DEDICATION

ABSTRACT

GROUNDWATER MODELING OF THE NAIVASHA BASIN, KENYA.

By

Robert Ramírez Hernández

DEPARTMENT OF WATER RESOURCES

The Naivasha Basin has been characterized in recent times by an intense agricultural development. This situation has caused the water consumption to increase. Because the amount of groundwater used for this purpose, has increased substantially the groundwater levels in the basin have been affected together with the flow directions. A groundwater model has been constructed and calibrated in order to estimate the amount of flow from the Malewa River to the well field area as well as from Lake Naivasha. Groundwater balance calculations have been made with the model and estimates of this quantities obtained. A sensitivity analysis of the model has been made and the necessity for better knowledge of parameter spatial distribution has been confirmed. Future situations, which are likely to occur, have been modeled and the results interpreted.

Pumping tests have been carried out and their results analyzed using available methods. The results have revealed the great variability of hydraulic parameters in the basin. Finally, the model results have been evaluated from an environmental point of view.

ACKNOWLEDGEMENT

I want to acknowledge the persons that in one way or in another helped me in the realization of this work. Especially to Mr. Kovacs because its patient, dedicated and invaluable attention during these twenty months. I acknowledge the precise comments and careful manner in which Mr. Gieske reviewed the drafts of this thesis.

To all my colleagues and friends thanks a lot.

CHAPTER 1. INTRODUCTION

1.1.- INTRODUCTION

One of the best known lakes in Kenya's Rift Valley is Lake Naivasha, which lies about 90 km northwest from Nairobi, capital of Kenya. Lake Naivasha is one of the world's most famous for bird watching and it is an extremely important source of fresh water for both agriculture and human consumption. Besides its importance as a source of fresh water other activities are developed as, for example, the fishery and horticultural industry and tourism.

This explains why Lake Naivasha is better documented than the other lakes in the Rift Valley which are more saline, but the relatively great number of investigations that have been carried out concerning the lake and its environment have been spread among a great number of scientific institutions, and sometimes it is difficult to gather and organize the existing information needed for a new research activity. Modeling studies related to the lake and its catchment are quite recent and little experience has been accumulated in this context for the area.

1.2.- PROBLEM DEFINITION

The Naivasha catchment is characterized by complex geological conditions making groundwater studies more difficult compared with surface water studies. The area consists mainly of volcanic strata and lake sediments derived from this volcanic material. Layers of diatomeas are also found. The catchment is also affected by two fault systems (Wiberg, 1974), a system that is developed along NNW – SSE trending directions and a more recent N-S system.

These two systems play an important role in the hydrogeological conditions of the catchment. In the zones surrounding Lake Naivasha these conditions are not so evident and the influence of the tectonics is much less in comparison with the rest of the basin. A regional groundwater model may give insight to the hydrogeological conditions of the catchment as well as provide a better understanding of other hydrological processes, such as recharge, lake water balance, groundwater-surface water interactions, etc.

1.3.- OBJECTIVES

- Organization of an adequate database according to available information. This database must be composed by data from previous works and data collected during fieldwork.
- Creation of a GIS database that acts as the input for the construction of the MODFLOW conceptual model.
- Construction of a regional hydrogeological numerical steady state model.
- To make groundwater flow calculations using the calibrated model in order to determine the amounts of flow from the Malewa River and Lake Naivasha to the aquifer.

1.4.- METHODOLOGY

The methodology applied is as follows:

- Analysis of available information.
  - Checking and updating of well databases.
  - Selection of appropriate wells and well parameters.
  - Study of previous work and literature.
  - Selection of appropriate information.
  - Preparation of a new data base.
  - Use of GIS methods.

- Field observations and desk tasks.
  - Pumping test data collection and evaluation.
  - Description of well lithological logs.
  - Geological observations.

- Modeling.
  - Construction of a conceptual model.
  - Selection of a specific steady state numerical model.
  - Model run.
  - Model calibration.
  - Interpretation of results and conclusions.

A flow chart for this methodology is presented in Fig. 1.

1.5.- LOCATION

Naivasha basin is located between coordinates (185000 , 990000) and (240000 , 997000) according to U.T.M. Zone 37, topographic sheets 133/1, 133/2, 133/3, 133/4, 119/4, scale 1:50 000 Survey of Kenya (1975). The basin forms part of the Gregory Rift Valley, flanked by scarp lines to the east represented by Aberdare Range and Kinangop Plateau and to the west by Mau Escarpment. To the north the catchment is bounded by the Bahati Uplands and to the south by the Olkaria Complex. The basin has a total area of 3387 km^2 of which 132 km^2 belong to the lake.

1.6.- CLIMATE AND VEGETATION

The Naivasha basin is characterized by moderate temperatures and a semiarid climate in areas surrounding the lake, while the climate is semi-humid to humid in mountainous areas where well developed rain forest is present. Annual values of precipitation for the hilly zones are high, ranging from 1250 mm to 1500 mm with similar or lower rates of evapotranspiration (Clarke et al., 1990) while the lower rainfall values average 430 mm at Magadi and 930 mm at Nakuru for the valley floor. Under these conditions potential evapotranspiration exceeds precipitation, often by several times. The annual rainfall average at Lake Naivasha is 650 mm (Wiberg, 1976).

Despite this, the lake and the catchment in general is in the rain shadow of winds coming from the west and, more importantly, from the east, causing a very high areal and temporal variations of the precipitation pattern in the area. The lake level in October 1998 stood at an altitude of 1885 m a.m.s.l.

Two main rainy seasons are recognized in the basin, a long-term rainy season and a short term one. The former is located in the months of March, April and May and the latter one falls in the months of October and November.

Vegetation in flat areas is represented mainly by bushes and vegetation of savanna, with the exception of areas surrounding the lake, the swamp zone, where Papyrus spp and Eucalyptus spp are the most representative species. The highlands support well-developed tropical rain forest, which partially covers the Aberdare, Mau, Kikuyu, and Kinangop escarpments. Precipitation in the highland areas is significantly higher than within the Rift, where the vegetation has a semi-arid character.

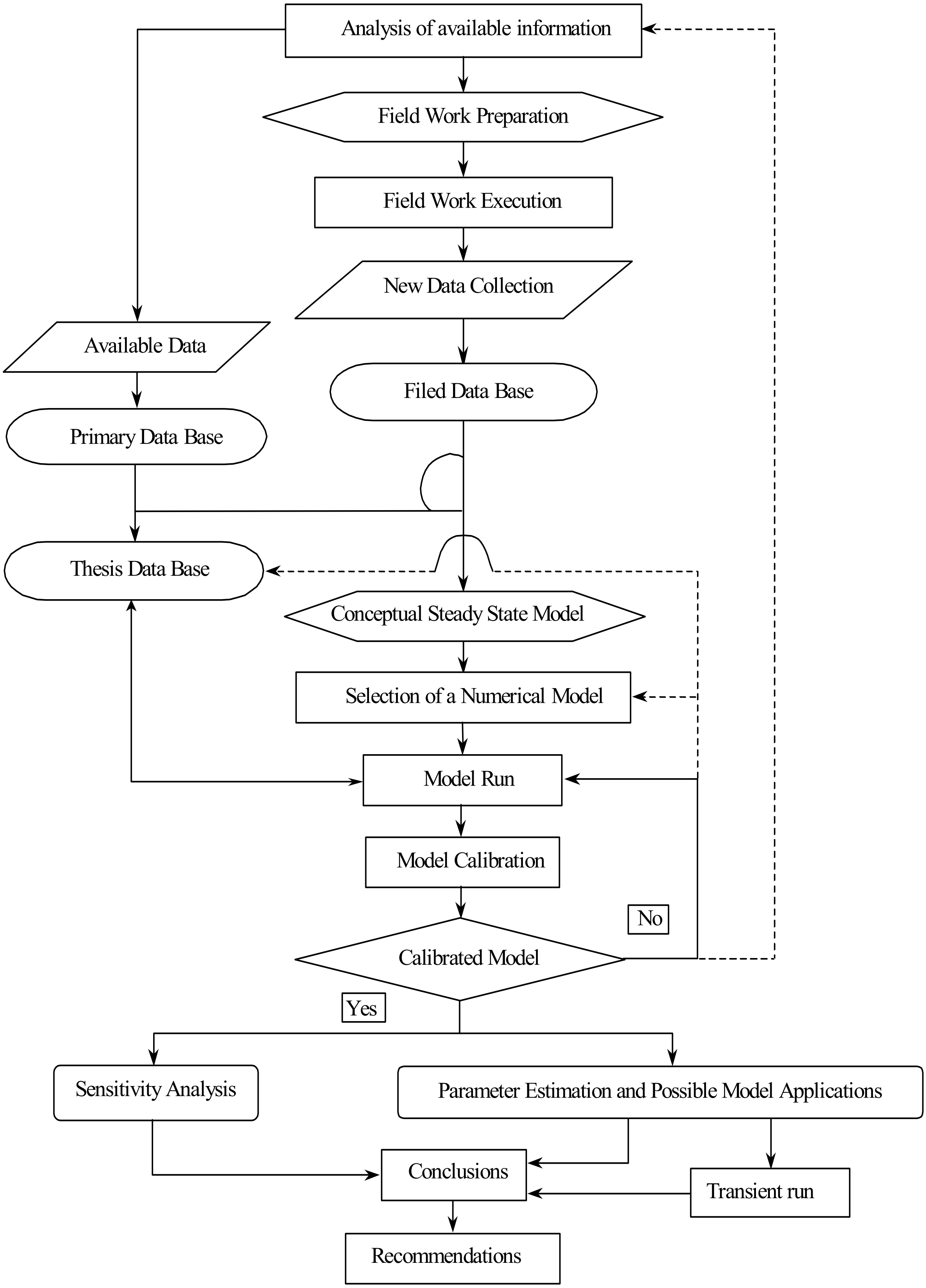


Figure 1. Flow chart of the modeling process.

CHAPTER 2. GEOLOGY AND HYDROGEOLOGY

2.1.- GEOLOGY

One of the most remarkable features of the Earth's crust is the Rift Valley of Africa. The valleys, rather than just a single valley, form a more or less continuous scar from Israel and Jordan in southwestern Asia to Mozambique in southeastern Africa (Ase et al., 1986). One prominent part of the valley system is the so-called "Gregory Rift Valley" in Kenya. Flanked by scarp lines to both eastern side, Aberdare Range, and western side, Mau Escarpment, it reaches relative altitudes of 1000 meters or more. The main part of the Rift Valley faulting took place during the Middle Pleistocene times (Wiberg, 1974), possibly along older fault lines and continued into Upper Pleistocene, although some investigations suggest that the development of the fault system continues nowadays (Ase et al., 1986).

Naivasha basin incorporates Lake Naivasha, the Ndabibi plains, which lie to the west of the lake, and the Ilkek plains, which lie in the vicinity to the north. Lake Naivasha dominates the Naivasha basin and during a 1983 survey its level stood at an elevation of 1989.3 meters (Ase et al., 1986). The lake is the most important hydrological feature of the study area. The Ndabibi plains extend up to 9 km west of the lake and separate the Olkaria and Elburru Volcanic Complexes. The plains are about 1980 meters above sea level (Clarke et al., 1990) along their western margin and slope gently eastward to the lake. The Ilkek plains extend up to 23 km north of Lake Naivasha and they range in width from 13 km near Naivasha Town to a minimum of 4 km near Gil Gil Town. The plains slope gently southward from a maximum elevation of 2000 meters in the north. Ridges of volcanic rocks occur at the east of Ilkek settlement, and several have prominent fault scarps along their western sides.

2.1.1.- STRUCTURAL AND LITHOLOGICAL FEATURES

Structural features such as faults in the rocks often optimize storage, transmissivity and recharge, particularly when they occur adjacent to or within a surface drainage system. In the region a series of parallel faults run in north-northwestern direction along the eastern face of the Mau Escarpment. This gives the hillslope a terraced appearance, marked by several well-defined steps. The same feature can be observed along the western face of the Kinangop Plateau, finalizing in the largest fault of the Plateau called *South Kinangop Fault Scarp*, Fig. 2.2. Faults will have the highest impact on hard and massive rock types, elastic formations such as tuffs and weakly consolidated deposits will bend rather than break. As a result, they tend to suppress the radius of influence and the magnitude of the damage caused by tectonic events. In relatively plastic rocks, the porosity will not increase in the area affected by the fault. Hard layers such as lavas on the other hand, will be broken by fractures and joints, thus giving rise to increased secondary porosity.

Faulting in the area is an on-going process. It is reported that, some time ago, several pipe bursts occurred on Oserian Farm (Kijabe, 1998). This fact could not be immediately explained since the bursts all appeared simultaneously. However, the events may have been the result of slight earth movement. The scale of this process ranges from hundreds of kilometers to the order of meters. It could be observed that small fractures and folds have occurred in zones considered more or less stables near Lake Naivasha, especially in Karati River. Among the most prominent structural features of the region are volcanoes: Longonot and Suswa to the south of the study area. Also at Olkaria volcanic complex numerous craters and volcanic cones are found. Magmatic activity reaches an exceptional beauty at Hell's Gate national park where columnar basalts are impressive features in the landscape. In the area the most common types of rocks are volcanic or are related to volcanic activity, the surrounding of Lake Naivasha is the exception where lacustrine material is predominant, Fig. 2.1.

2.1.2.- EFFECT OF FAULTING SYSTEM ON THE MODELED ZONE

Although the faulting process has been intense and of long duration, its effect on the shallow aquifer formation in the modeled zone does not appear to be of great importance with the exception of the boundaries of the model which have been selected mainly according to the existence of fault lines. Inside the modeled area there is a predominance of sedimentary material composed by fluvial and lacustrine deposits and pyroclastic material.

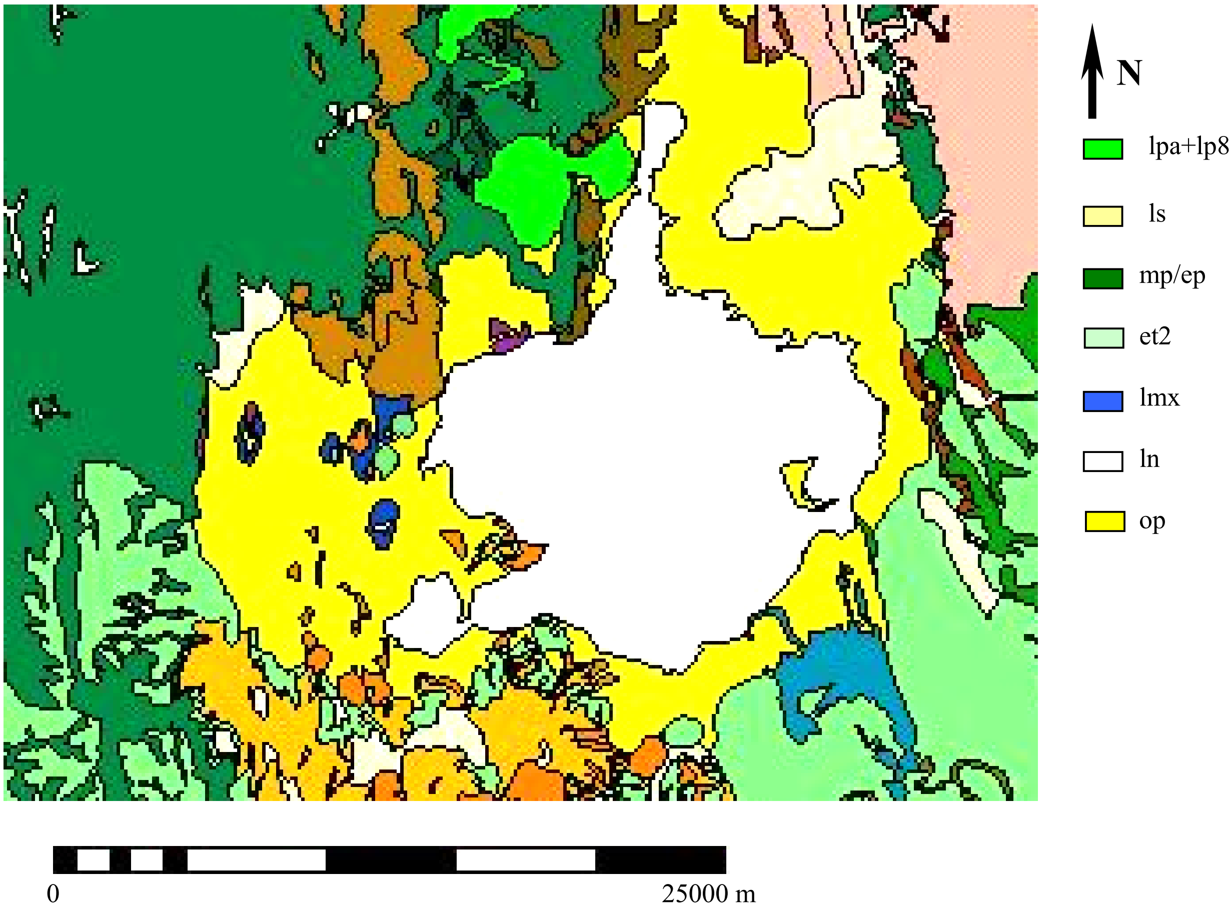


Figure 2.1. Simplified geological map of the study area (after Clark et al 1992).

Table 2.1 Legend explanation for geological map.

Legend Explanation		
Symbol	Description	Age
Lpa + lp8:	Longonot ashes and Akira pumice.	Mid / Late Pleistocene-Holocene
ls:	Lacustrine sediments.	Mid / Late Pleistocene-Holocene
mp / ep:	Maiella pumice, trachyte, ashes, Elburru pumice.	Mid / Late Pleistocene-Holocene
et2:	Elburru trachytes, lava flows, pyroclastic material.	Mid / Late Pleistocene-Holocene
lmx:	Longonot basalts, trachytes, lava flows.	Mid / Late Pleistocene-Holocene
ln:	Lake Naivasha.	Mid / Late Pleistocene-Holocene
op	Olkaria comendites, pyroclastics	Mid / Late Pleistocene-Holocene

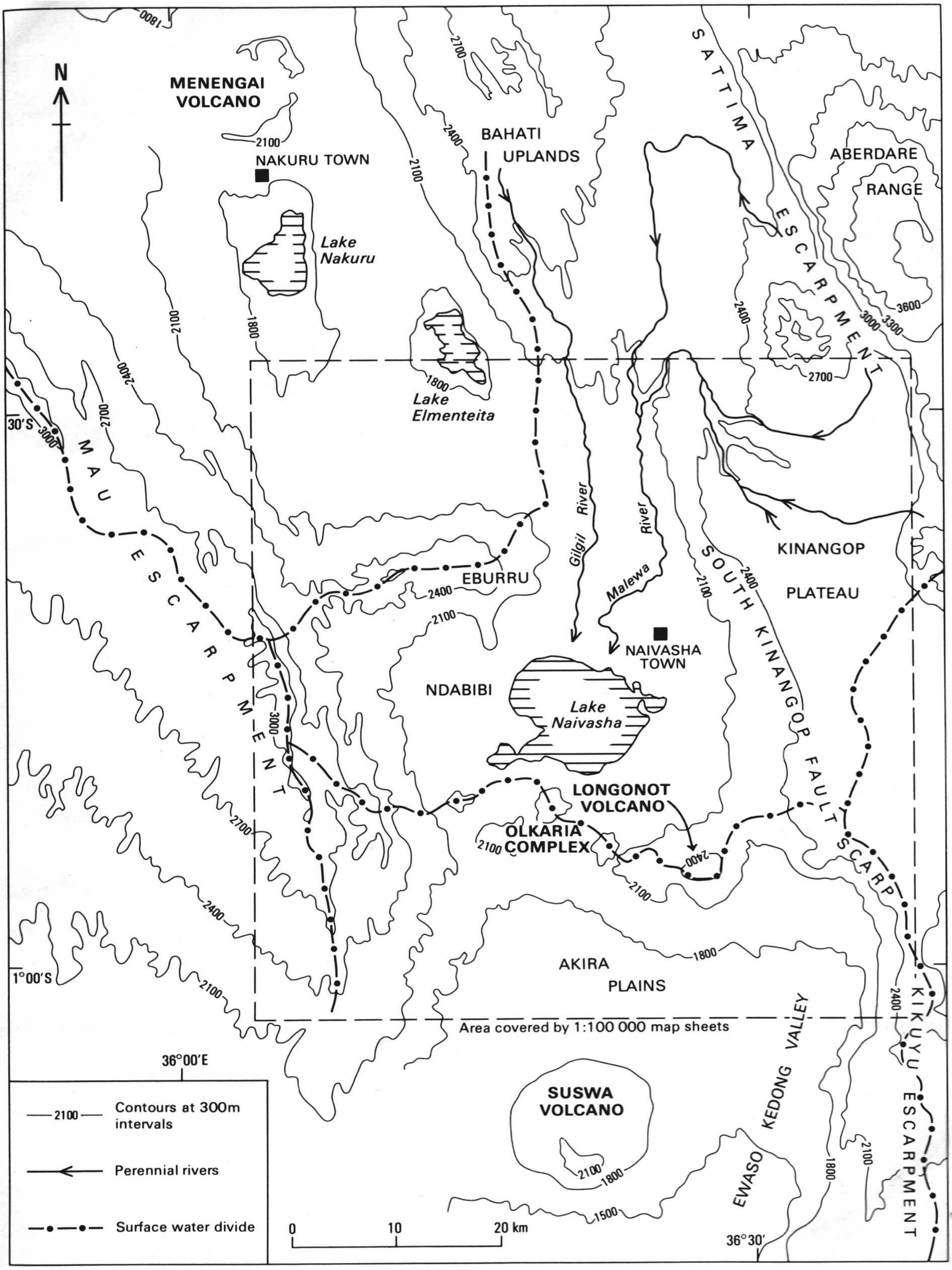


Figure 2.2. Study area and main physiogeographic features (after Clarke et al., 1990).

As mentioned before the effects of tectonic activity upon these types of formations will be suppressed and the influence of faults can be safely neglected. However during the fieldwork in the area of interest no evidence was found that could indicate that the fault system plays an important role in the shallow groundwater flow.

2.2.- HYDROGEOLOGY

The hydrogeology of an area is determined mainly by the nature of the geology, topography and climate, these aspects are related to characteristics such as the parent rock, structural features, weathering and patterns of precipitation and finally the human activity.

The relief, creating two different hydrogeological environments markedly affects the hydrogeology of the region. The first is localized in highland areas, characterized by deep groundwater tables and steep groundwater gradients. This type of environment is also described by its larger rainfall values as compared with the valley, approximately 1200 mm versus 500 mm a year. Naibor Ajijik to the west of Mau Escarpment, with a mean rainfall figure close to 1000 mm, and a geology marked by alternating pyroclastic and sedimentary layers, is an example of this type of environment in which the potential for groundwater development is high (Kijabe, 1998).

The second type of environment is localized in the valleys and is characterized by a shallow water table, which gently slopes to the lake, low values of precipitation as compared with mountains and low values of recharge. With a mean rainfall value of 500 mm a year and an estimated groundwater recharge of about 50 mm Lake Naivasha is the most representative of this type of environment in the region.

Human activities have influenced groundwater levels in recent years. For example, nearby Lake Naivasha water abstraction for agricultural production has caused a drop in the water table, thus changing flow direction. Human impact on hydrogeology can have considerable proportions.

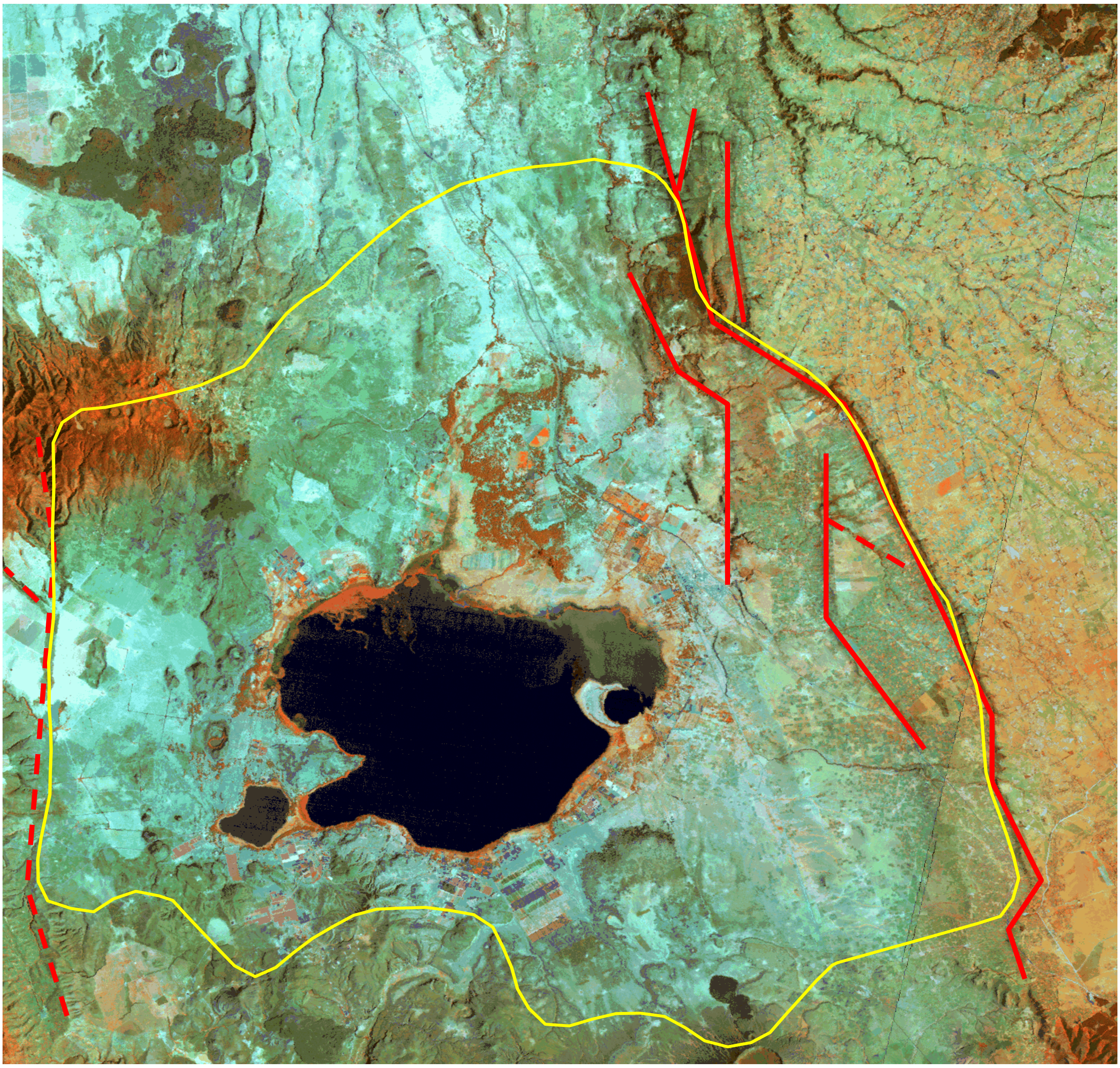
Within volcanic rocks, groundwater primarily occurs within fissure zones, fractures and bedding. Lava flows rarely possess significant pore space; their porosity is purely secondary, such as cracks, joints and fissures. However, pyroclastic and especially sedimentary deposits as opposed to massive volcanics, do have a primary porosity and the cavities between the mineral grains or clasts are usually open and interconnected. Consequently, they can contain and transmit water. Whereas deposits of clean, unconsolidated sands are highly transmissive, the permeability rapidly decreases in the presence of clays, even if their portion is very small. Heavy clays, which may be marked by porosity as high as 50%, are generally impermeable. Tuff layers and ashes are fine-grained pyroclastic deposits with generally unfavorable hydraulic characteristics similar to those of clayey sediments.

The floor of the Rift Valley forms a topographical shoulder in the Naivasha area, with the general elevation decreasing to the south. Consequently, there is a hydraulic gradient along the axis of the Rift, which accounts for the outflow of groundwater from Lake Naivasha. The water tables are considerably higher on the often steep and elevated boundary escarpments to the east and west. Lateral groundwater flow into the Rift from these source areas might certainly be an extremely important recharge mechanism for the deep groundwater flow.

2.2.1.- GROUNDWATER OCCURRENCE

Groundwater occurrence is controlled by the geological conditions of the area and the available water for recharge. Fresh volcanic rocks are known to be virtually compact and with no intergranular or primary porosity although secondary porosity may be well developed locally. The permeability of the volcanic rocks underlying de Rift Valley are generally low, although, there is considerable local variation and layers with poor hydraulic characteristics can be followed by layers with good hydraulic properties.

Transmissivity values may vary greatly in the region mainly in zones where fine volcanic material was deposited. Because this volcanic deposition was accompanied with other materials of different origin it is common to find intercalated zones of high and low transmissivities. Places where rivers were an important component of the geological environment in the past are characterized by abundant alluvial deposits that form aquifers. Therefore it is expected that these areas be characterized by good hydraulic properties. However, in places where volcanism has been a major ingredient of the geological history low values of transmissivities are found quite often.



Proven faults.

Supposed faults.

Model boundary

Figure 2.3. Color composite of LANDSAT images of the study area, bands 2, 3 ,1.

Faults are considered to have two main effects in fluid flow: to act as conduits or to act as barriers. The hydraulic role of faults is still not well understood because there is little evidence that a fault behaves in a particular way (Clarke et al., 1990). Despite of this lack of evidences from satellite imagery studies it can be seen that the largest faults in the area may act as, at least for shallow aquifers, barriers for groundwater flow into the aquifer in hydraulic connection with the lake, see Fig. 2.3. This assumption is explained in more detail in Chapter 4.

2.2.2.- RECHARGE AND GROUNDWATER FLOW

The groundwater level in the upper aquifer is governed by the lake level and water abstractions (Wiberg, 1976). Because of the raised lake level since 1950's and the increasing use of groundwater, the gradient and thus the recharge from the lake is likely to have increased. Apart from the groundwater recharge from the lake and highland areas there is evidence of recharge from the rivers (Wiberg, 1976), this situation could be corroborated during the fieldwork carried out by the author. Despite the fact of the high evapotranspiration rates existing in the Rift floor some of the recharge is believed to come from irrigation of cultivated fields. As noted by the author during his fieldwork, large quantities of water, coming from the lake as well as from the aquifer, are being used for this purpose. Estimated quantities of groundwater recharge due to crop irrigation losses might be important but this amount has not been quantified yet.

Groundwater movement from recharge areas to discharge areas depends on many factors such as available water for recharge, aquifer characteristics and the mode of recharge. Recharge of the deep groundwater flow system in the area is thought to be mostly from the Aberdare Range and Kinangop Plateau in the east and Mau Escarpment in the west. In these regions the actual monthly rainfall values exceed the potential evapotranspiration for a few months per year. Important sources of recharge for the shallow aquifer are the rivers and the lake. Some of the recharge may be due to highland surface runoff discharging into the valley. The existence in mountainous areas of high annual values of precipitation associated with the presence of a forest cover creates favorable conditions for rainfall infiltration and groundwater recharge. Moreover the valley also receives recharge by infiltration of local precipitation and by infiltration of crop irrigation losses.

Different authors have described two main flow directions in the area (Ojiambo, 1992., Clarke et al., 1990). The first one is lateral flow from elevated areas; this flow is west, northwest and south from Aberdare Range and Kinangop Plateau and east from Mau Escarpment. The author has the opinion that only a small part of this water discharges into the lake. The idea is that an important portion of this water flows in a direction north-south through a fault system by means of which water infiltrates to deep zones and leaves the catchment by groundwater flow. The second flow direction is believed to occur axially from the lake to the north and south respectively, however this assumption contradicts field observations. Therefore contrary to Ojiambo (1992) and Clarke et al., (1990) there appears to be no shallow groundwater flow to the north. The direction of deep groundwater flow is still uncertain. However, it seems to be mainly to the south, see Fig. 2.4. The lake has no surface outlet; therefore the water losses occur through evapotranspiration and groundwater outflow.

2.2.3.- GROUNDWATER QUALITY

Generally, groundwater in volcanic areas is of bicarbonate type with low total dissolved solids. The water quality in Naivasha area is usually good enough for domestic purposes, cattle watering and crop irrigation, however, locally fluoride concentrations are in excess of the World Health Organization guideline values of 1.5 ppm. This high fluoride content is thought to originate from volcanic and fumarolic activities in the Rift Valley region (Wiberg, 1976). Poor water quality has always been associated with deep groundwater in areas far from Lake Naivasha, however this situation can also be observed in areas not so far away from the lake. A deep well (compared to those surrounding it) of 120 m exists at Three Ostrich Farm. The odor and taste of the water in this well make it unsuitable for human consumption.

The situation for the surface water bodies in the region is different. While volcanic activity and recent tectonic movements may affect deep aquifers, surface waters are mostly affected by human daily activities. The principal economic activity of the region is agriculture; this activity is closely related to the use of fertilizers and pesticides to increase production. The use of fertilizers and pesticides generates favorable conditions for contamination of surface water bodies. On the other hand the lack of a well-organized plan for treatments of urban and agricultural effluents is another potential source of water contamination. As it could be observed during the fieldwork, most of the effluents discharge without treatment directly into the lake.

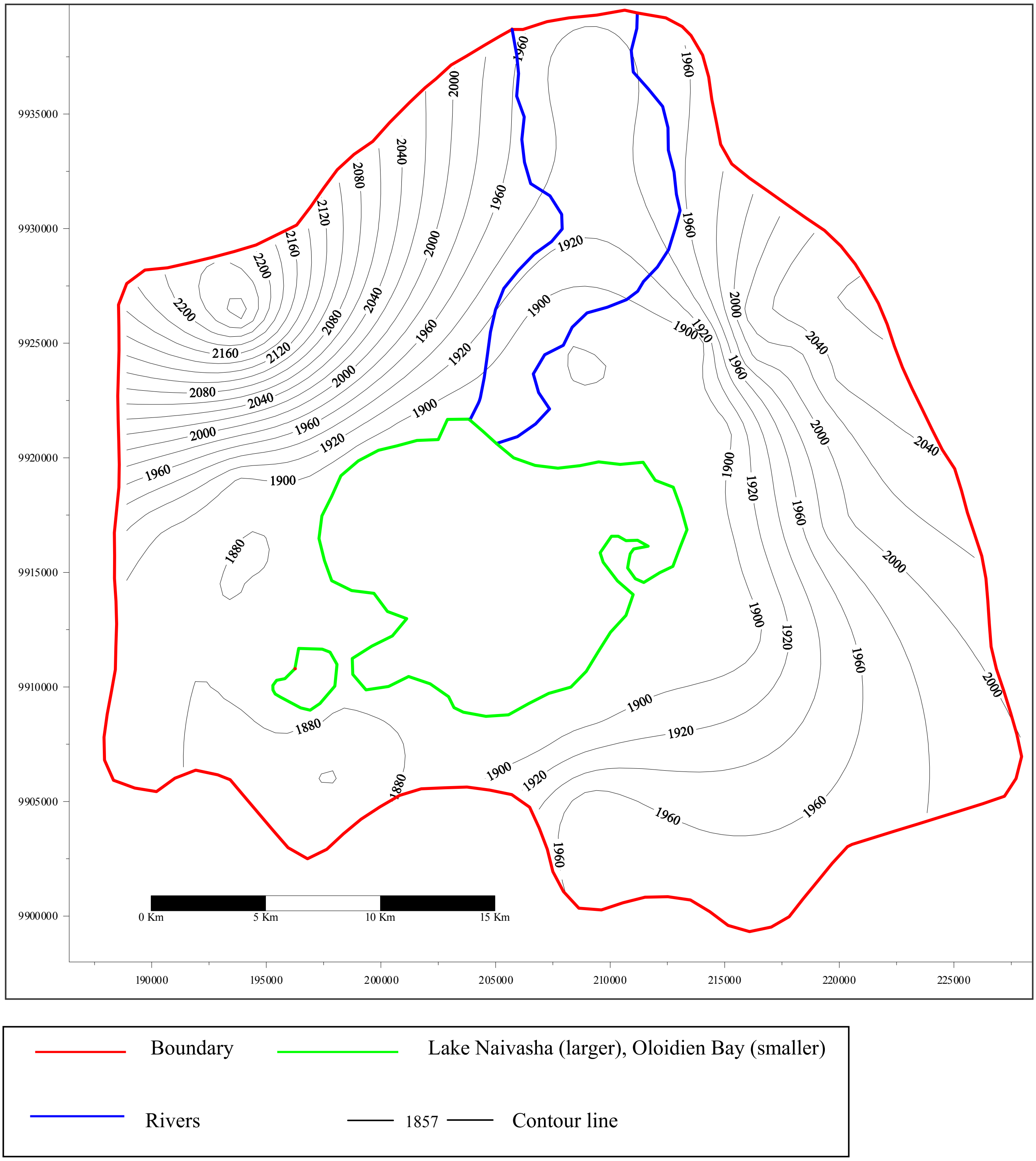


Figure 2.4. Contour map of observed heads, contour every 20 meters.

CHAPTER 3. DATA PROCESSING

3.1.- INTRODUCTION

Because the information used for the model construction was originally coming from different sources and formats a process of checking, selection and organization had to be carried out. One important step in data processing was to remove wells where levels might be erroneously measured because of problems with the datum. Another important part was to organize all necessary information in tables. GIS was also used to get a better general picture of the study area and to decide on model boundaries.

3.2.- IMAGE PROCESSING

The images used in the work were LANDSAT TM, all bands; they were recorded on different dates, the western part on 21 January 1996 and the eastern part of the basin on 25 February 1987. They were imported from ERDAS into ILWIS format. Having the images in the necessary format the next step was to georeference them, this step was performed using topographic maps of the zone scale 1:50 000 and the option *georeference: tie points* in ILWIS. The final part of this process was to create a complete mosaic of the catchment and surrounding areas, which was achieved using the option *glue* of ILWIS.

The next step in image processing was to select the area of interest. For this aim a color composite image (Fig. 2.3) and the geological map (Fig. 2.1) were used. The original images covered a larger area than that needed and therefore a sub-area had to be selected. This was accomplished using the operation *submap* in ILWIS. This new image proved to be useful in reducing the computing and processing times.

Another important aspect of this new image was its usefulness in constructing and validating the conceptual model of the zone. On top of it segment, polygon and point maps were overlain in order to check the model boundaries and location and distribution of observation wells in the area. This image was also used to select the correct zones of groundwater abstraction. Finally, this image served as background for the different variations of the conceptual model during the calibration process.

3.3.- VECTOR MAPS PROCESSING

Different vector maps were available, a high detail one with topographic elevations for the lake and a contour map with topographic elevations scale 1:50 000 for the rest of the area. Refinement of these vector maps was made by automatic and hand made analysis. Automatic refinement was carried out using ILWIS. Because none of them entirely represented the region of interest and due to difficulties with the leveling work carried out by surveyors in the past, different points with absolute altitudes have been used in different years and publications, Ojiambo (1992). A suitable digital topographic map, representing altitude in the region, was obtained through combination of the previously mentioned vector files.

3.4.- MAP RASTERIZATION

With the aim of performing operations between maps in later stages all maps must be in a raster format. Because in the creation of a *Digital Elevation Model* errors are introduced in the calculations for the highest points in an area, points in an existing point map, obtained from a table, were used as altitude control points. The previously glued map for elevations was rasterized and combined together with the point map, also in a raster format, to calculate the final DTM from which absolute water level for the wells inside the basin were obtained.

3.5.- CALCULATION USING TABLES AND DTM

As stated at the beginning of the chapter all point information was available in tables. Using *table operations* in combination with the DTM raster map it was possible to obtain the altitude at every well location and therefore to obtain the absolute water levels for the wells. Because inaccuracies introduced during the topographic map preparation as well as digitizing the reliability of the absolute surface elevations might not be high in some places. This situation has an immediate effect on the absolute water level calculations. Fortunately in areas away from the lake where the topographic map scale 1: 50 000 was used the errors are not be important and therefore the absolute water level calculated and used for the model calibration may be considered to be correct.

3.6.- POINT MAPS

Although not mentioned point maps played an important role in developing this thesis. They were used to check surface elevations and to verify the distributions of observation points in the area. Location of pumping wells was possible by using these maps. Another use of this map was to plot sample points where EC readings and chemical analysis were carried out, mainly to the north of the lake. This was important because those sample points were used in the selection of the model boundary to the east of the study area.

3.7.- INTERPOLATION SCHEME FOR WATER LEVELS

In hydrogeology it is costly and time consuming to gather a large amount of information about a natural process, besides, in natural sciences the number of samples that can be taken is infinite. Because that sampling is impossible to accomplish, a finite number of point samples are taken. However it is of interest to know the spatial behavior of the parameters under study, therefore an interpolation scheme has to be used in order to reach this goal. Because of the complexity of the area and the characteristic distribution of the points a manual linear interpolation scheme has been preferred in most cases. Although in this way the hydrogeologist judgement is involved in the interpolation it has been seen that points in the area near the lake tend to be discarded simply because the interest has been to construct a regional potentiometric map. The author has the opinion that information about the area near Lake Naivasha has been overlooked many times when making a head distribution map.

An automated interpolation scheme has the disadvantage that the hydrogeologist judgement may not be incorporated during the interpolation process but on the other hand it will try to find the best estimate for every point considered in the interpolation with the smallest associated error. To obtain this map, from which interpretations may be made, a linear interpolation method was used, kriging, which produced good results. Although this method minimizes the estimation variance it is impossible to obtain error-free estimates. The head distribution map obtained with this method is somewhat similar to that obtained using the groundwater flow model.

3.8.- PUMPING TEST DATA ANALYSIS

One of the parameters required for the construction of the groundwater flow model is aquifer hydraulic properties. The most common way of obtaining these properties is through pumping test data analysis. Many methods for analysis and evaluation of pumping test data have been developed. Kruseman and De Ridder (1994) presented a book in which most of them are well explained. Of these methods only a small number were of interest for the present data analysis because of the characteristics of the tests. By use of quantitative methods the hydraulic properties of the leaky aquifer system have been determined.

3.8.1.- AQUIFER SYSTEM CHARACTERISTICS

The aquifer system in the area is characterized by the presence of interbedded permeable and less permeable layers. It has been seen that near the lake a shallow groundwater table exists with a depth below surface varying from one meter to six or seven meters. This first water-bearing layer is considered an unconfined aquifer. There are two main reasons why this succession of permeable and less permeable layers has been considered as a unique leaky system aquifer. First of all in the area no information exists that could help in separating the aquifers and secondly these clay and silt layers are of reduced thickness in comparison with layers composed by sand, conglomerates, volcanic and others materials. Materials of different compositions form the aquifer in different areas. It is quite common to find areas where transmissivities are very high, 5000 m^2/d, and others where this value may be of the order of 0.01 m^2/d.

Areas where alluvial deposits are found are characterized by good hydraulic properties, however, areas where volcanic ashes and fine-grained volcanic rocks together with fine lacustrine sediments were present the hydraulic properties were quite poor. Also good hydraulic properties can be observed in wells located in fractured volcanic rocks. These wells are characterized by high discharges rates with very little water table depletion. There are wells placed in zones where the transmissivities may be in the order of thousands m^2/d, this is the case of “Three Ostrich Farm” and “La Belle Inn” boreholes. Specifically in the last well recovery measurements could not be done because its recovery was immediate after stopping the pump, the water level recovered to original within less than two minutes after the pump was switched off. Two pumping tests were carried out there although in the second one readings were not made because the interest was put in the recovery. Pumping tests have been carried out in five boreholes, together with the pumping tests the data about three recovery tests were also available, all this work was made during the fieldwork, their location is shown in Fig. 3.1. Well logs of the previous mentioned wells are shown in Fig. 3.2 and 3.3. More detailed information about geophysical logs in the zone can be found in Gressando (1999).

3.8.2.- PUMPING TEST ANALYSIS METHODS

When performing a pumping test the presence of at least one observation well is desirable. In this case none of the five tested boreholes had an observation well. Therefore, analysis methods developed specifically to deal with this situation had to be used. In all cases one of the methods used for the pumping data analysis was *Jacob’s straight-line method for confined leaky aquifer,* in the case of the recovery tests the *Theis’s recovery method* was preferred because its simplicity and suitability. Both methods use the same equation to calculate the transmissivity of the aquifer at the well location, Equation. 3.1.

$$K \cdot D = \frac{2.3 \cdot Q}{4\pi \cdot \Delta S_w} \qquad [3.1]$$

$K \cdot D$ Transmissivity

Q Discharge

ΔS_w Drawdown per log cycle for pumping test, Residual drawdown for recovery.

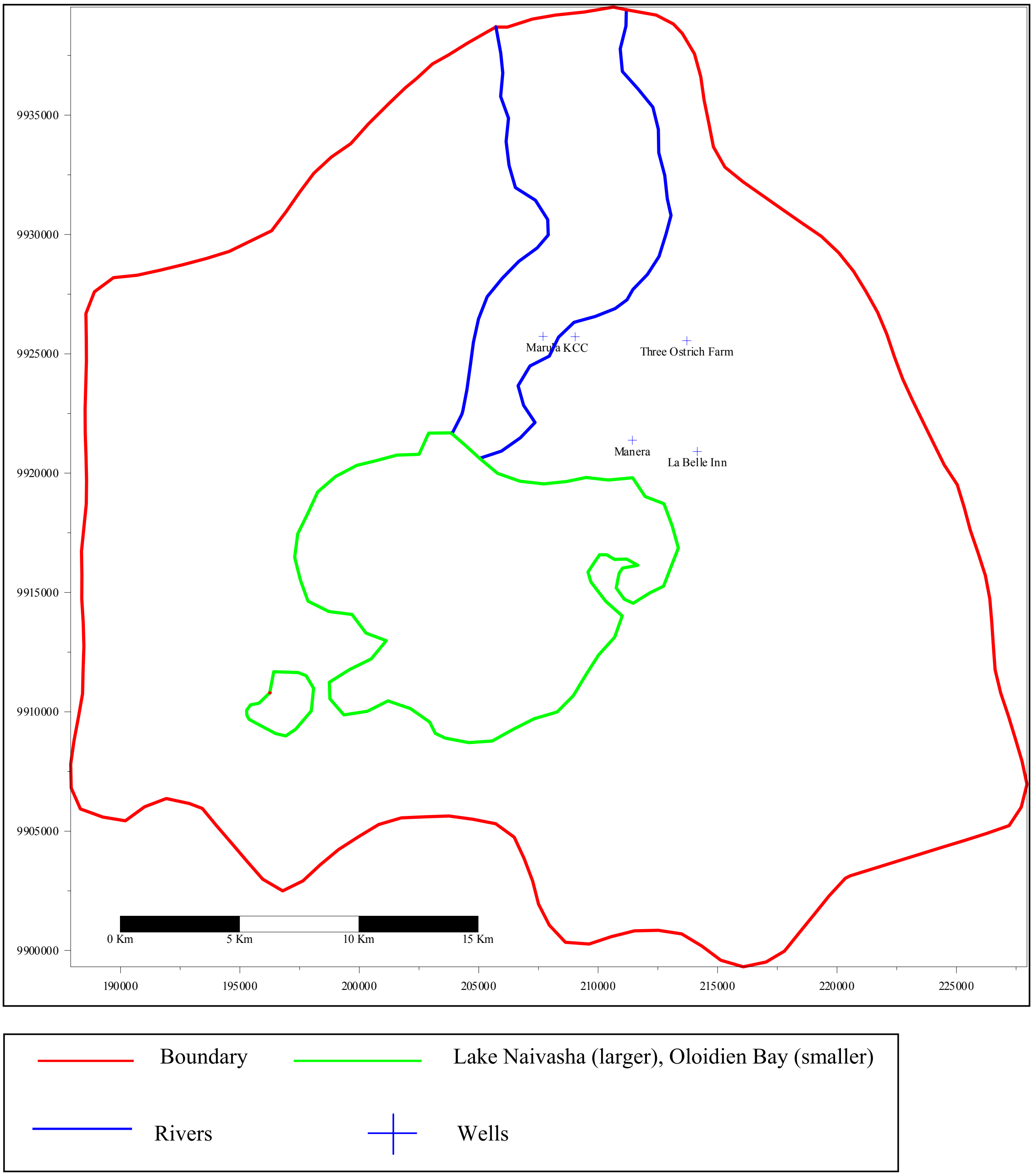
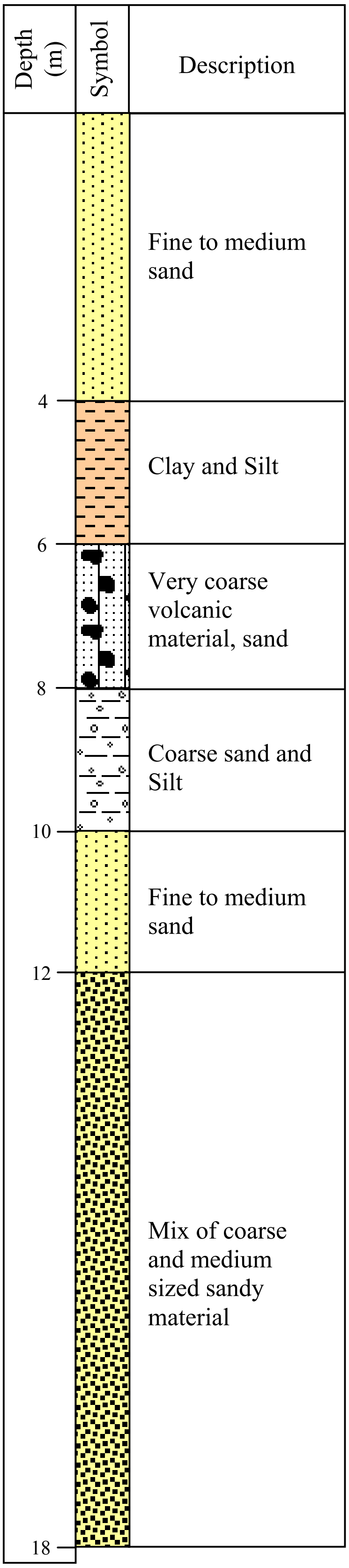
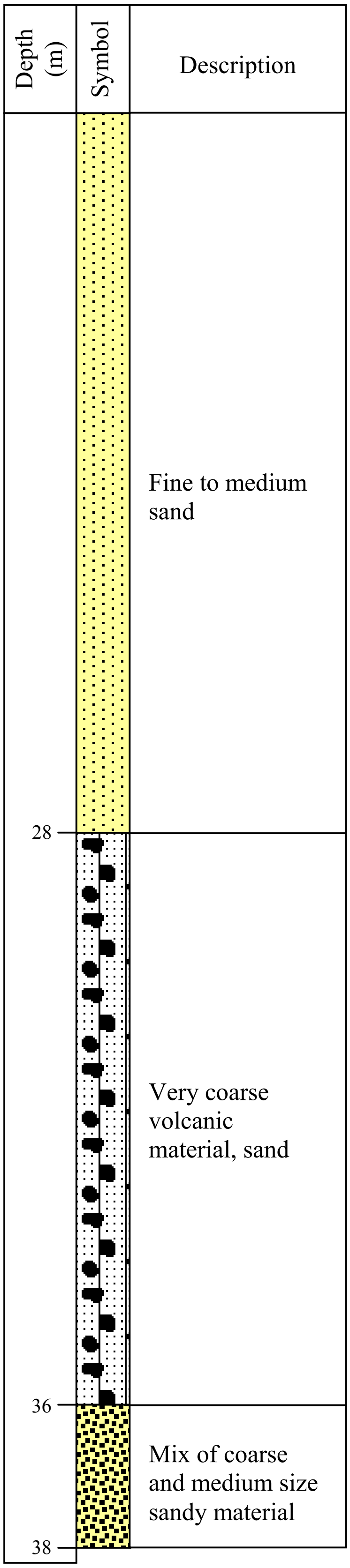


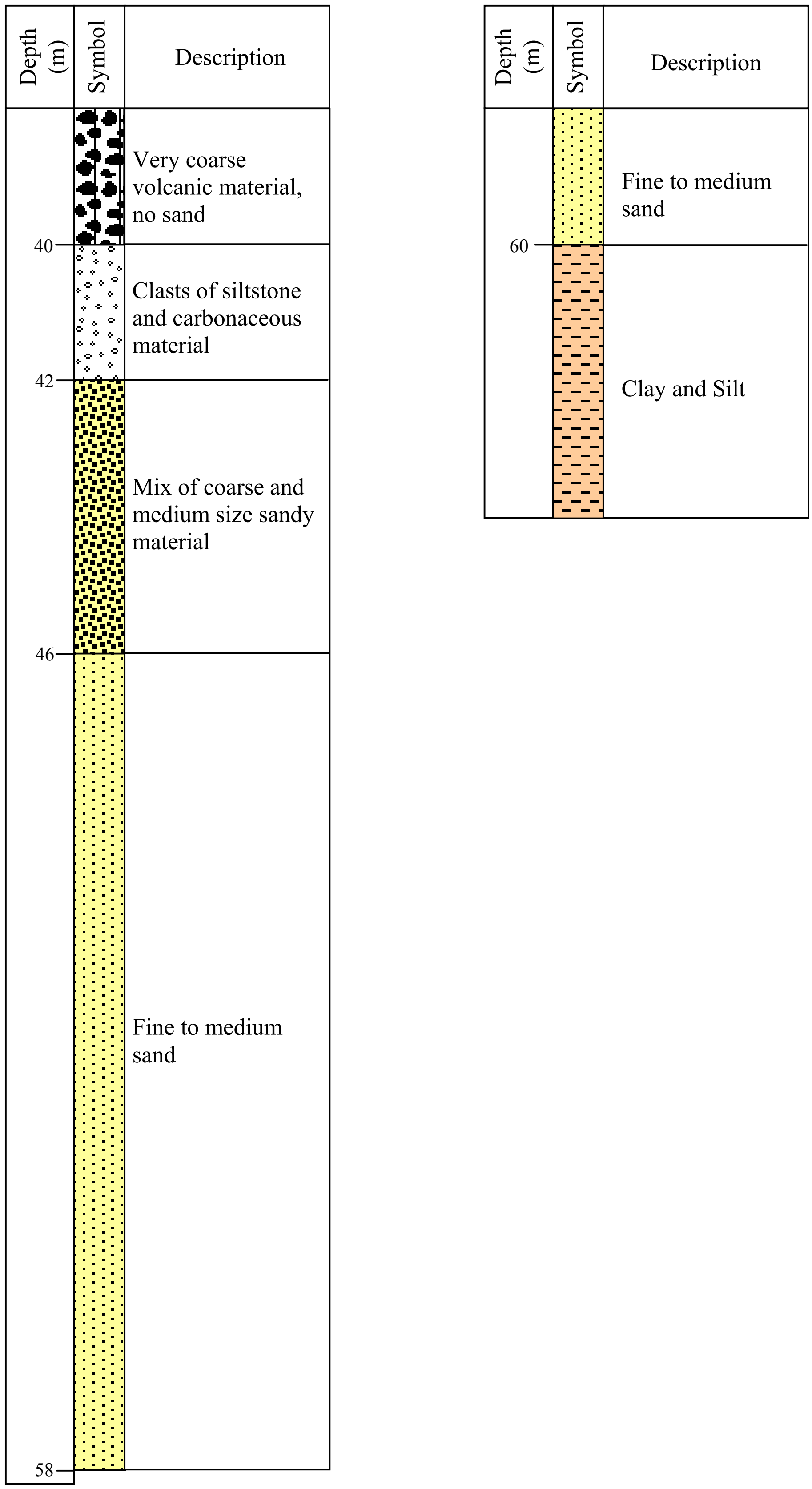
Figure 3.1. Location of wells where pumping test were carried out



continue to the right

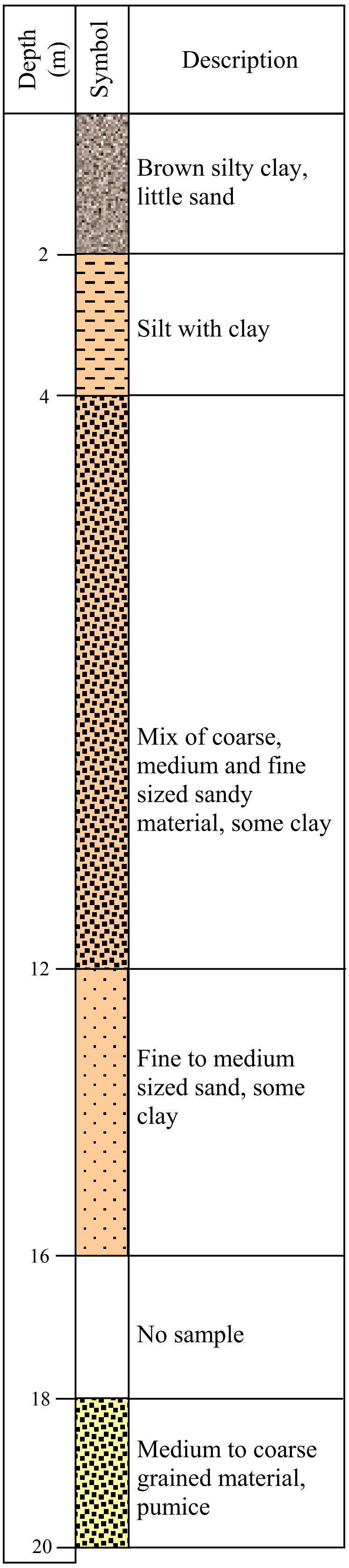


continue next page, left figure

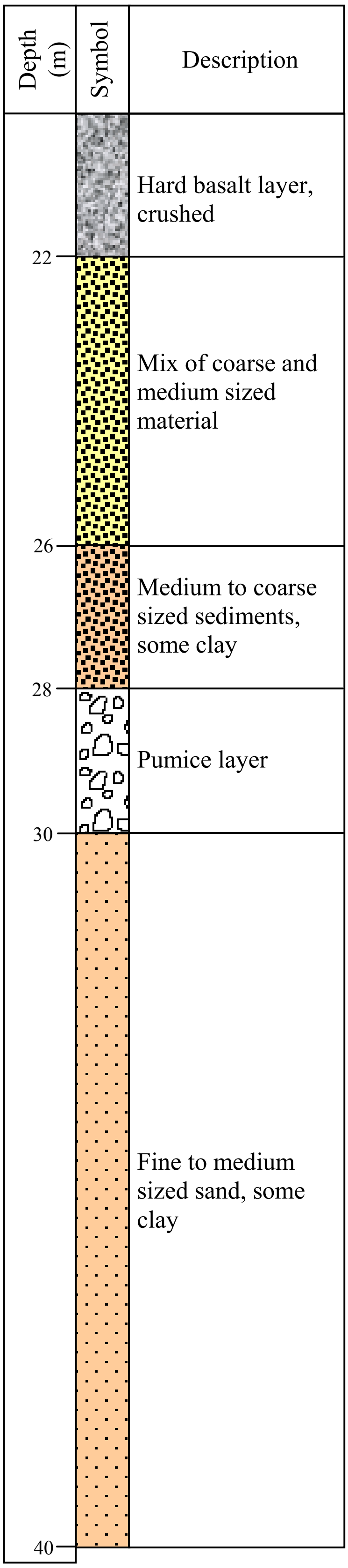


continue to the right

Figure 3.2. Well log for Three Ostrich Farm well.



continue to the right



continue next page, left figure

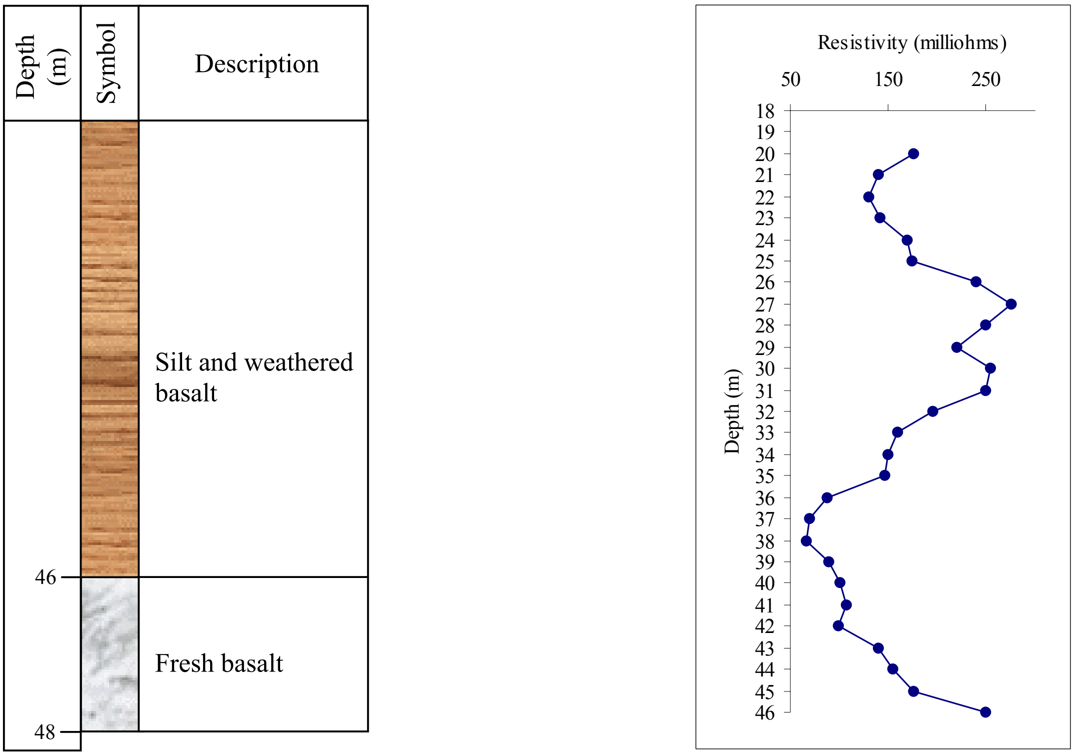


Figure 3.3. Well log and resistivity profile for "La Belle Inn" well.

The second method used for the analysis of the pumping test data was *Hantush's method for leaky aquifer*. To apply this method an assumption had to be added: An observation well existed and was located at the edge of the pumping well. In order to evaluate also the recovery *Theis-Jacob's recovery method* was applied. Because of the assumption that measurement were taken in an observation well the storativity values obtained by *Hantush's method* are not reliable. The results of both methods are shown together with the results obtained by applying methods for the situations where observation wells are not available. For the case of Three Ostrich Farm well Cooper & Jacob was preferred. These tests are included in the package AQUITEST (Roehrich).

3.8.2.1.- PUMPING AT MARULA FARM

Marula well was pumped for eight hours with a constant discharge of 10 m^3/h. In this well the drawdown values were not high with a total water level decrease of 0.36 m during the pumping time. After the pump was switched off recovery measurements were taken for a period of ten hours when the level almost reached the original static level. A description of the well, water level during the pumping and recovery times are presented in Annex I. In the table below values of drawdown, residual drawdown and transmissivities are shown. Values calculated with the assumption of the observation well at the edge of the pumping well are also shown. In Fig. 3.4 pumping time is presented, Fig. 3.5 represents recovery time. Results of AQUITEST are enclosed in the same annex.

Table 3.1. Values of calculated transmissivities, well Marula.

Measurement during:	Parameter	Value (m)	Calculated T (m^2/d)
Pumping	Drawdown	0.27	168
Recovery	Residual Drawdown	0.20	216
Pumping*			269
Recovery*			220

* Using AQUITEST.

A simple inspection of the previous table shows the good agreement between the transmissivity values calculated by all the four methods.

3.8.2.2.- PUMPING AT KCC MILK FACTORY

This pumping test was characterized for its short period. Conditions were not favorable at this location to make a pumping test of a long duration as the previous one. Nevertheless, and according to Heij et al., (1990) the best results can be obtained with data collected during a short recovery period, after a short pumping period. The pumping time was twenty minutes and recovery ten minutes. The discharge during the pumping was 6 m^3/h. A description of the well, water level during the pumping and recovery times is presented in Annex II. The results of the analysis are presented in the table bellow, Table 3.2. The Jacob's straight-line method and the Theis's recovery method graphs are shown, Fig. 3.6 and 3.7. Results in AQUITEST are found in the same annex.

Table 3.2. Values of calculated transmissivities, well KCC milk factory.

Measurement during:	Parameter	Value (m)	Calculated T (m^2/d)
Pumping	Drawdown	0.28	96
Recovery	Residual Drawdown	0.33	84
Pumping*			48
Recovery*			75

* Using AQUITEST.

3.8.2.3.- PUMPING AT "LA BELLE INN" WELL

This test was characterized by an average discharge of 13.5 m^3/h. However a rapid water level stabilization was observed. Discharge related measurements were made in an underground reservoir. Geometric characteristics of this reservoir, reservoir levels and well water levels, during the pumping time are presented in Annex III. In this well it was not possible to record levels during recovery time because it occurred within less than two minutes after the pump stopped. Because of the rapid stabilization of the water level after pumping started only two points may be used to fit a straight line through them. In order to have a more reliable result more points would be required, in view of this no fit was attempted. However, transmissivity in this point is ≥ 600 m^2/day. Fig. 3.8 shows the drawdown level graph. In order to show the effect of leakage the result of the *Hantush's method for leaky aquifer with no aquitard storage* is presented (Annex III).

3.8.2.4.- PUMPING AT THREE OSTRICH FARM

The record in this well was obtained using a *diver*, which is a pressure data logging device capable of measuring the pressure caused by the water column above it. This type of records is very precise. For this well recovery measurements could not be made. It is observed that at the start of pumping there were variations in the abstraction rates, this situation provoked sudden variations in the water level in the well. After the abstraction rate stabilized the level started to stabilize too. Although the abstraction rate for this well is the largest, 120 m^3/h, in comparison with the other pumping tests the drawdown in the well is rather small. The record for this well is shown in Annex IV, results of the pumping test interpretation are presented in Fig. 3.9. and Table 3.3. The result obtained with AQUITEST is enclosed in the same annex.

Table 3.3. Values of calculated transmissivity, well Three Ostrich Farm.

Measurement during:	Parameter	Value (m)	Calculated T (m^2/d)
Pumping (Hantush)	Drawdown	0.09	6600
Pumping* (Jacob)			7600

* Results in AQUITEST.

3.8.2.5.- PUMPING AT MANERA FARM

In Manera Farm another pumping test was carried out. In this case the discharge used was 22.5 m^3/h. Water level was recorded using an electric dipper. The total pumping time was ninety minutes and the recovery time was hundred and fifty minutes. At the end of the recovery time the water level had almost reached the initial level before pumping. Information about this test is presented in Annex V. Results of the analysis are shown in Table 3.4 and Fig. 3.10 and 3.11. Good agreement was reached between the results obtained with the different methods. The results obtained with AQUITEST are shown in the same annex.

Table 3.4. Values of calculated transmissivities, well Manera Farm.

Measurement during:	Parameter	Value (m)	Calculated T (m^2/d)
Pumping	Drawdown	0.12	816
Recovery	Residual Drawdown	0.14	696
Pumping*			670
Recovery*			728

* Using AQUITEST.

A summary of calculated transmissivity values for every well is presented in the table below together the with values calculated by AQUITEST. For the applications of these tests additional conditions must be satisfied in each case (Kruseman et al., 1994). The calculations for the conditions are shown in Annex VI.

Table 3.5. Summary of calculated transmissivities.

Well	Test	T (m^2/d)	T (m^2/d) (a)
Marula Farm	Pumping	168	269
	Recovery	216	220
KCC	Pumping	96	48
	Recovery	84	75
Three Ostrich Farm	Pumping	6600	1020
Manera Farm	Pumping	816	670
	Recovery	696	728

[a] Values calculated by AQUITEST.

Other pumping tests and their interpretations have been carried out in the area by a private company (VIAK, 1975). The results of these tests together with the methods used are shown below, Table 3.6.

Table 3.6. Transmissivity, Storativity and Leakage coefficients as calculated by VIAK (1975).

Test	Method	Well	T (m^2/d)	S	K'/D' (d^{-1})
Step-Drawdown	Gustafsson	BH1	500		
		BH2	345		
		BH3	207		
		BH4	285		
Recovery	Theis	BH1	371	$3.1 \cdot 10^{-4}$	
		BH3	233	$2.3 \cdot 10^{-2}$	
Pumping	Jacob	BH1	233	$2.24 \cdot 10^{-3}$	
		BH3	224	$1.13 \cdot 10^{-3}$	
		BH4	198		
Pumping	Hantush	BH1	233	$2.2 \cdot 10^{-3}$	$9.9 \cdot 10^{-4}$
		BH3	207	$1.2 \cdot 10^{-3}$	$3.8 \cdot 10^{-4}$
Pumping	Theis-Walton	BH3	216	$1.1 \cdot 10^{-3}$	$2.2 \cdot 10^{-4}$

These tested wells are located in the same area as those where pumping test where carried out during the field work. It is observed that transmissivity values shown in Table 3.5 are of the same order of magnitude as those calculated by VIAK (1975).

CHAPTER 4. MODEL SETUP

4.1.- INTRODUCTION

Numerical models have become important tools in modern hydrogeology. There are two areas of this discipline where we need to rely upon models of the real hydrogeologic system: to understand why a groundwater flow system is behaving in a particular way and to predict the future behavior of such a system. Moreover due to their flexibility and easy implementation the use of models permits us to generate and simulate hypothetical situations of the flow system to gain insight in it. In studying the groundwater flow system two definitions are of major importance, one is the *conceptual model* and the second is its translation into a *numerical model*. The conceptual model is nothing else than a simplified representation of reality because it can not reflect all the complexity of the real system even if this conceptual model is based on a large amount of information about the system it is describing, "*Our conceptual model will always be less complex than the real system, a conceptual model could never fully describe all details of the real system*" (Fetter, 1994).

4.2.- CONCEPTUAL MODEL

The formulation of a conceptual model is determined in some extent by the amount of information available, the model scale, the purposes of the model and the complexity of the zone under study. The conceptual model is a representation of the understanding of the real physical system. In general the following aspects define the conceptual model:

- Type of aquifers.
- Number of aquifers considered during modeling.
- Relation among aquifers.
- Aquifer geometry.
- Location, magnitude and spatial distribution of the aquifers.
- Location and distribution of surface water bodies.

- Boundary conditions.
- Hydraulic properties of aquifers. This topic includes transmissivity, hydraulic conductivity, storativity, leakage coefficient, etc.
- Natural and/or artificial recharge zones.
- Pollution sources.

4.2.1.- AQUIFER SYSTEM

As it was described before (Chapter 3) the aquifer system is characterized by the presence of thin and less permeable layers of clay, silt and basalt interbedded with thicker layers of sand and coarser material. The real situation represents a multiple leaky aquifer system with a top aquifer unconfined, although, in some parts of the modeled area this top unconfined aquifer is not present, Fig. 4.1. However, this leaky system is underlain by impermeable layers formed by clay, silt and hard volcanic rocks such lavas and trachytes.

To build the conceptual model for this particular case the concept of hydrostratigraphic units has been applied. This concept implies that geologic units of similar hydrogeologic properties may be combined into a single hydrostratigraphic unit or a geologic formation may be subdivided into aquifers and confining units. The concept of hydrostratigraphic units is most useful for simulating geologic system at a regional scale (Anderson et al., 1992).

In Fig. 4.1 a diagrammatic geologic representation of the modeled zone is presented, this figure also shows how complex the zone is. In turn Fig. 4.2 represents a simplified scheme with the purpose of modeling. For this purpose it is assumed that the aquifer is confined and has an average thickness of 50 m.

In the conceptual model two rivers, Gil Gil and Malewa, and Lake Naivasha have been included because of their relationship with the aquifer system. The lake occupies roughly the central part of the area to be modeled, while the rivers extend from north to south and discharge into Lake Naivasha.

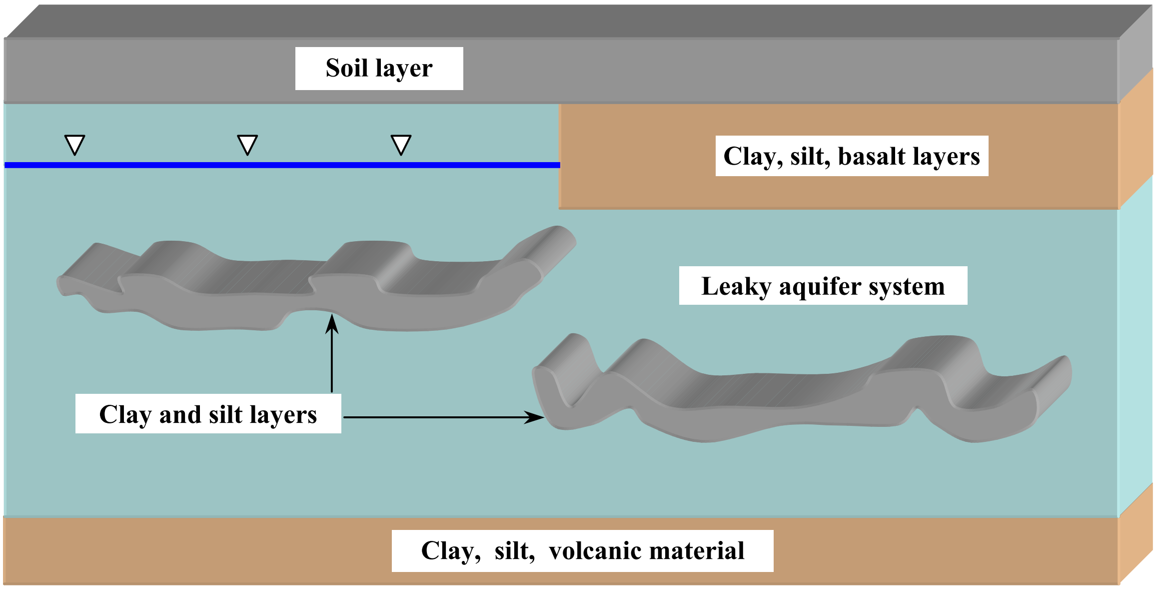


Figure 4.1. Diagrammatic geologic section of the leaky aquifer system.

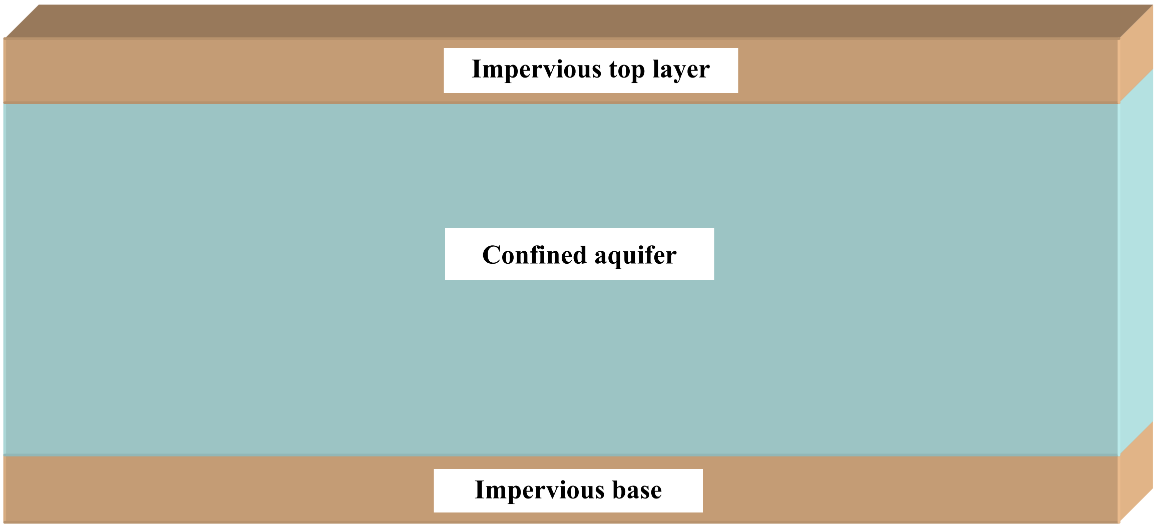


Figure 4.2. Diagrammatic section for modeling purposes.

4.2.2.- BOUNDARY CONDITIONS

One of the components of a mathematical model are the boundary conditions. Boundary conditions are mathematical statements specifying the dependent variable or the derivative of the dependent variable (Anderson et al., 1992).

The boundaries in the model were selected following geological features and structures, geomorphological features and hydrogeological evidences. Among the geological features faults, including the rift faults, are the most important, see Fig. 2.2 and 2.3. These fault lines are *physical boundaries*. Surface water divides belong to the *hydrological boundary* type. They were also taken into account to select part of the model boundaries. Finally the small but important evidence of some groundwater EC values assisted in the definition of the boundaries. Lakes and rivers were taken as constant head boundaries.

To the East: In the eastern part a long and pronounced fault scarp is present, South Kinangop fault scarp, Fig. 2.1. EC routing studies made by Graham (1998) showed that the differences in EC readings on both sides of the fault were almost comparable, the same observation was made during the fieldwork (EC routing was made by Behar, 1999). This situation indicates that the groundwater inflow from the eastern part of the fault is almost negligible and as such was considered a *no flow boundary*. Another evidence taken into account to select this fault as a no flow boundary was the fact that it can be easily observed in the LANDSAT image that the river flow directions are generally away from the fault, Fig. 2.3.

To the West: In the western side of the area another fault line can be seen on both the image and the geological map. Although no definite evidence exists to consider that alignment as a physical model boundary it is believed, after the inspection of the geological map and the color composite image, that this feature is really a fault which acts as a no flow boundary. This fault is likely to be associated with Elburru volcanic complex present to the north and which constitutes a surface water divide.

To the North: This boundary has been selected following a water surface divide. It is assumed that through this boundary no groundwater flow occurs, therefore a no flow boundary has been placed there for the model.

To the South: In the south Longonot volcano and Olkaria complex represent geomorphological and geological features and structures that provided evidences to consider this part of the model as a *no flow boundary,* Fig. 2.2 and 2.3. This zone coincides with a surface water divide, represented by a volcano and lavas flows. It is thought that through this zone, more specifically to the southwest, a deep groundwater outflow occurs. This is considered the way through which water from Lake Naivasha escapes to the Olkaria complex.

4.2.3.- GEOMETRIC CHARACTERISTICS

As can be seen in the conceptual model and from the assumed boundary conditions the aquifer extends from north to south with irregular boundaries. Its average length is about 38 km while its width is about 37 km in the widest part and approximately 12 km in its narrowest part. Its average thickness is considered to be 50 meters and extends from north to south.

4.2.4.- HYDRAULIC CHARACTERISTICS

The hydraulic properties, e.g., transmissivity, of the aquifer used in the model for its first run were obtained from the analysis of pumping test data collected during the fieldwork. Furthermore transmissivity values obtained last year during a modeling exercise were also used. The magnitude and spatial distribution of hydraulic properties of the aquifer are not well known for the zone and for the last model runs have been estimated and calibrated during the process of model calibration. These hydraulic characteristics depend mainly on aquifer type and geological conditions.

4.3.- SURFACE WATER BODIES

Surface water bodies are represented by Lake Naivasha itself and a by smaller lake to the southeast of Lake Naivasha. This small lake is named Oloidien Bay. It was previously connected to Lake Naivasha, Fig. 4.3. These two lakes were not connected in the last few years and have been considered as two independent surface water bodies in the present study. However for the effect of modeling there is no difference between considering them as two independent lakes or just as one larger lake. Lake Naivasha acts as source of recharge to the aquifer in zones where the water table has dropped below the lake’s level.

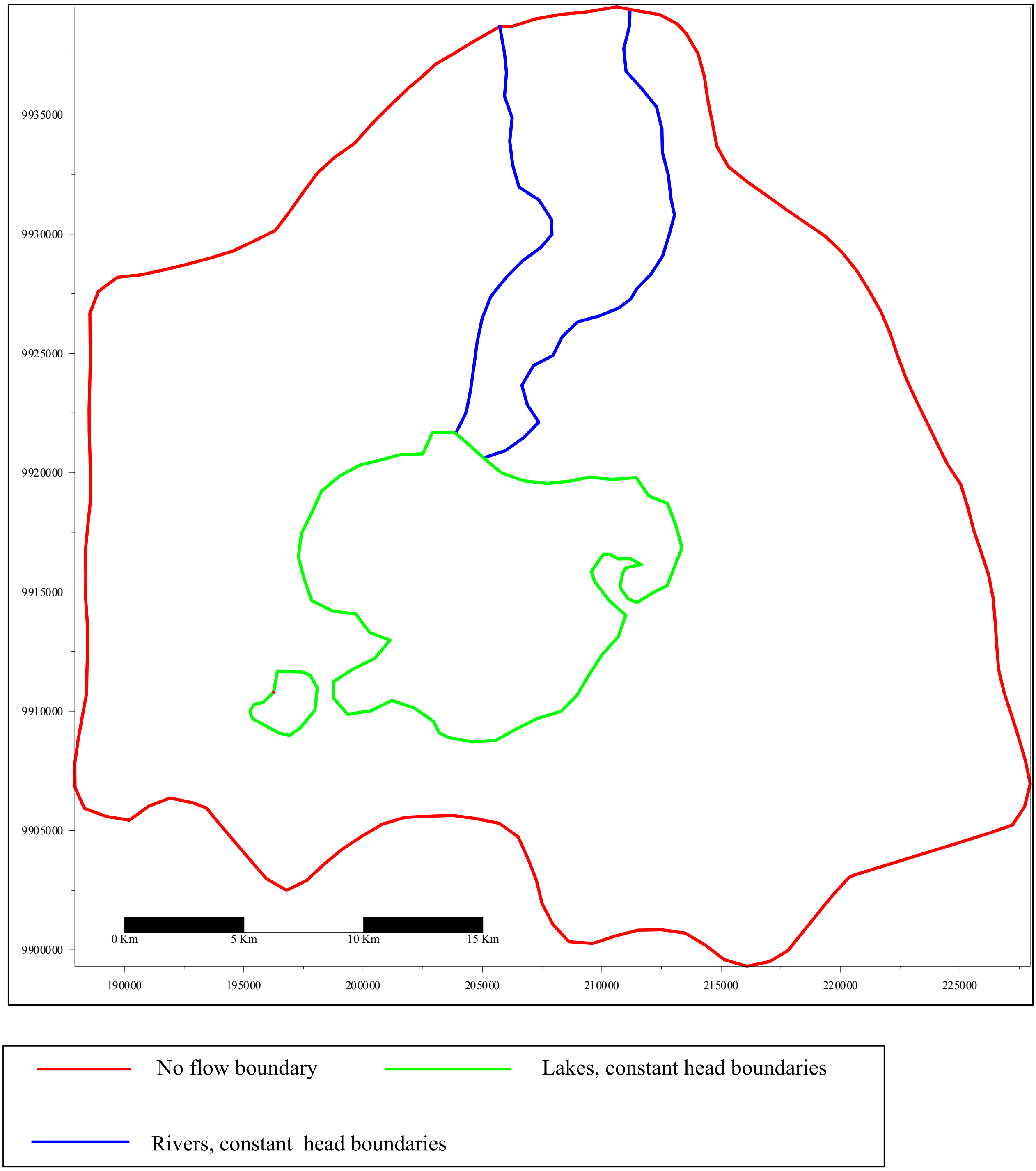


Figure 4.3. Conceptual model definition.

4.4.- DATA AVAILABLE

The data set available for this simulation consists of heads measured in wells the area during this fieldwork. Aquifer geometry has been deduced from interpretation of well logs carried out during the fieldwork (1998), thus as literature review (McCann, 1976). Values of transmissivities have been obtained from pumping test data analysis and extracted from previous works (VIAK, 1975; McCann, 1976). The missing heads were obtained by interpolation in the software used for modeling; the interpolation method used is based on the nearest neighbor principle. The purpose of the interpolated values is just to help the software in reaching the convergence criterion faster. An initial recharge value has been taken from a previous work made in the zone, (Wiberg, 1976), as 50 mm / year. This value was used to start running the model and lately modified during calibration. The exception was the area near Elburru Volcanic Complex where a value of 150 mm / year was used.

4.5.- NUMERICAL MODEL

Except for very simple situations analytical solutions are used to solve flow problems. Numerical models are more versatile and with the widespread availability of computers, are now easier to use than some of the more complex analytical solutions (Anderson et al., 1992). In this particular case the GMS 2.1 software has been chosen to implement, edit and translate the conceptual model into a numerical model. Facilities provided by the software were used to convert the conceptual model from a high-level feature object-based definition to a grid-based MODFLOW numerical model, which was considered to have the necessary capabilities to solve the problem in question.

4.5.1.- DISCRETIZATION

Here just space discretization is described. As pointed out before MODFLOW was the numerical model used to solve the flow equation. This package is a three-dimensional finite difference model and is included in GMS 2.1. When using the conceptual model approach in GMS a frame and a grid are used to define the boundaries of the conceptual model, through a simple process of *cell activation* the grid is correctly positioned on the conceptual model and later on used to create the grid-based MODFLOW numerical model. The grid dimensions were established at 1 km x 1 km. The model consists of 42 rows and 39 columns. The space discretization grid is shown in Fig. 4.4.

4.5.2.- MODEL INPUTS

In general model inputs consist of recharge, evapotranspiration and other artificial stresses imposed on the aquifer, hydraulic parameters, boundary conditions and time discretization in the case of transient simulation. The model was primarily run and calibrated in steady state and later run in transient condition. For the transient stage different stress period lengths were used (Chapter 6).

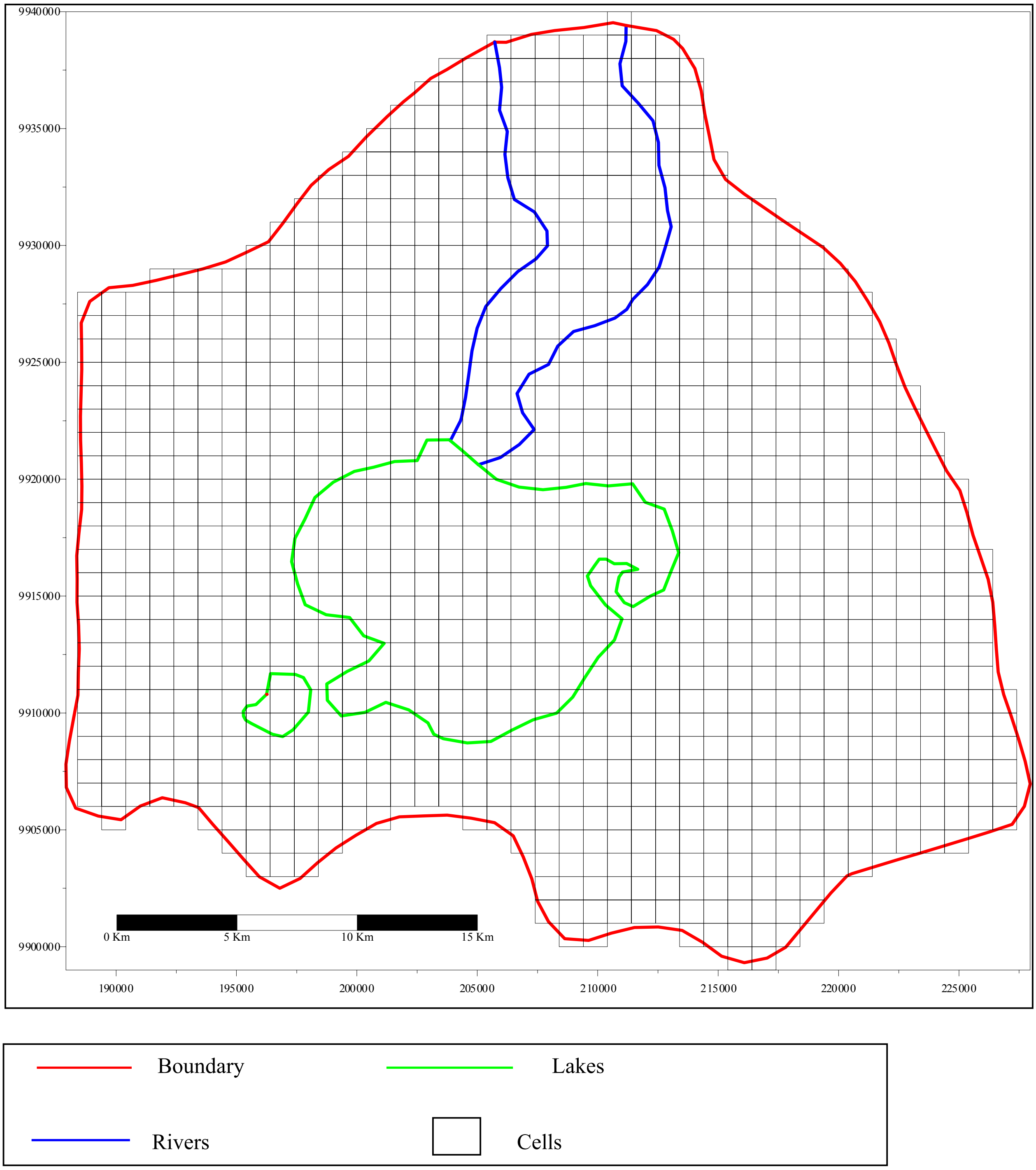


Figure 4.4. 2D Representation of the model space discretization.

CHAPTER 5. STEADY STATE CONDITIONS

5.1.- INTRODUCTION

A model is initially calibrated by taking the initial estimates of the model parameters and solving the model to see how well it reproduces some known condition of the aquifer (Fetter, 1994). A model calibration in steady state conditions is a basic first step in many modeling exercises. The existing records about water levels in wells and boreholes in the area are not consistent, therefore transient simulation calibration was not be possible. Because a reliable head distribution is necessary for a correct model calibration only those wells and boreholes in which water levels were measured during the 1997 and 1998 field work were used in the model. The complete modeling process is shown in Fig. 5.1.

5.2.- WELL DISTRIBUTION

The modeled area has been characterized by an intense agricultural development during the last 10 years; this situation has been associated with an inevitable increase in water demand. Due to the fact that volcanic material is widely spread in the region and to low values of precipitation during the year not all zones are suitable for an agricultural development at large scale. At those farms far away from the lake and the rivers, where fresh water is available, the only source of water left is groundwater. This is why the wells are found in clusters in the area rather than being homogeneously distributed. In some parts of the modeled area there are no wells at all. Fig. 5.2 shows the well distribution for calibration.

5.3.- MODELING APPROACHES

Two modeling approaches are included in the GMS package and both were used to run and calibrate the model: The conceptual model approach and the grid modeling approach. Each of them has advantages and disadvantages, and therefore they were used whenever convenient. When large areas are going to be modified, zone by zone editing

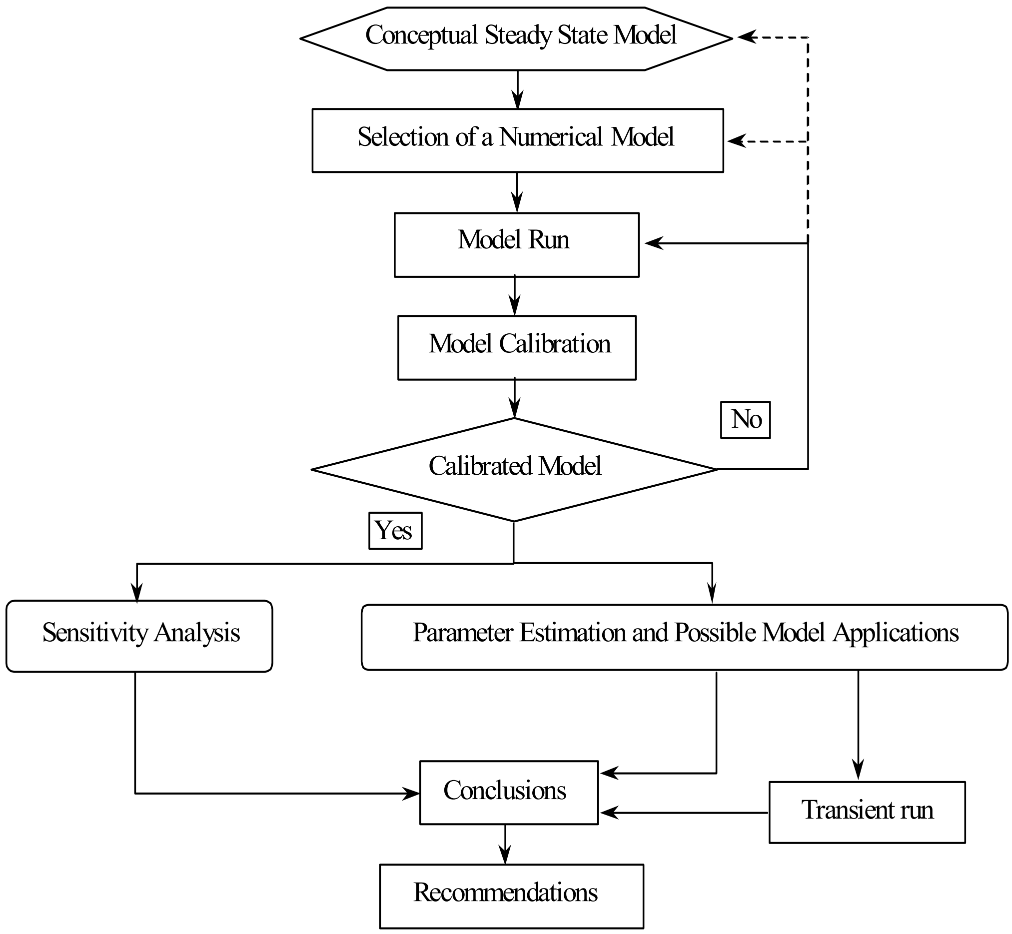


Figure 5.1. Flow chart during modeling

is more convenient but if small areas need changes in their parameter values, a cell by cell approach is more suitable. A brief description of both approaches is given below.

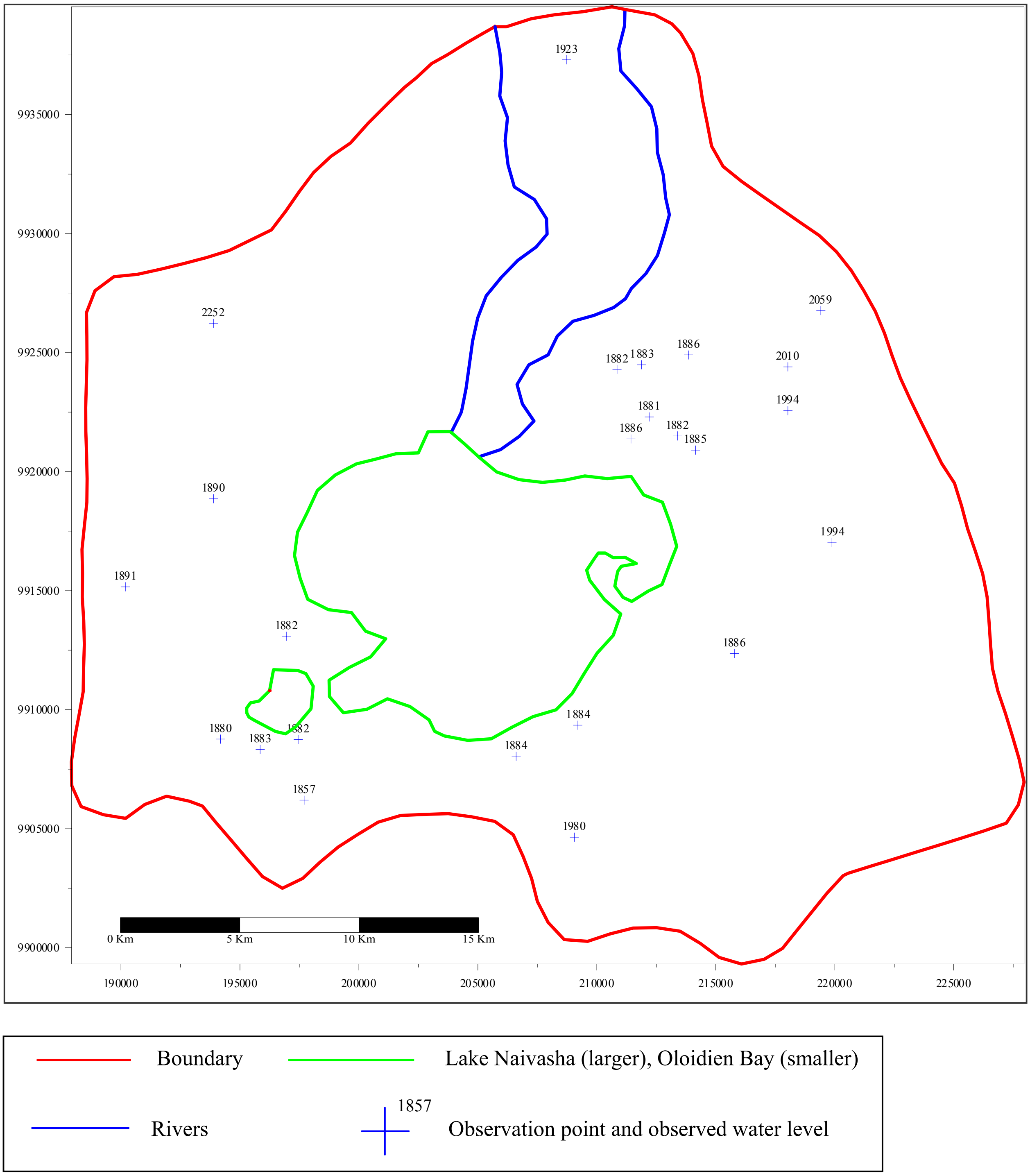


Figure 5.2. Location and distribution of wells for the calibration.

5.3.1.- CONCEPTUAL MODEL APPROACH

The conceptual model approach involves using GIS tools to develop the conceptual model of the study area. The location of the sources/sinks, layer parameters such as transmissivities, layer boundaries, and all other data necessary for the simulation can be generated at this level. Once this is complete the grid is generated and the conceptual model is converted to the grid model and the entire cell by cell assignments are performed automatically. In this approach all the experience of the modeler and the knowledge about the area (represented under the forms of arcs, polygons, points) is transformed into a grid-based model. The previous process is accomplished by defining zones for each one of the areal parameters included in the simulation such as recharge and evapotranspiration.

The definition of different zones was based mainly on the combination of visual interpretation of Landsat images, field observations and the fieldwork carried out during October 1997 and 1998. During the calibration process the model results showed that changes in the conceptual definition had to be introduced in order to be able to calibrate the model.

5.3.2.- GRID APPROACH

This approach involves working directly with the 3D grid and applying sources or sinks and other model parameters on a cell by cell basis. This approach is more suitable for very simple models, which is not the case in this study. Although at the start of the simulation the conceptual approach is more appropriate for model-making, a grid-based approach is more suitable during the final part of the process since minor changes are required in order to make the calculated heads fit the observed ones; another factor that forced the use of this approach was the space discretization which in this case is used to investigate the regional behavior of the aquifer.

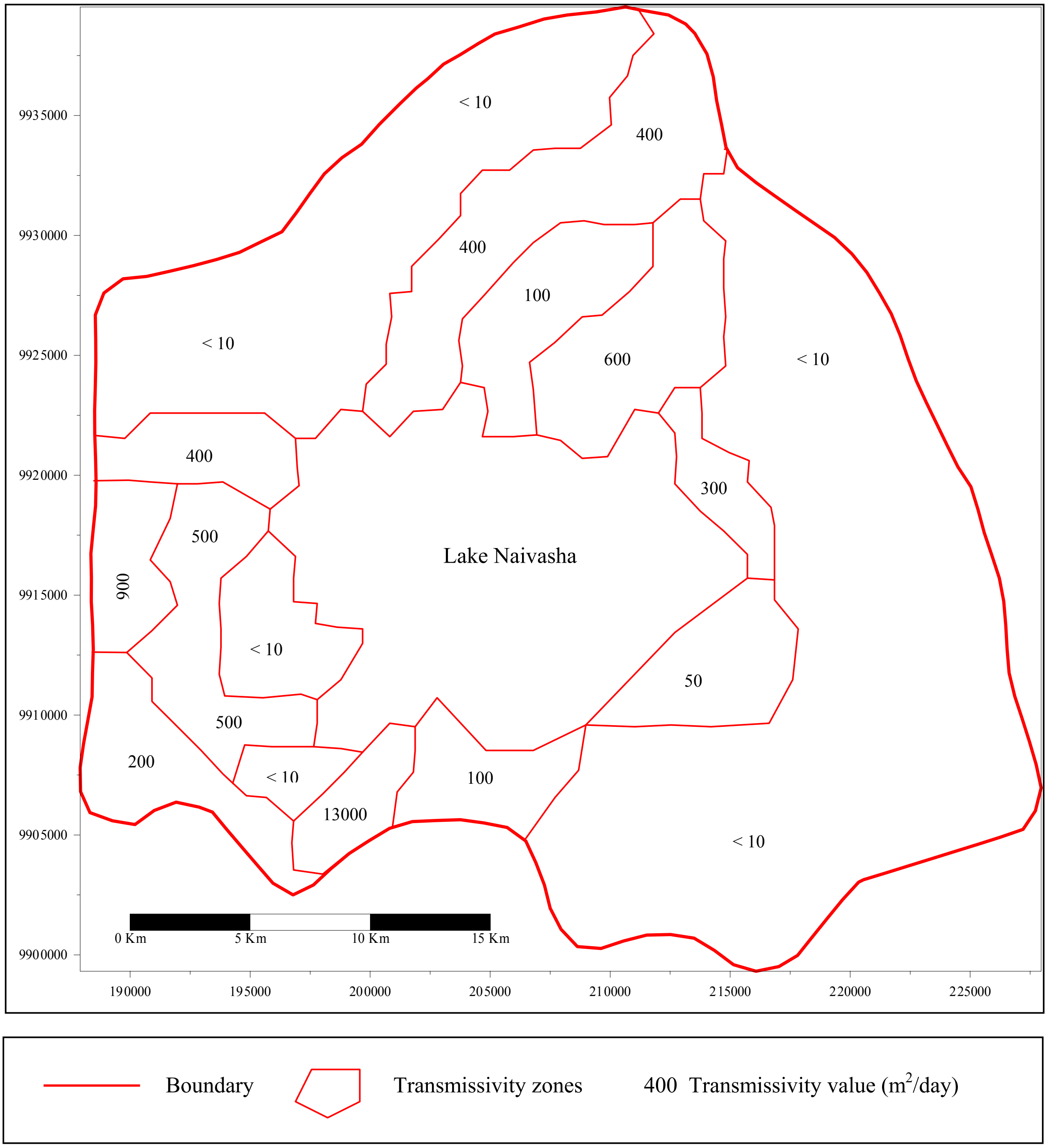


Figure 5.3. Different zones of transmissivities.

5.4.- SOURCES / SINKS TERMS

As explained before the agricultural activity has increased dramatically during the last 10 years and therefore the water consumption. Large amounts of water for irrigation, industry, livestock and human consumption have been used during this time. The main source of water for some farms and other dependencies is groundwater. This situation, associated with the prevalence of a semiarid climate in the region and low values of annual rainfall (500 mm) and recharge (50 mm), may lead to substantial changes in aquifer water level. The effect of the combined abstractions is introduced in the model using the sink term.

The information about the abstraction rates existing in the farms was extracted from previous work in the area (Huaccho, 1998). Although the declared rates for each farm were used in the model it seems that larger rates have existed during the past years. Five abstraction wells were used for the most affected area, to the northeast. The total discharge of these wells amounts to 18 000 m^3/day.

5.5.- MODEL CALIBRATION

Calibration is accomplished by finding a set of parameters, boundaries, stresses that produce simulated heads and fluxes matching observed values in the field. A complication in groundwater problems is that the distribution of heads is always incomplete and flux calculations are not always known accurately. Estimates of flux have associated errors that are usually larger than errors associated with head measurements (Anderson et al., 1992). Nevertheless, it is advisable to use estimates of flow as calibration values in addition to heads in order to increase the likelihood of achieving a unique calibration.

The model must be first calibrated before it can be used for prediction, that is, the model parameters should be adjusted until the simulation is consistent with the analyst's understanding of the groundwater system and the available data. Most models are initially calibrated against the steady-state heads, the head distribution is usually well known. Another calibration target is the flux estimates. This procedure is shown in the

Fig. 5.4. Computed head values must match closely those measured at observation points. The observed differences between the measured and calculated values can give an indication of those places where adjustment must be done first.

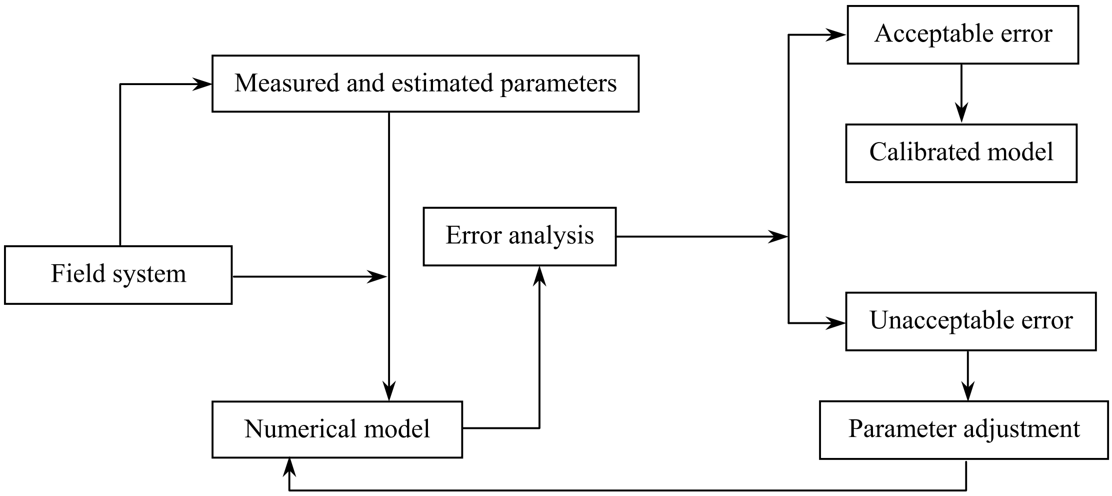


Figure 5.4. Trial and error calibration procedure (after Anderson et al., 1992).

There are two basic methods of model calibration: the first one is *trial and error adjustment of parameters* and the second *automated parameter calibration.* Although automated parameter calibration has some advantages when compared with trial and error method the former one was used. Several runs had to be made before the model was completed calibrated.

5.5.1.- CALIBRATION PROCEDURE

In trial and error procedure initial values of heads are assigned to cell together with other parameters included in the model. Heads calculated by the model are compared with those observed, this process of parameter adjustment is repeated until a required fit is reached. In a calibration procedure there are three accepted steps (Anderson et al., 1992):

1. To first change the values in cells where the highest deviation occurs.
2. To change just one parameter in each run.

3. To determine if any change of that parameter has a positive or negative effect in other cells.

Heads can have errors associated due to the measuring device, the operator, the topographic survey in the area especially in hilly areas, scale errors because wells with large screens and point levels are needed for 3D models. In the case of a two-dimensional areal model this does not affect the model. Another source of error is the small-scale variability inherent to hydraulic parameters in horizontal and vertical directions. In the model transmissivity is vertically averaged and does not take into account this small-scale variability. Besides the above-mentioned sources of errors others can be mentioned such as truncation and round-off errors.

Finally, the model was considered to be calibrated with an acceptable error. This error was estimated taking into account the data quality and the area characteristics. An example of the calibration target is given in the Table 5.1 below. *Interval* is the range within which a calibrated point head can vary.

Table 5.1. Example of calibration target.

Well id	Name	X	Y	Z	Wlevel	interval	confid.
10	C210	209200	9909350	1834.4	1884.1	2	0.95
11	C630-D	197700	9906200	1806.8	1856.8	2	0.95
.	.	.	.	.	.	2	0.95
.	.	.	.	.	.	2	0.95

The model was considered to be calibrated once the differences between the calculated heads minus the observed heads were in a range of ± 2 meters. Table 5.2 below shows values of errors during the calibration. A graph of error versus simulation is also included, Fig. 5.5. Equations 5.1, 5.2, 5.3 were used to calculate these statistics.

Table 5.2. Error variation during the calibration process.

Run number	Mean error	Mean abs. error	Root mean sq. error
1	1.87	2.24	4.18
2	1.65	2.02	3.57
3	-0.5	1.48	2.21
4	0.45	0.87	1.16
5	0.13	0.73	0.87

$$Me = \frac{1}{n}\sum_{i=1}^{n}(h_{cal} - h_{obs})_i \qquad 5.1$$

$$Mae = \frac{1}{n}\sum_{i=1}^{n}\left|(h_{cal} - h_{obs})_i\right| \qquad 5.2$$

$$Rmse = \left[\frac{1}{n}\sum_{i=1}^{n}(h_{cal} - h_{obs})_i^2\right]^{0.5} \qquad 5.3$$

h_{cal}	Calculated head
h_{obs}	Observed head
Me	Mean error
Mae	Mean absolute error
$Rmse$	Root mean square error

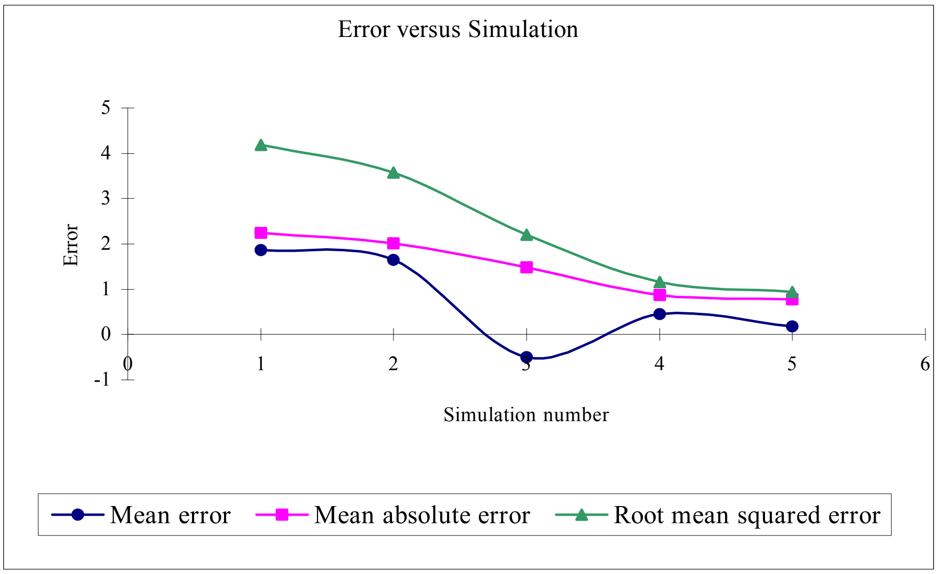


Figure 5.5. Simulation errors versus number of simulation.

The final model run gave the best results in this stage. A graphical representation of the match between the observed and calculated heads is given below, Fig. 5.6. At this point the model is considered to be calibrated as long as the simulated heads match the observed heads, the simulation summary is given in Table 5.3.

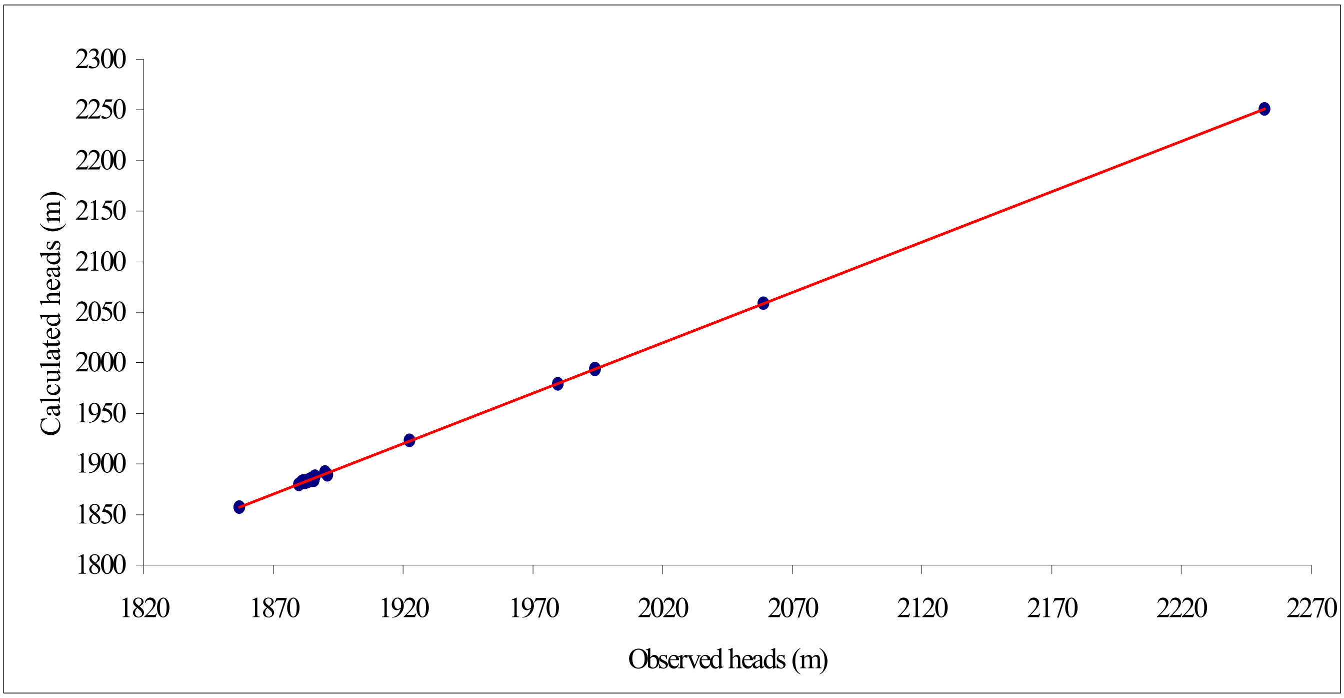


Figure 5.6. Results of the last model run, observed versus calculated heads.

Table 5.3 Error summary of the calibrated model.

Error summary	
Calculated parameter	Value (m)
Mean error	0.13
Mean absolute error	0.73
Root mean square error	0.87

Another way to check the simulation results is a visual comparison between the observed head distribution and that calculated by the model, Fig. 5.14 and 5.15. An acceptable agreement exists between both maps. Table 5.4 shows the difference between observed and calculated heads. At this point it is necessary to note that the disagreement in the west part of the model is resulting from scarcity of data against which the model may be calibrated. Another area where information is scarce is to the southeast.

5.6.- GROUNDWATER FLOW CALCULATIONS

It is clear that in areas where an intense agricultural development has taken place the water levels have dropped. In this particular case many water levels measured in wells have been found to be even below the lake level. This situation lead to groundwater flow from the rivers and the lakes to the aquifer. The difficulty is to quantify these flows. Water balance calculations are mainly affected by uncertainties in precipitation values, evapotranspiration values, groundwater inflow to and outflow from the catchment and stream flows. The best known parameter for the catchment is the stream flow into the lake.

Table 5.4. Observed and calculated heads for observation points.

Well	Observed head (m)	Calculated head (m)	Difference (m)
ADC Ndabibi	1882.10	1881.98	-0.13
C-10887	1881.50	1882.53	1.03
C-1404	1890.87	1889.52	-1.35
C-1488	1994.00	1993.44	-0.56
C-210	1884.40	1885.00	0.60
C-2522	1890.00	1891.75	1.74
C-3675	1881.10	1882.20	1.10
C-465	1922.53	1923.16	0.63
C-630-D	1856.80	1857.40	0.60
C-7829	1884.06	1884.78	0.72
C-939	1994.00	1994.33	0.33
Kongoni_1	1879.80	1879.59	-0.21
Kongoni_2	1883.12	1882.31	-0.81
La Belle Inn	1884.98	1884.02	-0.96
Manera	1885.67	1883.90	-1.77
Milk_Fact	1882.61	1883.37	0.76
Mirera/Suswa	1886.12	1887.77	1.65
N-18	1979.60	1979.34	-0.26
N-23	1881.54	1882.96	1.41
N-54	2059.00	2058.79	-0.21
TOF_3	1885.50	1885.15	-0.35
UW1	1882.08	1882.06	-0.02
C-4600	2252.00	2251.10	-0.90

McCann (1974) estimated an outflow value of $34 \cdot 10^6$ m^3/year; Ase (1986) estimated an outflow between 45-$50 \cdot 10^6$ m^3/year; Gaudet and Melak (1981) estimated an amount of $44 \cdot 10^6$ m^3/year; Ojiambo (1992) calculated a value of $38 \cdot 10^6$ m^3/year. An outflow value of approximately $55 \cdot 10^6$ m^3/year has been estimated by Mubui (1999), that value was used to run and calibrate the model. As can be seen from the presented figures substantial variations exist in the estimated lake outflows. It is understood that any water balance calculations made with the model will be affected by uncertainties derived from the poor knowledge of the outflow rates as well as of the parameters. Flow calculations concentrated on an area to the northeast of the lake, Fig. 5.7., because its importance for agricultural development. Flow calculations in the area were made in order to determine the amount of lake water draining to the aquifer because of groundwater abstraction. Ojiambo (1992) presents a detailed summary of these calculations. Results in this paper are expressed on daily and monthly basis.

5.6.1.- FLOW FROM THE LAKE INTO THE AQUIFER

Because the water level in the well field, located to the NE of Lake Naivasha, has dropped, groundwater is flowing from the lake into this well field. By using the model an estimate of this amount of flow can be obtained. In order to accomplish this, a flow direction vector map was created first. This map was made using one of the functionalities incorporated in GMS. It is observed that to the northeast of the lake there is an important seepage zone, through which water is flowing into the aquifer, Fig. 5.8. An outflow rate of 2000 m^3/day equivalent to 60 000 m^3/month was obtained. In other areas this quantity is much less and amounts to approximately 180 m^3/day, 5400 m^3/month. The amount of flow of 60 000 m^3/month is just a small portion of the total abstraction rate in the area which rises up to 540 000 m^3/month (Calculated amount based on declared abstraction rates). The abstraction rates in other areas are negligible compared with the well field, about 320 m^3/day, 9600 m^3/month (For another abstraction area see Fig. 5.7).

5.6.2.- FLOW FROM THE RIVER INTO THE AQUIFER

From the inspection of Fig. 5.9 it is seen that groundwater is also flowing from the Malewa River into the aquifer. It was believed that the major component of the groundwater inflow into the well field came from the Malewa River. This belief was corroborated by model calculations. The cells marked by red color squares were used to estimate the amount of flow from the river into the aquifer.

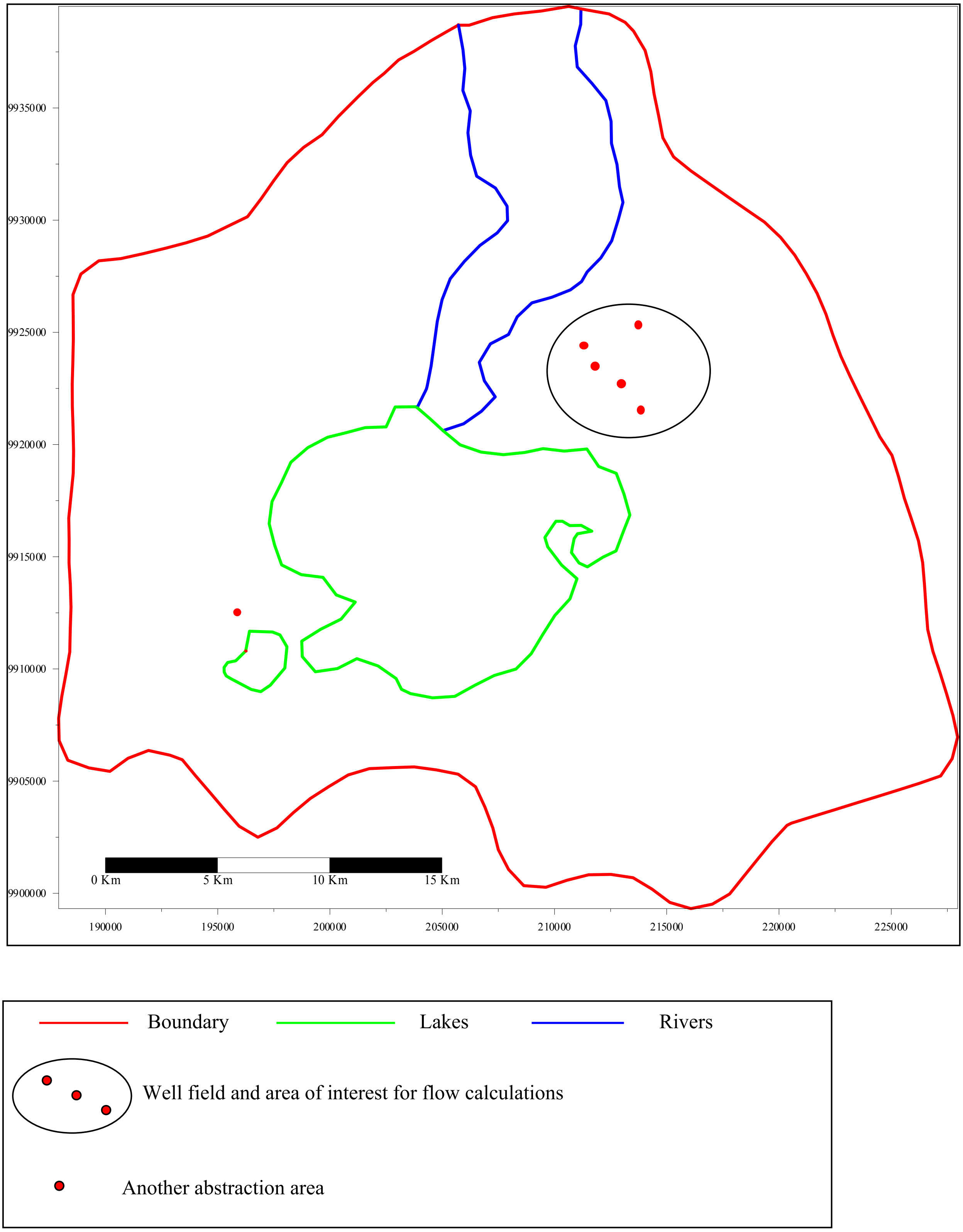


Figure 5.7. Area of interest for water flow calculations.

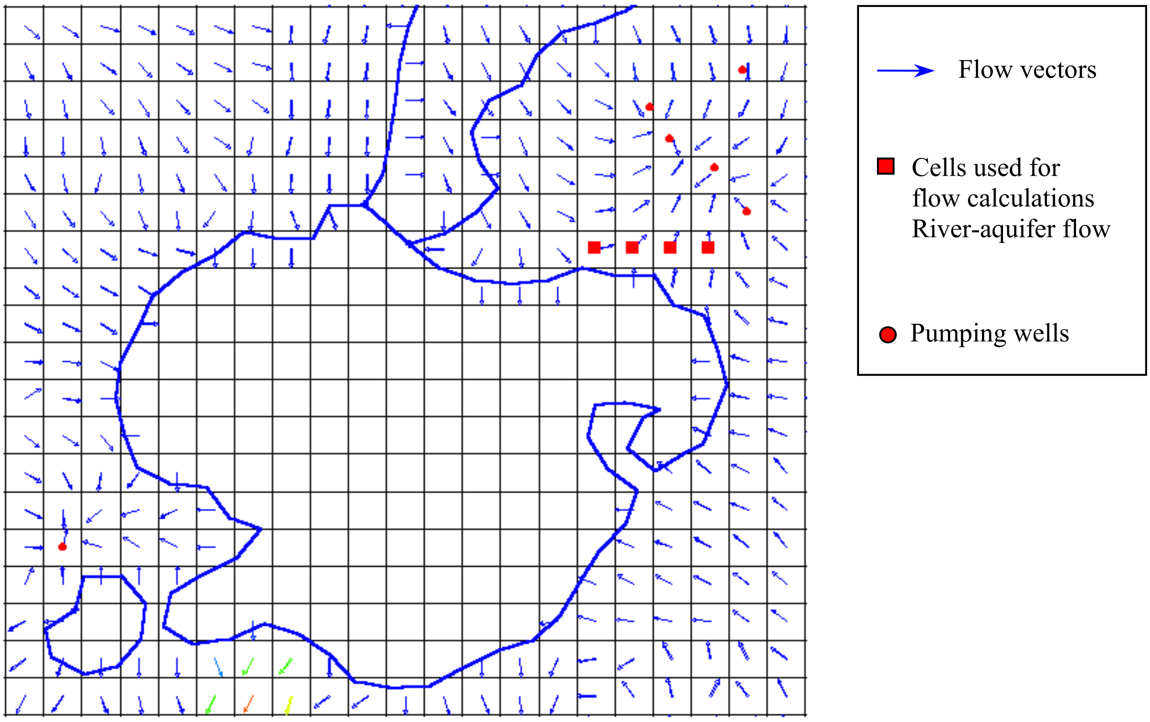


Figure 5.8. Cells used in the flow calculations for the interaction Lake-Aquifer.

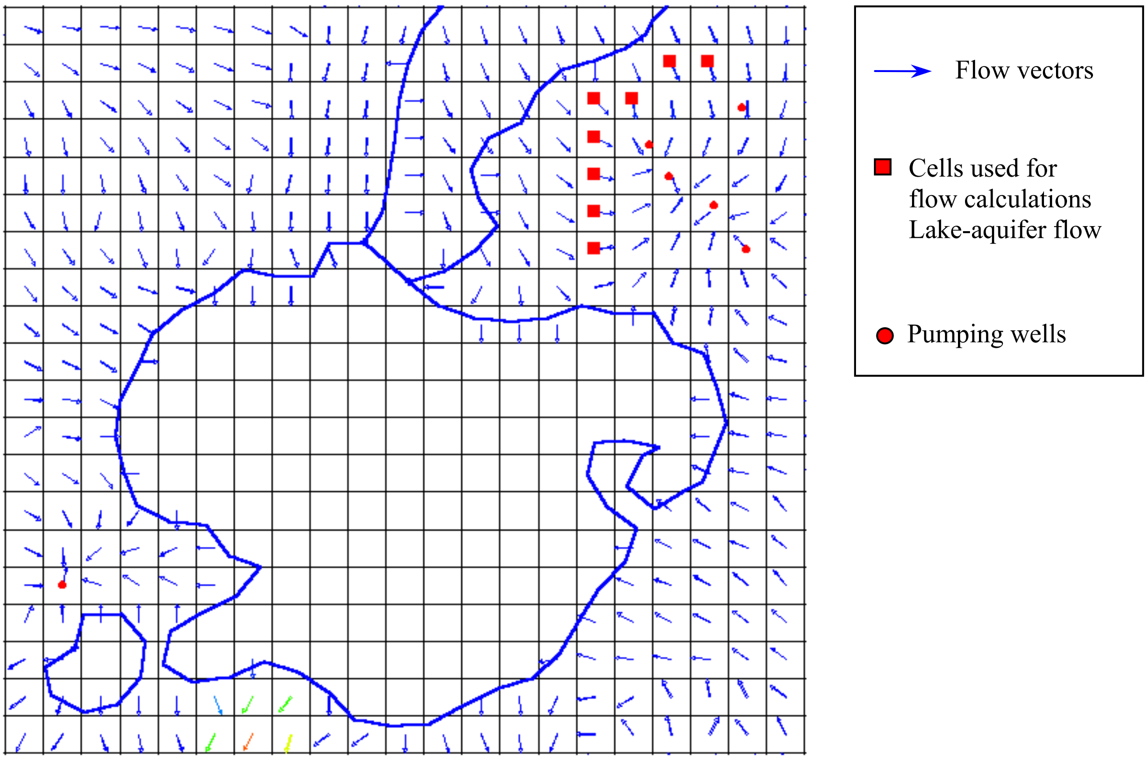


Figure 5.9. Cells used in the flow calculations for interaction River-Aquifer.

The model estimated value amounts to 14 000 m^3/day, which is equivalent to 420 000 m^3/month. Based on the quantities presented in the previous paragraphs an estimate for the water budget was made. All this values and results are presented in Table 5.5. Every inflow component for the water balance is shown in Fig. 5.10.

Table 5.5. Water balance calculations for the *area of interest* (*30 days month*).

Component	Amount [m^3/day]	Amount [m^3/month]
Groundwater abstraction	18 000	540 000
Recharge	1800	66 000
Inflow from Lake Naivasha	2000	60 000
Inflow from Malewa River	14 000	420 000
Inflow from the east	200	6000
WATER BALANCE		
Q_{in} = 18 000 m^3/day	Q_{out} = 18 000 m^3/day	$\Delta Q = 0$
Q_{in} = 540 000 m^3/month	Q_{out} = 540 000 m^3/month	

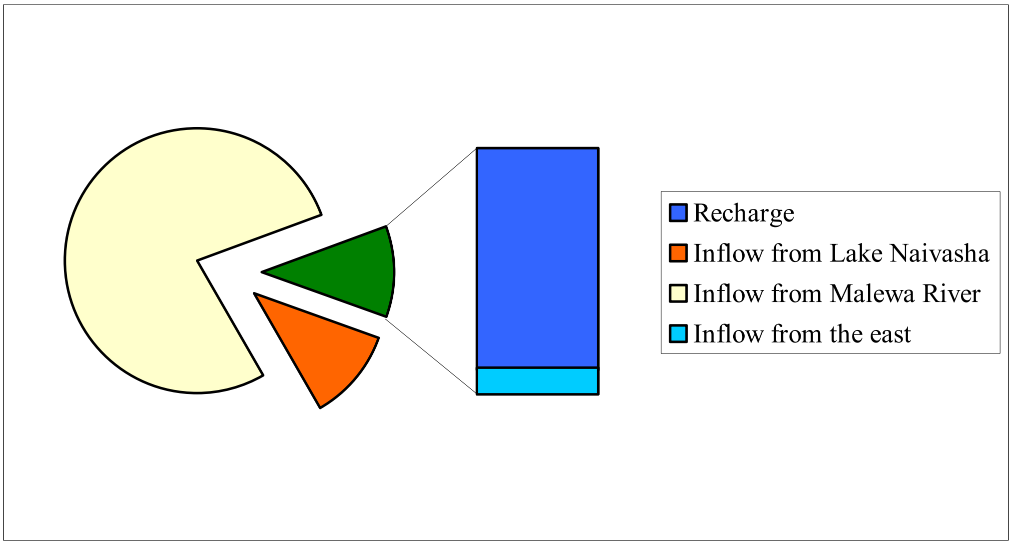


Figure 5.10. Components of the groundwater inflow to the well field.

The model also gives insight about the order of magnitude of the groundwater inflow into the lake. The value obtained according to the model amounts to 75 000 m^3/day, equivalent to $2.25 \cdot 10^6$ m^3/month (30 days month). A rough calculation about this amount may be done with the application of Darcy's law. Taking into consideration the average gradient in the area, assuming that Lake Naivasha represents a well in the center of the study area and taking an average value of transmissivity it is possible to estimate a value for this groundwater inflow, this situation is schematized in Fig. 5.11. Calculations are made using Equation 5.4 below.

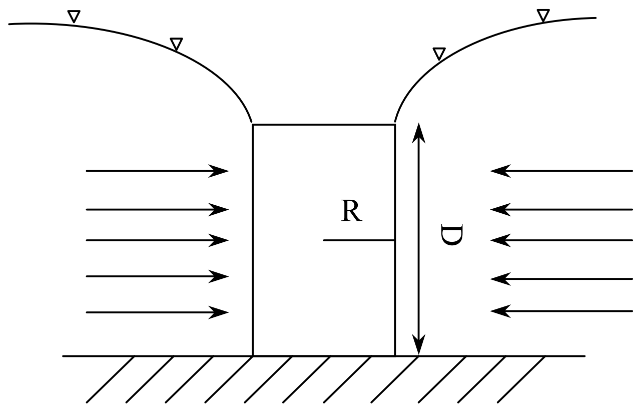


Figure.5.11. Scheme for lake groundwater inflow using Darcy's law.

$$Q = K \cdot A \cdot I \qquad 5.4$$

$$Q = K \cdot 2\pi \cdot R \cdot D \cdot \frac{dh}{dr} \qquad 5.4.1$$

Assuming an average $T = K \cdot D = 300\ \text{m}^2 / \text{day}$,

Assuming a $\frac{dh}{dr} = 0.01$ and $R = 5000$ m then

$$Q = 2\pi \cdot 300 \cdot 5000 \cdot 0.01$$

$$Q \approx 94000\ \text{m}^3 / \text{day}$$

It is seen that this value is of the same order of magnitude as the value calculated by the model which is 75 000 m^3/day.

5.7.- SENSITIVITY ANALYSIS

The purpose of sensitivity analysis is to quantify the uncertainty in the calibrated model caused by uncertainties in the estimates of the aquifer parameters, stresses, and boundary conditions (Anderson et al., 1992). Sensitivity analysis is typically performed by changing one parameter value at a time. The procedure followed here was:

- Start changes in transmissivity values.
- To make changes in the values up to 10, 20, 30, 40, 50 % larger than those for the calibrated model.
- To make changes in the values up to 10, 20, 30, 40, 50 % smaller than those for the calibrated model.
- To keep constant transmissivities and make changes in recharge values.
- To make changes in the values up to 10, 20, 30, 40, 50 % larger than those for the calibrated model.
- To make changes in the values up to 10, 20, 30, 40, 50 % smaller than those for the calibrated model.
- Observe the area most affected by the changes.
- To make a combined graph for transmissivities recharge changes versus error.
- Remarks of this procedure

5.7.1.- REMARKS ABOUT SENSITIVITY

In general the areas more affected by increases in transmissivities were in the east, southeast and west of the modeled area. Those zones are characterized by low transmissivities. The zone with the well field appears to react slowly to these increasing values of transmissivities and only when this increment is of the order of 40 % the levels change substantially. It is seen that the model is more sensitive to a decrease in values of transmissivities than it is to an increase in such values. A decrease of 40 % provokes a strong reaction in the well field. A decrease of 50 % in transmissivity affects the whole model, to the east the water levels went up but in the well field they decreased. To the west water levels were also increasing. Results of changes in error values with changes in transmissivities are shown in Tables 5.6 and 5.7 and Fig. 5.12.

Table 5.6. Error values for changes in transmissivities, 10-50 % higher values.

Run number	Mean error	Mean abs. error	Root mean sq. error
610	-2.98	4.18	8.58
620	-5.58	7.13	15.52
630	-7.77	9.67	21.4
640	-9.65	11.85	26.44
650	-11.28	13.74	30.81

Table 5.7. Error values for changes in transmissivities, 10-50 % smaller values.

Run number	Mean error	Mean abs. error	Root mean sq. error
609	3.93	4.76	10.04
608	8.68	10.21	22.78
607	14.79	17.42	39.17
606	22.94	27.08	61.03
605	34.35	40.59	91.63

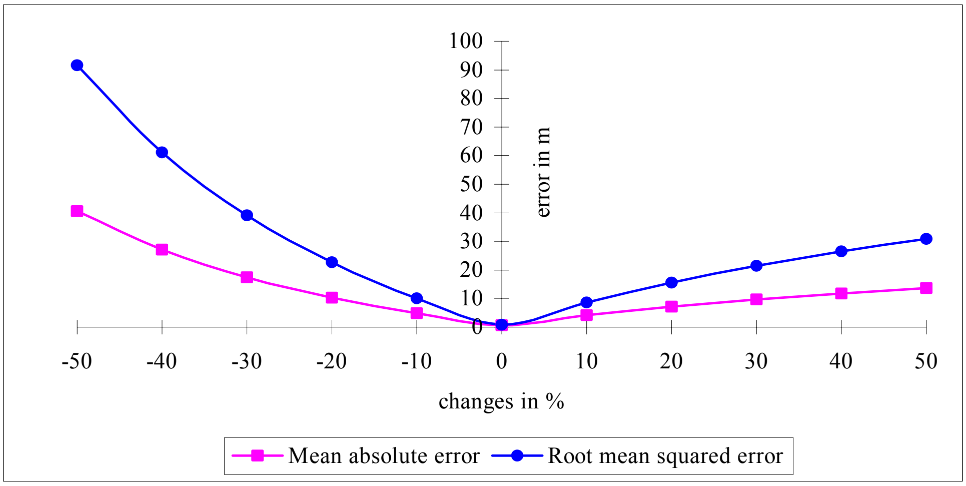


Figure 5.12. Errors versus change in transmissivities

A similar procedure was followed for recharge values. The original recharge value used in the model was a long-term average and little information about areal recharge distribution in the area was available. The zones most affected by changes in the model are located to the east and northwest of the lake. With an increase of 30 % or more the complete model shows a tendency to increase the water levels while, the opposite is observed with a decrease of 30 % or more. These results are shown in Tables 5.8 and 5.9 and Fig. 5.13.

Table 5.8. Error values for changes in recharge, 10-50 % higher values.

Run number	Mean error	Mean abs. error	Root mean sq. error
710	4.02	4.33	9.1
720	7.92	8.18	18.33
730	11.82	12.04	27.58
740	15.72	15.91	36.82
750	19.61	19.78	46.07

Table 5.9. Error values for changes in recharge, 10-50 % smaller values.

Run number	Mean error	Mean abs. error	Root mean sq. error
709	-3.77	4.39	9.47
708	-7.67	8.2	18.7
707	-11.57	12.03	27.94
706	-15.46	15.87	37.19
705	-19.36	19.7	46.44

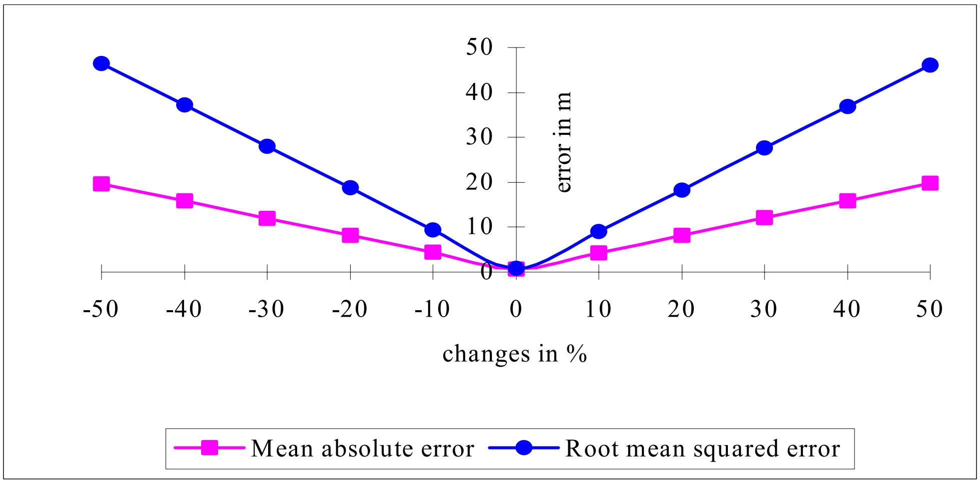


Figure 5.13. Errors versus change in recharge.

It can be observed from Fig. 5.13 that the model response to different values of recharge is almost symmetric. Again the areas with poor results are those in the east, southeast and northwest. It is interesting to note that all those wells are located in highland areas. As the final step of the sensitivity analysis a comparison between the model response under the changes in values of transmissivity and recharge is presented, Fig. 5.14.

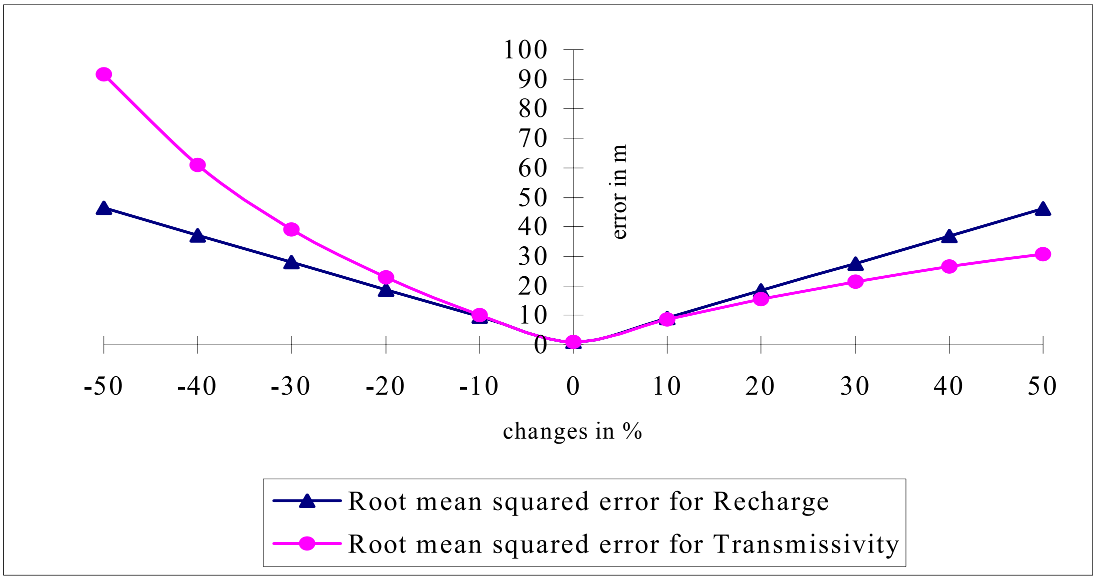


Figure 5.14. Model response to changes in values of transmissivity and recharge.

From the analysis of the previous figure it can be seen that the model is more sensible to low values of transmissivities than to recharge. An opposite behavior is observed for increasing values of both parameters. The model is slightly more sensitive to an increase in recharge than to transmissivities, however, the influence of higher values of recharge is not so pronounced as the low values of transmissivities are.

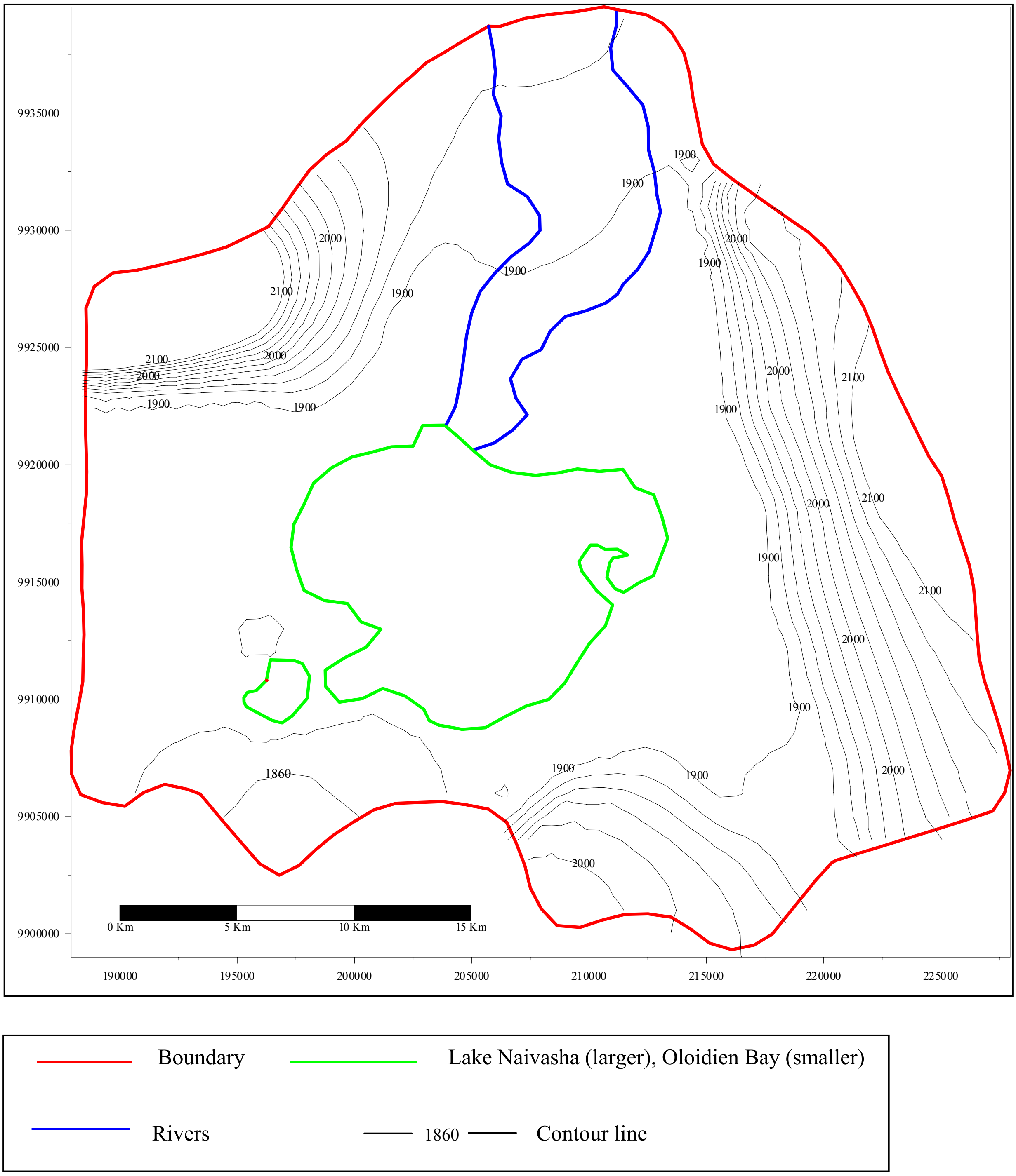


Figure 5.15. Contour map of modeled heads, contours every 20 meters.

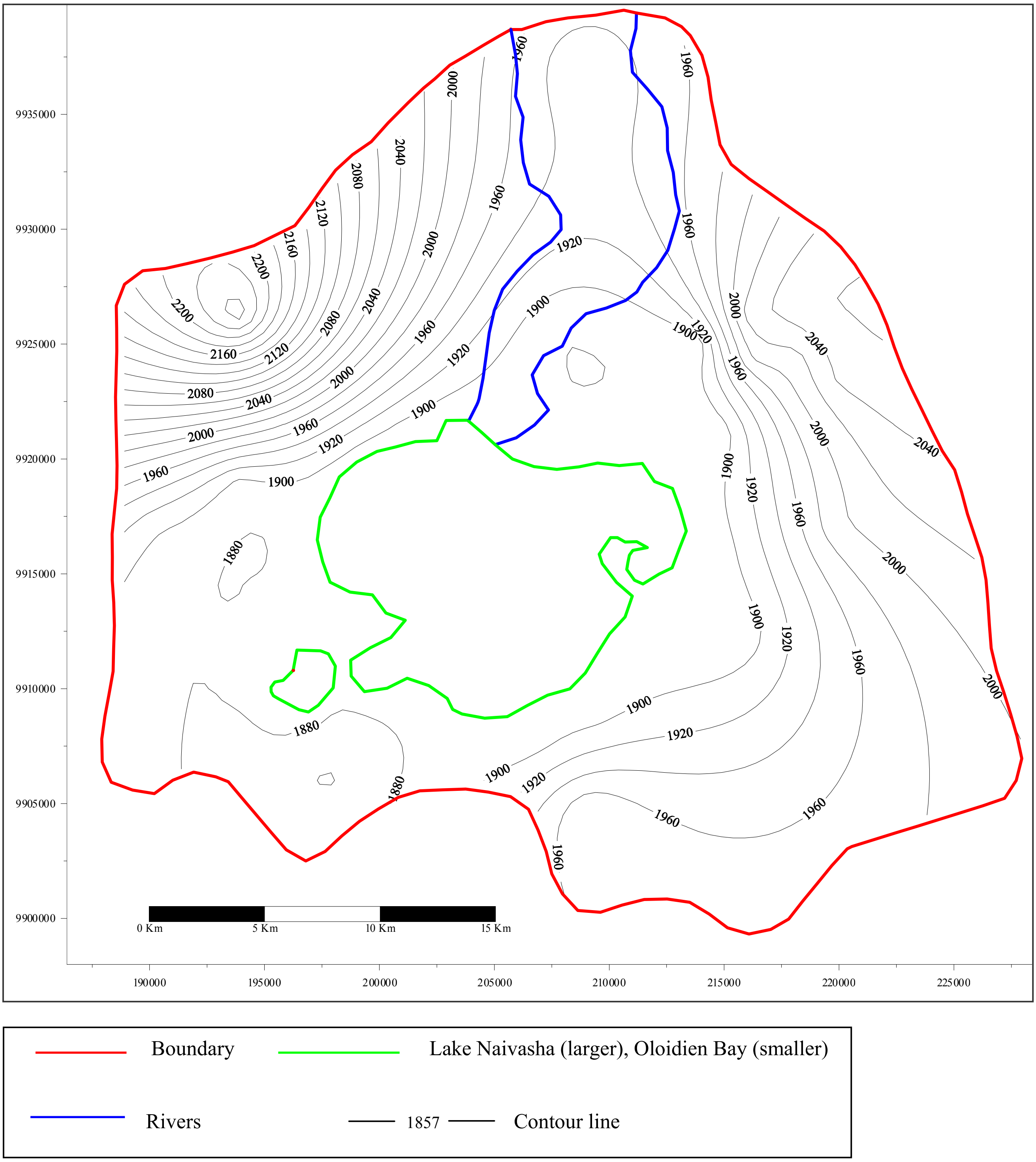


Figure 5.16. Contour map of observed heads, contours every 20 meters.

CHAPTER 6. TRANSIENT CONDITIONS

6.1.- INTRODUCTION

This chapter describes model results in transient conditions. Calibration for these transient runs has not been attempted. Characteristics related to geometry, space discretization and boundary conditions are described in the previous chapter.

The purpose of this transient modeling is to determine how water level has changed in the aquifer with time and if the model is capable of explaining these changes. In order to obtain the initial head distribution for these transient simulations the pumping wells in the well field were switched off and the water levels recorded to be used as initial heads in the simulations. The average water level, in the absence of abstraction, was about 5.8 meters higher compared to the observed levels in the field, with a range from 1.5 meters to 8.1 meters, Table 6.1. However, the same table shows that the average water level when abstraction occurred is only 0.17 meters higher. The former range of values is thought to be in the actual order of changes in the area, which have not been recorded, but seem to be in agreement with field observations and would be measured in the zone in the absence of groundwater exploitation. To perform this type of simulation storativity values are also necessary; the spatial distribution of this parameters is not well known. Different scenarios with variations in abstraction rates, storativity value and stress periods have been modeled. The interpretation of the results for every scenario is based on observations in the well field.

6.2.- CHARACTERISTICS OF THE FIRST SCENARIO

This scenario was made with the purpose of determining the time the levels in the well field took to reach those observed at present, this scenario was run with the abstraction wells working. The storativity value for this simulation was 0.15. One stress period of 10 years was used for the simulation, divided in 10 time steps of equal length.

Table 6.1. Comparison of observed and calculated heads with and with no wells.

Well	WL No Abst	WL Obs.	Diff (a)	WL Abst.	Diff (b)
C-10887	1889.60	1881.50	8.10	1882.50	1.00
Mil_Fact	1890.20	1882.60	7.60	1883.37	0.77
TOF_3	1891.00	1885.50	5.50	1885.15	-0.35
C-3675	1888.50	1881.10	7.40	1882.20	1.10
Manera	1887.20	1885.70	1.50	1883.90	-1.80
N-23	1888.60	1881.50	7.10	1882.96	1.46
La Belle	1888.60	1885.00	3.60	1884.02	-0.98
Average			5.83		0.17

(1) WL no Abst: Calculated water level with no abstraction

(2) WL Abst: Calculated water level with abstraction

(3) WL Obs: Observed water level

Diff (a): Difference between (1) and (3)

Diff (b): Difference between (2) and (3)

Although no accurate record exists of the lowering of the water table during the last ten years it seems that the model gives some insight into this phenomenon. A few field observations demonstrated that the water table in the area has dropped at least four meters in the last ten years. Temporal variations due to climatic conditions, which could also affect the distribution of heads in the zone, are not considered in the model. Local variations can also be expected due to changes in the well top elevations because of the lack of accuracy in the absolute surface elevation or because of the overexploitation of wells for irrigation purposes. To show the temporal variations of heads two wells were chosen and their levels plotted in graphs of heads versus time, Fig. 6.1 and 6.2. the initial water level in each well is: 1889.6 meters for well C-10887 and 1887.2 meters for well Manera respectively.

The figures below show how the levels have changed with time in the zone where the well field is located. At the same time they might be used to get a general idea of the water level drawdown during the last ten years.

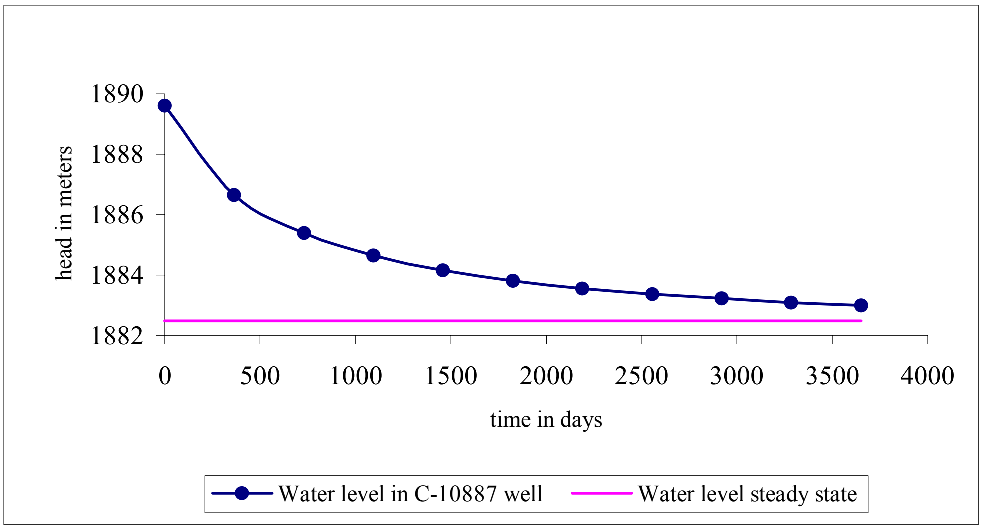


Figure 6.1 Variation of water level in well C-10887 after 10 years of pumping.

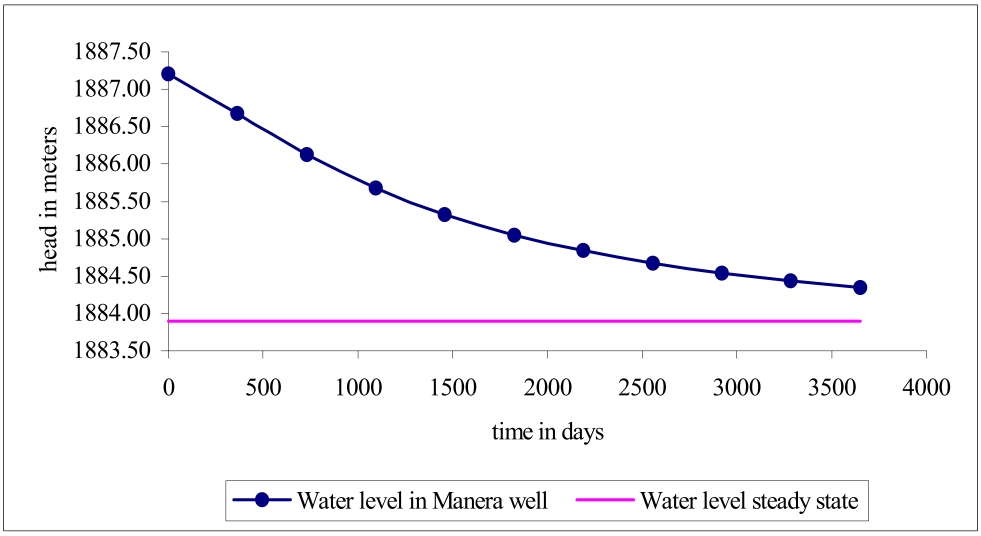


Figure 6.2. Variation of water level in well Manera after 10 years of pumping.

It can be seen that after a period of ten years the wells have reached the quasi-steady state condition once more, although the final levels are slightly different from those observed in the field. It seems that the model may reproduce the field situation.

6.3.- CHARACTERISTICS OF THE SECOND SCENARIO

The second scenario is made with the purpose of studying the recovery time of the levels in the aquifer, especially in the well field. Two stress periods have been used, every period of ten year, and every stress period was divided in ten time steps. During the first stress period abstraction was imposed to the model, in the second stress no abstraction was used. A storativity value of 0.15 was used. This scenario is characterized by a rising in water levels after the wells have been switched off. In this case what may be observed is the time the aquifer would need to recover to pre-abstraction levels. The results of the model are shown in Fig. 6.3 and 6.4.

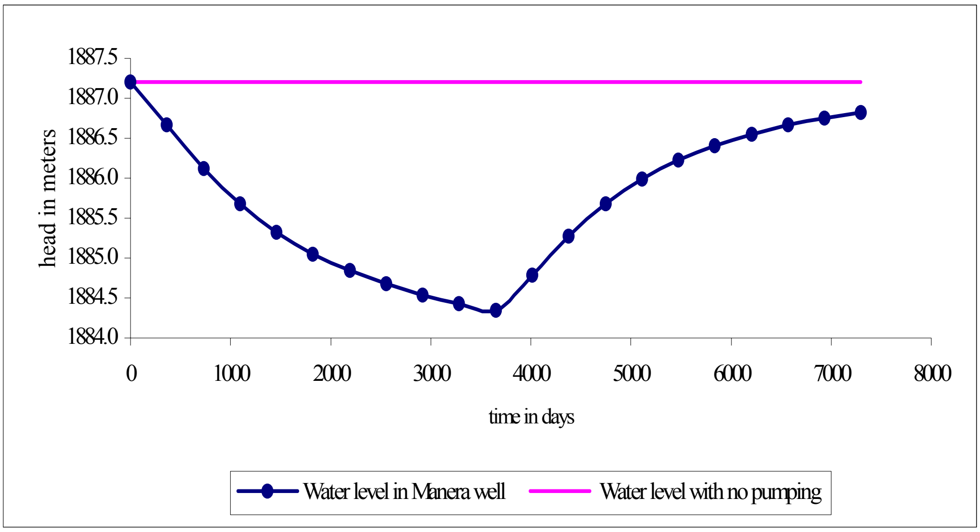


Figure 6.3 Recovery of water level in Manera well.

The water levels in the wells are recovering to their original levels after approximately the same time period it took to reach the quasi-stable condition under the influence of pumping wells.

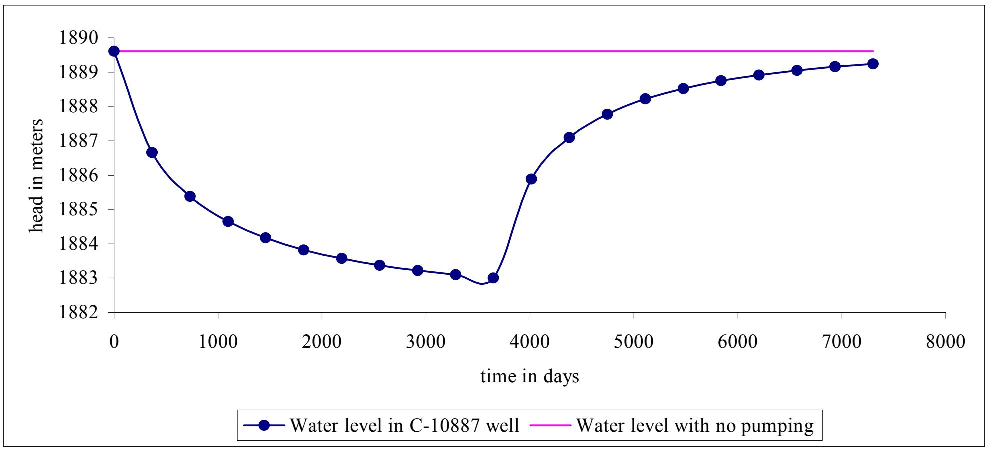


Figure 6.4 Recovery of water level in C-10887 well.

6.4.- CHARACTERISTICS OF THE THIRD SCENARIO

In this scenario changes were introduced respect to time and abstraction rates. The total simulation time in this case is twenty years, divided in to stress periods of ten years each. Every stress period has been divided in ten time steps of equal length. The abstraction rates in the second stress period have been duplicated. The storativity value was 0.15.

This scenario was created to determine if the water levels in the well field would drop below the observed levels in a period of ten years from now if the abstraction rates were increased to double the present ones. It can be seen that during the first stress period the water levels in the wells calculated by the model are still above the observed ones. However at the end of the second stress period this situation has changed, see Fig. 6.5 and 6.6, the model level is below the observed groundwater level.

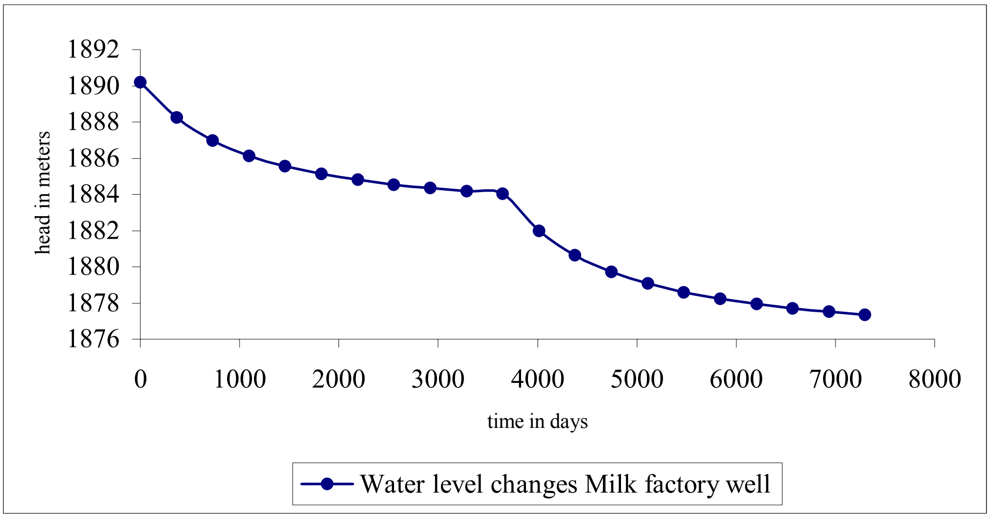


Figure 6.5. Changes in water level with changes in abstraction rates, *twenty years*.

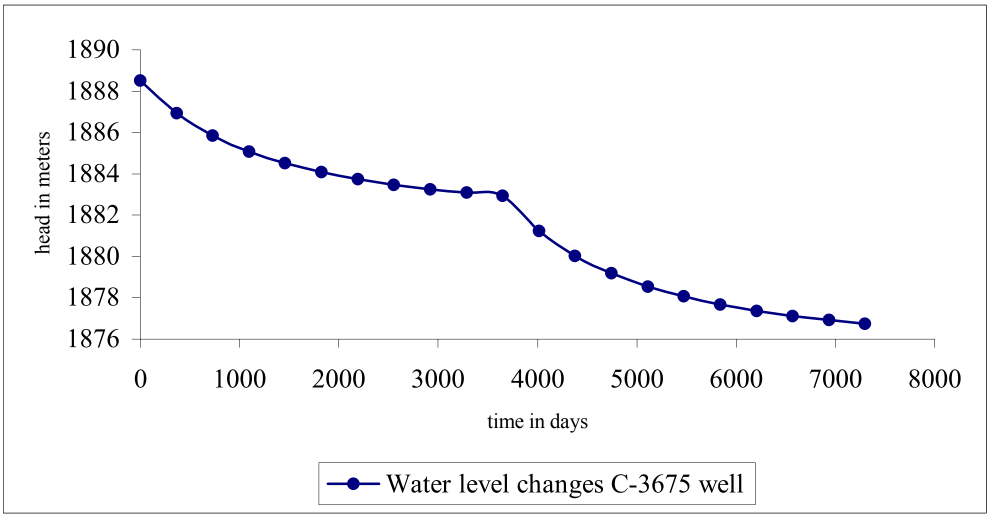


Figure 6.6. Changes in water level with changes in abstraction rates, *twenty years*.

6.5.- CHARACTERISTICS OF THE FOURTH SCENARIO

The objective of this scenario is to show how a decrease in storativity value from 0.15 to 0.01 may produce similar a result to that of pumping the wells for ten year. The time discretization for this scenario has been as follows: one stress period of five years with equal time steps, five time steps for the stress period. During the stress period the pumping rates used were the declared ones in a shorter time. With this combination of abstraction rates and storativity value the level in the wells would reach the present ones. This is a clear indication that storativity values play an important role in groundwater level changes in the zone, Fig. 6.7 and 6.8. The time required for the levels to be in the order of the ones observed in the field would be approximately 3.5 years for Milk factory and C-3675 wells respectively. At the end of the simulation it is observed that the level in the wells are under the influence of the lake level. A comparison between levels calculated by the model using storativities of 0.15 and 0.01, is also shown in the figures below.

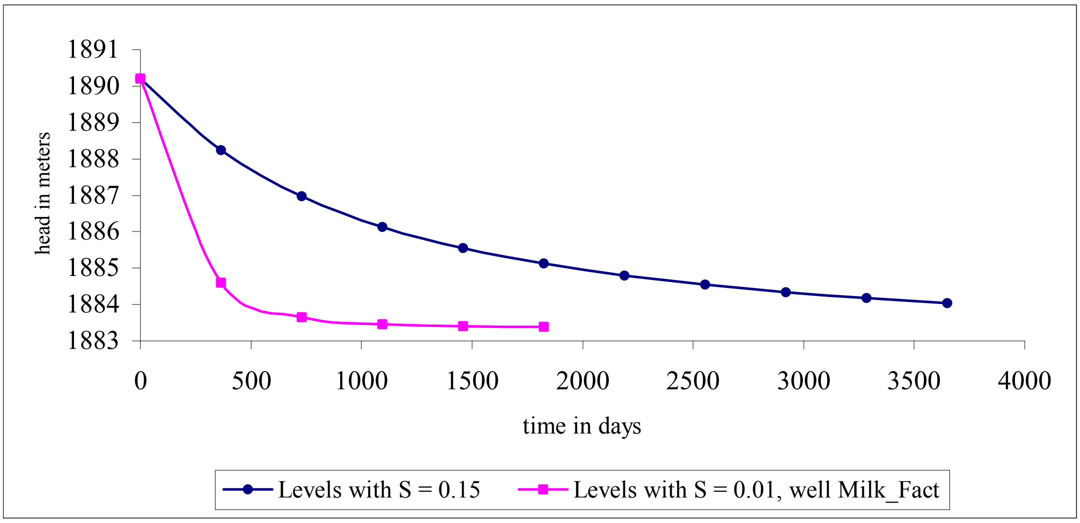


Figure 6.7. Variation of water levels with two different storativity values.

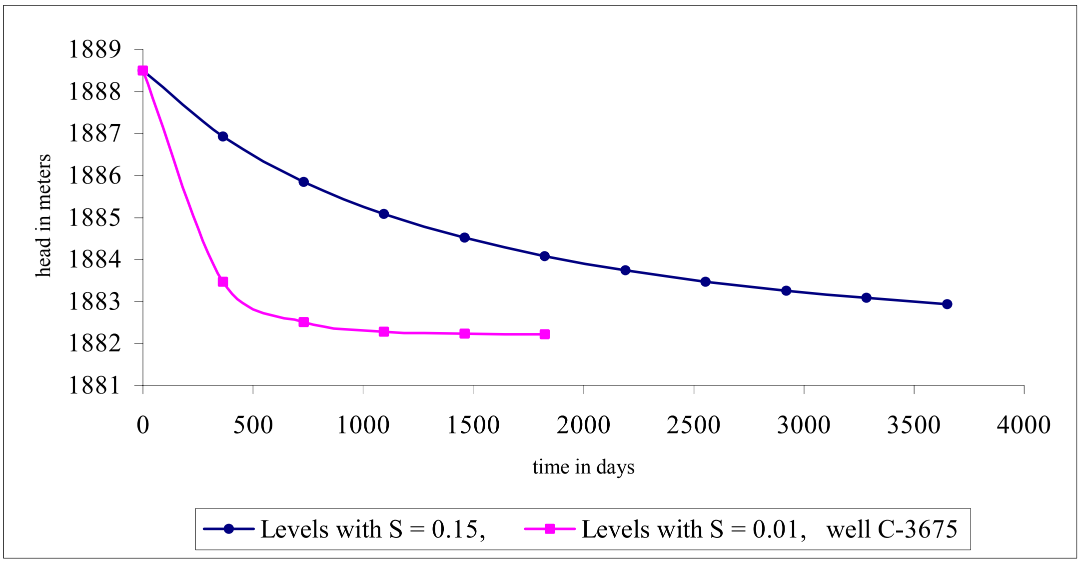


Figure 6.8. Variation of water levels with two different storativity values.

6.6.- CHARACTERISTICS OF THE FIFTH SCENARIO

The purpose of this scenario is to calculate water levels assuming a storativity value of 0.01 and an increased abstraction rate. In order to do it two stress periods of five years each have been used, five time steps of equal length were chosen. In the second stress period the abstraction rates have been increased to double. It is observed a pronounced lowering of the water table in the well field, Fig. 6.9 and 6.10.

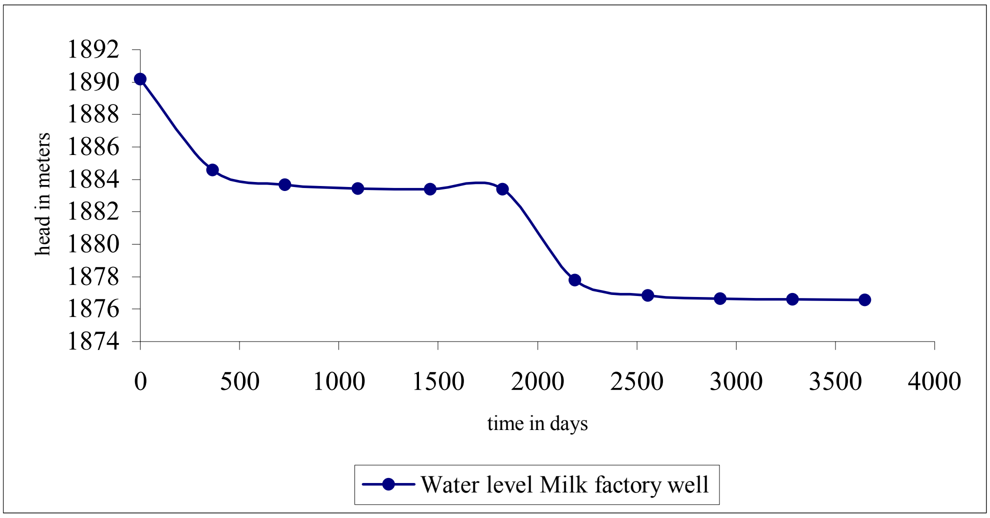


Figure 6.9. Water levels after modifying storativity and abstraction.

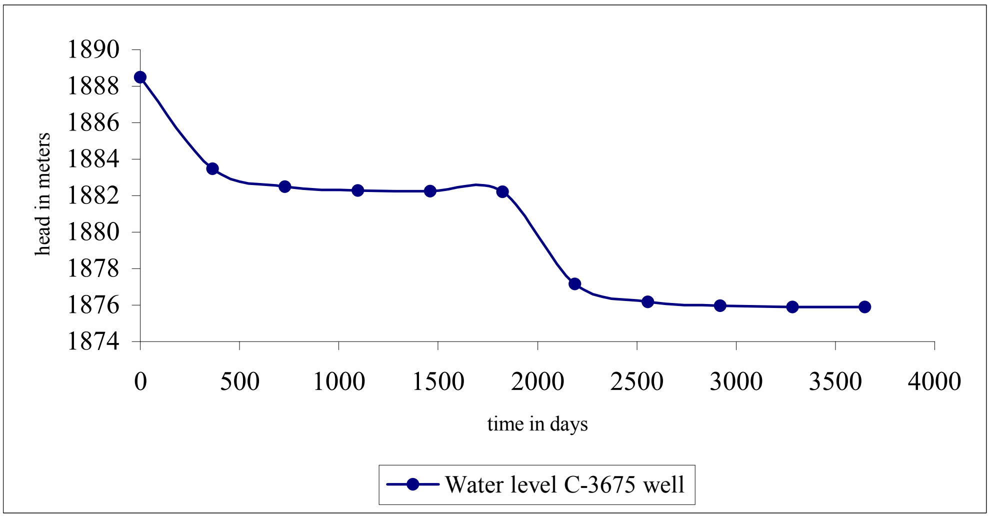


Figure 6.10. Water levels after modifying storativity and abstraction.

An analysis of the Fig. 6.9 and 6.10 shows how a change in storativity values can modify the results of the model. It is seen that immediately after the start of pumping there is a rapid drop of levels in both wells, approximately thousand two hundred days after the start of pumping the wells have reached a steady state. Another important conclusion drawn from the graph is that even if the storativity values are low the water to the wells will be drained from the lake and the rivers keeping in that way a permanent interaction among the lake, the rivers and the aquifer.

6.7.- REMARKS ABOUT TRANSIENT CONDITIONS

Different results have been shown with different model set up through the five scenarios. These results must be interpreted with caution because of the lack of model calibration in these transient conditions, the absence of data in some parts of the modeled area and because hydraulic parameters, e.g., storativity coefficient are not well known in the zone.

Another aspect to be taken in consideration is the variations of lake level in time. Although in all conditions the lake level was considered constant this may change in the future as it occurred in the past. Different model responses may be obtained if the lake level falls or rises. Hence the results of the models must be regarded as tentative because they represent general characteristics of the head distribution in the study area. However, the results may still give a general view of what the situation would be if the abstraction rates continue to increase.

Conditions for Jacob's straight line method

For single - well test in leaky aquifers

$$\frac{25 \cdot r_c^2}{K \cdot D} < t_p < \frac{c \cdot S}{20}$$

r_c Radius of the unscreened part of the well where the water level is changing

$K \cdot D$ Trasmissivity

c Resistance coeficient

t_p Pumping time

Conditions for Theis's recovery method

For single well - test in leaky aquifers

$$t_p > \frac{25 \cdot r_c^2}{K \cdot D}$$

$$t_r > \frac{25 \cdot r_c^2}{K \cdot D}$$

$$t_p + t_r \leq \frac{c \cdot S}{20}$$

t_r Recovery time

Well	Test	r_c (m)	T (m^2/day)	t_p or t_r (min)	Comparison
Marula	Pumping	0.75	168	$t_p > 121$	480 > 121
	Recovery		216	$t_r > 94$	600 > 94
KCC	Pumping	0.1	96	$t_p > 4$	20 > 4
	Recovery		84	$t_r > 4$	10 > 4
Three Ostrich Farm	Pumping	0.15	6600	t_p > less 1	
Manera	Pumping	0.12	816	t_p > less 1	
	Recovery		696	t_r > less 1	

In no case the term $\frac{c \cdot S}{20}$ could be calculated.