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APPLICATIONS OF A FINITE-DIFFERENCE MODEL FOR AQUIFER SIMULATION TO AN AREA IN EAST CENTRAL ALBERTA

by

(C) LEWIS G. FAHNER

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

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Date 22 April, 1982

ABSTRACT

Demand for ground water and the resulting need to expand our unde[illegible] of ground water systems has led to the extensive use of models of sever[illegible] Limitations of physical-type and analog-type models have res[illegible]ent of more powerful numerical techniques.

The Trescott, Pinder and Larson (1976) model, one [illegible] finite-difference models available, has been modified to r[illegible] Alberta Amdahl V8 computer. The model has then been used to r[illegible]a for an extensive pumping test conducted on an aquifer in east-[illegible]test, with a pumping phase exceeding 34,000 minutes in duration, w[illegible] sufficient length to identify boundary and recharge effects. Simulat[illegible] bee[illegible] run in order to test the validity of several different conceptual models w[illegible] of selecting one most appropiate for the simulation of a twenty-year produ[illegible]od including appropiate recharge effects.

A favorable comparison between real and simulated drawdown in pumping and observation wells could not be improved upon to any great extent by the inclusion of various forms of recharge. It would seem that the field test does not supply all data necessary to effectively quantify the recharge to the ground water system.

The effectiveness of a pumping well at intercepting recharge, which occurs as direct precipitation on an adjacent recharge area, is also evaluated using the Trescott et al (1976) model. The influence of transmissivity, storativity, rate of recharge and pumping rate have been considered utilizing a hypothetical ground water basin. Stress, in the form of pumping, has been added to the ground water system at steady-state and the results analysed. Varying parameters such as transmissivity and storativity may result in an increase or decrease in the efficiency with which a pumping well can intercept ground water moving through an aquifer system, while varying rate of recharge has little effect on such efficiency.

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I. INTRODUCTION

The increasing demand for ground water and the competition for the resource provides an important incentive for expanding our knowledge and understanding of ground water systems. One area that continues to receive attention by researchers is modeling, where predictive techniques have been used for aquifer evaluation since the early 1900's (Thiem, 1906).

There are a variety of different kinds of models, which may be categorized as conceptual, physical, analog or mathematical. Generally the conceptual model is used to represent, in a generalized way, our understanding of how various components of a ground water system actually function. The formulation of a conceptual model is usually a preliminary step in the development of physical, analog or the more powerful mathematical models. The physical models are scaled down replicas of real systems. A sand-box model for example, may be made up of layers of of sand representing aquifers and clay representing confining layers, scaled in proportion to an actual field situation. Physical models tend to be very limited in their applicability to practical problems but are often very useful for illustrative purposes. Analog models such as the resistor-capacitor network type or the conductive-paper type are based on an analogy between the flow of ground water and the flow of electric current. These models have been used extensively for a variety of practical problems and are particularly useful when computer facilities are not available.

The mathematical approach involves describing, in mathematical terms, response of an aquifer system to pumping for example. Governing differential equations can be solved using a variety of mathematical techniques to provide a description of drawdown or hydraulic head distributions within a region of interest. The analytical techniques, which are based on classical mathematical techniques, usually can be applied only to relatively simple field situations because they often are based on assumptions such as an infinite areal extent for the aquifer, radial flow conditions, and constant aquifer parameters.

The computer-based procedures, which involve a numerical solution of the flow equation, represent a powerful set of tools for solving practical problems. The numerical techniques are not subject to the limiting assumptions, which characterize the analytic models. For example, they can incorporate a variety of complex boundary conditions, and permit aquifer parameters to vary as a function of both space and time. In addition, it is a simple matter to consider multiple discharge or recharge wells. Such flexibility and generality is extremely difficult or impossible to attain with either analytic or physical models.

Numerical models are easier to set up than resistor-capacitor models, avoiding the selection of appropiate resistors and capacitors and the painstaking soldering of numerous connections. Advantages of the numerical approach are evident for three-dimensional situations, which are particularly difficult to simulate with an analog model of either type. Changes to a numerical model are simple in comparison to the analog or physical model. Such versatility is particularly useful during the model calibration phase of the study involving matching of simulation results to the field results.

The successful application of predictive numerical techniques to real problems requires sets of parameters characterizing the system under consideration. While a good deal of information is available for the common parameters such as transmissivity and storativity, characterization of the quantity of recharge to an aquifer system has been a longstanding hydrogeological problem. To effectively utilize the ground water resources of an aquifer and to predict the results of various withdrawal schemes, it is necessary to have some knowledge of the rate at which the aquifer system is being recharged as well as the effectiveness of the particular withdrawl scheme in intercepting recharge.

Recharge from precipitation varies both spatially and temporally. This variation has made the quantification of recharge difficult although a variety of attempts have been made to determine how much recharge occurs in a given area. Published rates of recharge, expressed as a percentage of the local annual precipitation, vary from about 2 percent for a small park-land area near Devon, Alberta (Farvolden et al., 1963) to over 18 percent for a sandy area in Goshen County, Wyoming, U.S.A. (Rapp et al., 1957). Toth (1968) estimated recharge near Olds, Alberta to be about 9 percent.

This study is concerned with computer modeling of aquifer systems. It's goals are to adapt the Trescott et al.,(1976) model to the University of Alberta Amdahl computer, to re-evaluate a detailed aquifer test conducted in an area near Kirkpatrick Lake, in east central Alberta, and to use the model to gain a better understanding of aquifer recharge.

The specific objectives of this study with regard to flow modeling are to:

1. obtain and interpret available hydrogeological data from the study area and formulate a conceptual model of the ground water regime,
2. use parameters obtained from conventional interpretative procedures and general field observations to construct and calibrate a numerical model by comparing computer predicted results to existing field results,
3. simulate various pumping strategies with a view to establishing the probable long-term response of the system and
4. evaluate, with the numerical model, how various ground water withdrawal schemes and aquifer parameters influence the interception of water moving through an aquifer system when recharge results from direct precipitation on adjacent areas. The approach to meeting this objective is based on a sensitivity analysis of a series of hypothetical cases.

II. GEOLOGY

The purpose of this section is to present a detailed description of the geology of the Kirkpatrick Lake area (Fig. 1). Information presented here has been obtained from lithological and geophysical logs available from the oil industry as well as private and government publications, reports and maps. Upper Cretaceous strata of the Bearpaw and Belly River Formations comprise the near-surface bedrock. These rocks are uncomformably overlain by the Pleistocene glacial deposits. A generalized stratigraphic column for this area is presented in Figure 2.

The Bearpaw Formation is the uppermost bedrock unit in the entire study area. It consists of an interbedded sequence of marine shale and sandstone. Rocks near the top of the sequence crop out south and east of the study area along Sounding Creek and Monitor Creek respectively. Test drilling indicates that the Bearpaw Formation is approximately 150m thick although the total thickness is not represented in the study area because the upper part has been removed by erosion. The exposed Bearpaw strata are typically weathered to a light brown color and are very soft. Ironstone concretions are common (Irish, 1967).

The Bearpaw Formation has been divided into two members in the study area (Fig. 2). The upper member is referred to as the Paintearth Member and the lower member is referred to as the Young Creek Member (Lines, 1963). The Paintearth Member is comprised of sandstone which may be argillaceous, bentonitic or glauconitic. The sandstone zones are typically separated by grey shale. At several locations the boundary between the two members is a layer of chert pebbles (Lines, 1963).

According to Williams and Dyer (1930) a definite and traceable sand member exists about midway in the Bearpaw Formation. This very fine to medium-grained sand, referred to as the Bulwark sandstone, is variable in thickness generally ranging from 5m to 7m. The exact extent of the Bulwark sandstone is unknown, although a similar sandstone at roughly equivalent elevations is known to exist about 13km south (Williams and Dyer, 1930).

Subsequent to the work by Williams and Dyer, water well drillers began to report that the Bulwark sandstone could produce either white or brown water which suggest that it was not a single sandstone but was at least two distinct units. More recent work

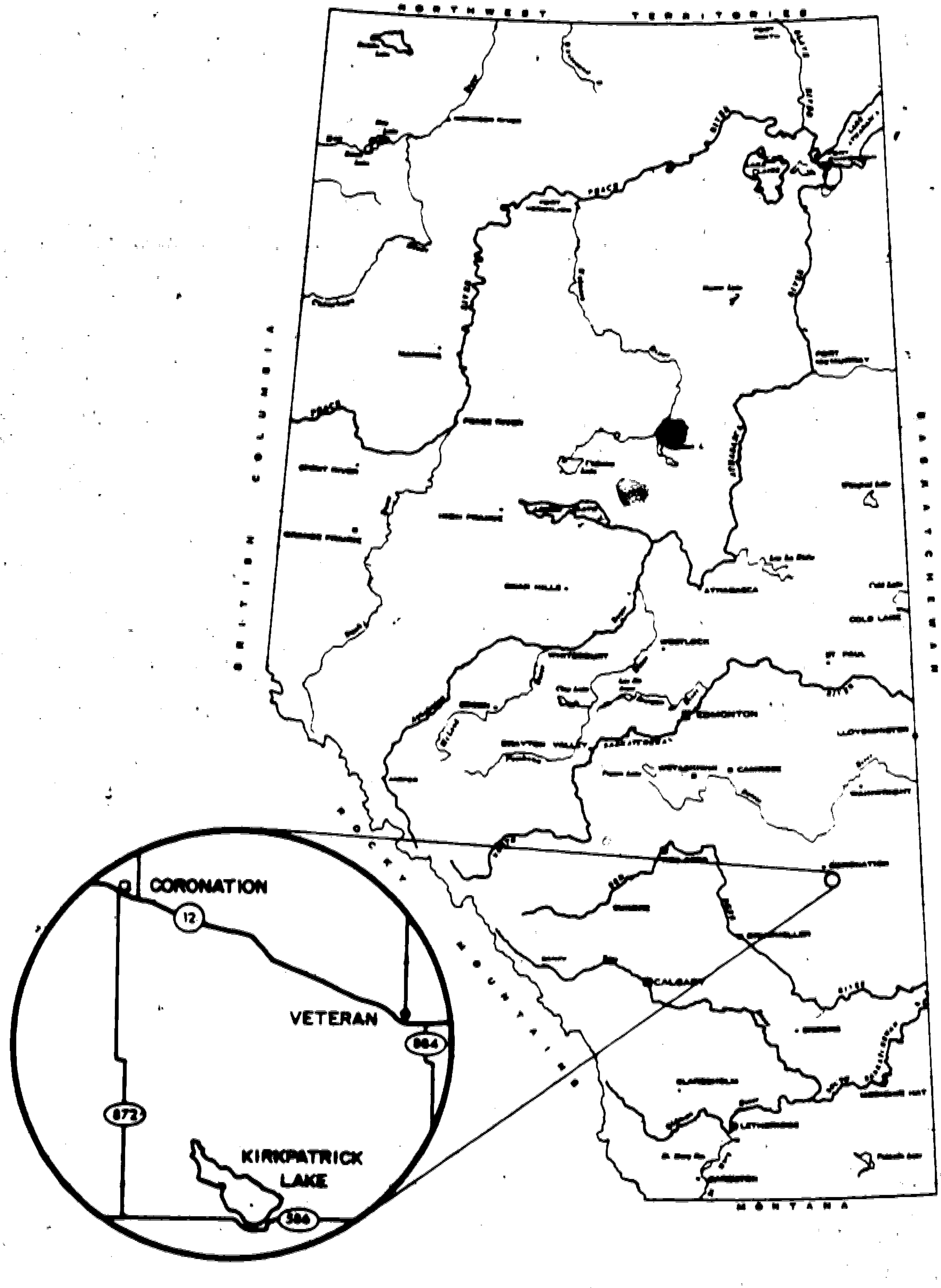


Figure 1. Location of study area

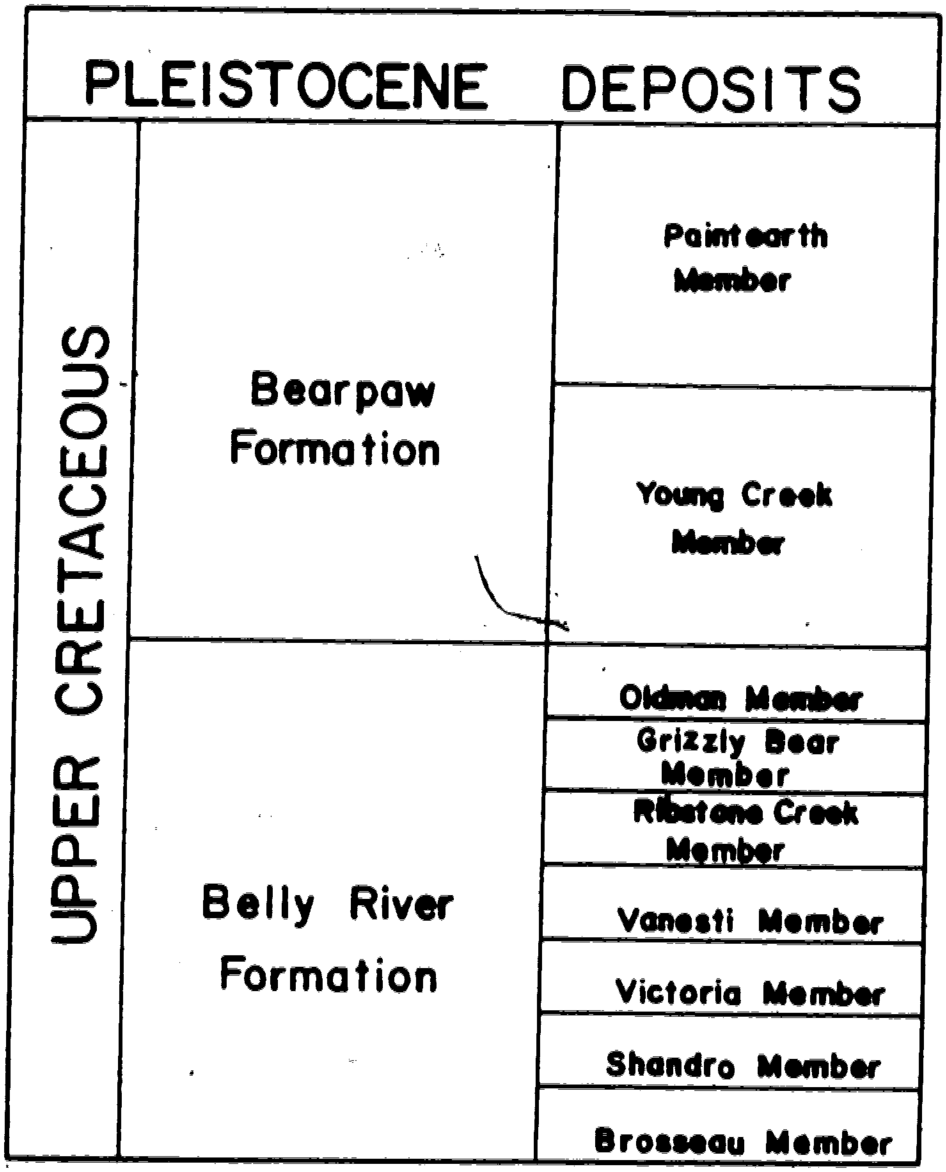


Figure 2. Generalized stratigraphic column for the Kirkpatrick Lake area (Adapted from Clissold, 1972; Shaw and Harding, 1965; Lines, 1963)

by Curry (1968) and Clissold (1972) has demonstrated the existence of at least 3 distinct sandstone units in the study area (Fig. 3). As a result, the usefulness of "Bulwark" as a stratigraphic unit is questionable. Clissold (1972) referred to the upper, middle, and lower sandstone units of the Paintearth member in the study area as zone I, zone II, and zone III respectively to avoid futher confusing use of the term Bulwark.

The Young Creek member of the Bearpaw Formation is predominantly grey shale or silty shale with some argillaceous or bentonitic sandstone (Lines, 1963). Bentonitic beds ranging from a few centimetres to as much as 9m in thickness have been reported in this member (Given and Wall, 1971). The base of the Bearpaw Formation is defined by a layer of chert pebbles (Lines, 1963) and apparently rests uncomformably on carbonaceous shale of the underlying Belly River Formation.

The Belly River Formation in general consists of an interbedded non-marine sequence of pale grey sandstone and buff-colored sandy shale (Irish, 1967). The interfingered character of the predominantly non-marine Belly River Formation and the underlying marine Lea Park Formation results in as many as ten members, which are arbitrarily placed in the Belly River (Shaw and Harding, 1954). Thus, the Belly River Formation in east-central Alberta and particularly in the study area appears to represent a mixed marine-continental origin. The continental or deltaic deposits forming the predominantly sandstone members such as the Brosseau, the Victoria and the Ribstone Creek appear to have been derived from the west or southwest and thin to the east and northeast (Shaw and Harding, 1954). Tongues of predominantly marine shale such as the Shandro, the Vanesti and the Grizzly Bear members present in the study area generally thin to the west. Minor coal seams or coal stringers are present where the Belly River Formation becomes more continental in nature.

The distinct sandstone and shale members of the Belly River Formation, which are present in the study area, are shown in Figure 2. Individual members have been described by various authors such as Allan, (1918) and Shaw and Harding (1954) as follows:

Brosseau member - fine-grained, grey calcareous sandstone with some sandy, brownish shale particularly to the east.

Shandro member - dark grey shale containing calcareous and arenaceous concretions.

Victoria member - fine to medium-grained grey sandstone and brownish-grey,

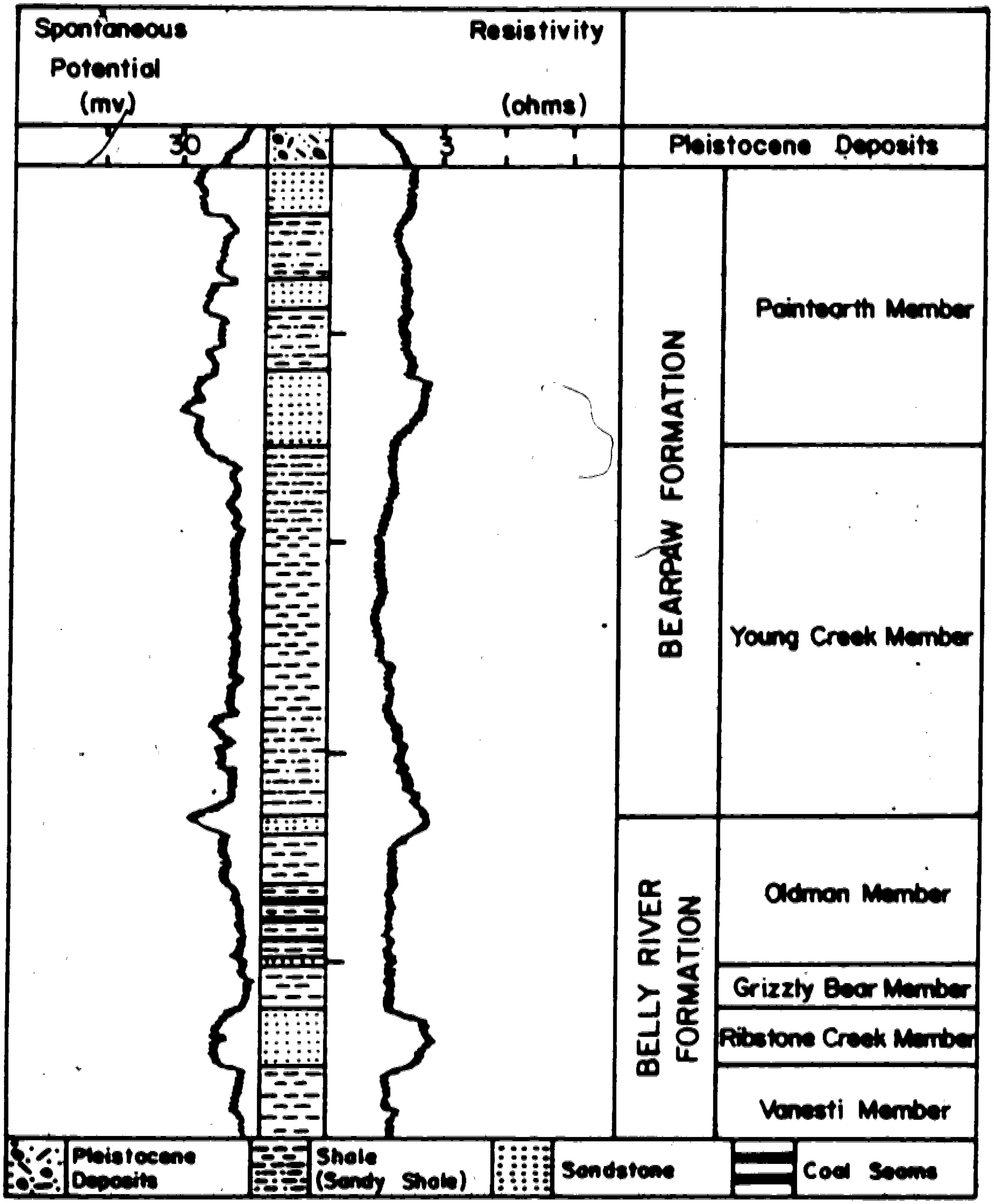


Figure 3. Typical drillhole lithology and corresponding electric log response in study area (Adapted from Clissold, 1972; Shaw and Harding, 1963)

carbonaceous, silty shale with local thin coal seams.

Vanesti member - grey shale, silty shale, clayey shale and fine sandstone.

Ribstone Creek member - soft grey sandstone with some carbonaceous shale and coal.

Grizzly Bear member - dark blue-grey shale with some ironstone and sandstone modules.

Oldman member - pale grey sandstone and buff-colored sandy shale (Irish, 1967).

The Oldman member, which has formational status in southern Alberta, represents the northeastward pinch-out of a typical Belly River facies and as a result resembles the undivided Belly River Formation (Shaw and Harding, 1954).

Information on the Belly River Formation in the study area must come from drillhole information because the unit does not crop out. Lithologic logs from existing drillholes and an electric log from an oil company test hole about 15km to the north and on strike with the bedrock of the study area indicate that the Belly River Formation is approximately 100m thick in the area.

Surficial deposits in the general area may be classified as residual deposits, unsorted glacial deposits, reworked glacial deposits or transported deposits of alluvial, lacustrine or colluvial origin (Wyatt et al., 1938). Very little of the unconsolidated material in the study area is residual with most deposits being glacial in origin and subsequently reworked by wind and/or water. The Viking moraine (Warren, 1937) occurs as an elevated feature north-east of the study area and much of the alluvial gravel, sand, silt and clay forming part of the flats around Kirkpatrick Lake has been derived from this topographically elevated feature. The Viking moraine is composed of unstratified clay with few pebbles or boulders that often comprise a terminal moraine on the prairies of Alberta.

The surficial deposits in the study area are mapped as ground moraine, alluvial deposits, aeolian deposits or lacustrine deposits as shown in Figure 4. The ground moraine is generally thin with a surface characterized by scattered pebbles and boulders of Precambrian origin. Occasional patches of gravelly outwash occur. The alluvial deposits tend to be well-sorted with some sand and gravel lenses. Some boulders may be present. Both the aeolian and lacustrine deposits are fairly well-sorted with few boulders. Bedding may be present in either type of deposit.

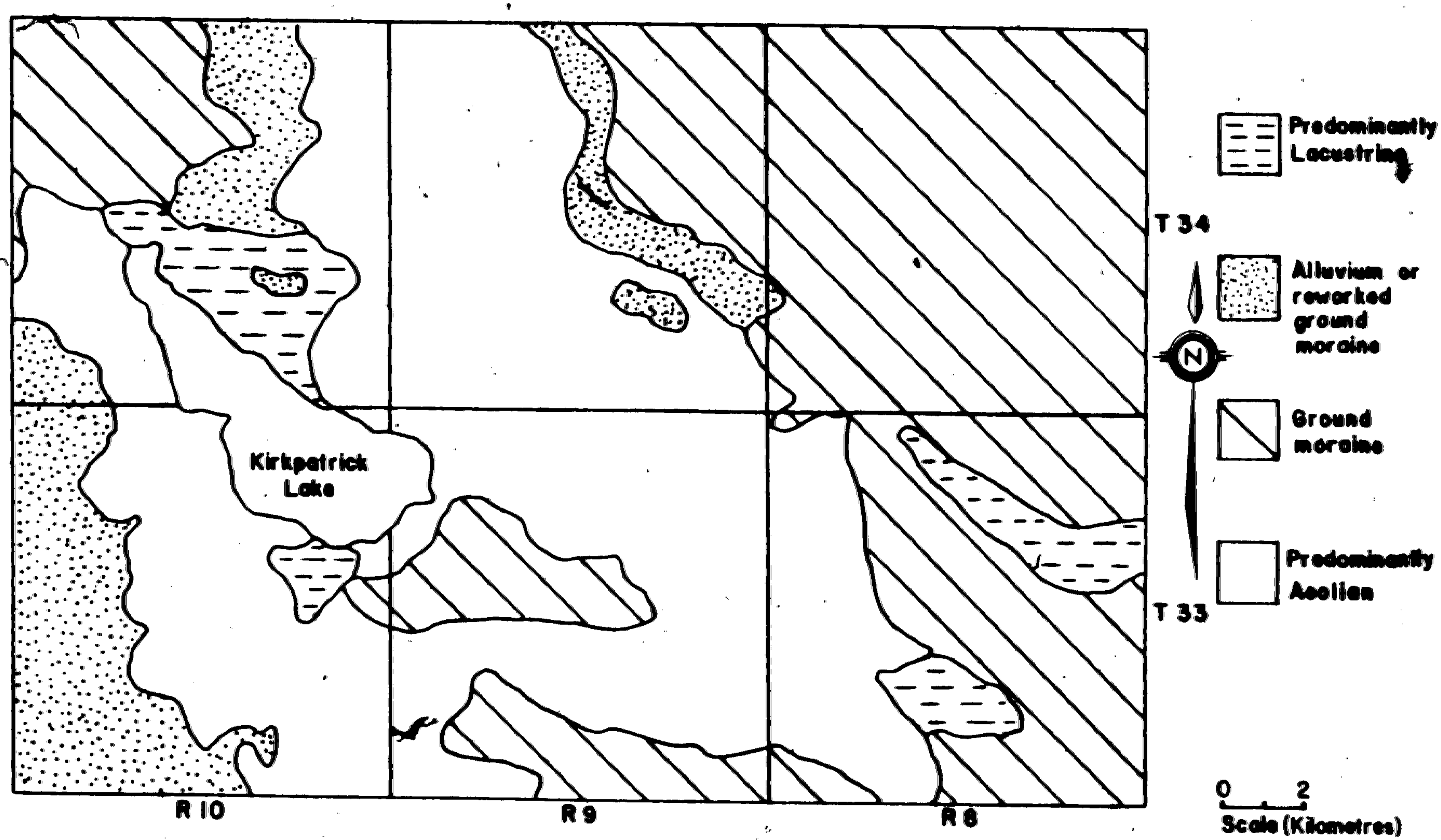


Figure 4. Surficial deposits as determined from soil survey sheet (Wyatt et al., 1938) Modified from Clissold (1972)

The sequence of surficial deposits range in thickness from 0 to 12m (Clissold, 1972). However, a water well driller's log from 16-21-34-9-W4 reports 40m of unconsolidated deposits. This thickness suggests that a bedrock channel, perhaps preglacial in origin, exists in the eastern portion of the study area.

The fence diagram shown in Figure 5 presents in detail the spatial relationships among the various stratigraphic units. The study area itself is characterized by the presence of a broad bedrock high. The bedrock low to the east may limit the lateral extent of some of the sandstone units particularly the uppermost sandstone.

Bedrock units in the study area strike northwest- southeast and dip at a low angle to the northeast. This interpretation is based on drill hole data compiled by Clissold (1972) for the base of the uppermost sandstone aquifer in the study area. The dip direction is opposite to that normally expected in this part of the Alberta Syncline (Elliott, 1960) and may represent a localized flexure on one limb of the broad syncline. This structure could limit the areal extent of the aquifers.

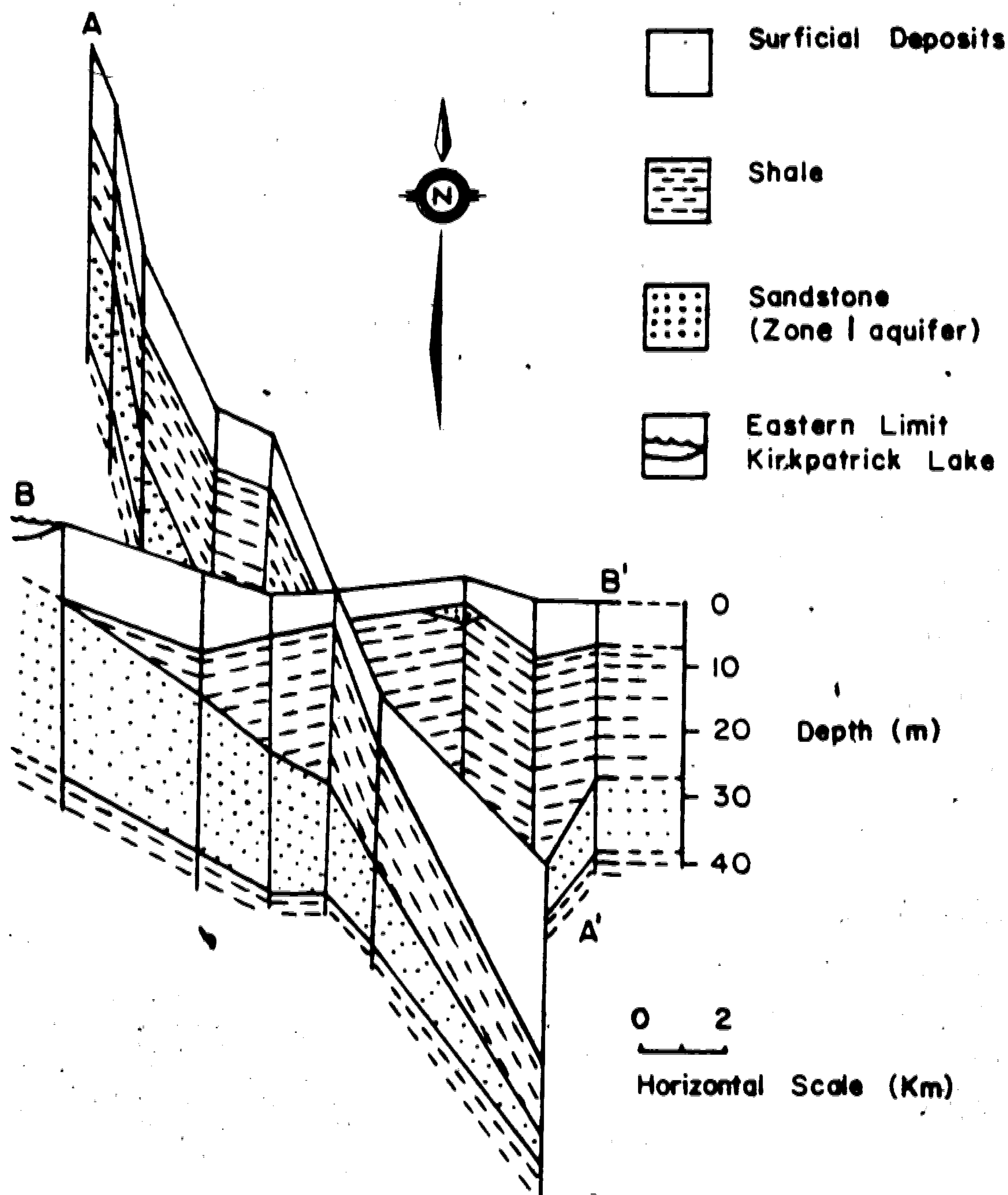


Figure 5. Fence diagram constructed from drillhole data (For lines of section see Fig. 6)

III. HYDROGEOLOGY

In this section, the potential ground water yield and chemistry for various geologic units in the study area are described. The data have been obtained from water well drillers' reports on file at the Groundwater Division of the Alberta Research Council, from ground water resource evaluation reports prepared by consultants in support of ground water licence applications as well as from complaint investigation reports written by members of the Groundwater Division of the Alberta Research Council.

According to Clissold (1972) all test holes drilled in the study area encountered aquifers which, individually, or collectively produced ground water in sufficient quantities to meet domestic requirements. Yields from wells completed in coarse-grained surficial deposits such as outwash sand and gravel range up to $1.0 \times 10^{-3} m^3/s$ although yields of $6.0 \times 10^{-3} m^3/s$ have been reported (Kunkle, 1962). The higher yields likely represent short term pumping capibilities. In general, ground water yields from surficial deposits are poorer than average where the deposits are thinner and are better than average where they are thicker.

The chemical quality of the ground water from surficial deposits aquifers is generally considered poor with total dissolved solids, hardness and sulphate content exceeding Canadian drinking water quality guidelines (National Health and Welfare, 1979). Total dissolved solids may range up to 2400 mg/l with hardness and sulphates ranging up to 600 and 1400 mg/l respectively. The higher total dissolved solid content occurs where surficial deposits are thinnest (Borneuf, 1978).

The Bearpaw and Belly River Formations contain the main bedrock aquifers in the study area. Extensive sandstone units up to 20m thick are present in both formations. At some locations east of the study area, sandstone in the Belly River represents the only reliable bedrock source of ground water. However, the depth of the Belly River Formation and the apparent lack of permeable strata in at least the upper part make it of minor significance as an aquifer in the study area (Clissold, 1972). Typically, Belly River aquifers are found at depths greater than 140m.

Information on the ground water potential of the three sandstone members of the Bearpaw Formation has been available for many years as the result of ground water investigations for domestic supplies. The most extensive investigation of the ground water potential of these three sandstone aquifers has been conducted by Clissold (1972). The purpose of that work was to locate a long-term supply of water for secondary petroleum recovery. The three aquifers are known to be of limited lateral extent but are more or less continuous across the entire study area. However, the upper and lower units are subdivided by shale units.

The study by Clissold (1972) covered an area of approximately 200 km^2 immediately to the east of Kirkpatrick Lake. The work verified the existence of the three Bearpaw aquifers at depths of less than 125m. As previously mentioned, Clissold (1972) proposed informal names for each of the units with zone I representing the upper sandstone in the area, zone II representing the middle sandstone and zone III representing the lower sandstone. Clissold's (1972) terminology has been adopted in this work to identify the various sandstone units in the Bearpaw Formation.

Zone I was most intensely investigated by Clissold (1972) because early in that program there were indications that only zone I could yield the quantity of water required for injection purposes. The top of the Zone I sandstone is intersected at depths ranging from 10 to 34m. It has a maximum thickness of 20m. The sandstone strikes NNW-SSE and dips at about 1m per 5km to the northeast. Cross-sections constructed in east-west and northwest-southeast directions show that zone I subcrops beneath the surficial deposits under Kirkpatrick Lake and thins to the northeast and south (see Figures 2 and 6).

The chemical quality of the ground water from the bedrock varies both laterally and vertically. Although no analyses of ground water within the study area are available from the Belly River Formation, two such analyses are available for well-water further south and east. One analysis is for ground water obtained from the Birch Lake member and one is for ground water obtained from the Ribstone Creek member. Analyses indicate ground water has relatively high Cl^- ion concentration, high alkalinity and a total dissolved solids content of about 2000 mg/l.

The chemical quality of the ground water from zones I, II, and III is shown in Table 1 along with representative compositions for two aquifers in the Belly River Formation. In general bicarbonate, total alkalinity and chloride contents increase with depth, while sulphate ion concentrations decrease. It appears that zone II ground water is better in quality than that from either shallower or deeper aquifers.

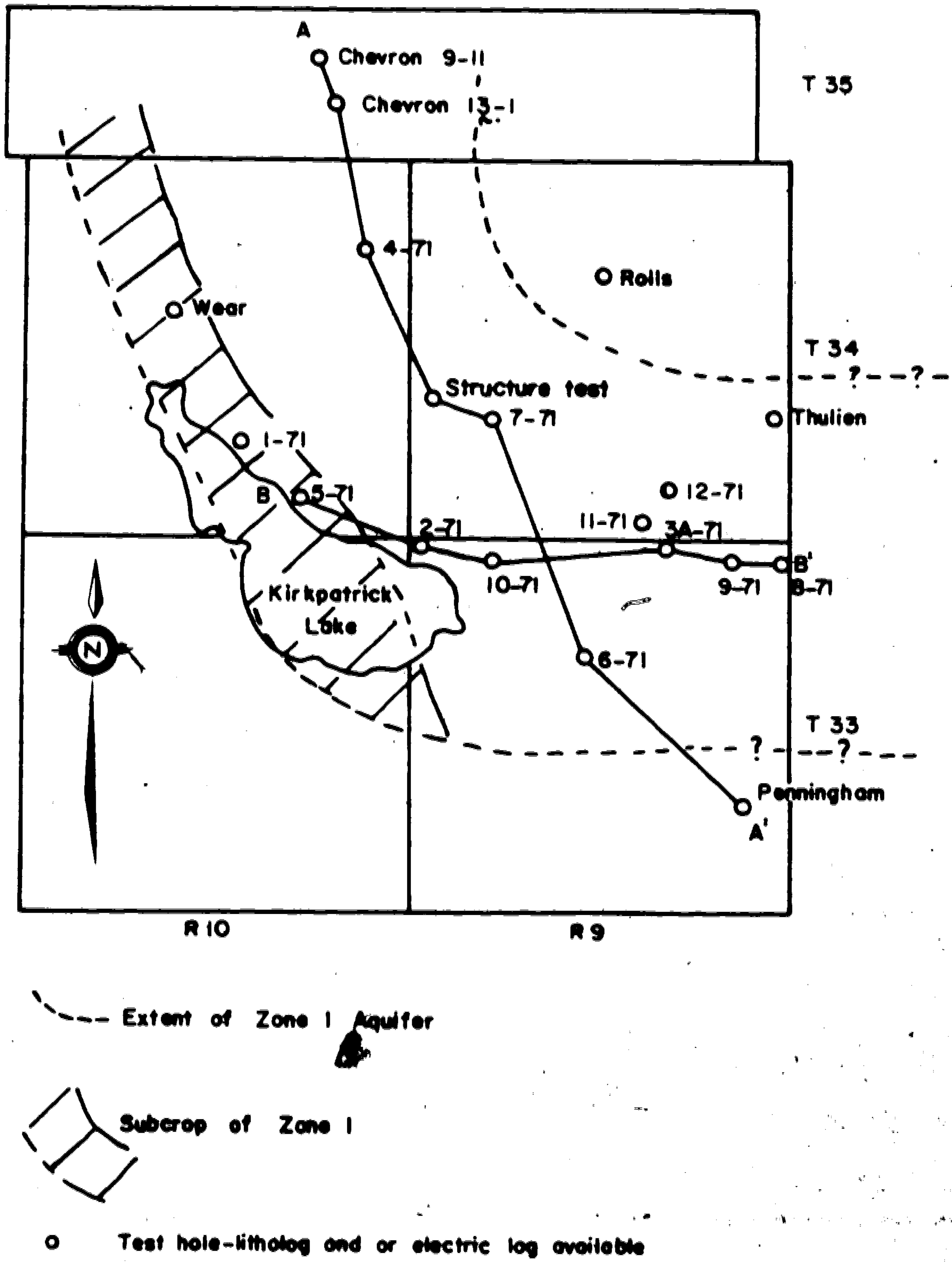


Figure 6. Testholes near Kirkpatrick Lake (Lines of section for fence diagrams shown)

TABLE 1. Ground water chemistry from various aquifers in the study area (Clissold, 1972; Kunkle, 1962).

Constituent[1]	BP[2]	BP[2]	BP[2]	BP[3]	BP[3]	BP[4]	BR[5]	BR[6]
TDS	1656	3062	1424	1632	948	1228	1990	2026
IL[7]	204	326	216	220		134		32
Na & K	522	890	389	520		459		
Ca	19	30	19	5		5		
Mg	3	5	5	1		.2		
CO_3	nil	33	29	43		33		
HCO_3	370	227	162	200		556		
TA[8]	303	242	182	235		515	396	400
SO_4	850	1650	690	890	8	160	5	nil
Cl	6	52	10	14	212	224	915	959
TH[9]	60	96	68	17		21	15	25
Fe						0.55		1.5
F								
pH	8.0	8.4	8.8	9.0		8.6	8.4	

Constituents[1] = concentrations in mg/l
BP[2] = Bearpaw Formation, Zone I aquifer
BP[3] = Bearpaw Formation, Zone II aquifer
BP[4] = Bearpaw Formation, Zone III aquifer
BR[5] = Belly River Formation, Birch Lake Member
BR[6] = Belly River Formation, Ribstone Creek Member
IL[7] = Ignition loss
TA[8] = Total alkalinity
TH[9] = Total hardness

IV. EXISTING AQUIFER TEST DATA

Existing aquifer test data for the Kirkpatrick Lake area consist of results for short-term tests by water well drillers, which are carried out in conjunction with domestic well construction, and more extensive tests by Currie (1968), LeBreton (1969) and Clissold (1971 and 1972) related to evaluation of ground water resources for industrial purposes. The most significant testing in the study area was conducted by Clissold (1972) in an attempt to locate and evaluate ground water supplies for secondary petroleum recovery operations. A total of twenty-nine constant-rate aquifer tests of 60 to 1499 minutes pumping duration were conducted on zones I, II and III prior to a long-term constant-rate aquifer test of 34,510 minutes duration in zone I.

Fifteen of the 29 constant-rate aquifer tests evaluated the water production capibilities of zone I, 7 evaluated zone II, 5 evaluated zone III and 2 evaluated the potential of the Belly River Formation or combined zone III and Belly River aquifers. The 15 aquifer tests on zone I were distributed throughout the area being studied in order to determine the extent of zone I and to obtain information which would allow the selection of the most appropriate site for the production well and subsequent long-term pumping test. Twenty-six of these tests included water level measurements during recovery. One step-drawdown pumping test was also conducted using a well completed in zone I. This test was designed to determine the most appropriate pumping rate for the zone I aquifer. The results indicated that a properly constructed well could be pumped at rates as high as $1.06 \times 10^{-2} m^3/s$ for a short time at least with drawdown being within acceptable limits (Clissold, 1972).

Graphical analysis of the pumping phase data and, where possible, the recovery phase data using the Jacob Method (Cooper and Jacob, 1946) are included in Clissold's (1972) report. (See Tables 2 and 3 for a summary of the relevant data). Transmissivity values for zone I, the most extensively tested and promising aquifer ranged from 2.6×10^{-3} to 4.7×10^{-3} m^2/s for the pumping phase of the test and from 3.8×10^{-3} to 3.4×10^{-3} m^2/s for the recovery phase. A map showing aerial distribution of transmissivity based on interpretation for the early part of the drawdown data and the early part of the recovery data is shown in Figure 7. This map indicates that the transmissivity increases in a north-easterly direction from Kirkpatrick Lake (Clissold, 1972). A water well driller's

TABLE 2. Summary of well data (Clissold, 1972)

Well No.	Location	Elevation'	WL'	Test No.	Interval Tested'	Aquifer	PR'	PD'	RD'	T'	T*
1-71	12-10-34-10-4	769.2	766.4	1	9.84-24.40	Zone I	1.39	120	120	3.78	3.82
			762.9	2	28.90-36.30	Zone II	2.27	120	120	8.80	2.63
			756.6	3	35.97-79.27	Zone III	2.27	1000	1300	8.10	5.38
			756.6	4	36.00-91.50	Zone III	2.36	120	120	7.35	7.35
			702.3	5	202.0-244.0	BR''		2.00			
			702.6	6	202.0-228.7	BR	0.76	60	13	1.04	
2-71	13-31-33- 9-4	769.8	765.9	1	11.30-33.50	Zone I	2.36	1499	3000	20.6	18.4
			764.8	2	34.10-70.10	Zone II	2.36	1200	1200	3.77	8.03
			756.2	3	61.90-120.0	Zone III	2.36	120	120	4.71	6.68
3-71	16-34-33- 9-4	772.3	766.0	1	10.70-35.00	Zone I	2.36	120	120	155	155
			766.1	2	10.70-42.70	Zone I	3.40	1000	80	272	120
			764.0	3	43.30-65.50	Zone II	2.36	120	120	5.56	5.43
			764.1	4	43.50-97.60	Zone II	2.36	1440	120	12.0	1.55
			698.4	5	100.6-244.0	BR	0.45	64		0.91	
3A-71		772.0	765.7	1	27.70-38.00	Zone I	3.90	120		125	
3B-71			765.1	1	28.40-41.20	Zone I	4.30	120	16	102	
3C-71			764.6	NIL							

Elevation' in metres above sea level
WL' = Water level in well in metres above sea level
Interval tested' in metres below ground level.
PR' = Pumping rate during test. ($\times10^{-3}m^3/s$)
PD' = Pumping duration (min.)
RD' = Recovery duration (min.)
T' = Transmissivity determined from drawdown data ($\times10^{-3}m^2/s$)
T* = Transmissivity determined from recovery data ($\times10^{-3}m^2/s$)
BR'' = Belly River Formation

TABLE 3. Summary of well data (Clissold, 1972)

Well No.	Location	Elevation[1]	WL[2]	Test No.	Interval Tested[3]	Aquifer	PR[4]	PD[5]	RD[6]	T[7]	T[8]
3D-71	16-34-33- 9-4	772.2	768.9	Nil							
4-71	12-25-34-10-4	774.6	764.5	1	14.30-42.70	Zone I	2.36	120	120	475	347
			764.5	2	14.30-42.70	Zone I	3.93	1200	120	376	359
			753.6	3	45.10-83.80	Zone II	1.97	120	120	1.04	1.04
			754.7	4	43.60-96.00	Zone III	1.97	120	120	2.43	3.24
5-71	12- 2-34-10-4	769.5	766.1	1	18.90-36.60	Zone I	3.78	120	120	123	134
6-71	2-28-33- 9-4	772.1	766.0	1	10.70-36.60	Zone I	3.40	120	120	95.5	103
7-71	13- 8-34- 9-4	777.0	765.5	1	9.10-42.70	Zone I	3.93	760	770	204	204
8-71	9-36-33- 9-4	777.5	765.3	1	27.60-48.80	Zone I	3.57	120	120	31.0	31.0
9-71	9-35-33- 9-4	769.7	765.7	1	39.90-73.00	Zone II	2.49	120	120	6.02	2.43
9A-71		769.7	765.7	Nil							
10-71	12-32-33- 9-4	769.5	766.2	1	19.70-36.60	Zone I	3.78	120	120	141	141
11-71	2- 3-34- 9-4	771.7	765.6	1	21.70-39.60	Zone I	3.18	120	120	192	59.8
12-71s[9]	9- 3-34- 9-4	774.1	766.0	1	11.90-45.40	Zone I	3.93	120	120	296	339
12-71d[10]			764.9	1	49.70-63.40	Zone II	2.36	120	110	8.91	6.83
TF[11]	8-12-34- 9-4	775.2	764.9	Nil							

Elevation[1] in metres above sea level
WL[2] = Water level in well in metres above sea level
Interval tested[3] in metres below ground level
PR[4] = Pumping rate during test. ($\times 10^{-3} m^3/s$)
PD[5] = Pumping duration (min.)
RD[6] = Recovery duration (min.)
T[7] = Transmissivity determined from drawdown data ($\times 10^{-5} m^2/s$)
T[8] = Transmissivity determined from recovery data ($\times 10^{-5} m^2/s$)
12-71s[9] = Shallow well
12-71d[10] = Deep well
TF[11] = Thullen farm well

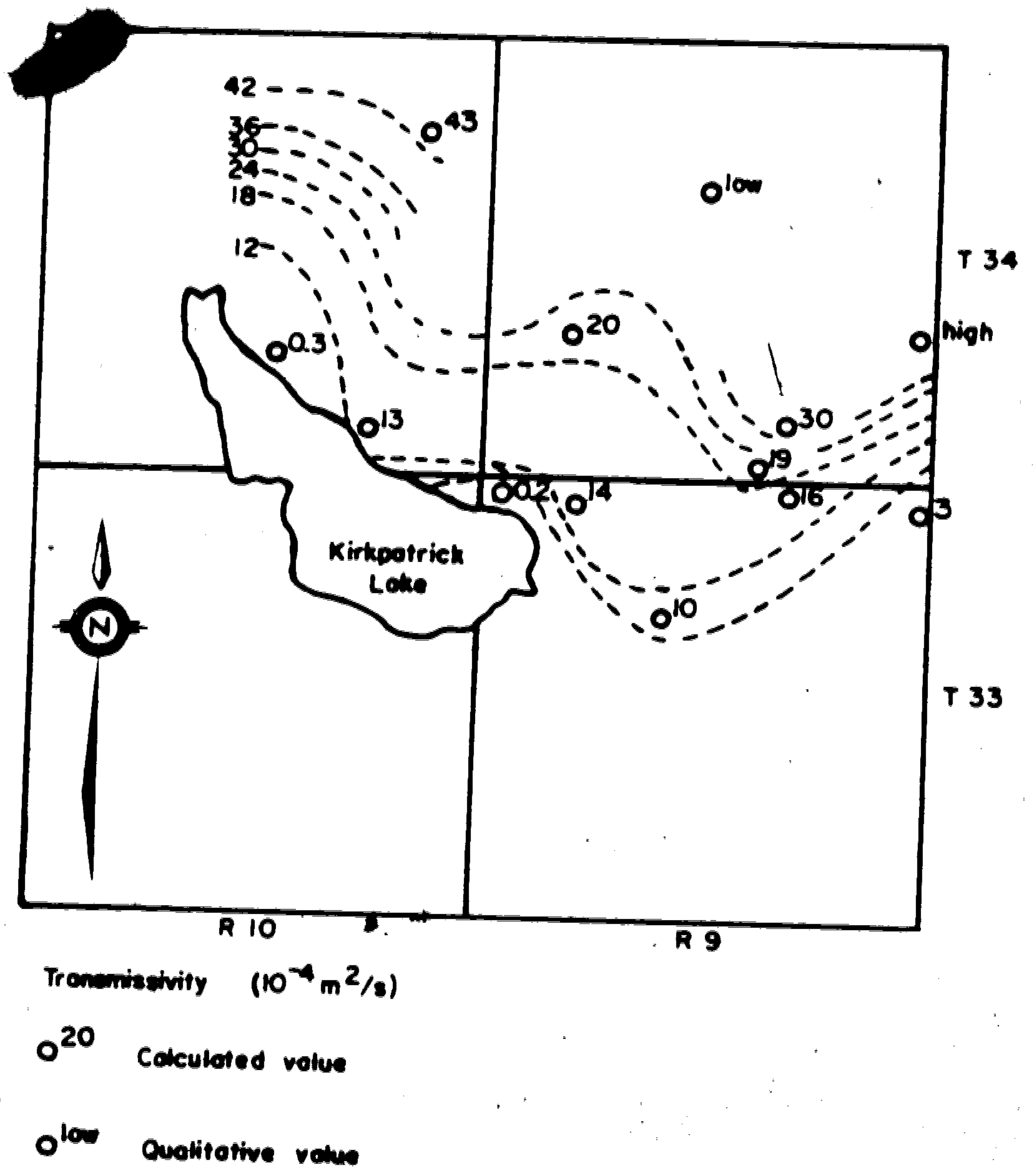


Figure 7. Areal distribution of transmissivity in zone I (Adapted from Clissold, 1972)

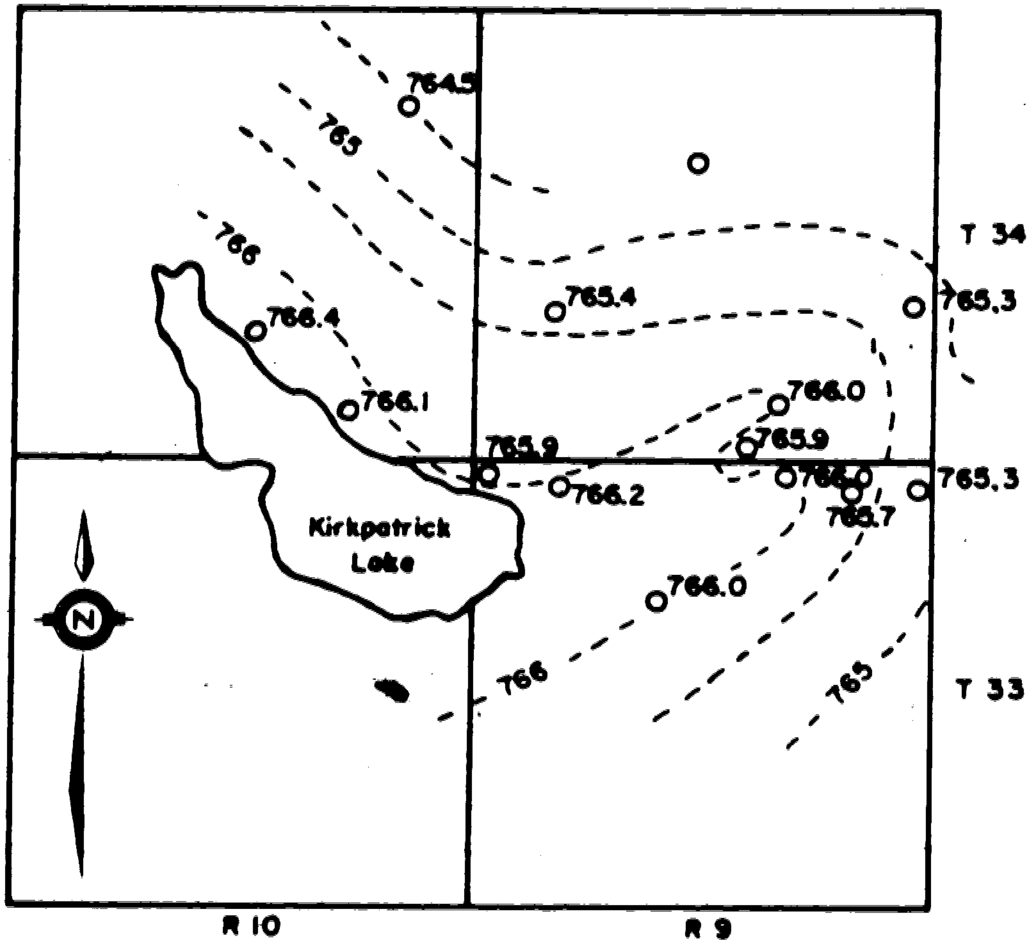
log of a domestic well in 16-21-34-9-4 indicates that the zone I aquifer is thin at that location and yields poorly when bail tested. The transmissivity is thus assumed to decrease further to the northeast based on the qualitatively low value at that well. Because no observation wells were used with any of the short term pumping tests on wells completed in zone I no storativity values are available.

The static water levels in the wells were measured, usually after the test holes stood undisturbed over night, and a map of non-pumping water levels has been prepared (See Table 2 and 3; Figure 8). This map shows a trend of decreasing water level northeast from Kirkpatrick Lake. This flow pattern may be an indication of recharge from the lake to the zone I aquifer, which subcrops beneath the lake.

During the long-term pumping test water levels in 11 of the observation wells in addition to the pumped well declined in response to the pumping (Clissold, 1972). Of these 11 wells, 10 were completed in the zone I aquifer and the remaining one was completed in the zone II aquifer. Although this well was completed in zone II it was at a site where a dual yet separate zone I and zone II aquifer completion was attempted in the same well. The fact that the water levels were very close to the same elevation in zone I as measured in the outside casing and in zone II as measured in the inside casing suggests communication between the two aquifers, perhaps as leakage along the casing. This contention is further supported by water level data from other sites. When separate, closely spaced wells were completed in zone I and zone II the water elevations were different and tended to decrease with increasing depth (Clissold, 1972). It therefore seems reasonable to assume that the water level decline in the well completed in zone II is the result of a poor casing seal and is not a response to pumping the zone I aquifer.

Barometric pressure was monitored during the pumping test and small water level changes in observation wells were corrected for the influence of atmospheric pressure variations during the test (See Table 4).

Two additional observation wells appeared to respond to long-term pumping although no consistent direct relationship was apparent. The water level in the well completed in the aquitard above zone I slowly rose prior to the test and continued to do so during the entire test. The rate of water level rise varied and it was not until the pumping phase of the test was concluded that the water level began to level off and



$\circ^{700}$ Water level in metres above sea level

Contour interval = 0.5 m

Figure 8. Nonpumping water levels in zone I (Adapted from Clissold, 1972)

TABLE 4. Observation wells in which water levels declined during pumping

Well number	Status	r[1]	Drawdown[2]
3A-71	P[3]	0	18.24
3B-71	O[4]	6.1	15.01
3C-71	O	6.1	5.82
11-71	O	533.5	3.73
12-71S[5]	O	952.7	3.26
12-71D[6]	O	952.7	0.23
9A-71	O	1646.3	1.86
6 -71	O	3475.6	0.22[8]
8 -71	O	3521.3	0.61
TF[7]	O	4039.6	0.20[8]
10-71	O	4436.0	0.17[8]
7 -71	O	5579.3	0.12[8]

r[1] = Distance from pumping well (m)
Drawdown[2] = Drawdown (m)
P[3] = Pumping well
O[4] = Observation well
12-71s[5] = Shallow well
12-71d[6] = Deep well
TF[7] = Thulien farm well
0.22[8] = Drawdown corrected for atmospheric pressure changes

finally began to decline.

The water level in well 3-71, completed in zone II, rose during the first few minutes of the test then began to slowly decline reaching the pretest level after 600 minutes. For the remainder of the test, the water level fluctuated within a few centimetres of the pretest level. The water level decreased about 5 centimetres within 20 minutes of the termination of the pumping and then began to rise slowly.

The analyses of the drawdown data from the long-term aquifer test consisted of time-drawdown plots and evaluations for each well and distance-drawdown for 8 well pairs after a pumping duration of 30,000 minutes. A summary of the transmissivity and storativity values calculated by Clissold (1972) from the time-drawdown data is presented in Table 5.

The time-drawdown curves indicated that the hydraulic characteristics of zone I appear to vary as a function of time. The data summarized in Table 5 are taken from the early part of the drawdown curve in each case and thus represent the hydraulic characteristics in the immediate vicinity of the test well. Figure 9 is a map showing the aerial distribution of transmissivity as determined from the early part of the data from the long-term aquifer test. The trend of increasing transmissivity towards the northeast from Kirkpatrick Lake is not as apparent as it was in the plot of transmissivity values taken from the short-term pumping tests on the individual wells (Figure 7). The differences are to be expected as the pumping tests on individual wells give a local transmissivity representative of the aquifer in the area immediately adjacent to the test well while, the long-term pumping test on the centrally located well and the subsequent drawdown analyses with individual observation wells result in an average transmissivity of the aquifer between the pumping and observation wells.

Storativity values determined from the early part of the time-drawdown curves of the observation wells range from a low of 1.4×10^{-4} to a high of 1.6×10^{-3}, about one order of magnitude. The areal distribution of storativity is shown in Figure 10. Storativity, in general, increases with increasing distance from the pumping well. This increase of about one order of magnitude was initially interpreted as leakage but later computer simulations suggest this was not the case.

TABLE 5. Summary of transmissivity and storativity values obtained from semilog and log-log analysis of time-drawdown data.

Well No.	Status	r[1]	T[2]	T[3]	S[4]
3A-71	P[8]	0	16.76		
3B-71	O[9]	6.1	19.00		
3C-71	O	6.1	17.28	15.54	1.4
11-71	O	533.5		46.64	1.6
12-71S[5]	O	952.7		27.64	1.4
12-71D[6]	O	952.7	639.17		
9A-71	O	1648.3		36.27	2.8
6-71	O	3475.6		63.91	16
8-71	O	3521.3		14.69	3.7
10-71	O	4436.0		14.69	8.4
TF[7]	O	4039.6		95.01	2.4
7-71	O	5579.3		69.91	13

r[1] = Distance from pumping well (m)
T[2] = Transmissivity obtained from semilog plot ($x10^{-4} m^2/s$)
T[3] = Transmissivity obtained from log-log plot ($x10^{-4} m^2/s$)
S[4] = Storativity ($x10^{-4}$)
12-71s[5] = Shallow well
12-71d[6] = Deep well
TF[7] = Thulien's farm well
P[8] = Pumping well
O[9] = Observation well

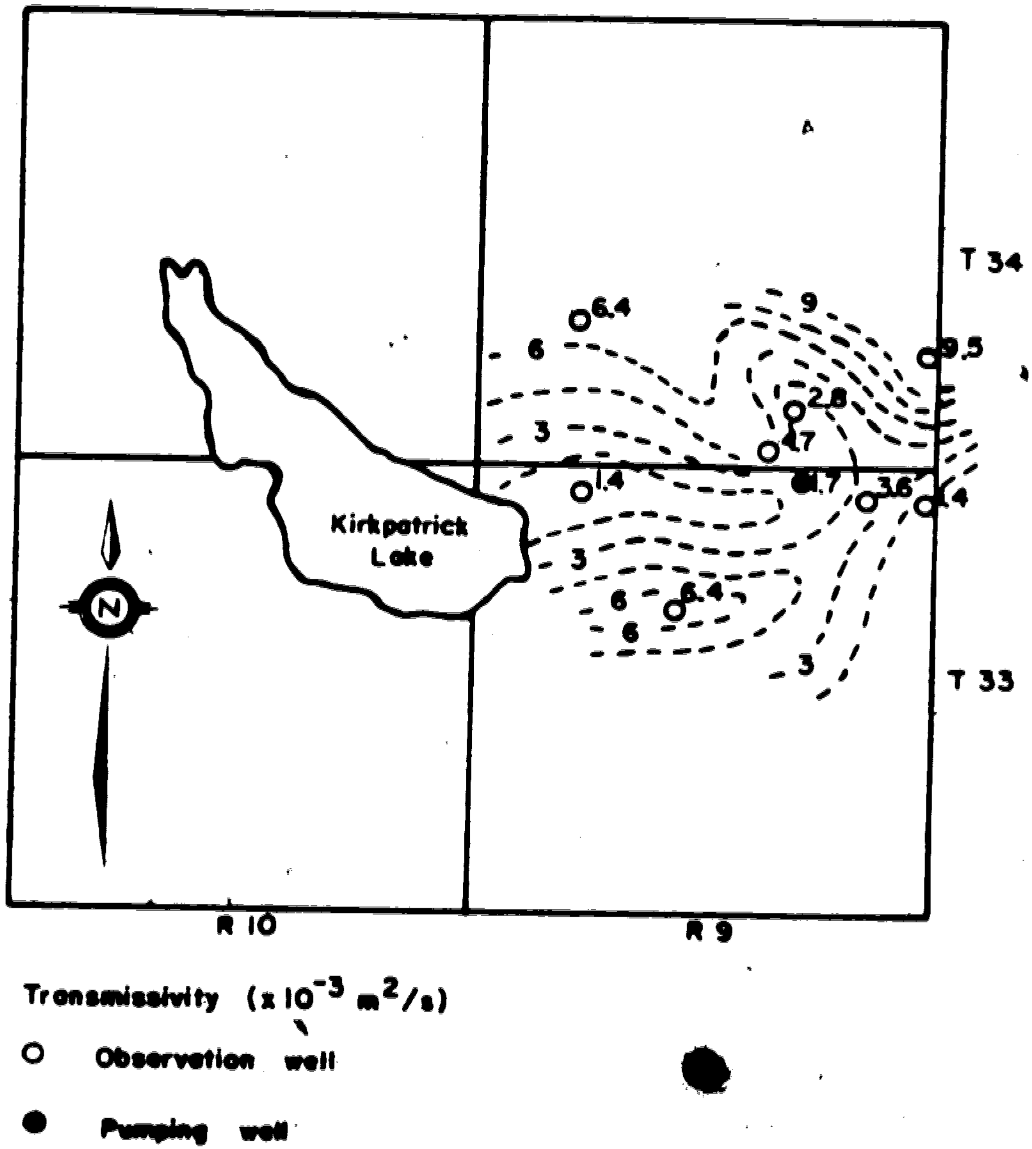
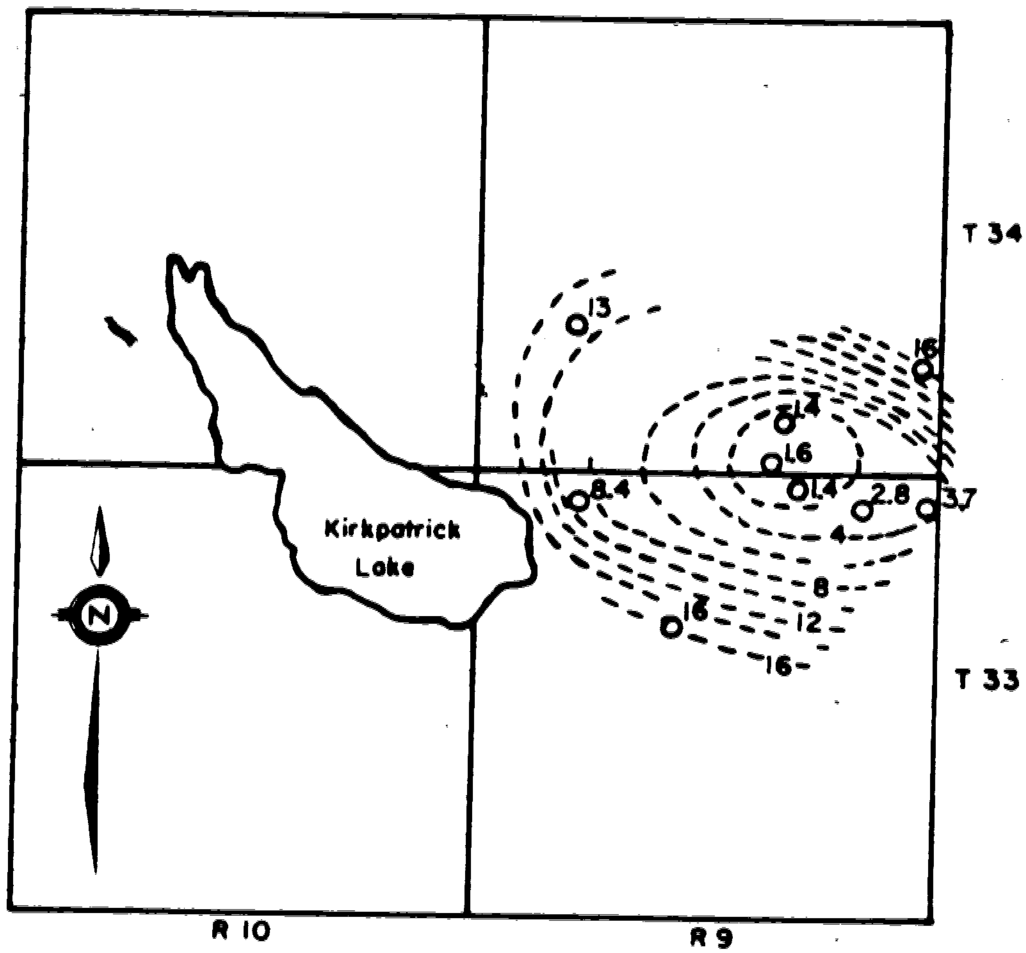


Figure 9. Areal distribution of transmissivity in zone I as determined from early drawdown data from long-term pumping test (Adapted from Clissold, 1972)



O Observation well ($x\,10^{-4}$)

Contour interval = 2

Figure 10. Areal distribution of storativity in zone I (Adapted from Clissold, 1972)

V. THEORETICAL DEVELOPMENT AND IMPLEMENTATION

The purpose of this section is to describe the basic theory of modeling techniques and to explain the features of the Trescott et al (1976) model, including options available to the user. The following partial differential equation describes general ground water flow in a two-dimensional aquifer system.

$$\frac{\partial}{\partial x}\left(Txx\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(Tyy\frac{\partial h}{\partial y}\right) = S\frac{\partial h}{\partial t} + W(x,y,t) \qquad 1$$

where T= components of the transmissivity tensor in the coordinate directions (L^2t^{-1})

h=hydraulic head (L)

S=storativity (dimensionless)

W(x,y,t,)=volumetric flux of recharge or withdrawl per unit surface area of the aquifer (Lt^{-1})

Equation (1) has been simplified by assuming that the coordinate axes x and y are coincident with the principal components of the transmissivity tensor T_{xx} and T_{yy}.

Although (1) may be solved by analytically, these simplified solutions are applicable only to idealized situations with, for example, uniform aquifer parameters, constant pumping rates from a small number of wells and simple boundary conditions. Numerical solutions using a computer provide the only means of dealing with more complex situations, which more accurately reflect the actual hydraulic setting.

To use a finite difference method to solve the partial differential equation, the aquifer is first divided up into a two-dimensional array of rectangular blocks. It is assumed that the properties of the aquifer are uniform within each of the resulting blocks. The continuous derivatives in equation (1) are replaced with finite difference approximations at the nodes centering on each of the blocks. With N blocks the result is N equations with N unknowns, which are the hydraulic head values defined in the center of each block.

For all the simulations presented here, the Trescott, Pinder, and Larson (1976) model is utilized. It is the most versatile model for evaluating two-dimensional aquifer problems. The model can consider a heterogeneous and anisotropic aquifer with regular or irregular boundaries. The aquifer can be confined, unconfined or a combination of both. The model allows for multiple discharge and/or recharge wells, evapotranspiration and leakage from confining beds in which storage effects may be approximated.

Model simulation requires the collection of field data in order to define the region and develop the parameters characterizing the aquifer system. It is necessary to have sufficient data to provide a good understanding of the boundary conditions with regard to both type and location and the character of confinement for the aquifer. Generally, the entire suite of geologic and hydrologic data is required to interpret these features of the system. The magnitude and distribution of transmissivity and storativity values is best determined from aquifer tests, which should be distributed throughout the area of interest. The aquifer tests in addition to verifying the aquifer type, should also provide estimates of possible leakage from any confining beds. It will be possible to develop a conceptual model of the aquifer system at this stage, which presents a unifying picture of the parameters of the actual field system. With the addition of a grid, the system can be defined in a form necessary for input to the computer model.

Once the data set runs successfully, its validity can be partially verified by means of a history match, that is, a comparison of model predictions with actual field observations of a pumping test. Should the match not be as good as expected or desired it may be improved by varying one or more of the parameters such as transmissivity or storativity, keeping them within reasonable limits, and re-running the model. With a calibrated working model it is possible to make predictions of future aquifer behaviour in response to various aquifer stresses.

The specific steps in the simulation process are as follows:

1. Prepare a map of the aquifer complete, with location and type of all boundaries. Although the actual boundaries of the aquifer system may lie beyond those of the project area, they should, if possible, be included in the model. Where actual boundaries are very distant it may be necessary to specify imaginary boundaries.
2. Locate all pumping and/or recharge wells on the map of the aquifer. Also locate any streams or lakes which may be recharging the aquifer.
3. Prepare a block-centered grid with any wells located as close to their actual positions as possible. They should also be as close to a node as possible. The ratio of $\Delta x_j / \Delta x_{j-1}$ and $\Delta y_i / \Delta y_{i-1}$ should not exceed 1.5 to avoid convergence problems. The grid should be oriented such that

there are a minimum number of nodes located outside the aquifer boundaries.

4. Determine T and S values and their distribution from field data and plot them, on the grid system superimposed on the map of the aquifer. For an anisotropic aquifer the grid should be oriented parallel to the principal direction of the transmissivity tensor.

5. Initial head conditions must be selected prior to model simulation. Often the changes in head resulting from a simulation are adequate and the computed head values are not important. In such cases the model does not need to start from steady-state conditions in which flow is occurring. The model does however have the capability of treating more complex problems of head distribution.

6. Prepare a data set of the initial hydrogeological parameters to be used for the computer model.

7. Type the data set and run the program.

Specifically, the Trescott et al (1976) model uses a block-centered, variable size grid such as that in Figure 11 and equation (1) can be expressed as the approximation:

$$\frac{1}{\Delta x_j}\left[\left(T_{xx}\frac{\Delta h}{\Delta x}\right)_{i,j+\frac{1}{2}}-\left(T_{xx}\frac{\Delta h}{\Delta x}\right)_{i,j-\frac{1}{2}}\right]+\frac{1}{\Delta y_i}\left[\left(T_{yy}\frac{\Delta h}{\Delta y}\right)_{i+\frac{1}{2},j}-\left(T_{yy}\frac{\Delta h}{\Delta y}\right)_{i-\frac{1}{2},j}\right]$$

$$=\frac{S_{i,j}}{\Delta t}\left(h_{i,j,k}-h_{i,j,k-1}\right)+W_{i,j,k} \qquad 2$$

where x = space increment in x direction for column j (L)

y = space increment in y direction for row i (L)

i = index in the y direction

j = index in the x direction

k = time index

W(x,y,t) the source term is computed in the model as:

$$W_{i,j,k}=\frac{Q_w[i,j,k]}{\Delta x_j\ \Delta y_i}-q_{re}[i,j,k]-q'_{i,j,k}+q_{et}[i,j,k] \qquad 3$$

where $Q_w[i,j,k]$ = well discharge (L^3t^{-1})

$q_{re}[i,j,k]$ = recharge flux per unit area (LT^{-1})

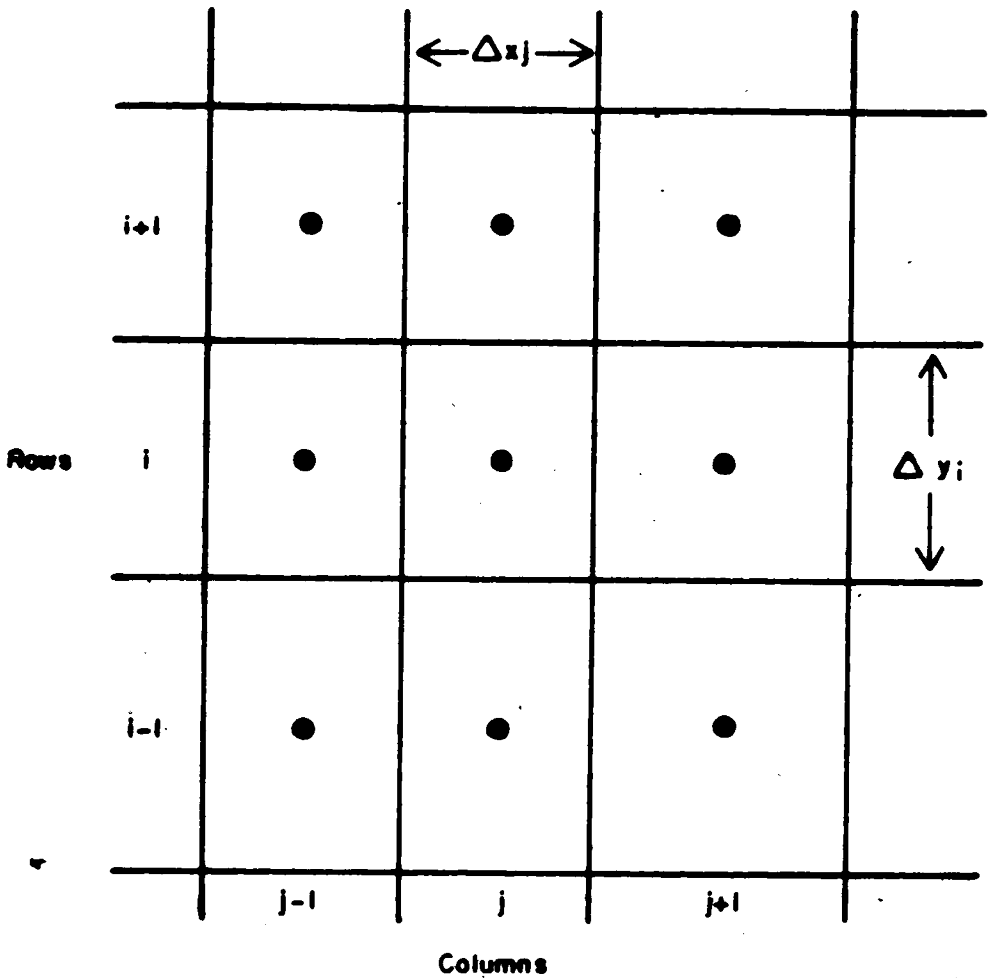


Figure 11. Representative block-centered variable-size finite difference grid with index scheme for node (i,j) From Trescott et al., 1976

$q'_{i,j,k}$ = flux per unit area from a confining layer (Lt^{-1})

$q_{et}[i,j,k]$ = evapotranspiration flux per unit area (Lt^{-1})

A confining layer, stream-bed or lake-bed from which leakage is taking place and the effect of storage as being considered is approximated by an equation modified after Bredehoeft and Pinder (1970). The equation is as follows:

$$q'_{i,j,k} = (h_{i,j,o} - h_{i,j,k}) \frac{K'_{i,j}}{\left(\frac{\pi K'_{i,j} t}{3m^2_{i,j} S_s[i,j]}\right)^{\frac{1}{2}} m_{i,j}} \cdot \left\{1 + 2 \sum_{n=1}^{\infty} \exp\left[\frac{-n^2}{\left(\frac{K'_{i,j} t}{3m^2_{i,j} S_s[i,j]}\right)}\right]\right\} + \frac{K'_{i,j}}{m_{i,j}} (\hat{h}_{i,j,o} - h_{i,j,o}) \qquad 4$$

where $h_{i,j,o}$ = hydraulic head in the aquifer at the start of the pumping period (L)

$\hat{h}_{i,j,o}$ = hydraulic head on the other side of the confining bed (L)

$K'_{i,j}$ = hydraulic conductivity of the confining bed (Lt^{-1})

$m_{i,j}$ = thickness of the confining bed (L)

$S_s[i,j]$ = specific storage in the confining layer (L^{-1})

$(K'_{i,j} t / m^2_{i,j} S_s[i,j])$ = dimensionless time

t = elapsed time of the pumping period (t)

In this model, evaporation is considered as a linear function of depth below the surface of the land by using the following equation:

$$q_{et\ i,j,k} = \begin{cases} Q_{et} & [h_{i,j,k} \geq G_{i,j}] \\ Q_{et} - \frac{Q_{et}}{ET_z}(G_{i,j} - h_{i,j,k}) & [ET_z > (G_{i,j} - h_{i,j,k}) ;\ h_{i,j,k} < G_{i,j}] \\ 0 & [ET_z \leq (G_{i,j} - h_{i,j,k})] \end{cases} \qquad 5$$

where Q_{et} = maximum evapotranspiration rate (Lt^{-1})

ET_z = depth below the surface at which evapotranspiration ceases (L)

$G_{i,j}$ = elevation of the land surface (L)

Three methods of numerical solution are available for the Trescott et al (1976) model. These are:

1. Line successive overrelaxation (LSOR)
2. Alternating-direction implicit (ADI) and
3. Strongly implicit procedure (SIP) which are in order of increasing complexity with the more complex numerical solution generally allowing for more rapid convergence and the solving of more complex problems.

All three methods make use of matrix solutions to solve the algebraic equations which are generated for each node.

Trescott et al (1976) concluded that SIP is the more powerful of the three interative techniques for most applications. It is particularly useful because it converges quickly and does not require the parameter maniputation that the ADI and LSOR techniques do to obtain optimum performance of rate of convergence and accuracy. SIP does have a trade-off for its apparent benefits in that it requires more arrays and hence more computer storage.

The model used in this study has been adapted for the Amdahl 470V/8 computer employing the FORTRAN G compiler and using the SIP method of numerical solution. The source code and fixed-dimension arrays require 100K bytes of memory assuming all solve routines are compiled with the FORTRAN G Level 21 compiler. This can be reduced to 88K bytes by compiling only one solve routine. If all options are included, storage requirements are (100 + X/256) K bytes where X is the dimension of vector Y in the main program.

$$Y \text{ dimension} = (15 + Na)\ Nx \times Ny$$

where Na = total number of arrays required for the options as outlined in Table 6

Nx = number of columns

Ny = number of rows

Examination of drill hole lithologs and the time-drawdown pumping test data from the test holes completed in the zone I aquifer indicate that in the study area the aquifer is confined. The extent of the aquifer has been determined from the drill-hole data and by deductive reasoning. Drill hole data suggest zone I subcrops beneath the surficial

TABLE 6. Number of arrays (Na in equation (6)) required for various model options.

MODEL OPTION	NUMBER OF ARRAYS
Water table	3
Conversion (also requires arrays for water table option)	1
Leakage	3
Evapotranspiration	1
SIP	4

deposits in the vicinity of Kirkpatrick Lake in the western part of the study area. Information obtained from oil company test drilling to the north indicates that the aquifer extends well beyond the limits of the study area in that direction. Drill hole 8-71 and the Thulien well (10-12-34-9-4) intersect the zone I aquifer so it can be assumed that it extends beyond the eastern limit of the study area although how far exactly is not known at this time. Zone I tested poorly during the bailing of the well at 16-21-34-9-4 and at 12-14-33-9-4, so it can be concluded that zone I has thinned to the northeast and to the south at these localities and may represent the limits of the aquifer in these directions. These assumed hydraulic boundaries occur well within the study area. The aerial extent of the zone I aquifer within the study area is shown in Figure 6. For purposes of the simulations, the western, southern and northeastern boundaries are considered to be no flow boundaries. The northern and eastern boundaries are imaginary no flow boundaries as the actual boundaries may occur well beyond the limits of the study area. However, in most cases these two imaginary boundaries are sufficiently distant from the pumping well to prevent them from influencing the model simulation. The grid system developed for the area to be modeled is shown in Figure 12. The node spacings have been kept small in the vicinity of the wells, gradually increasing away from the well yet not exceeding a spacing factor of 1.5 (See Fig. 12) for adjacent nodes as suggested in the model documentation. The larger spacing away from the wells reduces the number of nodes required and yet does not seriously affect the accuracy nor limit the proposed use of the simulation results.

Transmissivity values determined at the various test holes from the early time-drawdown data are plotted on a map of the aquifer, contoured (See Fig. 7) and used to prepare a grid of transmissivity values for the simulation. Early time-drawdown data are used as initial grid values because the transmissivity values assigned to the grid must not reflect the influence of any significant recharge or barrier boundaries, or the boundaries would have a secondary influence when added explicitly to the model during simulations. The transmissivity values, upon which the model simulations were ultimately based range about two orders of magnitude, from 0.02×10^{-3} to $2.3 \times 10^{-3} m^2/s$ (See Figure 13). In general, the value of transmissivity increases towards the north and the northeast, away from Kirkpatrick Lake.

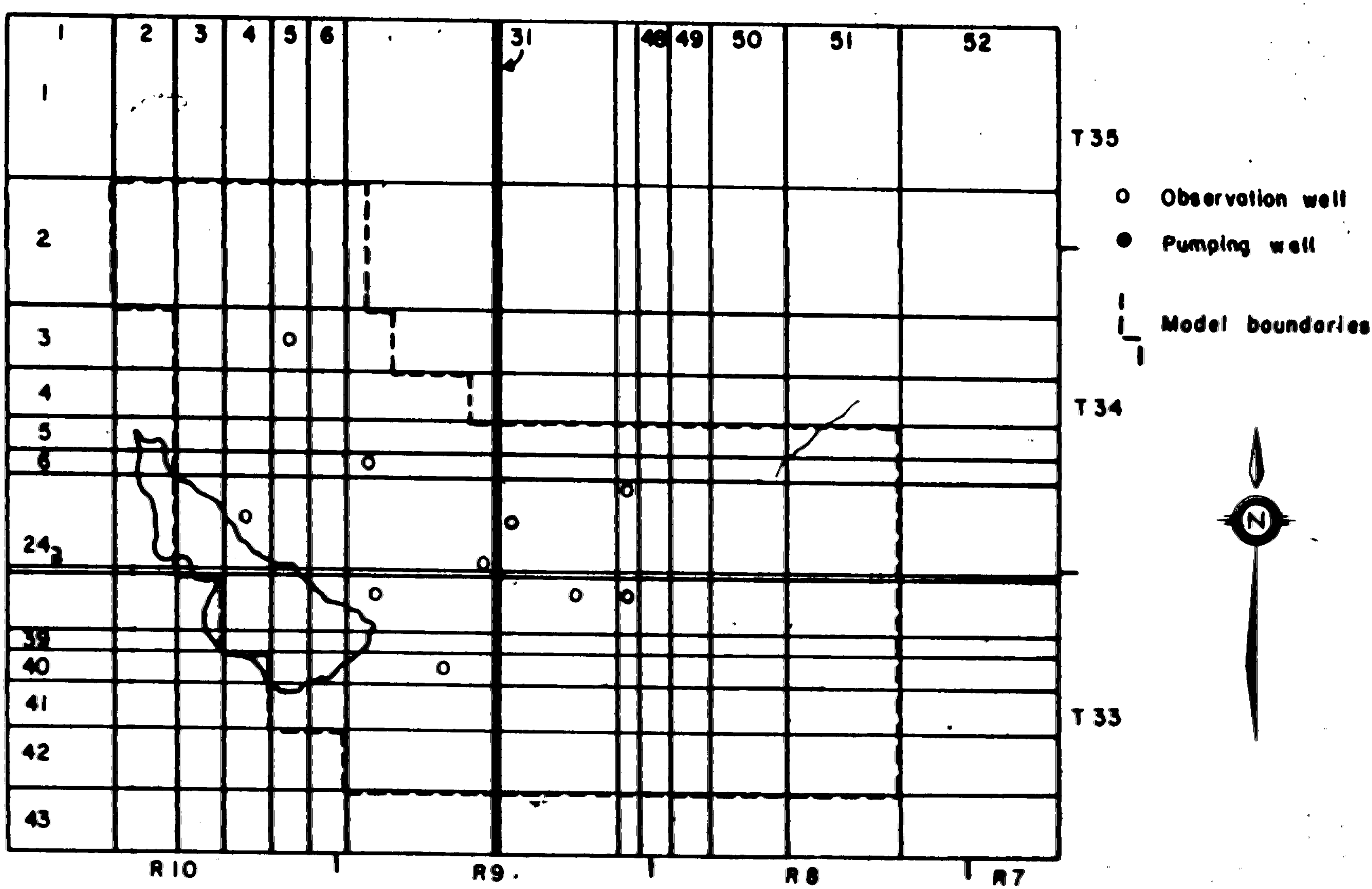


Figure 12. Grid system for the study area (small grid widths not numbered)

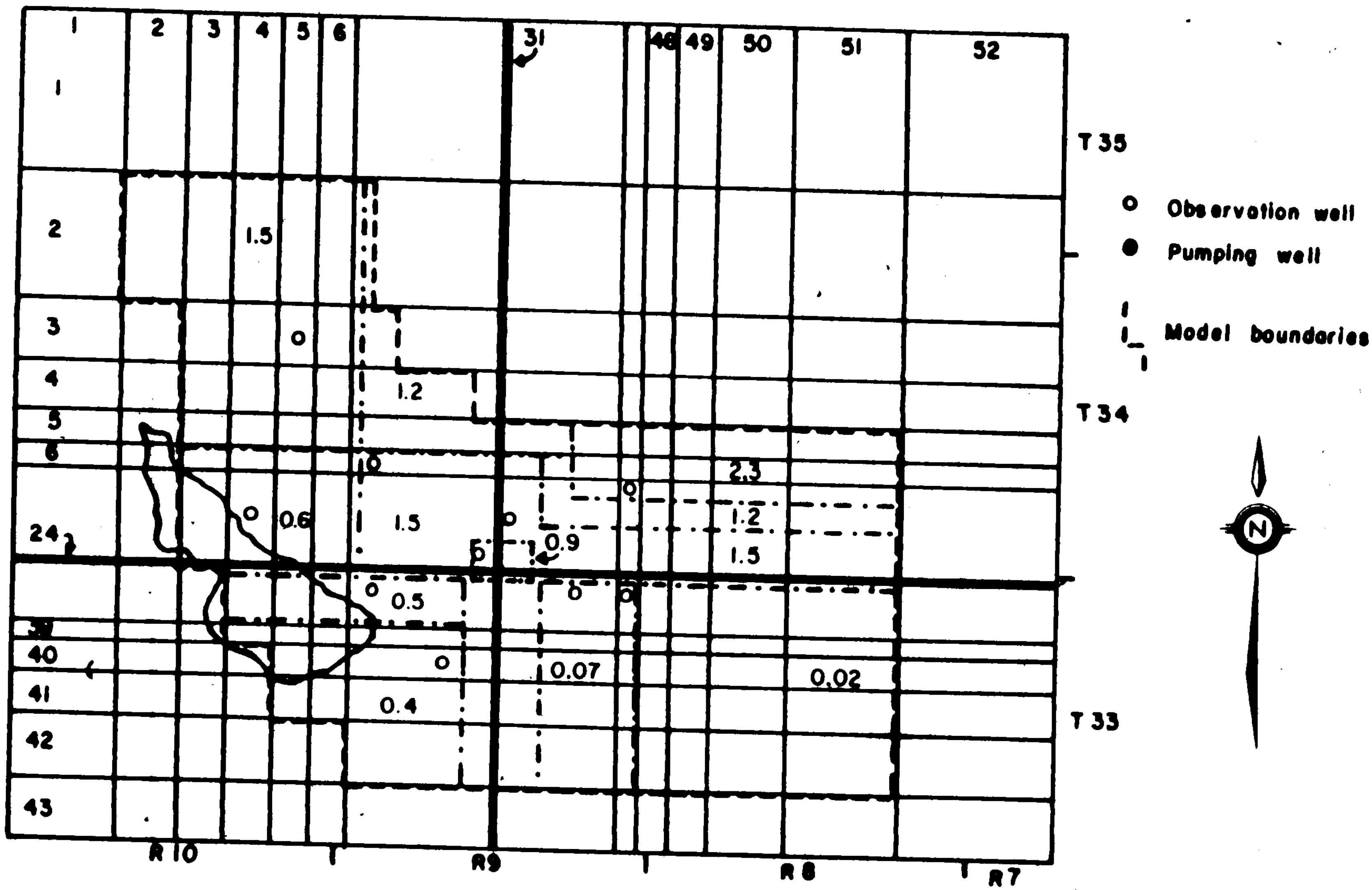


Figure 13. Transmissivity for the model ($\times 10^{-3} m^2/s$)

The values used in the preparation of a grid of storativity (See Fig. 10) could only be determined at locations where pumping tests were conducted with observation wells. The storativity grid used in the model was prepared using only the storativity values determined from early results of the long-term pumping test. The values ultimately used in model simulations which vary approximately one order of magnitude, from 6×10^{-4} to 80×10^{-4}, are shown in Figure 14. Storativity, in general, is lower in the vicinity of the production well and to the southeast. It tends to increase to the southwest, the northwest and the northeast.

The transmissivity and storativity values on Figure 13 represent those values which were established through trial and error variation in an attempt to obtain the parameters which would result in the best correspondence between computer predicted drawdown and actual drawdown. The values ultimately used for model simulations varied less than one order of magnitude from those derrived from the analyses of field data. A comparison of Figures 7 and 13, and Figures 10 and 14 indicate that in general, storativity values had to be decreased and transmissivity values had to be increased in order to obtain the best match between field and model data.

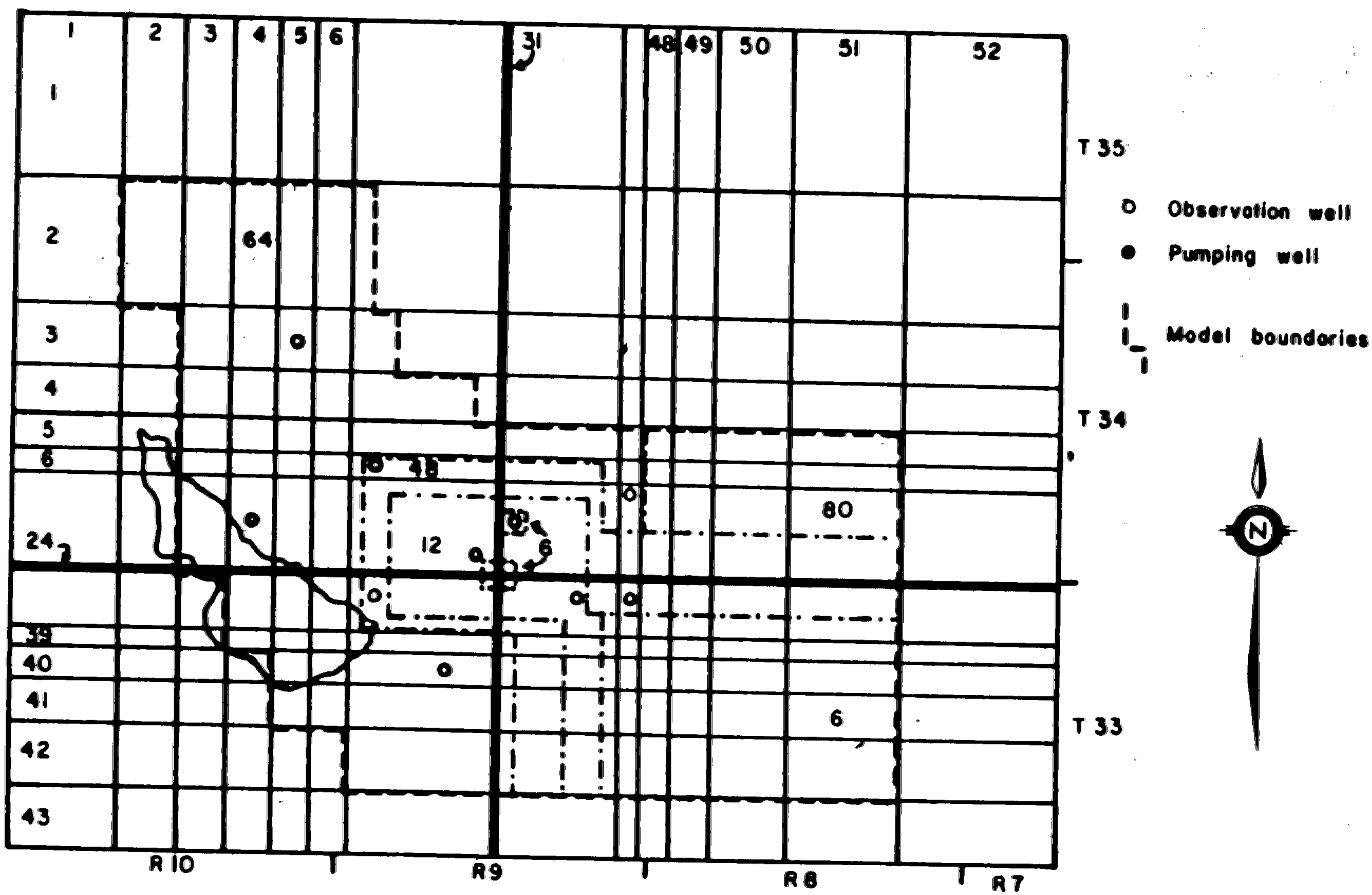


Figure 14. Model storativity (x10^{-4})

VI. MODEL SIMULATION AND RESULTS

A. Kirkpatrick Lake Study Area

This section describes first, the comparison of analytically and digitally derived solutions, second the various models, which are each potentially representative of the hydrogeological conditions in the study area, and third the application of the model to the investigation of the efficiency with which a well can intercept water moving through an aquifer.

Before any attempts were made to utilize the simulation model, tests were carried out to ensure that it had been modified to operate on the University of Alberta Amdahl computer. The test data used to verify the model are shown in Figure 15. Boundaries for the domain are assumed to be impermeable, and transmissivity and storativity values are constant for the non-leaky, artesian, test aquifer. The boundaries are located sufficiently far from the pumping well that the aquifer can be assumed to be infinite for the time of simulation. The model verification is carried out by comparing results of the simulated drawdown at two observation points with the drawdown calculated from the analytic solution for the same problem. Early-time divergence between analytically derived drawdown values and values determined by computer simulation is greater than late-time divergence. This early-time divergence is consistent with results obtained by others during similar simulations. Prickett and Longquist (1971) suggest that this error results from discretizing time for successive time steps during simulation. Thus, calculated drawdowns are somewhat distorted during early parts of the pumping phase. Accuracy can be improved if necessary by decreasing the size of the time increment. However, in practice it is convenient to place less emphasis on results from at least the first few time steps. The close correspondence of the calculated drawdown from the two methods indicate that the model is operating satisfactorily. (See Table 7, Figures 16 and 17).

Attempts to select field conditions on which to base simulations suggest, initially at least, that five different but equally possible arrangements may be appropriate. As a result, five sets of conditions are simulated during modeling in an attempt to evaluate the possible responses of the system to pumping and provide a basis for estimating the most likely long-term behavior of the system. The five scenarios for recharge can be

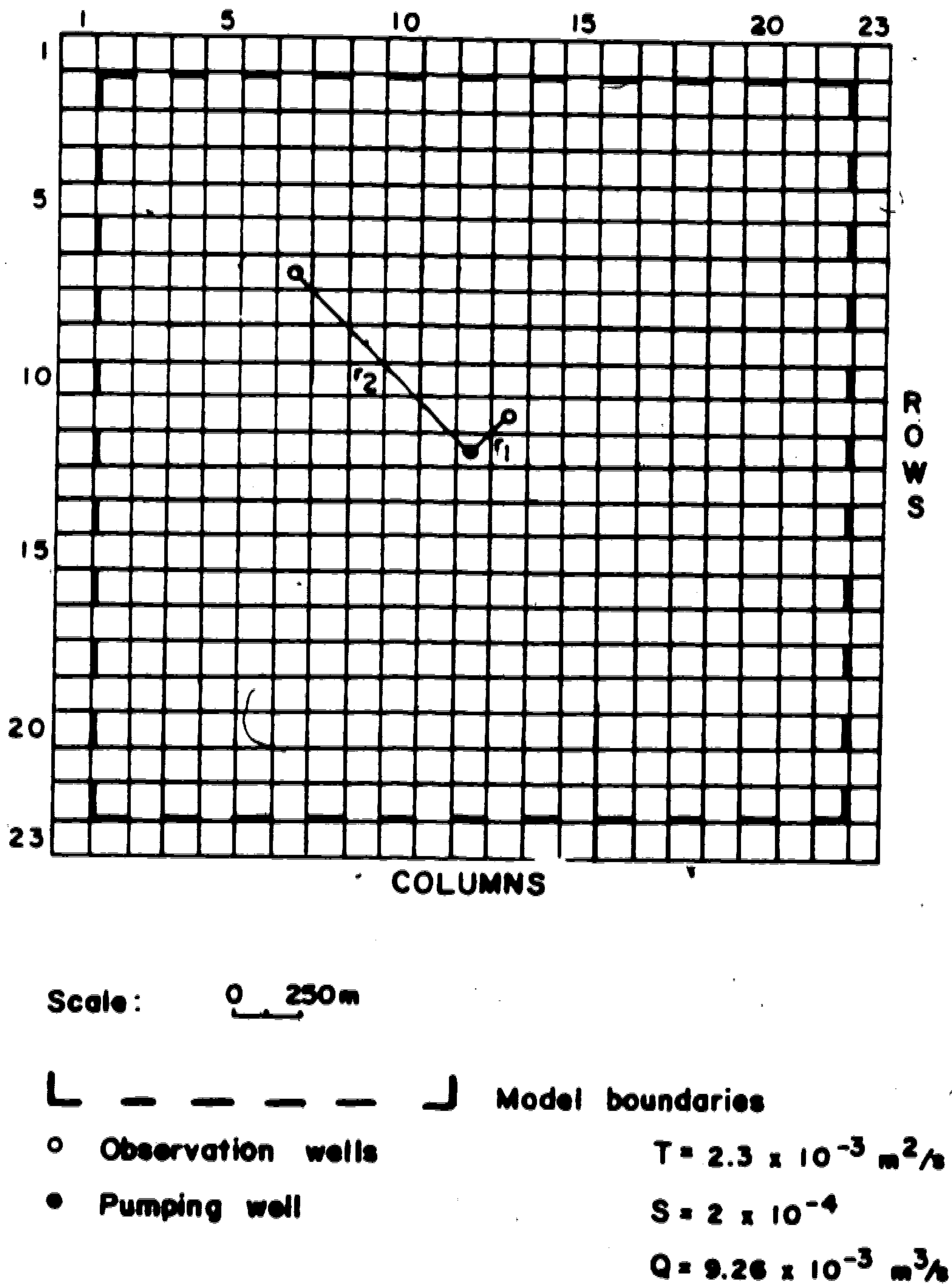


Figure 15. Grid for hypothetical test problem

TABLE 7. Comparison of analytically calculated drawdown with computed drawdown

t[1]	u[2]	W(u)[3]	r[4]	Calculated drawdown(m)	Computed drawdown(m)
0.56	140[5]	3.71	176.8	1.18	1.00
1.41	56	4.61		1.46	1.40
2.67	29	5.29		1.68	1.70
4.57	17	5.80		1.85	1.80
7.42	11	6.24		1.98	2.00
11.69	6.7	6.73		2.14	2.20
18.10	4.3	7.17		2.28	2.30
27.70	2.8	7.60		2.42	2.40
0.56	239[6]	0.03	2312.2	0.01	0.00
1.41	95	0.24		0.08	0.10
2.67	50	0.56		0.17	0.20
4.57	29	0.93		0.30	0.30
7.42	18	1.31		0.42	0.40
11.69	11	1.74		0.55	0.50
18.10	7.4	2.10		0.67	0.70
27.70	4.8	2.51		0.80	0.80

t[1] = Time since pumping began (min.)
u[2] = Argument
W(u)[3] = Theis well function
r[4] = Distance from pumping well (m)
140[5] = $x10^{-4}$
239[6] = $x10^{-8}$

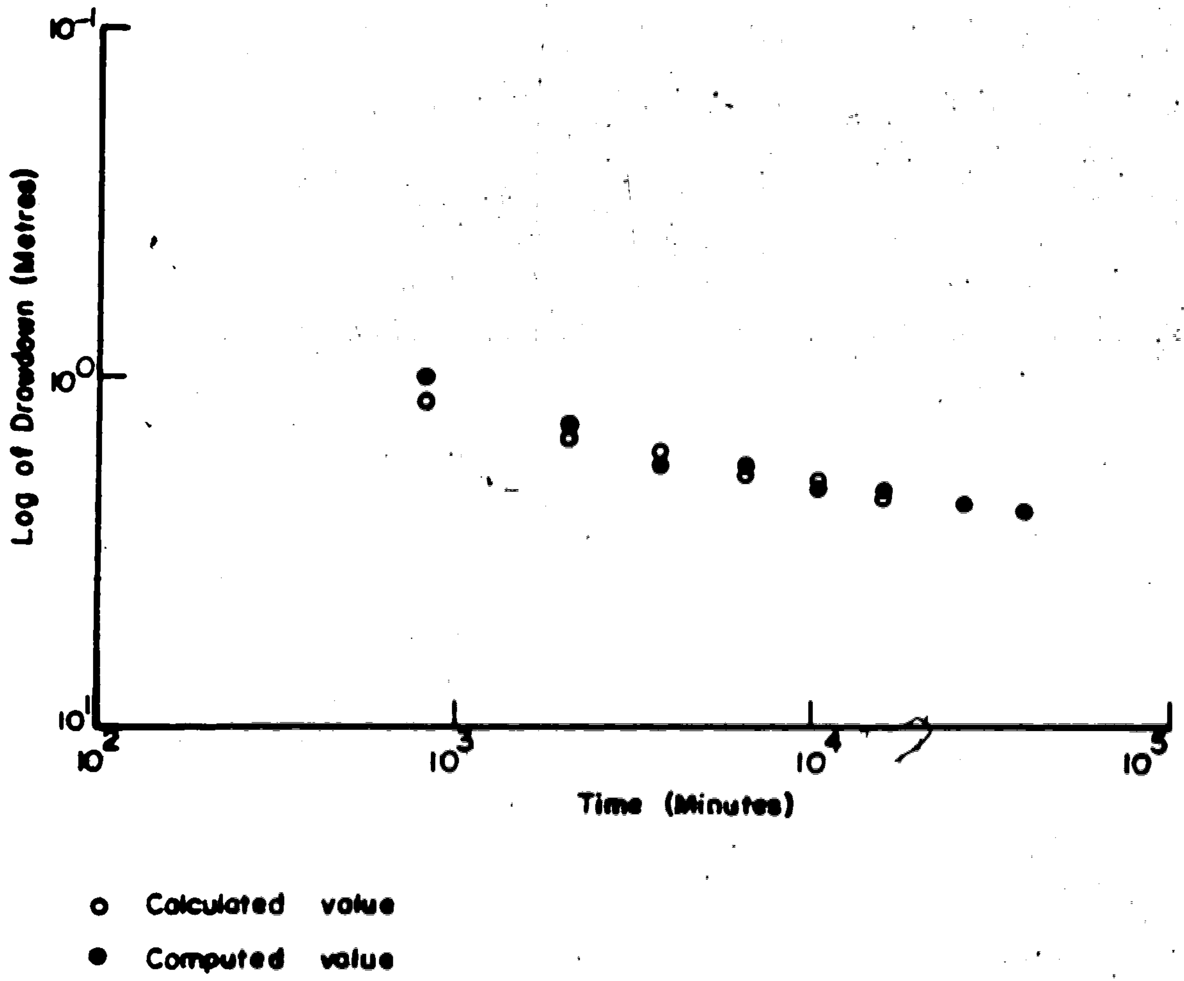


Figure 16. Comparison of Theis values of drawdown with model predicted drawdown for r=176m

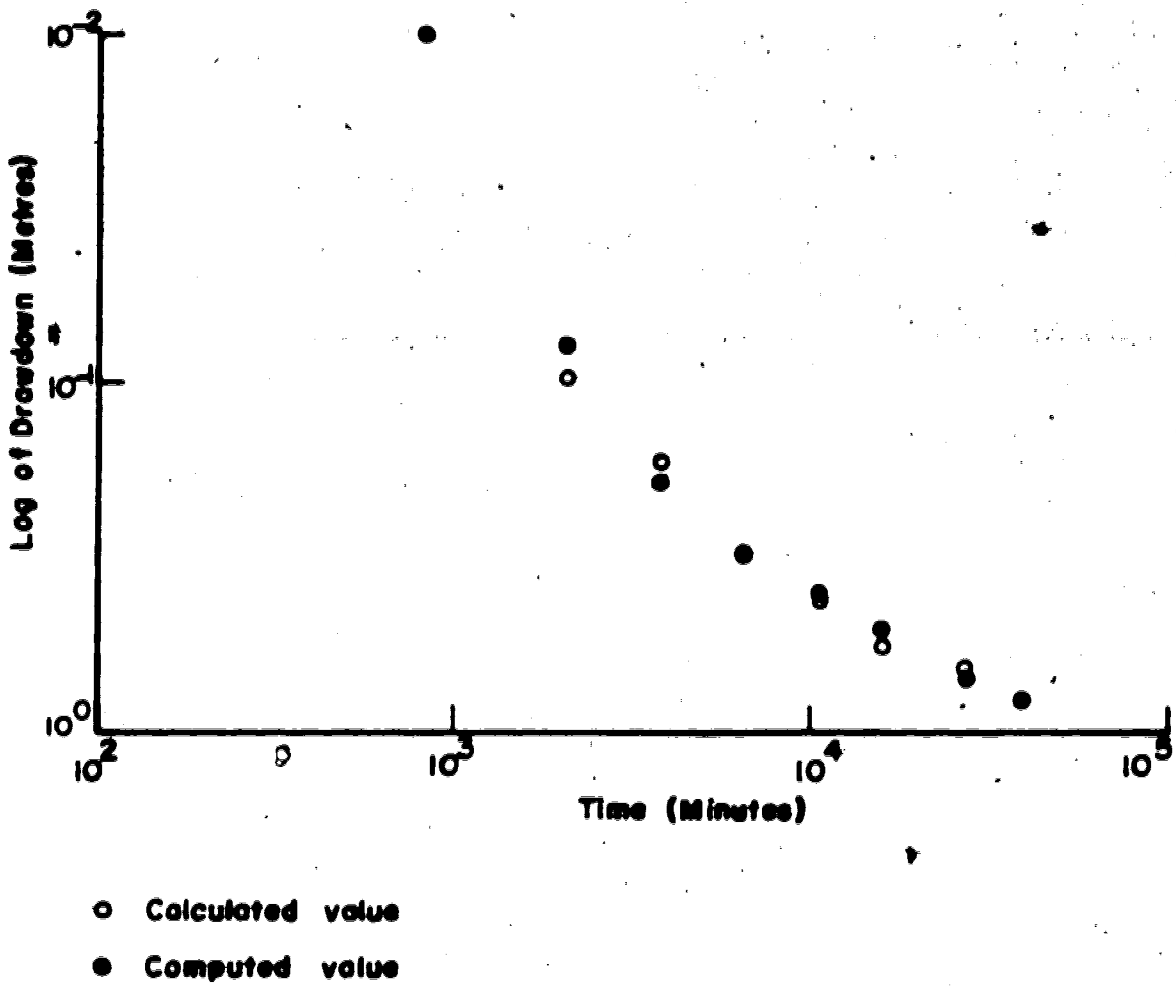


Figure 17. Comparison of Theis values of drawdown with model predicted drawdown for r=2312.2m

summarized follows:

Model 1 – Withdrawal of ground water from a system where recharge is negligible.

Model 2 – Withdrawal of ground water from a system where induced recharge from a lake is approximated by a constant boundary.

Model 3 – Withdrawal of ground water from a system with leakage from the overlying confining shale unit.

Model 4 – Withdrawal of ground water from a system with the lake a constant head, and leakage from the overlying confining shale unit.

Model 5 – Withdrawal of ground water from a system where recharge from the lake is controlled by the hydraulic conductivity of the sediments of the lake bottom.

The five model systems differ from each other only in the type of recharge conditions. Recharge to the aquifer can be modeled either using boundary conditions providing flux or leakage through confining units.

Model 1 represents a case where no recharge is assumed to occur. The south, west and northeast boundaries are considered to be impermeable and hence hydraulic barriers. Those to the north and east are located sufficiently far from the pumping well to effectively remove their influence during simulation.

Model 2 represents a case where recharge from Kirkpatrick Lake is assumed to occur. The main difference between model 1 and model 2 is that in model 2 the lake is considered to be penetrating the aquifer where it subcrops and is thus in direct hydraulic connection with the aquifer. During simulation the lake, which is the western boundary, is represented as a constant head boundary and may recharge the aquifer along a 5 km front, once the cone of depression reaches the boundary. The southern and northeastern boundaries are considered to be impermeable, and the northern and eastern boundaries are located sufficiently far to effectively remove their influence during the simulation.

Model 3 represents a case where recharge in the form of leakage is assumed to occur. The difference between model 3 and model 1 is the leakage from the confining shale unit which occurs in model 3. The confining shale unit has been assigned a vertical hydraulic conductivity of 10^{-11} m/s which could be typical for a bentonitic and silty shale (Kruseman and DeRidder, 1970) such as occur in the study area. Model 3 and model 1 have identical boundary conditions.

Model 4 represents a case where recharge from the lake, which is represented as a constant head as well as from an overlying confining shale unit, is assumed to occur. The boundary conditions for model 4 are similar to model 2. Additional recharge is now possible as leakage from the overlying confining shale. Model 4 is essentially a combination of models 1, 2 and 3.

Model 5 represents a case where leakage can occur as induced infiltration from Kirkpatrick Lake with actual quantities controlled by the hydraulic character of the lake bed sediment. The aquifer in which the pumping well is completed subcrops beneath the lake and, for the model 5 simulation is considered to be in hydraulic connection with the lake. Field investigation indicates that the mucky, organic rich lake bottom sediment is predominantly silt and clay. Initially, because of lack of actual field data, the lake bottom sediment is assigned a thickness of one metre and a hydraulic conductivity of 1×10^{-7} m/s for simulation purposes. The southern and northeastern boundaries are considered to be impermeable and therefore hydraulic barriers. Those to the north and east are located sufficiently far to effectively remove their influence during simulation.

The calibration of the model is intended to demonstrate which set of hydraulic parameters and boundary conditions satisfactorily represents the actual hydrogeologic conditions which occur in the field. The five sets of model systems are designed to represent a spectrum of possible situations that should include the actual conditions. Because of the good quality of data available for the area, it is felt that most of the uncertainty in representing the systems rests with the recharge conditions.

The computed drawdowns for each of the five models are compared (See Table 8) to the actual drawdown values obtained from a 24 day pumping test, conducted at a rate of $1.06 \times 10^{-3} m^3/s$, during which the water levels of 11 observation wells, completed in the same aquifer as the pumping well, were monitored (Clissold, 1972). Figure 18 presents, in graphical form, the variation between observed and model 1 computed drawdown values. Except for one obvious point, corresponding to observation well 3C-71 located about 6 m. from the pumping well, the match between simulated drawdown values and observed drawdown values is satisfactory for each of models 1 to 5. These results represent the best match after attempts made to improve the match between observed and computed values by trial and error varying of transmissivity,

TABLE 8. Summary of actual drawdown and drawdown computed by models 1 to 5 for the pumping and observation wells after 24 days of pumping

Well No.	DD'	Computed drawdown (m)				
		1	2	3	4	5
3A-71	18.23	19.7	18.6	18.7	18.8	18.7
3B-71	15.01	14.9	14.7	14.9	14.9	14.9
3C-71	5.71	8.4	8.3	8.4	8.4	8.4
11-71	3.73	4.1	4.2	4.1	4.1	4.1
12-71	3.23	2.7	2.7	2.7	2.7	2.7
9-71	1.80	1.6	1.7	1.6	1.6	1.6
6-71	0.13	0.2	0.2	0.2	0.2	0.2
8-71	0.48	0.4	0.4	0.4	0.4	0.4
TF²	0.07	0.3	0.4	0.3	0.4	0.3
10-71	0.10	0.2	0.2	0.2	0.2	0.2
7-71	0.04	0.1	0.1	0.1	0.1	0.1
5-71	0.00	0.0	0.0	0.0	0.0	0.0
4-71	0.00	0.0	0.0	0.0	0.0	0.0

DD' = Actual drawdown (m)
TF² = Thulien farm well

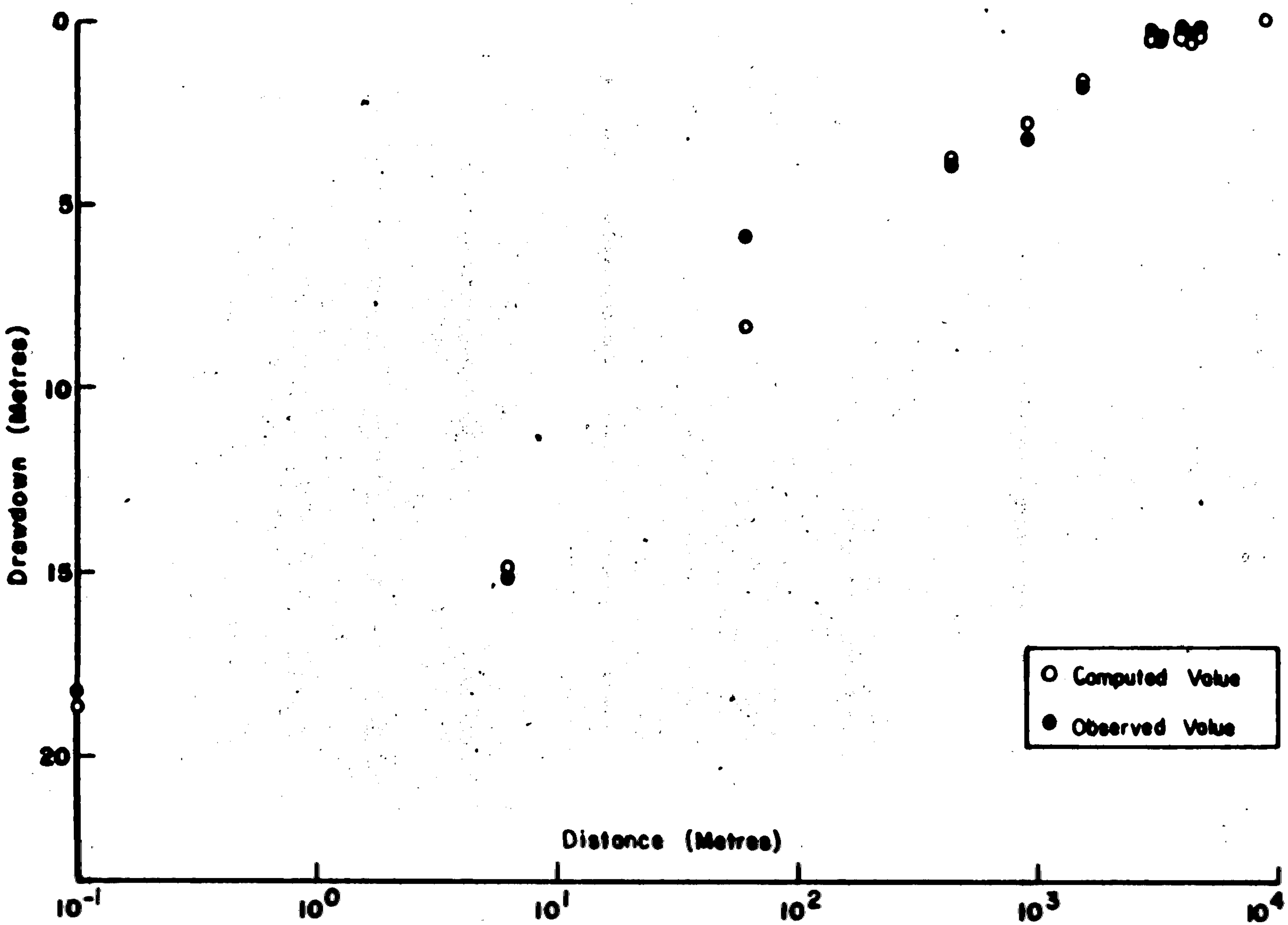


Figure 18. Comparison of observed and computed drawdown values using model 1

storativity and leakage within reasonable limits.

The simulated drawdown at the pumping well exceeds actual drawdown by about 1.5m (8%) for a simulation in which recharge is negligible (model 1). In simulations which have some recharge (models 2,3,4 and 5), the simulated drawdown at the pumping well exceeds actual drawdown by about 0.6m (3%). The actual drawdown at the closest observation well (3B-71) exceeds the computed drawdown by about 0.1m, or less than 1%. Simulated drawdown for observation well 3C-71 is consistently much higher than the actual drawdown for all five cases, and as a result some problem with the original field data is suspected. Actual drawdown values for observation wells 11-71, 12-71, 9-71, 6-71 and 8-71 deviate about 10% from computed values in all five cases. The general trend of the computed drawdown values is similar to that of the actual drawdown for observation wells 10-71, 7-71 and Thulien's farm well located approximately five kilometres from the pumping well. The results do however, deviate up to 0.1m from the actual values, an error approaching 50%. Attempts to improve the match were unsuccessful. No model seems to represent the actual hydrogeologic conditions any better than the others and as a result does not yield computed drawdown values which fit the observed values any better, at least not after a period of 24 days of pumping.

Original field data (Clissold, 1972) are no longer available to review in an attempt to explain the anomalous drawdown value for observation well 3C-71. It may be significant that when the observation wells were installed, well 3C-71 was the only one with which a preliminary short-term pumping test was not conducted. Although the reason for the lack of a test test could not be confirmed, it is possible that some difficulties were encountered during the construction and development of the well. One possible explanation for the apparent anomalous behavior of observation well 3C-71 could be related to the fact that it has been completed at a location where the bedrock is the shallowest observed in any of the test holes drilled. The shallow bedrock could have been fractured by the over-riding glacial ice of the Wisconsin ice advance resulting in an area where recharge from the surface occurs very readily. Very rapid recharge to the well through a fracture network could result in less drawdown in the observation well than the simulation predicts.

Figure 19 shows a comparison of observed and computed drawdown using model 1 after 4, 8, 12, 16, 20 and 24 days for the pumping well and observation wells. It should be noted that the correspondence between observed and computed drawdown after pumping of the production well for periods of 4, 8, 12, 16 and 20 days is about the same as that for the full pumping period of 24 days. Similar results, although not included here, are achieved using models 2, 3, 4 and 5.

The results of the simulation trials show that each of models 1 to 5 can equally well simulate the hydrogeologic response of the aquifer system at least up to a time of 24 days. It appears that the aquifer test was not of sufficient duration, when combined with the sufficiently low recharge rate, to establish whether Kirkpatrick Lake will interact with the aquifer system over the longer term. Yet, a 24 day pumping test would be considered by most hydrogeologists to be a relatively long-term test. These results call attention to the general problem of being unable, with a single test, to evaluate pertinent boundary effects.

In order to evaluate the long-term response of the aquifer to pumping at a rate of $1.06 \times 10^{-3} m^3/s$, each of the five models was used as the basis for a 10 year simulation. Table 9 is a summary of the decline of water level in the pumping well with time for each model. Using model 1 of the system (no recharge), calculated drawdown exceeds the available head of 21.4m in the pumping well in just one year of pumping and reached a total decline of 42.68m in ten years. With model 2 there is a reduced rate of total water level decline, although drawdown also exceeds that available in just one year with a total decline of 26.23m after ten years. Model 3 exceeded available drawdown in one year, ultimately declining to a level of 38.83m, not significantly different from results of model 1. With model 4, drawdown in the pumping well exceeds available drawdown in one year but resulted in the least total drawdown of any of the models used, 25.75m at the end of 10 years simulation. With model 5, drawdown exceeds that available in one year and gives rise to the second lowest total drawdown after a 10 year simulation, 27.1m. Model 5 is selected for a longer-term simulation in order to determine the time required for steady state conditions to develop. Steady-state conditions are attained by year 16 with no further changes in head at later times. The maximum drawdown was 27.1m resulting in a water level which was 5.7m below the upper limit of the aquifer.

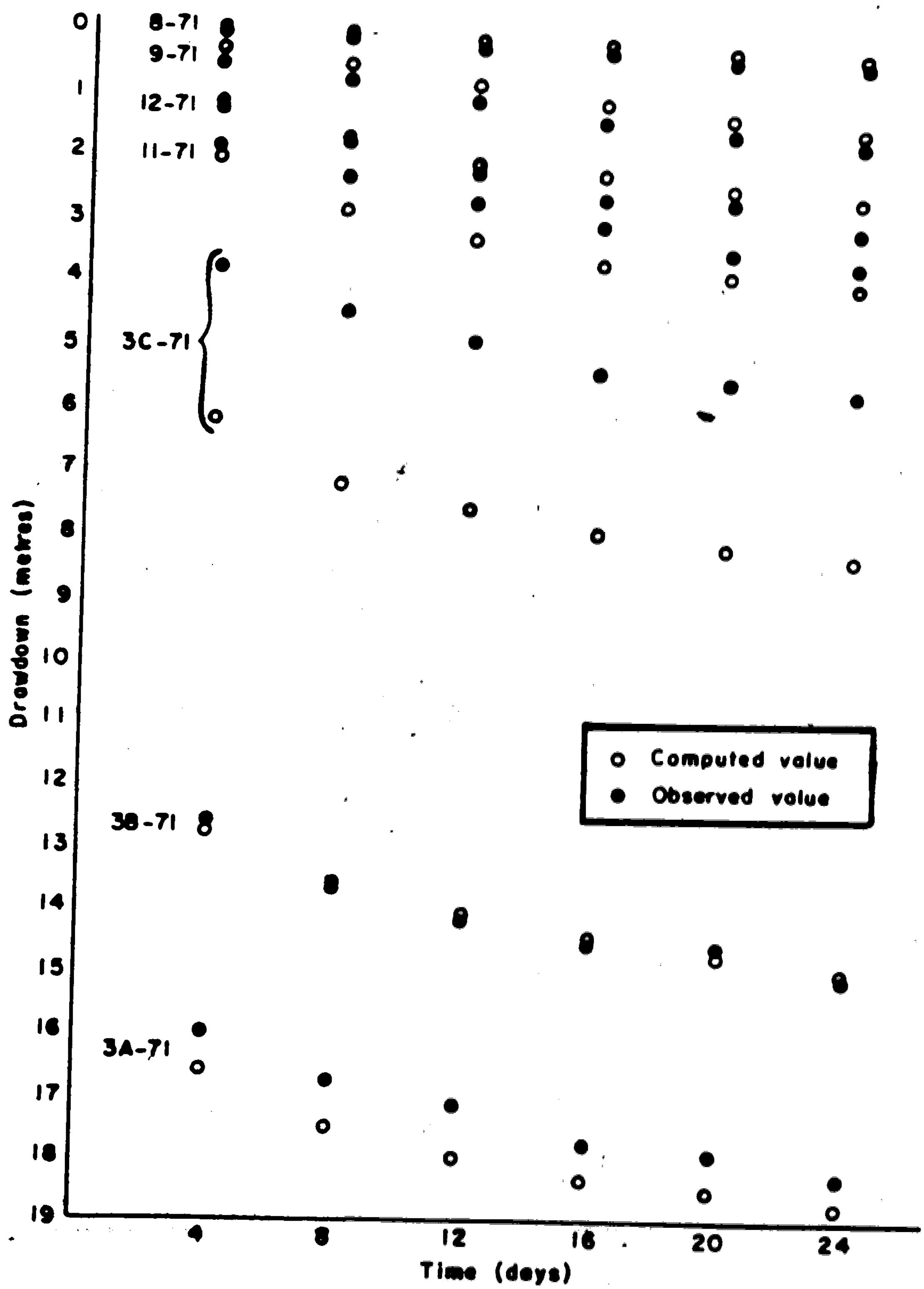


Figure 19. Comparison of observed and computed drawdown values from model 1 after 4, 8, 12, 16, 20, and 24 days of pumping

TABLE 9. Predicted drawdown' at the pumping well over a period of 10 years

t'	1	2	3	4	5
0.20	19.5	19.4	19.5	19.5	19.5
0.51	21.1	21.1	21.0	21.0	21.1
0.96	22.5	22.3	22.4	22.2	22.5
1.65	24.2	23.3	24.0	23.2	23.6
2.68	26.5	24.3	26.2	24.2	24.6
4.22	30.0	25.2	29.2	24.9	26.0
6.53	35.1	25.8	33.3	25.5	26.8
10.00	42.7	26.2	38.8	25.7	27.1

drawdown' = In metres for each of five simulations
t' = Time since pumping began (years)

It is unfortunate that the calibration technique did not verify that one of the five models considered during simulation would be more appropriate than the others for use in predicting the effects of continued ground water withdrawal and hence be suitable as a ground water resource management tool. Model 5 was selected for the 20 year simulation as this model, qualitatively at least, is possibly the most appropriate for the following reasons. Firstly: the static water levels in test holes completed in the zone I aquifer declined away from Kirkpatrick Lake towards the north-east suggesting that recharge from the lake is a distinct possibility. Secondly, the study area is characterized by saline soils indicating considerable evapotranspiration and if the Thornthwaite method is used, the area is characterized by having a net deficit of soil moisture (LeBreton, 1969) and, theoretically at least, little or no recharge. Hence, model 5, with leakage from Kirkpatrick Lake the only source of recharge to the zone I aquifer could reasonably be selected for long-term simulations of the effects of ground water withdrawal in the study area. The 20 year time period has been chosen for the extended simulation as this was the expected duration of the pumping of ground water for injection purposes. Long term production of ground water at the rate proposed will, based on model predictions, result in the decline of the available head to the point where dewatering of the aquifer will take place. At this time it is likely that the rate of decline of the water level will accelerate as the effective transmissivity decreases. This decline may be slowed or halted once the cone of depression has increased enough in size to begin accepting recharge from Kirkpatrick Lake. However, by this time the cone of depression will be large enough to interfere with the water supplies of domestic users in the area. The severity of interference can only be evaluated when the available heads for the wells being intefered with are known. Thus, it seems likely that long-term production of ground water at the proposed rate of $1.06 \times 10^{-2} m^3/s$ is, to say the least, impractical.

B. Theoretical Analysis of Recharge

Long-term ground water management strategies require some understanding of the effects various aquifer parameters have on the efficiency with which a well (or wells) can intercept recharge. The remainder of the simulations in this section are designed to evaluate how effective a pumping well can be in appropriating recharge in an active ground water flow system. The basic approach involves a sensitivity analysis of the response of a model of a large flow system to pumping. With recharge explicity defined, it is possible to relate it, with mass balance calculations, to the source of water withdrawn from the aquifer. Figure 20 describes the hypothetical ground water system, which forms the basis for this group of simulations. The grid system is comprised of twenty rows and twenty-five columns with a uniform distance between nodes of 1000 m. The five zones shown on Figure 20 represent five areas with differing rates of recharge. The actual magnitude of the recharge can be varied as desired for successive simulations. Zone I is part of the basin where no recharge is assumed to occur. Zone 3 has a recharge rate which is double that of zone 2. Zone 4 and Zone 5 have recharge rates which are three and four times respectively that of zone 2.

The hypothetical basin has no-flow boundaries to the north and west representative of a ground water divide and constant-head boundaries to the south and east, representative of a stream at those locations. Recharge in the form of direct precipitation is applied to the zones shown in Figure 20. As expected for steady state conditions, the hydraulic gradient declines away from the no-flow boundary in the zone of greatest recharge towards the constant-head boundary representing the stream. Stressing an aquifer in the basin by inserting a pumping well should result in the formation of a cone of depression which may intercept ground water moving through the aquifer possibly resulting in steady state head distribution. Comparison of the amount of water withdrawn from the system by the pumping well during the simulation with the amount of recharge introduced to the system as the result of direct precipitation, once steady state has been reached, should give an indication of how effective a well can be in the interception of recharge for that particular set of hydraulic conditions. The more effective the pumping well is, the greater will be the percentage of water derived from recharge.

All computer simulations involve the Trescott et al., (1976) model. For this type of simulation, it is necessary to input the hydraulic head distribution in the natural flow system as an initial condition. Thus, computed drawdowns are superimposed on the natural flow system. The first step in this simulation procedure is to compute initial hydraulic head distribution by setting storativity to zero, omitting any stress, such as

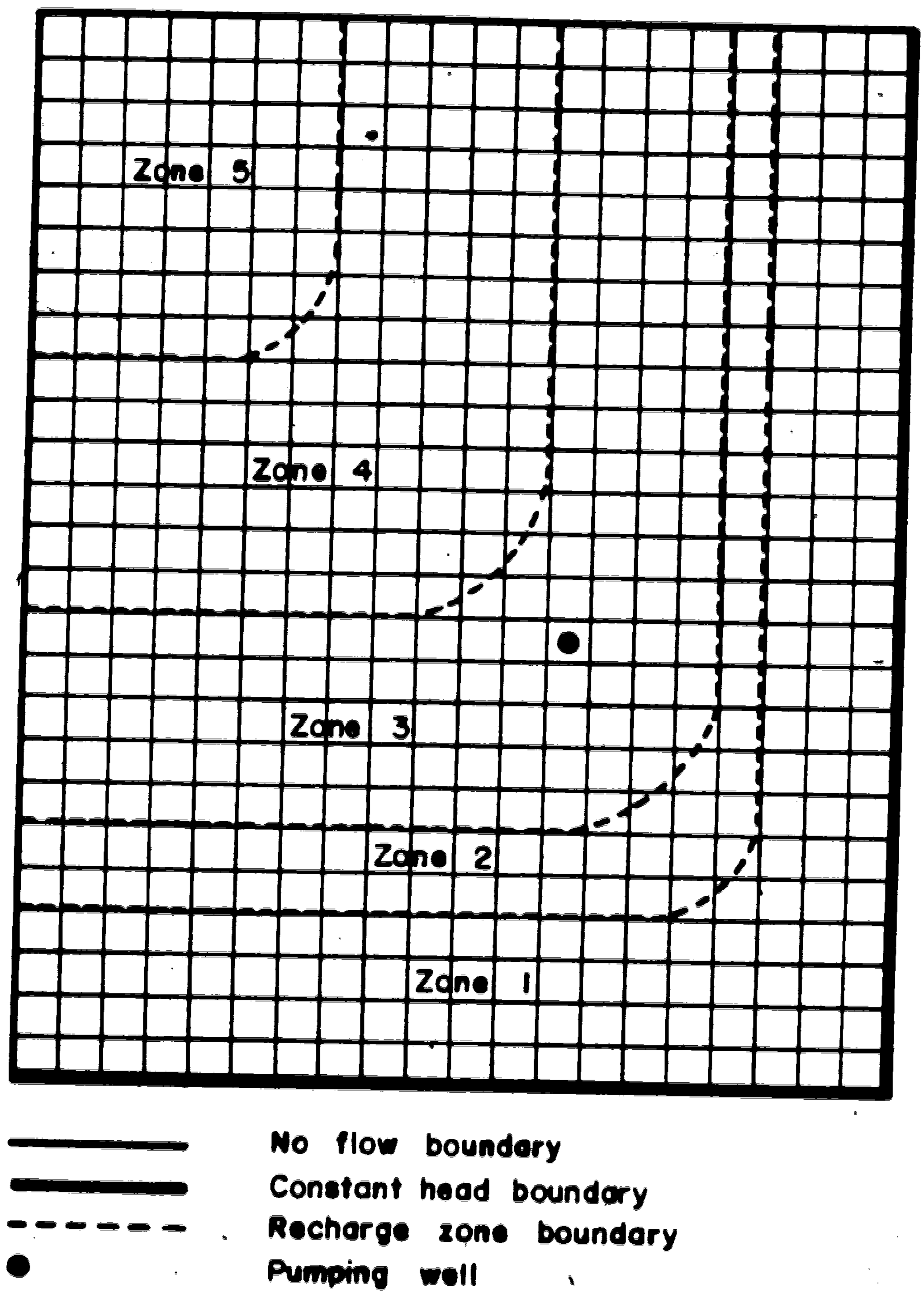


Figure 20. Hypothetical ground water basin

pumping, on the aquifer and allowing the simulation to proceed to steady state. Once the pre-pumping head distribution is obtained, it can be used as the initial head in the transient simulations comprising the second step in the modeling procedure.

The hydraulic parameters required for the transient simulations include recharge rate, storativity, transmissivity and pumping rate. The set of preliminary values, selected arbitrarily for the first simulation, are designed to be representative of actual geological systems. Transmissivity is $1.14 \times 10^{-3} m^2/s$, storativity is 1.0×10^{-3}, and recharge has been set at a maximum (that is, zone 5 on Figure 20) of $4.0 \times 10^{-10} m/s$, equivalent to ten percent of an assumed annual precipitation of 50 cm. Pumping rates are allowed to vary from $8.33 \times 10^{-4} m^3/s$ to $133.0 \times 10^{-4} m^3/s$. Simulations using the initial hydraulic parameters resulted in steady state conditions in a few days or a very few weeks at the most, even with the higher pumping rate. To obtain data from which conclusions might be drawn, the initial hydraulic parameters are varied in trial and error procedure in an attempt to choose those which would increase the time to reach steady state. Ultimately seven sets of hydraulic parameters are selected for simulations. Each of the seven simulations is run using five different pumping rates in an attempt to determine if pumping rate affects the effectiveness of the well at intercepting recharge for a given set of hydraulic parameters.

Tables 10, 11 and 12 present a summary of the simulation results, describing percent of recharge intercepted by a hypothetical pumping well for five different pumping rates and seven different sets of hydraulic parameters.

Storativity values are increased one order of magnitude for simulations one to two with other hydraulic parameters identical. The result is an increase in the length of time required for the simulation to reach steady state (See Table 13). The results are consistent for each of the five different pumping rates used for these simulations.

Simulations three and four are another pair of cases that vary only in the value of storativity. Again, the simulations with the higher storativity value require a longer period of time to attain steady state (See Table 13). The influence of storativity on the effectiveness of the well at intercepting recharge is probably explained by the shape of the cone of depression. Decreased storativity results in a cone of depression which increases in both radius and height. The resulting increase in area over which water

TABLE 10. Summary of computed percentage of pumped water which is intercepted recharge for various pumping rates and hydraulic parameters.

Time	Test 1					Test 2					Test 3				
(days)	PR¹	PR²	PR³	PR⁴	PR⁵	PR¹	PR²	PR³	PR⁴	PR⁵	PR¹	PR²	PR³	PR⁴	PR⁵
1.24	38	41	41	42	42	0	6	3	3	3	0	6	3	3	3
3.08	75	74	73	73	73	13	12	12	13	12	13	12	12	13	12
5.87	88	88	88	90	90	25	29	27	27	27	25	29	27	27	27
10.0	100	94	97	97	97	38	47	42	43	44	38	47	42	43	44
16.3		100	99	98	99	63	59	61	60	59	63	59	61	60	59
25.7			100	100	99	75	77	73	77	74	75	77	73	75	74
38.8					100	88	88	85	85	85	88	88	85	85	85
60.8						88	94	94	93	93	88	94	94	93	93
92.5						100	99	97	97	97	100	99	97	97	97
140							100	99	99	99		100	99	99	99
211								100	100	99			100	100	99
318										100					100
479															
719															
1080															
1620															
2430															
3650															

Pumping Rates

PR¹ 8.33 × 10^{-4} m^3/s
PR² 16.66 × 10^{-4} m^3/s
PR³ 33.32 × 10^{-4} m^3/s
PR⁴ 66.66 × 10^{-4} m^3/s
PR⁵ 133.32 × 10^{-4} m^3/s

TABLE 11. Summary of computed percentage of pumped water which is intercepted recharge for various pumping rates and hydraulic parameters.

Time (days)	Test 4 PR[1]	PR[2]	PR[3]	PR[4]	PR[5]	Test 5 PR[1]	PR[2]	PR[3]	PR[4]	PR[5]
1.24	0	0	0	0	0	0	0	0	0	0
3.09	0	0	0	0	0	0	0	0	0	0
5.87	0	0	0	2	0	0	0	0	2	2
10.0	0	6	0	2	0	0	6	3	6	5
16.3	0	6	3	5	4	13	12	12	13	13
25.7	0	12	9	9	9	25	24	24	24	24
39.8	13	18	15	18	17	38	35	36	37	37
60.8	25	29	27	30	29	50	53	52	50	51
92.5	38	41	42	42	42	63	65	64	66	65
140	50	59	55	57	57	75	77	76	78	77
211	63	71	70	69	68	88	88	88	87	87
318	75	82	82	81	81	100	94	94	94	94
479	88	88	88	90	90		100	97	97	98
719	99	94	94	96	96			100	100	99
1080	100	100	98	98	98					100
1620			100	99	99					
2430				100	100					
3650										

Pumping Rates

PR[1] = $8.33 \times 10^{-4} m^3/s$

PR[2] = $16.66 \times 10^{-4} m^3/s$

PR[3] = $33.32 \times 10^{-4} m^3/s$

PR[4] = $66.66 \times 10^{-4} m^3/s$

PR[5] = $133.32 \times 10^{-4} m^3/s$

TABLE 12. Summary of computed percentage of pumped water which is intercepted recharge for various pumping rates and hydraulic parameters.

Time (days)	Test 6 PR[1]	PR[2]	PR[3]	PR[4]	PR[5]	Test 7 PR[1]	PR[2]	PR[3]	PR[4]	PR[5]
1.24	0	0	0	0	0	0	0	0	0	0
3.09	0	0	0	0	0	0	0	0	0	0
5.97	0	0	0	0	0	0	0	0	0	0
10.0	0	0	0	2	1	0	0	0	0	0
16.3	0	6	3	5	4	0	0	0	2	1
25.7	0	12	9	9	9	0	6	0	3	2
39.8	13	18	15	18	17	0	6	6	6	5
60.8	25	29	27	30	29	13	12	15	12	11
92.5	38	41	42	42	42	13	24	21	24	21
140	50	59	55	57	56	25	35	33	34	33
211	63	71	70	69	68	50	47	46	46	47
318	75	82	82	81	81	63	59	61	60	60
479	88	88	88	90	90	75	71	73	73	72
719	99	94	94	96	96	88	82	82	84	84
1080	100	100	97	99	99	88	94	91	91	91
1620			100	99	99	99	94	97	97	96
2430				100	100	100	100	99	99	99
3650								100	100	99

Pumping Rates

PR[1] = 8.33×10^{-4} m^3/s
PR[2] = 16.66×10^{-4} m^3/s
PR[3] = 33.32×10^{-4} m^3/s
PR[4] = 66.66×10^{-4} m^3/s
PR[5] = 133.32×10^{-4} m^3/s

TABLE 13. Summary of computed time for a hypothetical well to reach steady state for various pumping rates and hydraulic parameters.

Simulation No.	T[1]	R[2]	S[3]	Number of years to reach steady state 08.33[4]	16.66[4]	33.32[4]	66.66[4]	133.32[4]
1	1.14	10	10^{-5}	0.03	0.04	0.07	0.07	0.11
2	1.14	10	10^{-4}	0.25	0.38	0.58	0.58	0.80
3	1.14	4	10^{-4}	0.25	0.38	0.58	0.58	0.80
4	1.14	4	10^{-3}	2.96	2.96	4.44	6.66	6.66
5	2.28	4	10^{-3}	0.87	1.31	1.97	1.97	2.96
6	1.14	2	10^{-3}	2.96	2.96	4.44	6.66	6.66
7	0.57	2	10^{-3}	6.66	10.00	10.00	NR[5]	NR[5]

T[1] = Transmissivity ($\times 10^{-3} m^2/s$)
R[2] = Recharge rate (percent) in zone 5
S[3] = Storativity
08.33[4] = Pumping rate ($\times 10^{-4} m^3/s$)
NR[5] = Steady state not reached

moving through an aquifer system can be intercepted makes it possible for the well to intercept recharge more effectively.

The quantity of recharge was also examined as a variable influencing the behavior of the aquifer. In simulations two and three, rates of recharge in zone 5 are ten percent and four percent of the annual rate respectively, with identical values of transmissivity and storativity. Although initial head distributions are different for each of the two simulations, there is no difference in the length of time required to reach steady state conditions for a specific pumping rate. Similar results are obtained for cases four and six where recharge rates in zone 5 are four percent and two percent respectively (See Table 13). It appears that varying the rate of recharge, while maintaining other parameters unchanged, does not effect the efficiency with which a pumping well can intercept water moving through an aquifer system. It is likely that there is a lower limit below which the recharge rate cannot be reduced for this result to hold.

Simulations four and five are based on identical rates of recharge and storativity but transmissivities are $1.14 \times 10^{-3} m^2/s$ and $2.28 \times 10^{-3} m^2/s$ respectively. The simulation with the higher transmissivity attained steady state more quickly than the simulation with a lower transmissivity for each of the five pumping rates. Similar results are obtained with cases six and seven which again differ only in transmissivity, ($1.14 \times 10^{-3} m^2/s$ and $0.57 \times 10^{-3} m^2/s$ respectively). Consistent with previous results, the simulation with the higher transmissivity attained steady state more quickly for each pumping rate.

Decreasing the transmissivity value while maintaining the same pumping rate results in a cone of depression which is increased in height but decreased in radius. Decreased transmissivity will also result in a reduction of the rate at which ground water will move through the aquifer system. The net result in the simulation when transmissivity is halved is a decrease in the efficiency with which a well can intercept recharge.

Results from simulations suggest that the efficiency of a pumping well to intercept recharge is not related to pumping rate when hydraulic parameters remain unchanged during simulations. Figures 21 and 22 present the results of a mass balance from two of the seven simulations, and indicate that generally, the efficiency of the pumping well at intercepting recharge does not vary more than five percent for pumping rates ranging over 1500% ($8.33 \times 10^{-4} m^3/s$ and $133.33 \times 10^{-4} m^3/s$). However, for a specific

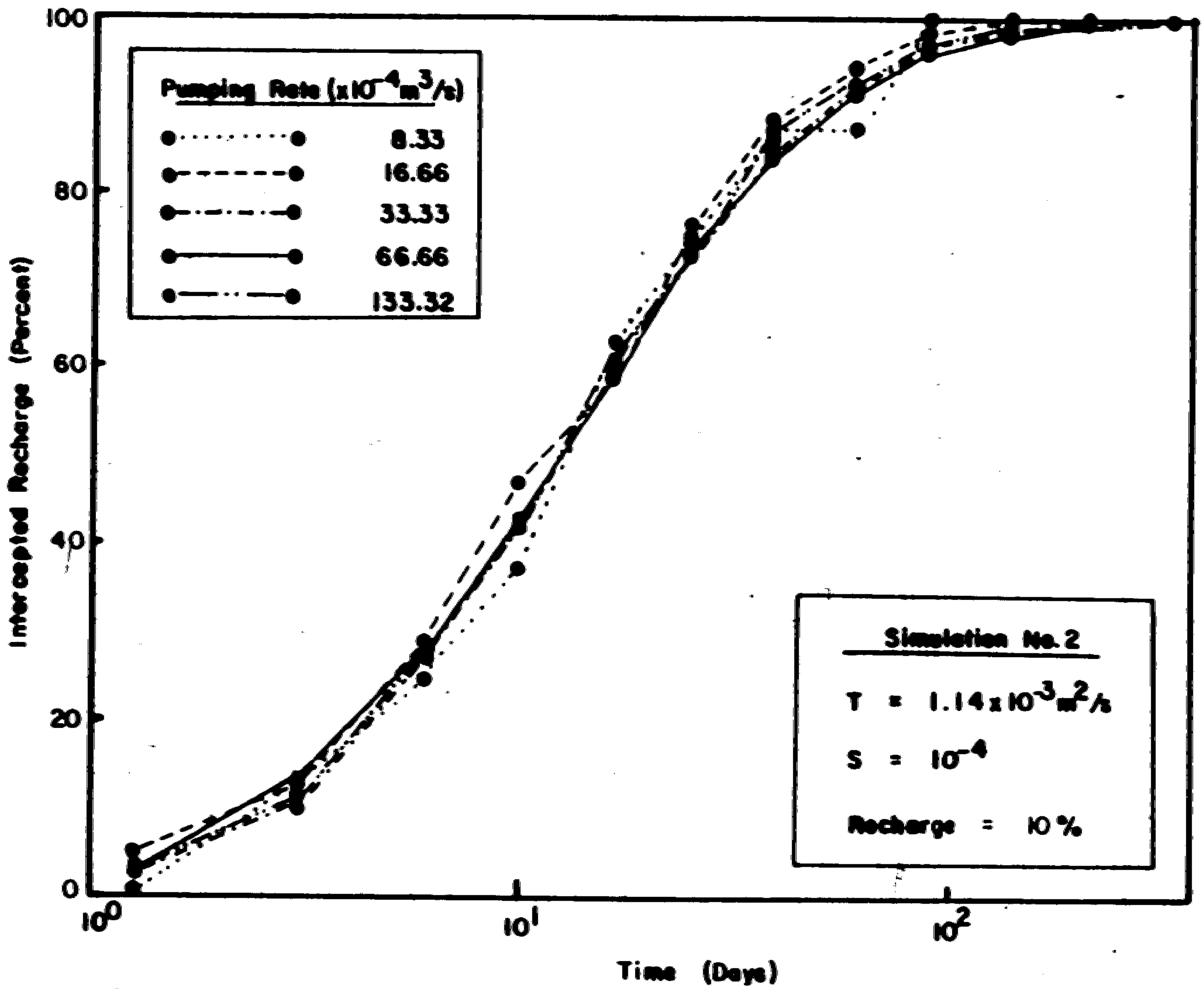


Figure 21. Change in amount of recharge intercepted with time for various withdrawl rates

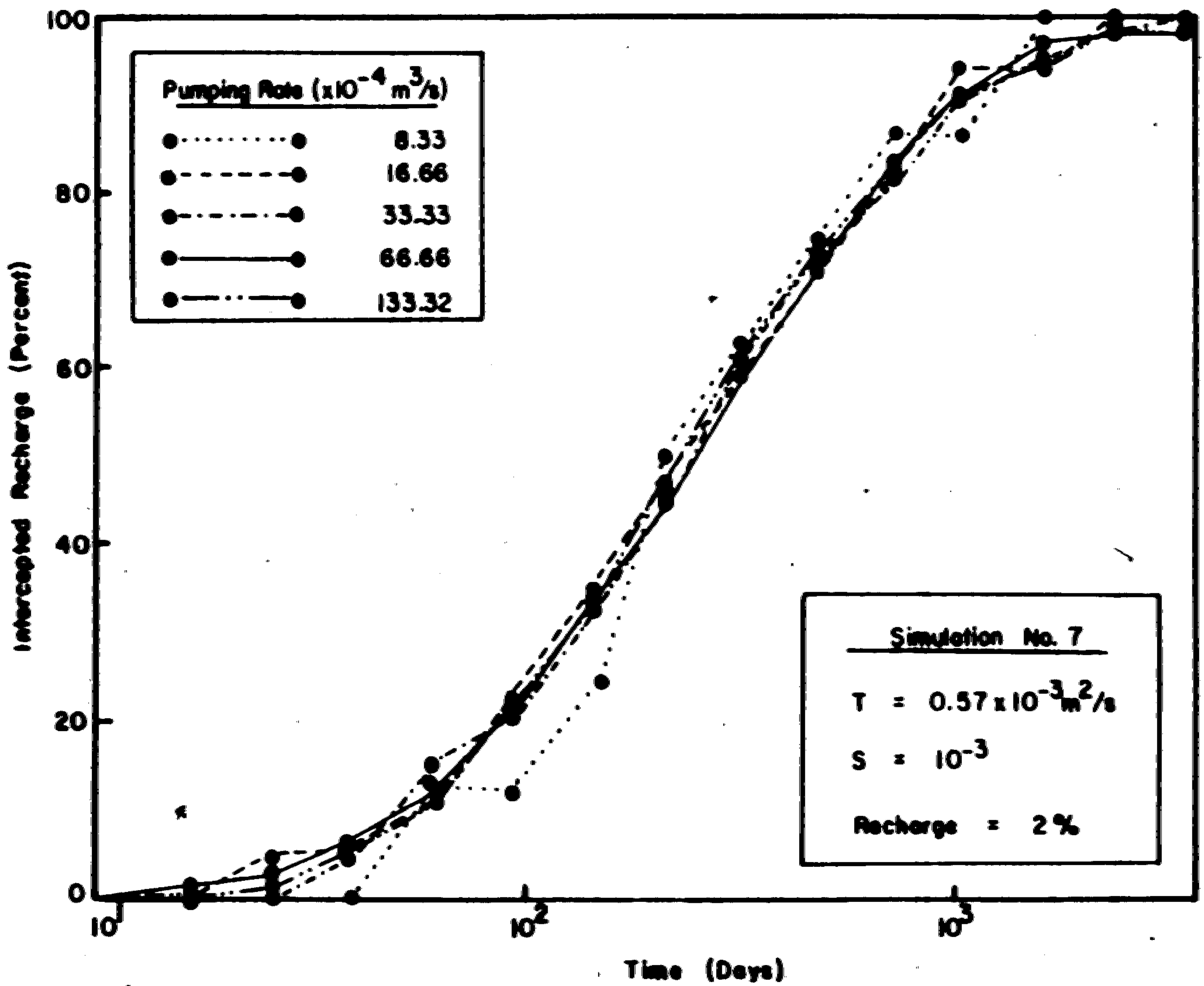


Figure 22. Change in amount of recharge intercepted with time for various withdrawl rates

pumping rate, varying hydraulic parameters results in variation in the length of time for the simulation to reach steady state conditions (See Figure 23). One exception noted is that if only the rate of recharge is varied (Tests 2 and 3, 4 and 6), there in no difference in the length of time required for the simulation to reach steady state conditions.

The seven sets of test data and the five pumping rates indicate that the efficiency with which a pumping well intercepts recharge varies according to the nature of the hydraulic parameters. Some variation of the parameters results in increased efficiency, some in decreased efficiency, and one parameter, the rate of recharge, at least appears to have little effect on the efficiency of the pumping well to intercept water flowing through an aquifer system.

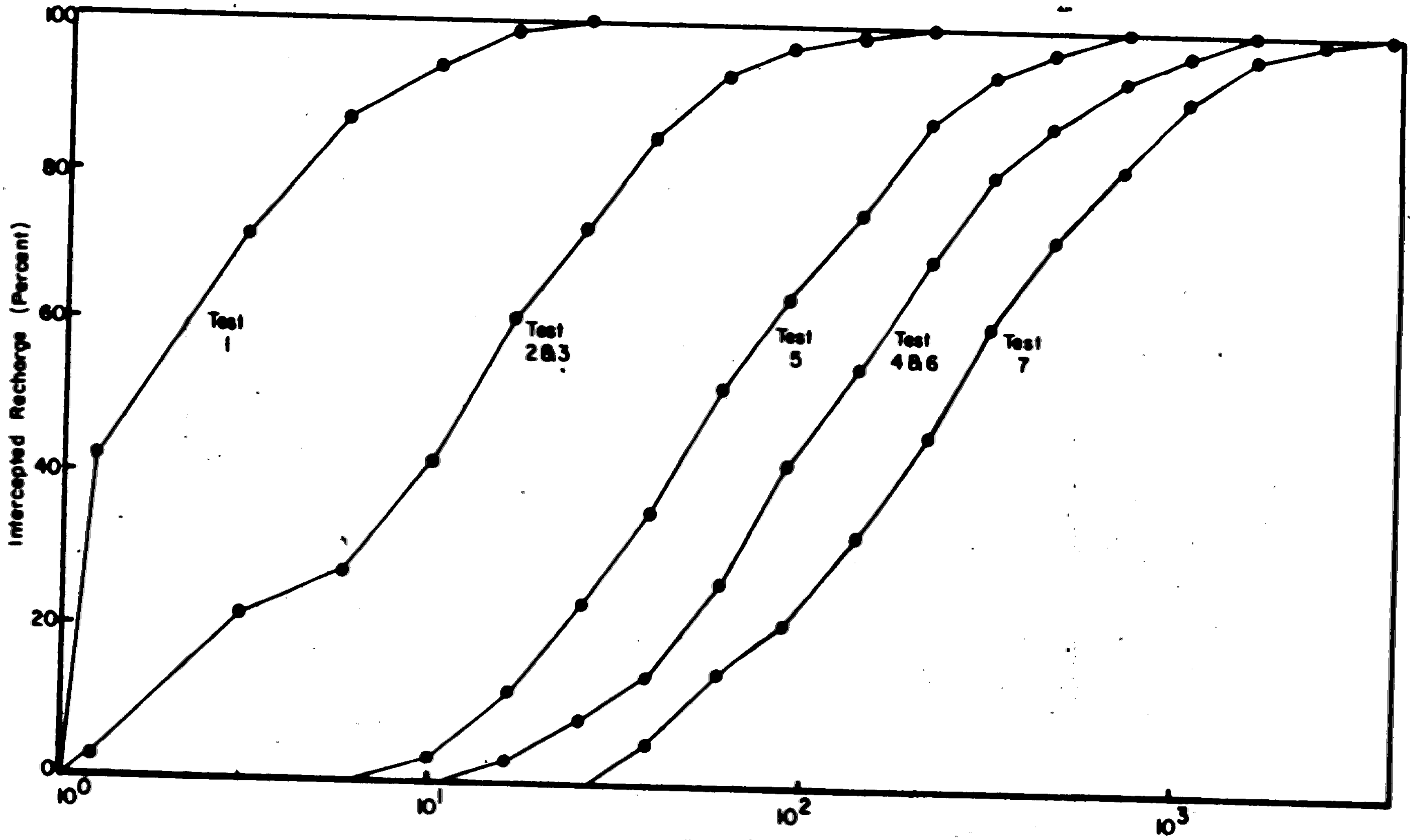


Figure 23. Percent recharge intercepted with time for a pumping rate of 33.32×10^{-4}

VII. CONCLUSIONS

1. It has been possible to adapt the Trescott et al (1976) finite difference model for aquifer simulation in two dimensions to the University of Alberta Amdahl computer and to use the model to re-evaluate a detailed aquifer test in the Kirkpatrick Lake study area located in east central Alberta.

2. The model which was calibrated with the data from a 24 day pumping test did not effectively differentiate between five different arrangements of hydraulic parameters. Although the pumping test utilizing 11 observation wells was of a time duration that locally at least would be considered very long, it did not adequately assess the boundary conditions nor the recharge conditions. It is apparent that for a particular class of problem, especially where the impact of pumping extends over a large region, long pumping tests on a single well combined with a large number of observation wells will not necessarily yield the best results or give valid information on which to base model development, whether analytical or numerical. It would be more beneficial to strategically locate a number of pumping wells each requiring at least one observation well in order to better evaluate aquifer and recharge parameters as well as to identify and locate boundaries.

3. Since the field test results do not allow proper assessment of recharge conditions, it is only possible to use the calibrated model qualitatively for any long-term simulations. With the static water elevations in the test holes completed in the zone I aquifer decreasing with increasing distance northeast from Kirkpatrick Lake, and the zone I aquifer appearing to subcrop beneath the lake, the simulation incorporating leakage from the lake may prove to be the most appropriate model with which to predict the response of the aquifer to the proposed rate of pumping. A 20-year simulation resulted in a drawdown of 27.1 m in the pumping well, reaching steady state at about 16 years. The drawdown at the pumping well exceeds the available head by about 25 percent.

4. Assuming recharge occurring as direct precipitation exceeds some basic minimum, varying the rate of recharge does not effect the efficiency with which a pumping well can intercept ground water moving through an aquifer system. Decreasing storativity improves the efficiency while halving transmissivity reduces the efficiency with which a pumping well can intercept recharge when other parameters are unchanged.

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