	8.59
		63	54.0	55.86	9.94	8.57
		64	55.1	55.90	9.98	8.60
		65	56.2	55.98	10.06	8.67
		70	61.6	55.94	10.02	8.64
		75	67.0	56.00	10.08	8.69
		80	72.2	56.06	10.14	8.74
		85	77.4	56.10	10.18	8.78
		90	82.6	56.18	10.26	8.84
		91	39.3	58.12	12.20	8.72
83.9	1.398	92	44.7	58.27	12.35	8.83
		93	48.4	58.35	12.43	8.89
		94	51.3	58.41	12.49	8.93
		95	52.6	58.46	12.54	8.97
		96	56.1	58.51	12.59	9.00
		97	58.1	58.57	12.65	9.05
		98	60.0	58.61	12.69	9.08
		99	61.8	58.60	12.68	9.07

Discharge, Q		t	$\bar{t}$	W/Level	s	s/Q
m^3/hr	m^3/min	100	63.5	58.63	12.70	9.09
83.9	1.398	102	66.7	58.64	12.72	9.10
		106	72.6	58.74	12.82	9.17
		110	77.9	58.81	12.89	9.22
		115	84.3	58.92	13.00	9.30
		120	90.3	58.99	13.07	9.35
		130	101.8	59.07	13.15	9.40
		140	112.9	59.10	13.18	9.43
		150	123.7	59.18	13.26	9.48
		160	134.3	59.25	13.33	9.53
		180	155.3	59.40	13.48	9.64
		210	186.2	59.52	13.60	9.73
		230	206.7	59.63	13.71	9.80
		260	237.2	60.26	14.34	10.26
		290	267.5	60.67	14.75	10.55
		320	297.8	61.20	15.28	10.93
		350	328.0	61.40	15.48	11.07
		410	388.4	61.68	15.76	11.27
		470	448.6	61.79	15.87	11.35
		530	508.8	61.87	15.95	11.41
		590	568.9	62.00	16.08	11.50
		650	629.1	62.12	16.20	11.59
		710	689.1	62.30	16.38	11.71
		770	749.2	62.46	16.54	11.83
		830	809.3	62.50	16.58	11.86
		890	869.3	62.54	16.62	11.89
		950	929.4	62.59	16.67	11.92
		1010	989.4	62.65	16.73	11.96
		1070	1049.5	62.68	16.76	11.99
		1130	1009.5	62.69	16.77	11.99
		1190	1169.5	62.70	16.78	12.00
		1250	1229.6	62.72	16.80	12.01
		1310	1289.6	62.77	16.85	12.05
		1370	1349.6	62.88	16.96	12.13
		1430	1409.6	62.93	17.01	12.16

Discharge, Q		t	$\bar{t}$	W/Level	S	s/Q
m^3/hr	m^3/min	1490	1469.6	62.87	16.95	12.12
83.9	1.398	1610	1589.7	62.63	16.71	11.95
		1730	1709.7	63.31	17.39	12.44
		1790	1769.7	63.37	17.45	12.48
		1910	1889.7	63.49	17.57	12.56
		2030	2009.7	63.47	17.55	12.55
		2090	2069.8	63.57	17.59	12.58
		2270	2749.8	63.60	17.68	12.64
		2330	2309.8	63.62	17.70	12.66
		2450	2429.8	63.66	17.74	12.69
		2510	2489.8	63.68	17.76	12.70
		2630	2609.8	63.71	17.79	12.72
		2690	2669.8	63.71	17.79	12.72
		27.50	2729.8	63.77	17.85	12.77
		2810	2789.8	63.73	17.81	12.74
		2870	2849.8	63.85	17.93	12.82
		2930	2909.8	63.82	17.90	12.80
		2990	2969.8	63.82	17.90	12.80
		3050	3029.8	63.86	17.94	12.83
		3110	3089.9	63.88	17.96	12.84
		3170	3149.9	63.93	18.01	12.88
		3230	3209.9	63.96	18.04	12.91

B Recovery Test

Static Water Level Prior to Pumping = 45.92m

Time, t" (min)	Time, t (min)	t/t"	W/Level (m)	Residual Drawdown s_r" (m)
0	3230		63.96	18.04
1	3231	3731	55.05	9.13
2	3232	1616	54.09	8.17
3	3233	1077.67	53.83	7.91
4	3234	808.5	53.48	7.56
5	3235	647	53.23	7.31
6	3236	539.33	52.97	7.05
7	3237	462.43	52.84	6.92
8	3238	404.75	52.71	6.79
9	3239	359.89	52.56	6.64
10	3240	324	52.42	6.50
12	3242	270.17	52.01	6.09
14	3244	231.71	51.83	5.91
16	3246	202.88	51.48	5.54
18	3248	180.44	51.37	5.45
20	3250	162.5	51.28	5.36
25	3255	130.5	51.06	5.14
30	3260	108.67	50.89	4.97
35	3265	93.29	50.75	4.83
40	3270	81.75	50.56	4.64
50	3280	65.60	50.38	4.46
60	3290	54.83	50.19	4.27
70	3300	47.14	50.03	4.03
80	3310	41.38	49.90	3.98
100	3330	33.00	49.68	3.76
120	3350	27.92	49.51	3.59
140	3370	24.07	49.36	3.44
170	3400	20	49.18	3.26
200	3430	17.15	49.15	3.23
230	3460	15.04	49.15	3.23

Time, t" (min)	Time, t (min)	t/t"	W/Level (m)	Residual Drawdown s" (m)
260	3490	13.42	49.12	3.20
290	3520	12.14	49.09	3.17
320	3550	11.09	49.03	3.11
380	3610	9.50	48.86	2.94
440	3670	8.34	48.74	2.82
500	3730	7.46	48.68	2.76
560	3790	6.77	48.43	2.51
620	3850	6.21	48.24	2.32
680	3910	5.75	48.10	2.18
740	3970	5.36	47.95	2.03
800	4030	5.04	47.86	1.94
860	4090	4.76	47.70	1.78
920	4150	4.51	47.64	1.72

ABSTRACTION WELL No. 11

A Step Drawdown/Constant Discharge Test: (Cooper-Jacob's Method)

Discharge Q		Time t (min)	Weighted Time $\bar{t}$ (min)	W/Level (m)	Drawdown s_w (m)	s/Q
m^3/hr	m^3/min	0	0	40.22	0.00	0.00
54.50	0.908	1	1	46.54	6.32	6.96
		2	2	47.00	6.78	7.46
		3	3	47.69	7.45	8.20
		4	4	48.05	7.83	8.62
		5	5	48.35	8.13	8.95
		6	6	48.38	8.16	8.98
		7	7	48.83	8.61	9.48
		8	8	49.05	8.83	9.72
		9	9	49.20	8.98	9.89
		10	10	49.32	9.10	10.02
		12	12	49.39	9.17	10.10
		14	14	49.62	9.40	10.35
		16	16	49.82	9.60	10.57
		18	18	49.93	9.71	10.69
		20	20	50.12	9.90	10.90
		25	25	50.45	10.23	11.26
		30	30	50.69	10.47	11.53
64.30	1.072	31	18.4	51.48	11.26	10.51
		32	21	51.61	11.39	10.63
		33	22.9	51.91	11.69	10.91
		34	24.5	52.02	11.80	11.01
		35	26.0	52.42	12.20	11.38
		36	27.4	52.71	12.49	11.65
		37	28.7	52.84	12.62	11.78
		38	30.0	52.94	12.92	11.87

Discharge, Q		t	$\bar{t}$	W/Level	s	s/Q
m^3/hr	m^3/min	39	31.2	53.05	12.83	11.97
64.30	1.072	40	32.4	53.13	12.91	12.05
		42	34.7	53.24	13.02	12.15
		44	37.0	53.38	13.16	12.28
		46	39.2	53.53	13.31	12.42
		48	41.3	53.60	13.38	12.49
		50	43.5	53.68	13.46	12.56
		55	48.8	53.89	13.67	12.76
		60	54	54.07	13.85	12.92
83.70	1.395	61	21.7	-		
		62	25.9	54.60	14.38	10.31
		63	28.8	55.93	15.71	11.26
		64	31.3	57.18	16.96	12.16
		65	33.4	57.29	17.07	12.24
		66	35.3	57.50	17.28	12.39
		67	37.0	-		
		68	38.7	57.62	17.40	12.47
		69	40.3	57.78	17.56	12.59
		70	41.8	57.81	17.59	12.61
		72	44.6	57.99	17.77	12.74
		74	47.3	58.10	17.88	12.82
		76	49.9	58.18	17.96	12.87
		78	52.5	58.24	18.02	12.92
		80	54.9	58.34	18.12	12.99
		85	60.8	58.56	18.34	13.15
		90	66.5	58.84	18.62	13.35
		95	72.1	58.98	18.76	13.45
		100	77.6	59.10	18.85	13.53
		105	82.9	59.30	19.08	13.68
		110	88.3	59.42	19.20	13.76
		120	98.8	59.66	19.44	13.94
		130	109.2	59.80	19.58	14.04
		140	119.5	59.99	19.77	14.17
		150	129.8	60.12	19.90	14.27

Discharge, Q		t	$\bar{t}$	W/Level	S	s/Q
m^3/hr	m^3/min	170	150.2	60.31	20.09	14.40
83.70	1.395	190	170.5	60.48	20.26	14.52
		210	190.8	60.66	20.44	14.65
		240	221.0	60.85	20.63	14.79
		270	251.2	61.02	20.80	14.91
		300	281.4	61.13	20.91	14.99
		330	311.5	61.23	21.01	15.06
		390	371.7	61.43	21.21	15.20
		450	431.8	61.58	21.36	15.31
		510	491.9	61.75	21.53	15.43
		570	552.0	61.85	21.63	15.51
		630	612.0	61.94	21.72	15.57
		690	672.1	61.93	21.71	15.56
		750	732.1	62.05	21.83	15.65
		810	792.2	62.10	21.88	15.68
		870	852.2	62.12	21.90	15.70
		930	912.2	62.14	21.92	15.71
		990	972.2	62.15	21.93	15.72
		1050	1032.3	62.15	21.93	15.72
		1110	992.3	62.15	21.93	15.72
		1170	1152.3	62.22	22.00	15.77
		1230	1212.3	62.25	22.03	15.79
		1290	1272.3	62.26	22.04	15.80
		1350	1332.3	62.26	22.04	15.80
		1410	1392.3	62.21	21.99	15.76
		1470	1452.4	62.26	22.04	15.80
		1530	1512.4	62.26	22.04	15.80
		1590	1572.4	62.29	22.07	15.82
		1650	1632.4	62.33	22.11	15.85
		1710	1692.4	62.38	22.16	15.89
		1770	1752.4	62.38	22.16	15.89
		1830	1812.4	62.38	22.16	15.89
		1890	1872.4	62.37	22.15	15.88
		1950	1932.4	62.38	22.16	15.89

Discharge, Q		t	$\bar{t}$	W/Level	S	s/Q
m^3/hr	m^3/min	2010	1992.4	62.37	22.15	15.88
83.70	1.395	2070	2052.4	62.36	22.14	15.87
		2130	2112.4	62.36	22.14	15.87
		2190	2172.4	62.36	22.14	15.87
		2250	2232.4	62.36	22.14	15.87
		2310	2292.4	62.36	22.14	15.87
		2370	2352.4	62.36	22.14	15.87
		2430	2412.4	62.36	22.14	15.87
		2490	2472.5	62.36	22.14	15.87
		2550	2532.5	62.36	22.14	15.87
		2610	2592.5	62.36	22.14	15.87
		2670	2652.5	62.37	22.15	15.88
		2730	2712.5	62.37	22.15	15.88
		2790	2772.5	62.37	22.15	15.88
		2850	2832.5	62.37	22.15	15.88
		2910	2892.5	62.37	22.15	15.88
		2970	2952.5	62.37	22.15	15.88

ABSTRACTION WELL NO. 12

A Step Drawdown/Constant Discharge Test: (Cooper-Jacob's Method).

Discharge Q		Time t (min)	Weighted Time t (min)	W/Level (m)	Drawdown s (m)	Specific Drawdown s/Q
m^3/hr	m^3/min	0	0	48.66	0.00	0.00
50.0	0.833	1	1	53.48	4.82	5.78
		2	2	53.64	4.98	5.98
		3	3	53.83	5.17	6.20
		4	4	53.92	5.26	6.31
		5	5	53.98	5.32	6.38
		6	6	54.10	5.44	6.53
		7	7	54.16	5.50	6.60
		8	8	54.22	5.56	6.67
		9	9	54.36	5.70	6.84
		10	10	54.39	5.73	6.88
		12	12	54.55	5.89	7.07
		14	14	54.58	5.92	7.10
		16	16	54.66	6.00	7.20
		18	18	54.76	6.10	7.32
		20	20	54.79	6.13	7.36
		25	25	54.81	6.15	7.38
		30	30	54.71	6.05	7.26
67.0	1.117	31	13.0	56.23	7.57	6.78
		32	15.8	56.27	7.61	6.81
		33	18.0	56.30	7.64	6.84
		34	19.8	56.69	7.43	6.65
		35	21.4	57.00	8.34	7.46
		36	22.8	57.79	8.13	7.28
		37	24.3	57.83	8.22	7.36
		38	25.6	57.02	8.36	7.49
		39	26.9	57.10	8.44	7.56
		40	28.1	57.11	8.45	7.57
		42	30.6	57.18	8.52	7.63

Discharge Q		t (min)	$\bar{t}$ (min)	W/level	s (m)	s/Q
m^3/hr	m^3/min	44	32.9	57.35	8.69	7.78
67.0	1.117	46	35.2	57.45	8.79	7.87
		48	37.4	57.52	8.86	7.93
		50	39.6	57.54	8.88	7.95
		55	45.0	57.67	9.01	8.06
		60	50.3	57.76	9.10	8.15
78.65	1.311	61	28.7	58.73	10.07	7.68
		62	32.3	58.80	10.14	7.74
		63	34.9	58.88	10.22	7.80
		64	37.0	58.92	10.26	7.83
		65	38.9	58.94	10.28	7.84
		66	40.6	58.96	10.30	7.86
		67	42.2	58.97	10.31	7.87
		68	43.7	58.02	10.36	7.90
		69	45.1	59.04	10.38	7.92
		70	46.5	59.05	10.39	7.93
		72	49.1	59.10	10.44	7.96
		74	51.7	59.17	10.51	8.02
		76	54.1	59.22	10.56	8.06
		78	56.5	59.28	10.62	8.10
		80	58.9	59.25	10.59	8.08
		85	64.5	59.40	10.74	8.19
		90	70.1	59.62	10.96	8.36
		100	80.8	59.73	11.07	8.45
		110	91.4	59.84	11.18	8.53
		120	101.8	60.00	11.34	8.65
		140	122.3	60.20	11.54	8.80
		160	142.7	60.33	11.67	8.90
		180	163.0	60.42	11.76	8.97
		200	183.2	60.50	11.84	9.03
		220	203.3	60.65	11.99	9.15
		240	223.4	60.74	12.08	9.22
		270	253.6	60.81	12.15	9.27
		300	283.7	60.89	12.23	9.33
		330	313.8	60.93	12.27	9.36
		360	343.9	61.00	12.34	9.41

Discharge Q.		t (min)	$\bar{t}$ (min)	W/level	s (m)	s/Q
m^3/hr	m^3/min	390	373.9	61.05	12.39	9.45
78.65	1.311	420	404.0	61.10	12.44	9.49
		480	464.1	61.20	12.54	9.57
		540	524.1	61.32	12.66	9.66
		600	584.1	61.40	12.74	9.71
		660	644.2	61.48	12.82	9.78
		720	704.2	61.47	12.81	9.77
		780	764.3	61.46	12.80	9.76
		840	824.3	61.52	12.86	9.81
		900	884.3	61.57	12.91	9.85
		960	944.4	61.65	12.99	9.91
		1020	1004.4	61.71	13.05	9.96
		1080	1064.4	61.73	13.07	9.97
		1140	1124.4	61.78	13.12	10.01
		1200	1184.4	61.80	13.14	10.02
		1260	1244.4	61.84	13.18	10.05
		1320	1304.4	61.90	13.24	10.10
		1380	1364.4	61.94	13.28	10.13
		1440	1424.5	61.96	13.30	10.15
		1500	1484.5	61.95	13.29	10.14
		1560	1544.5	61.99	13.33	10.17
		1620	1604.5	62.03	13.37	10.20
		1680	1664.5	62.62	13.36	10.19
		1740	1724.5	61.99	13.33	10.17
		1800	1784.5	61.98	13.32	10.16
		1860	1844.5	61.98	13.32	10.16
		1920	1904.5	61.98	13.32	10.16
		1980	1964.5	62.00	13.34	10.18
		2040	2024.5	62.03	13.37	10.20
		2100	2084.5	62.03	13.37	10.20
		2160	2144.5	62.04	13.38	10.21
		2220	2204.5	62.05	13.39	10.22
		2280	2264.5	62.05	13.39	10.22
		2340	2324.5	62.08	13.42	10.24
		2400	2384.5	62.10	13.44	10.24
		2460	2444.5	62.12	13.46	10.25

Discharge Q.		t (min)	$\bar{t}$ (min)	W/level	s (m)	s/Q
m^3/hr	m^3/min	2520	2504.5	62.13	13.47	10.27
78.65	1.311	2580	2564.5	62.14	13.48	10.28
		2640	2624.5	62.15	13.49	10.29
		2700	2684.5	62.15	13.49	10.29
		2760	2744.5	62.15	13.49	10.29
		2820	2804.5	62.15	13.49	10.29
		2880	2864.5	62.15	13.49	10.29

B Recovery Test:

Original Water Level Prior to Pumping = 48.66m

Time t" (min)	Time t (min)	t/t"	Water Level (m)	Residual Drawdown s" (m)
0	2880		62.15	13.49
1	2881	2881	54.00	5.34
2	2882	1441	53.89	5.23
3	2883	961	53.60	4.94
4	2884	721	53.40	4.74
5	2885	577	53.32	4.66
6	2886	481	53.19	4.53
7	2887	412.43	53.07	4.41
8	2888	361	52.98	4.32
9	2889	321	52.89	4.23
10	2890	289	52.82	4.16
12	2892	241	52.65	3.99
14	2894	206.71	52.55	3.89
16	2896	181	52.44	3.78
18	2898	161	52.34	3.68
20	2900	145	52.26	3.60
25	2905	116.2	52.08	3.42
30	2910	97	51.95	3.29
35	2915	83.29	51.80	3.14
40	2920	73	51.71	3.05
50	2930	58.6	51.51	2.85
60	2940	49	51.31	2.65
70	2950	42.14	51.21	2.55
80	2960	37	51.09	2.43
90	2970	33	50.98	2.32
100	2980	29.8	50.89	2.23
120	3000	25	50.73	2.07
150	3030	20.2	50.73	2.07