CONTROL OF GROUNDWATER BY UNDERGROUND DAMS

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ABSTRACT

CONTROL OF GROUNDWATER

BY UNDERGROUND DAMS

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In this study underground dams are briefly described and detailed information about the design and construction aspects is provided. Since the material, of which dam wall is composed, is the main variable influencing the groundwater behavior, various types of dam wall are discussed.

The use and usefulness of the underground dams as a means of sustainable development, and their performance in the management of groundwater resources are analyzed with the help of two example studies. In the first example a hypothetical idealized aquifer is considered, while in the second one, a real aquifer is selected.

For the performance evaluation, and for the analysis of the impact of the underground dams on the groundwater behavior, numerical simulation is opted. For that purpose, a well-known computer code, MODFLOW, A Modular Three-Dimensional Finite Difference Groundwater Flow Model of U.S. Geological Survey, (McDonald and Harbaugh, 1988) is used.

Keywords: Underground dam, Groundwater Storage, Numerical Simulation, MODFLOW

ÖZ

YERALTI SUYUNUN

YERALTI SUYU BARAJLARI İLE KONTROLÜ

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Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Kasım 2003, 80 Sayfa

Bu çalışmada yeraltı suyu barajları tanımlanmış, tasarım ve inşa konusunda detaylı bilgi sağlanmıştır. Baraj duvarını oluşturan madde yeraltı suyu davranışını etkilediği için farklı yeraltı suyu barajı tipleri tartışılmıştır.

Sürdürülebilir gelişme açısından yeraltı suyu barajı kullanımının yeraltı suyu kaynaklarının yönetimindeki performansı iki örnek çalışma ile analiz edilmiştir. Birinci örnekte hipotetik ideal bir akifer, ikincide ise gerçek bir akifer seçilmiştir.

Yeraltı suyu barajlarının yeraltı suyu davranışı üzerindeki performans değerlendirmesini yapmak ve etkisini analiz etmek için sayısal simülasyon yöntemi seçilmiştir. Bu amaçla iyi bilinen bir bilgisayar programı olan

MODFLOW, A.B.D. Jeolojik Araştırma Kurumunun Modüler Sonlu Farklar Yeraltı Suyu Modeli, (McDonald ve Harbaugh, 1988) kullanılmıştır.

Anahtar kelimeler: Yeraltı Suyu Barajı, Yeraltı Suyu Depolaması,
Sayısal Simülasyon, MODFLOW

TO MY MOTHER,FATHER AND BROTHER

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After completion of my undergraduate degree, it was an itch for me to make graduate study as an awakening, so I tried to put maximum effort in this work.

Patience is the keyword for this thesis. It may be as important as making the study.

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TABLE OF CONTENTS

ABSTRACT iii

ÖZ v

ACKNOWLEDGMENTS viii

TABLE OF CONTENTS ix

LIST OF TABLES xi

LIST OF FIGURES xii

LIST OF SYMBOLS xiv

CHAPTER

1. INTRODUCTION AND LITERATURE REVIEW………………… 1

   1.1 DESCRIPTION OF THE PROBLEM………………………… 1

   1.2 OBJECTIVES………………………………………………….. 3

   1.3 SCOPE OF THE THESIS………………………….................. 3

   1.4 LITERATURE REVIEW……………………………………… 4

      1.4.1 CASE HISTORIES…………………………………… 5

2. A REVIEW ON GROUNDWATER DAMS.. 8

   2.1 SUBSURFACE DAMS.. 8

   2.2 SAND STORAGE DAMS…………………………………... 14

   2.3 CHARACTERISTICS OF GROUNDWATER DAMS 20

3. THEORETICAL BACKGROUND 22

   3.1 MATHEMATICAL MODEL.. 22

   3.2 BOUNDARY AND INITIAL CONDITIONS............................ 28

   3.3 NUMERICAL SOLUTION... 29

4. HYPOTHETICAL CASE STUDY... 31

   4.1 Case 1-a 32

   4.2 Case 1-b ....…... 35

4.3 Case 1-c 37
4.4 Case 1-d 40
4.5 Unsteady Solution of Case 1.. 43
4.5.1 Duration in which steady state is reached for Q=7005 m^3/day 44
4.5.2 Duration in which steady state is reached for Q=0........... 46
4.5.3 Limit Extraction Duration when R=0.003 m/day and Q=7005 m^3/day.. 48
4.5.4 Limit discharge for 90 days extraction when R=0.003 m/day.. 48
5. REAL CASE STUDY 49
5.1 Description of the study area.. 49
5.2 Case 2-a 55
5.3 Case 2-b 58
5.4 Case 2-c 63
5.5 Case 2-d 67
5.6 Unsteady Solution of Case 2... 73
5.6.1 Duration in which steady state is reached for Q=900 m^3/day in Case 2-b... 73
5.6.2 Duration in which steady state is reached for Q=1200 m^3/day.. 73
5.6.3 Duration in which steady state is reached for Q=0(preceded by Q=1200 m^3/day).................................. 74
6. SUMMARY AND CONCLUSION.. 75
REFERENCES 77

LIST OF TABLES

TABLE

2.1 Average dam heights.. 13

4.1 Content of Cases 1-a, 1-b, 1-c, and 1-d... 31

4.2 Inputs of Case 1-a.. 33

4.3 Inputs of Case 1-b.. 35

4.4 Inputs of Case 1-c 37

4.5 Inputs of Case 1-d.. 40

5.1 Content of Cases 2-a, 2-b, 2-c, and 2-d.. 50

5.2 Inputs of Case 2-a.. 55

5.3 Inputs of Case 2-b.. 58

5.4 Inputs of Case 2-c.. 63

5.5 Inputs of Case 2-d.. 67

LIST OF FIGURES

FIGURE

2.1	Typical sub-surface dam..	8
2.2	Effect of subsurface dam on groundwater flow................................	9
2.3	Clay dike..	10
2.4	Concrete dam...	11
2.5	Stone masonry dam..	11
2.6	Reinforced concrete dam..	11
2.7	Plastic or tarred-felt sheets..	12
2.8	Injection screen..	13
2.9	Typical sand storage dam..	15
2.10	Concrete sand dam..	18
2.11	Stone masonry sand dam..	18
2.12	Gabion sand dam with clay cover...	19
2.13	Gabion sand dam with clay core...	19
3.1	Flow through the control volume...	24
4.1	Plan view of idealized rectangular aquifer with dam wall and wells..	32
4.2	Case 1-a, cross-section of water table without wells without dam.....	34
4.3	Case 1-b, cross-section of water table with two discharging wells Q_1=Q_2=5000 m^3/day………………………………………………	36
4.4	Case 1-c, cross-section of water table with dam wall………………	38
4.5	Case 1-c, cross-section of water table with dam wall relocated seaward at x=500 m……………………………………………….	39
4.6	Case 1-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=5000 m^3/day……………………………	41
4.7	Case 1-d, cross-section of water table with dam wall and two	42

discharging wells Q_1=Q_2=7005 m^3/day……………………………
4.8 Head vs. time for control point W_1 (2300,500)................................ 45
4.9 Head vs. time for control point P (2700,500)................................... 45
4.10 Head vs. time for control point W_2 (3000,500)................................ 45
4.11 Head vs. time for control point W_1 (2300,500)................................ 46
4.12 Head vs. time for control point P (2700,500)................................... 47
4.13 Head vs. time for control point W_2 (3000,500)................................ 47
5.1 The study area and its location in Turkey.. 51
5.2 Map of the aquifer and the potential dam construction site............... 52
5.3 Finite-difference grid used to model study area................................ 53
5.4 Approximation of aquifer boundaries.. 54
5.5 Case 2-a, top view of water table without wells without dam............ 56
5.6 Case 2-a, cross-section of water table without wells without dam..... 57
5.7 Typical cross-section of a coastal aquifer.. 59
5.8 Case 2-b, top view of water table with two discharging wells Q_1=Q_2=900 m^3/day………………………………………………….. 61
5.9 Case 2-b, cross-section of water table with two discharging wells Q_1=Q_2=900 m^3/day………………………………………………….. 62
5.10 Case 2-c, top view of water table with dam wall................................ 64
5.11 Case 2-c, cross-section of water table with dam wall......................... 66
5.12 Case 2-d, top view of water table with dam wall and two discharging wells Q_1=Q_2=4302 m^3/day 68
5.13 Case 2-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=4302 m^3/day…………………………… 69
5.14 Case 2-d, top view of water table with dam wall and two discharging wells Q_1=Q_2=1200 m^3/day…………………………… 71
5.15 Case 2-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=1200 m^3/day……………………………... 72

LIST OF SYMBOLS

b : aquifer thickness

b_1 : thickness of inflow vertical leakage boundary

b_2 : thickness of outflow vertical leakage boundary

C : dimensionless constant

d : particle size

h : piezometric head in the main aquifer

h_1 : piezometric head of inflow vertical leakage boundary

h_2 : piezometric head of outflow vertical leakage boundary

h_{gw} : groundwater level above impervious bottom

h_{sea} : mean sea level

K : hydraulic conductivity

K_1 : hydraulic conductivity of the inflow vertical leakage boundary

K_2 : hydraulic conductivity of the outflow vertical leakage boundary

K_s : hydraulic conductivity of soil

K_w : hydraulic conductivity of dam wall

K_{xx} : hydraulic conductivity in x axis

K_{yy} : hydraulic conductivity in y axis

L : length of aquifer along the flow direction

n_{ef} : effective porosity

n_t : total porosity

P : rate of volume of water consumed per unit horizontal area

q : specific discharge

q_x : specific discharge in x axis

q_y : specific discharge in y axis

q_z : specific discharge in z axis

q_{z1} : leakage into the control volume in z direction

q_{z2} : leakage out of the control volume in z direction

Q : discharge

Q_1 : discharge from well W_1

Q_2 : discharge from well W_2

R : recharge

R' : rate of volume of water produced per unit horizontal area

S : specific storage of the porous material

S_s : specific storage

S_y : specific yield

w : width of the aquifer

t : thickness of the dam wall

x : coordinate in x principal axis

y : coordinate in y principal axis

z : coordinate in z principal axis

ρ : density

γ : specific weight

μ : dynamic viscosity

η : bottom elevation

ΔA : horizontal area

Δh : headloss

ΔL : length in flow direction

ΔV_w : volume of water released from or added to storage

Δx : length in x axis of the control volume

$\Delta' x$: length of unit grid in x-axis

Δy : width in y axis of the control volume

$\Delta' y$: length of unit grid in y axis

CHAPTER 1

INTRODUCTION AND LITERATURE REVIEW

1.1 DESCRIPTION OF THE PROBLEM

Turkey is not a water rich country, and it is estimated that there will be only 1100 m^3 available water per capita annually by the year 2050 (Turfan, 2001). The sustainable development of water resources will be one of the key issues in the future. Underground dam will be one of the alternative ways of achieving the sustainable development.

In the hydrological cycle groundwater occurs whenever surface water occupies and saturates the pores or interstices of the rocks and soils beneath the earth's surface. The geological formations that are capable of storing and transmitting the subsurface water are known as aquifers.

An underground dam is any structure designed to intercept or obstruct the natural flow of groundwater through an aquifer and provide storage for water underground. Underground dams are intended to be used in regions with arid or tropical climates. Underground dams are used as small-scale water supply. They cannot be looked upon as universal method for water supply; however they can be treated as alternative solution when conventional methods are not suitable or applicable. By using underground dams for storing water, instead of conventional methods, the disadvantages of conventional water storage, such as high evaporation rates, pollution, siltation, health hazards may be avoided (Nilsson, 1988).

The proper siting of underground dams necessitates a thorough knowledge of hydrogeological conditions in the actual area. It is necessary to make generalizations and to use simple geophysical methods. Therefore in a study about underground dams it is important to reach simple and useful solutions (Hansson and Nilsson, 1986).

In this study, since groundwater dams are not used widespread and there exists few materials about the subject, information about groundwater dams especially about how to design and build the dam and other necessary information will be given. After necessary illustration is made about groundwater dams, two cases will be handled using MODFLOW (McDonald and Harbaugh, 1988). First case will be about a hypothetical idealized aquifer and second case is planned to be a real or almost real aquifer. The separate effects of factors such as wells and dam wall and their combined effects are planned to be discussed. The effect of building a groundwater dam on the variation of water table elevation is to be analyzed using case studies.

1.2 OBJECTIVES

The objectives of this study can be stated as follows:

- Presenting brief information about underground dams in many aspects including design and construction of different types

- Demonstration using MODFLOW including case studies

- Making comparisons among case studies and thereby reaching useful solutions about underground dams

1.3 SCOPE OF THE THESIS

This thesis is composed of six chapters. The first chapter covers the description of the problem, the objectives and literature review including case histories about the subject.

In the second chapter the necessary information about groundwater dams including design and construction and characteristics of groundwater dams are given.

In the third chapter theoretical background of the subject including the governing equation, its derivation, information for numerical solution and its tool MODFLOW are provided.

Fourth chapter contains simulation of a hypothetical aquifer, whereas in the fifth chapter simulation is made on a real or almost real aquifer. The different scenarios modeled using MODFLOW in the fourth and fifth chapters will give us the opportunity to make comparisons about the results and further recommendations about the matter in Chapter 6.

1.4 LITERATURE REVIEW

Damming groundwater for conservation purposes is certainly not a new concept. Groundwater dams were constructed on Sardinia in Roman times and damming of ground water was practiced by ancient civilizations in North Africa. More recently, various small-scale groundwater damming techniques have been developed and applied in many parts of the world, notably in South and East Africa and in India (Hansson and Nilsson, 1986).

Groundwater dams are looked upon as alternative means of water supply and groundwater damming is not a universally applicable method for water supply. The techniques used in groundwater dam applications are very old. However in the past decades there have been systematic studies. Injected cutoffs have been used to arrest the flow in large or deep-seated aquifers in North Africa and Japan (BCEOM, 1978; Matsuo, 1977) and to protect fresh water from pollution in Europe and the USA (Nilsson, 1988). Also there is another study on sub-surface dams by Ahnfors (1980) in India related with proper design and construction of the dam.

Another type of groundwater dam is a sand storage dam. The first recorded attempt was in 1907 in Namibia (Wipplinger, 1958). Wipplinger (1958) developed it further in the Hoanib River and proposed his ‘sand dam’. Sand storage dams are built in stages and they are costly in comparison to construction of full height directly. The economical aspects of sand storage dams for water conservation have been discussed by Burger (1970). Brief information about the design of sand storage dams is given in Beaumont and Kluger (1973). Design instructions of a very practical nature are given in Nissen-Petersen (1982). The book written by Nilsson (1988), called ‘Groundwater Dams for Small Scale Water Supply’ presents the results of a literature study combined with studies in Africa and India.

1.4.1 CASE HISTORIES

The most comprehensive information about groundwater dams is given in Nilsson (1988), which consists of most detailed concept including literature review. As it is mentioned in Nilsson (1988); there are several groundwater dams in the world including Europe, Africa, Asia and America.

In Europe there are several schemes in Germany, France and Italy where sub-surface dams have been used mostly to raise groundwater levels (BCEOM, 1978). Sub-surface dams serving the purpose of containing water in existing aquifers have been constructed in Greece (Garagunis, 1981) and sub-surface dams mainly functioning as protection against sea water intrusion into fresh water aquifers have been proposed in Yugoslavia (Pavlin, 1973) and Greece (Garagunis, 1981).

Africa is the continent where groundwater dams are notably used. Several very large sub-surface dams exist in northwestern Africa, notably in Morocco and Algeria. Groundwater dams are quite frequently used for water supply in East Africa. There exist sand storage dams in Machakos Region, Kenya and sub-surface dams close to Dodoma, Tanzania (Nilsson, 1988).

In South America, Brazil is another country where groundwater dams are frequently used. Moreover there is a long tradition of building groundwater dams in the arid southwestern parts of the United States and northern Mexico. Sub-surface dams called ‘tapoons’ have been constructed in sandy riverbeds in Arizona (Lowdermilk, 1953).

In Asia groundwater dams are used in India. Two sub-surface dams have been constructed in Kerala, South India; one by a private farmer and the other by the Central Ground Water Board of India. The private dam was constructed in Ottapalam in 1962-1964. The other dam built by government was completed in

1979. This dam was constructed across a narrow valley and has a catchment area of about 20 ha. The total length of the dam is about 160 m and the crest was kept 1 m below the groundwater level to avoid water logging in the upstream area. The main part of the dam is made up of brick wall but there are sections consisting of tarred felt and plastic sheets. The dam took three months to complete at a total cost of 7500 dollars. One third of it was for earthwork and the rest was for equipment and construction materials. The storage volume was estimated at 15000 cubic meters. There are also other sub-surface dams built in India, namely in Ootacamund in 1981 and by the Minor Irrigation Department in sandy riverbeds in Andhra Pradesh (Hansson and Nilsson, 1986).

There are other examples of groundwater dams in other parts of Asia. Subsurface dams have been proposed for construction in Thailand and at several sites in Japan by Matsuo (1975 and 1977), who also reports of a sub-surface dam constructed by means of jet injection on the Island of Kabain western Japan.

During the last few years, considerable attention has been given to the use of groundwater dams as a method of overcoming water shortage in regions with arid and tropical climates. This thesis is an attempt to make a systematic study on groundwater dams so as to make new contributions to the subject.

CHAPTER 2

A REVIEW ON GROUNDWATER DAMS

Groundwater dams are structures that intercept or obstruct the natural flow of groundwater and provide storage for water underground. There are basically two different types of groundwater dams, namely subsurface dams and sand storage dams. A subsurface dam is constructed below ground level and arrests the flow of a natural aquifer, whereas a sand storage dam impounds water in sediments caused to accumulate by the dam itself (Hansson and Nilsson, 1986).

2. 1 SUBSURFACE DAMS

The cross-section of a typical subsurface dam is given in Figure 2.1.

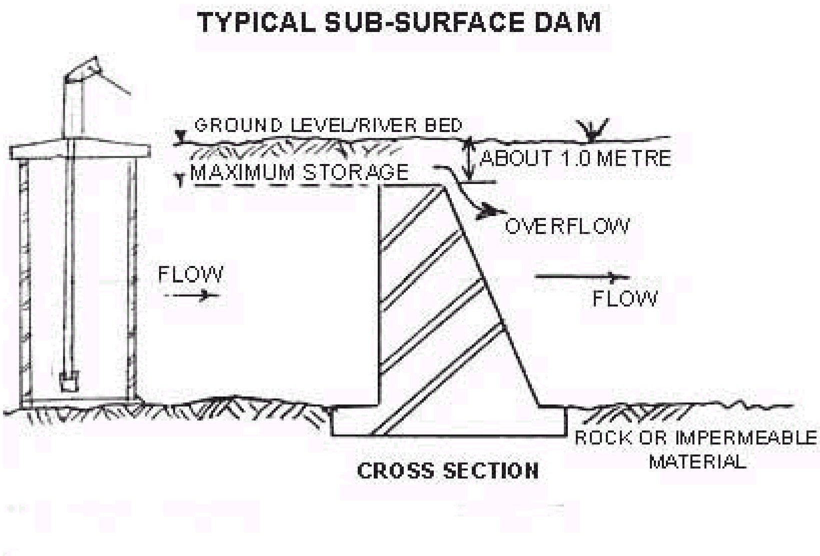


Figure 2.1 Typical sub-surface dam

The actual storage volumes of sub-surface dams range from a few hundred to several million m^3 due to differences in design. The effect of subsurface dam on groundwater flow is given in Figure 2.2. The design procedure of a sub-surface dam is as follows: a trench is dug across the suitable part of the valley, which reaches down to bedrock. In the trench an impermeable wall is constructed and the trench is refilled with excavated material. The excavated depths are generally not more than 3-6 m (Nilsson, 1988).

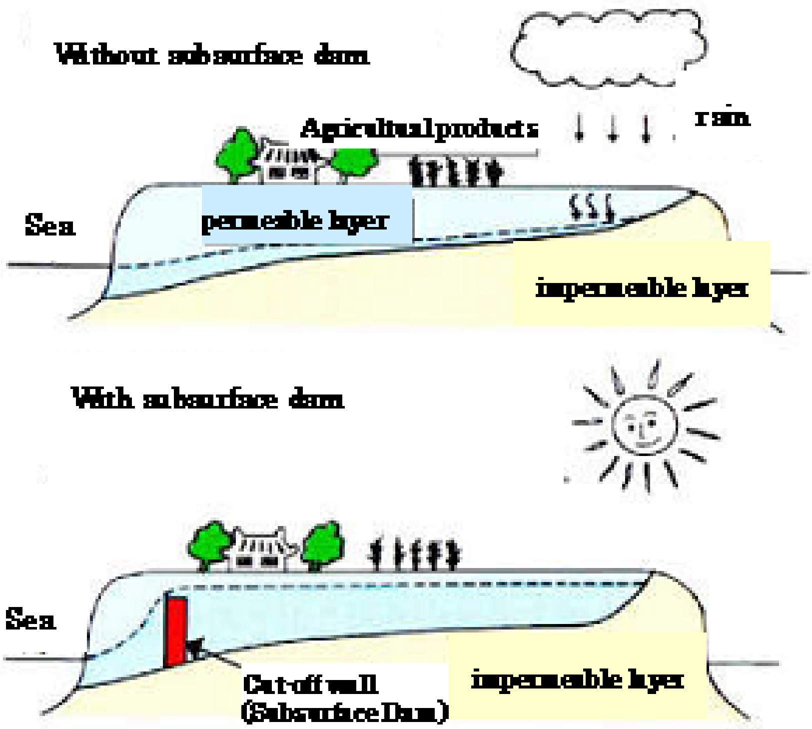


Figure 2.2 Effect of subsurface dam on groundwater flow

Sub-surface dams are generally built at the end of the dry season when there is minimum water in the aquifer. The existing flow has to be pumped out during the construction.

Various construction materials have been used for the impermeable screen such as clay, concrete, stone masonry, reinforced concrete, brick, plastic, tarred-felt, sheets of steel, corrugated iron or PVC (Nilsson, 1988).

The clay dike shown in Figure 2.3 is suitable for small schemes in highly permeable aquifers of limited depth, such as sandy riverbeds. The clayey soils are generally available close-by and with low cost can be mined and transported to the site. The clay layers should be compacted. This is usually done by hand using wooden blocks. The risk of erosion damage can be avoided by protecting the dike with plastic sheets.

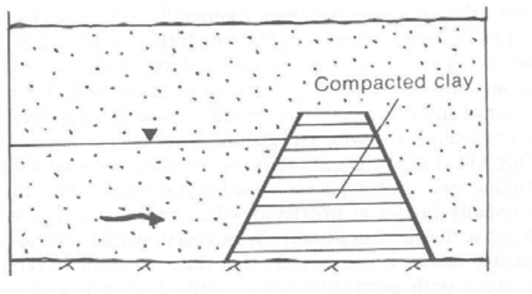


Figure 2.3 Clay dike (Nilsson, 1988)

A concrete dam shown in Figure 2.4 is an alternative involving rather more advanced engineering for which skilled labor is needed. It necessitates the use of formwork and the availability of sand and gravel. The stone masonry dam given in Figure 2.5 has the same property with the concrete dam in labor aspect. The advantage of using reinforced concrete is that very little material, namely steel rods or wire mesh is needed to achieve a very strong wall. But these materials are at reasonable cost and formwork has to be used. The reinforced concrete dam in Figure 2.6 has to be anchored to a solid reservoir bottom.

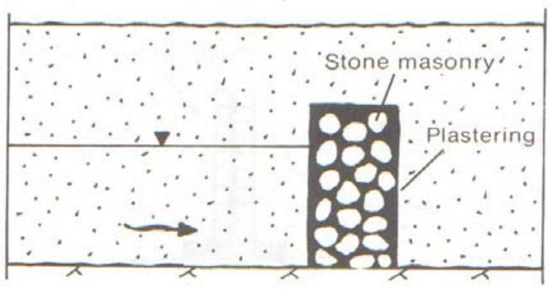


Figure 2.4 Concrete dam (Nilsson, 1988)

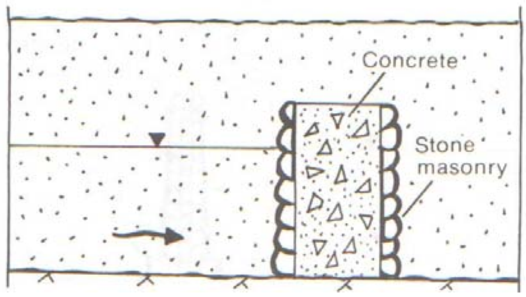


Figure 2.5 Stone masonry dam (Nilsson, 1988)

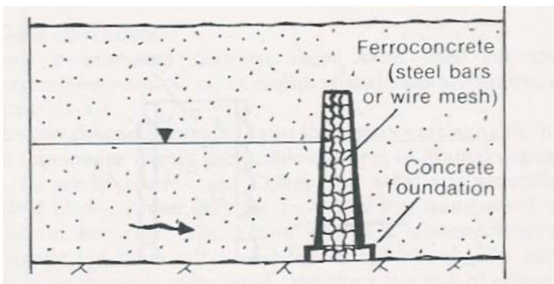


Figure 2.6 Reinforced concrete dam (Nilsson, 1988)

Bricks are generally available or may be manufactured from local clay. It is very simple to build a brick wall and make it watertight. The disadvantages of brick wall are the relatively high cost of bricks and stability problems.

Thin sheets of impermeable materials such as tarred felt given in Figure 2.7 or polyethylene is the least expensive choice as far as material cost is concerned. The mounting of sheets to wooden frames and the erection process is rather complicated. A minor rip, that can occur during the erection as well as refilling the trench, will cause leakage losses. If small sheets are joined, to overcome this problem, then the joints may become weak points that may break due to the water pressure. There are also doubts whether plastic material will withstand high groundwater temperatures and the activities of microorganisms in the soil.

Sheets of steel, corrugated iron or PVC can be used to build up an impermeable wall. In construction stages such as the welding of steel sheets skilled labor is needed. However the result is a sturdy and impermeable structure.

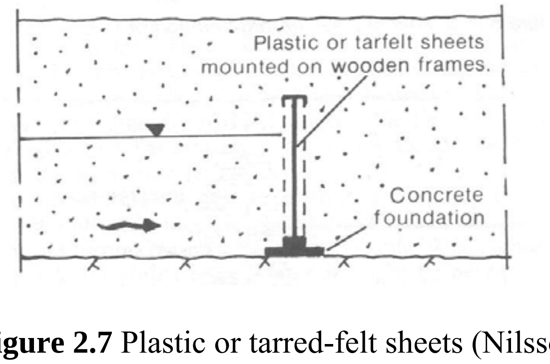


Figure 2.7 Plastic or tarred-felt sheets (Nilsson, 1988)

Also injection screens (Figure 2.8) have been used to arrest the flow in large or deep-seated aquifers in North Africa and Japan; and to protect fresh water from pollution in Europe and USA (Nilsson, 1988). There is also one example in Turkey in Çeşme to prevent seawater intrusion to fresh water aquifers (Sargın, 2003 and Kocabaş, 2003).

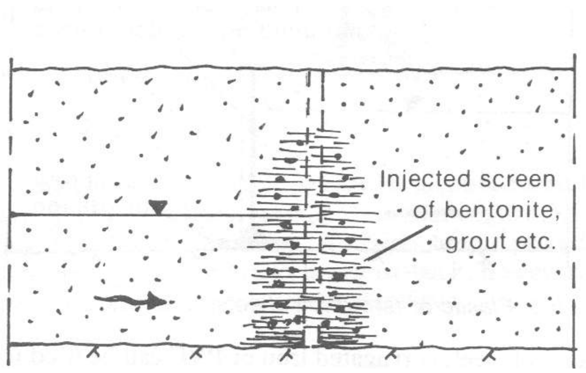


Figure 2.8 Injection screen (Nilsson, 1988)

The average heights of some subsurface dams are given in Table 2.1 (Nilsson 1988).

Table 2.1 Average dam heights

Dam type	Average height(m)
Injection screen	10
Brick wall	6
Concrete dam	6
Stone masonry dam	5
Reinforced concrete dam	4
Clay dike	3
Plastic sheets	2

The crest of a subsurface dam is usually kept at some depth from the surface to avoid water logging in the upstream area and partly to avoid erosion damage to the dam.

The well through which water is extracted may be placed in the reservoir or, for erosion protection reasons, in the riverbank. When aquifers with low permeability are dammed, construction of a series of large-diameter wells or collection chambers may be necessary. By this way a sufficient storage volume for pumping can be created.

It is possible to extract water from the reservoir by gravity if the community to be served by the scheme is located downstream of the dam site. This is managed if the topographical conditions are favorable. By using gravity extraction, problems with pump installation, operation and maintenance are avoided.

2. 2 SAND STORAGE DAMS

The origin of the sand storage dam is unknown but it may stem from the occasion that someone observed that a steady water supply of water could be obtained from an open-storage dam, which had been filled over years by coarse sediment. As stated in the report prepared by the ministry of agriculture and water of Saudi Arabia; an ingenious idea has been incorporated in dam provision in the Namibia desert (Namibia): ‘sand dams’ (surface dams with sand filled reservoirs) have been used to minimize evaporation losses, since 1907 (Wipplinger, 1958). This sand dam term is actually sand storage dam. A sand storage dam impounds water in sediments caused to accumulate by the dam itself. The height of a sand-storage dam is typically 1-4 meters. (Nilsson, 1988)

A sand storage dam (Figure 2.9) is built by raising the dam wall in stages.

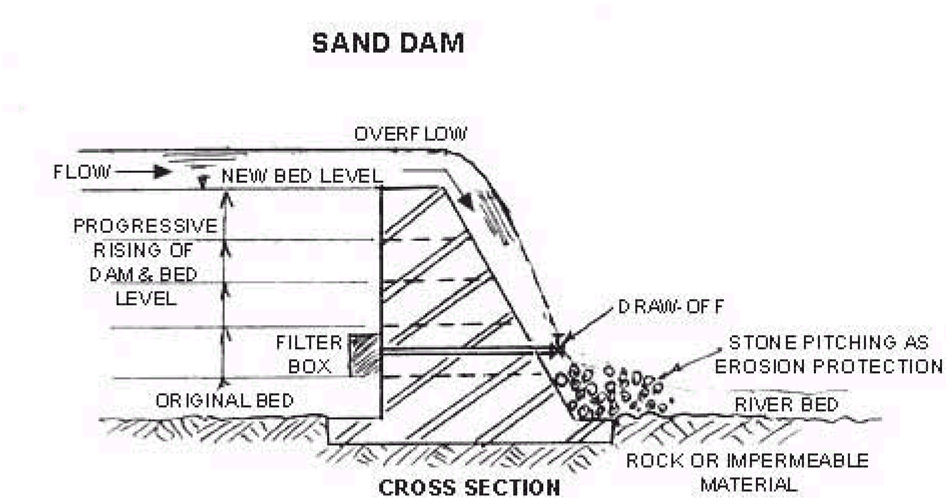


Figure 2.9 Typical sand storage dam

Ideally the clean coarse fraction of the sediment load transported downstream by successive floods should be trapped in the dam basin and the finer material washed over the dam wall (Beamont and Kluger, 1973). The resulting sand media produced will absorb floodwater, which can be withdrawn by boreholes and drains. The evaporation loss is considered negligible. This low evaporation loss term is important for semi-arid regions such as South West Africa. In that region, the infrequent rainfalls produce surface run-off, which lasts only from a few hours to a few days, and during the remainder of the year the riverbed may remain dry. Thus the concept of storing water in a sand media offers a tremendous potential.

Unfortunately, however, the deposition of fine material cannot be entirely prevented and this causes relatively impermeable layers. These layers have an adverse effect upon the efficiency of a sand storage dam.

Due to the effects of molecular attraction, capillarity and evaporation the volume of water that can be stored in a sand storage dam does not represent the actual volume of water that can be withdrawn from the sand media.

Molecular attraction causes a thin layer of water to adhere to a grain of sand. However in comparison to the total volume this volume can be considered as negligible.

Reduced grain size and size distribution increase the water storage capacity but at the same time also increase the volume of water held by molecular attraction and lost to the atmosphere by the combined effects of capillarity and evaporation. The increased particle size allows for more rapid infiltration of floodwaters, re-charging of the dam and greater rate of withdrawal of stored water during the dry seasons.

The actual design of a sand storage dam represents a compromise between allowing only coarse material in the dam basin, which entails many small increases in height of the dam wall over along period of time and permitting a certain amount of fine material to settle which enables the dam wall to be built up to the final height in larger stages over a shorter period.

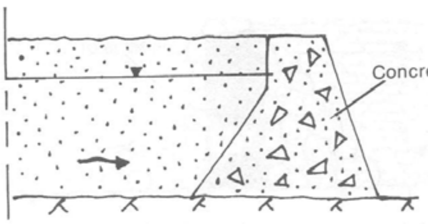
The design of a sand storage dam can be considered in two parts.

The first part is concerned with determining the overall size of a sand storage dam necessary to supply a given quantity of water. The ways in which particle shape, size, size distribution and type of material can be combined varies in a particular sediment deposit. Therefore tests are being conducted on site and in laboratory to evaluate water storage capacity and water movement in different sand media. By the help of these tests the characteristic storage capacity and yield of various sediments can be determined. Thereby the designer can determine the necessary size of dam.

The second part of the design concerns the flow control in the dam basin, which influences formation of sediment deposits. A sand storage dam is built in stages but the method of constructing the dam by adding a new stage each season means that costs will be higher. To overcome this problem, techniques such as siphons or provision of openings in the dam wall have been used. By using a siphon, water is discharged over the dam and flow velocity in the reservoir is maintained in a sufficiently high level. This method has been found to be technically inefficient and it is very costly (Burger and Beaumont, 1970). The other method is leaving a notch that allows the settling of the sediments only up to a certain height. The notch is then filled in before the next rainy season and the reservoir is allowed to be filled completely.

Some types of sand storage dams can be stated namely as; concrete sand storage dam, stone masonry sand storage dam, gabion sand storage dam with clay cover, gabion sand storage dam with clay core, stone-fill concrete sand storage dam and stone sand storage dam.

Concrete (Figure 2.10) and stone masonry dams (Figure 2.11) are the most common. They are sufficiently massive to take up the pressure from sand and water stored in the reservoir.



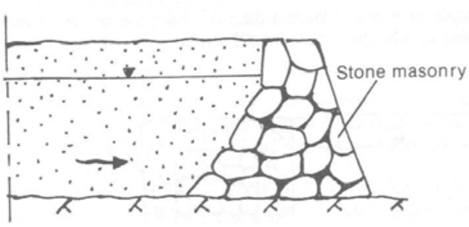


Figure 2.10 Concrete sand dam. (Nilsson, 1988)

Figure 2.11 Stone masonry sand dam (Nilsson,1988)

In gabion dams with clay cover (Figure 2.12) and gabion dams with clay core (Figure 2.13) the weight of the dam is made up of stone gabions or large blocks, which are sufficient to withstand the pressure.

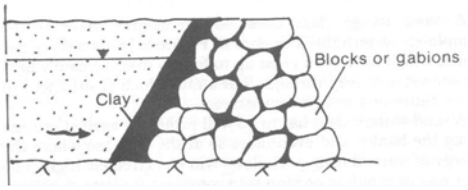


Figure 2.12 Gabion sand dam

with clay cover

(Nilsson, 1988)

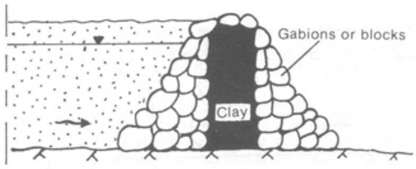


Figure 2.13 Gabion sand dam

with clay core

(Nilsson, 1988)

In stone-fill concrete dam the main dam body is made up of stones, which are covered by concrete walls for stability and tightness. There exists an example in Kenya which functions also as a bridge over a small stream (Nilsson, 1988).

A sand storage dam does not necessarily have to be completely watertight. Stone sand storage dam, for example, consists of flat stones which have been piled up to form a massive dam allows water to seep at a sufficient rate for downstream.

Erosion protection is important for sand storage dams. A sand storage dam has to be well protected against erosion along the banks and at the dam toe where energy of water during peak flows is very high. The best way of avoiding erosion is to construct the dam at natural rock bars. If this cannot be achieved, the dam should be extended several meters into the riverbank or complemented with wing walls of sufficient dimensions (Nilsson, 1988).

Sand storage dams are more suitable for gravity extraction than subsurface dams. Water is generally extracted by placing a drain at the reservoir bottom along the upstream side of the dam. The drain is connected to a well or a gravity supply pipe through the dam wall. If a well is built it can be made a part of the dam structure. The well should be placed at the deepest part of the dam section. The extraction is simply achieved by allowing seepage through the dam and collecting immediately at the downstream side or in a well along the course of stream.

2. 3 CHARACTERISTICS OF UNDERGROUND DAMS

Characteristics of underground dams are given as compared with river-dams.

a) For an underground dam, it takes a long time to store water because the water is not only stored at the dam site but also at the site far away, in upstream of the dam. The storage of the water at the upstream side will increase after the groundwater begins to overflow at the dam site.

b) Because the groundwater is stored far away upwards from the dam site, even if the dam is low, the volume of stored water is large. But the depth of the groundwater level restricts the depth of the dam.

c) Because the water is stored under the surface, the ground surface above the stored water area can be used as it is used to be.

d) With an underground dam, the excess water in a rainy season and unused water do not flow away but stored.

e) The temperature and the properties of the groundwater do not change through the year so that it is very convenient for the usage of cooling water.

f) The construction of an underground dam is very easy and simple. No strict quality control is needed which is needed for a river-dam. There is no disaster caused by the failure of an underground dam.

g) The underground dam can be constructed partly. Therefore it is very economical, since the result could be checked up before the completion of the whole dam.

h) In the underground dam with the utilizable depth of several meters, the range of fluctuation of the groundwater level is along several kilometers and about hundred million cubic meters of the groundwater can be used.

i) The underground dam is constructed not only for the effective use of the groundwater but also for controlling the groundwater level. For example it can prevent the fluctuation of the groundwater level in the surrounding area caused by the change in the water level of lakes or sea. (Matsuo, 1975)

In addition to these characteristics, groundwater dams can take important role in prevention of salt-water intrusion, since the hydraulic conductivity of dam wall is much less than the conductivity of the media.

Most of the characteristics of groundwater dams given above are related to the concept of sustainable development. Sustainable development is the development, which is both economically and ecologically sustainable (Archibugi and Nijkamp, 1989). Groundwater dams contribute to sustainability by providing additional water supply without causing disturbance in natural life.

CHAPTER 3

THEORETICAL BACKGROUND

3.1 MATHEMATICAL MODEL

Governing differential equation of the flow is obtained by combining continuity equation and Darcy's law. Darcy's law governs the apparent velocity of groundwater movement in porous medium;

$$q = K \frac{\Delta h}{\Delta L} \tag{3.1}$$

Where, q is the specific discharge Δh is the head loss and ΔL length in the direction of the flow path and K is the proportionality constant known as the hydraulic conductivity. The hydraulic conductivity depends not only on the medium of the formation but also on the properties of the fluid. By dimensional analysis,

$$K = Cd^2 \frac{\gamma}{\mu} \tag{3.2}$$

In Equation 3.2, C is a constant, d is a representative grain size, μ is the dynamic viscosity of the fluid and γ is the specific weight of the fluid.

The q value that is called apparent velocity in Darcy's law is the fictitious velocity through the whole cross-section, whereas the seepage velocity is the velocity of water traveling through pores.

For three-dimensional flow, in an isotropic media, the one-dimensional form of Darcy's law can be generalized as follows:

$$q_x = -K\frac{\partial h}{\partial x} \qquad q_y = -K\frac{\partial h}{\partial y} \qquad q_z = -K\frac{\partial h}{\partial z} \tag{3.3}$$

The minus sign is because the groundwater flow is in the direction of decreasing head.

The continuity equation, the basic principle also known as conservation of mass is used with Darcy's law to provide mathematical framework to find the head distribution within a region as a function of location and time.

For a leaky confined aquifer the representative control volume used in the derivation of the governing equation is shown in Figure 3.1.

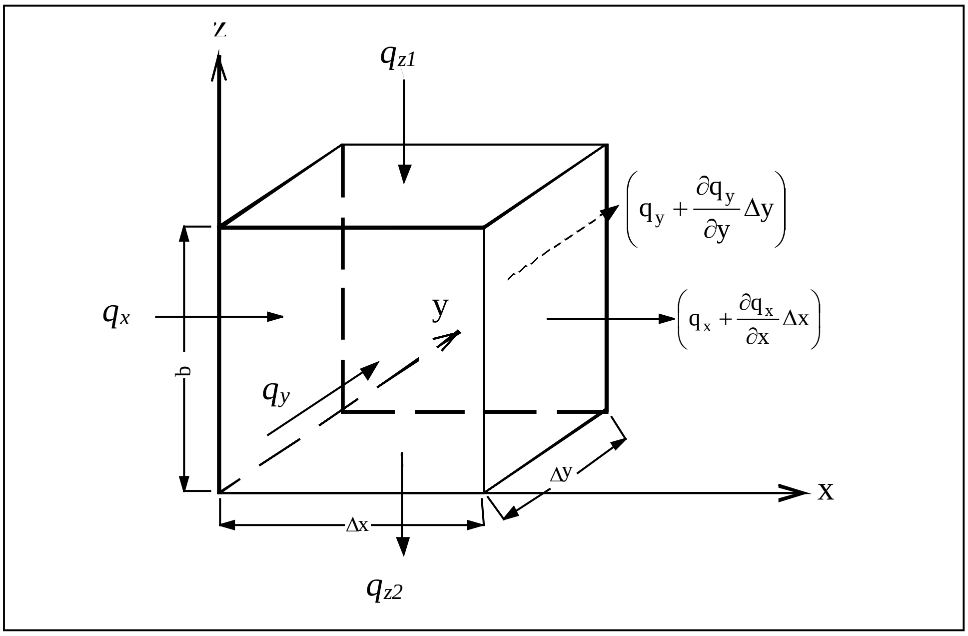


Figure 3.1 Flow through the control volume

q_{z1} and q_{z2} are leakage into and out of the control volume respectively

Development of groundwater flow equation follows from the application of the continuity equation for a control volume: the sum of all flows in to and out of the control volume must be equal to the rate of change in storage within the control volume. A general equation for conservation of mass for the volume may be expressed as;

[rate of mass input]- [rate of mass output]+ [rate of mass production(+) or consumption(-)] =[rate of mass accumulation] (3.4)

When control volume is considered rate of mass input and output terms in Equation 3.4 can be expressed in x direction as:

$$\rho q_x b\Delta y - \left\{\rho\left[q_x + (\frac{\partial q_x}{\partial x})\Delta x\right]b\Delta y\right\} = -\rho(\frac{\partial q_x}{\partial x})b\Delta x\Delta y \tag{3.4.a}$$

in y direction as:

$$-\rho(\frac{\partial q_y}{\partial y})b\Delta x\Delta y \tag{3.4.b}$$

And in z direction as:

$$\rho q_{z1}\Delta x\Delta y - \rho q_{z2}\Delta x\Delta y \tag{3.4.c}$$

Rate of mass production or consumption terms are related to the process responsible for sources and sinks. The source may be point (concentrated) such as recharge well or distributed (continuous) such as recharge from precipitation. The sink may be point (concentrated) such as pumping well or distributed (continuous) such as evapotranspiration. If it is defined;

R'=Rate of volume of water produced per unit horizontal area

P=Rate of volume of water consumed per unit horizontal area

Then; net production per unit time is;

$$\rho(R'-P)\Delta x\Delta y \tag{3.5}$$

Rate of mass accumulation is the process related to compressibility of water and expandability of porous matrix for confined aquifer. For unconfined aquifer it is related to filling of void space. If it is defined:

S=the specific storage of the porous material

ΔV_w=volume of water released from or added to storage

Then;

$$\Delta V_w == S\Delta A\Delta h \tag{3.6.a}$$

So rate of mass is:

$$\rho\frac{\Delta V_w}{\Delta t} = \rho S(\Delta A)\frac{\Delta h}{\Delta t} = \rho S\Delta x\Delta y\frac{\Delta h}{\Delta t} \tag{3.6.b}$$

If the Equations ;(3.4.a), (3.4.b), (3.4.c), (3.5) and (3.6.b) are inserted in Equation 3.4, which is the continuity equation:

$$-\rho b\left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y}\right)\Delta x\Delta y + \rho q_{z1}\Delta x\Delta y - \rho q_{z2}\Delta x\Delta y + \rho(R' - P)\Delta x\Delta y = \rho S\Delta x\Delta y\frac{\Delta h}{\Delta t}$$

(3.7.a)

By canceling ρ, Δx, Δy

$$-b\left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y}\right) + q_{z1} - q_{z2} + (R' - P) = S\frac{\partial h}{\partial t} \tag{3.7.b}$$

The above equation (3.7.b) is the continuity equation for 2D flow. Now Darcy’s law can be inserted in continuity equation; when principal axes are assumed to coincide with our axes, in 2D flow;

$$q_x = -K_{xx}\frac{\partial h}{\partial x} \tag{3.8.a}$$

$$q_y = -K_{yy}\frac{\partial h}{\partial y} \tag{3.8.b}$$

The leakage terms; q_{z1} and q_{z2} should be rewritten using K_1 and K_2; hydraulic conductivity of the inflow and outflow vertical leakage boundary respectively and using b_1 and b_2; thickness of inflow and outflow vertical leakage boundary respectively. Also h_1 and h_2 represent piezometric heads of inflow and outflow vertical leakage boundary respectively. Thus q_{z1} and q_{z2} can be rewritten as:

$$q_{z1} = K_1\frac{(h_1 - h)}{b_1} \tag{3.9.a}$$

$$q_{z2} = K_2\frac{(h - h_2)}{b_2} \tag{3.9.b}$$

When Equations (3.8.a), (3.8.b) and (3.9.a), (3.9.b) are inserted in Equation (3.7.b), Equation (3.10.a) is established representing the governing differential equation for 2-D flow in a leaky confined aquifer;

$$\frac{\partial}{\partial x}(bK_{xx}\frac{\partial h}{\partial x}) + \frac{\partial}{\partial y}(bK_{yy}\frac{\partial h}{\partial y}) + K_1\frac{(h_1 - h)}{b_1} + K_2\frac{(h - h_2)}{b_2} + R' - P = S\frac{\partial h}{\partial t} \tag{3.10.a}$$

The governing equation for 2-D flow in an unconfined aquifer is;

$$\frac{\partial}{\partial x}\left[(h-\eta)K_{xx}\frac{\partial h}{\partial x}\right]+\frac{\partial}{\partial y}\left[(h-\eta)K_{yy}\frac{\partial h}{\partial y}\right]+K_1\frac{(h_1-h)}{b_1}+R'-P=S_y\frac{\partial h}{\partial t} \quad (3.10.b)$$

where η is the bottom elevation.

The unknown h(x, y, t) in the above equations can be determined by an appropriate solution method and using boundary and initial conditions.

3.2 BOUNDARY AND INITIAL CONDITIONS

To describe a specific problem, the partial differential equation that describes flow in an aquifer must be supplemented by appropriate initial and boundary conditions. Several types of boundary conditions may be encountered. These are:

(a) Head is known for surfaces bounding the flow region (Dirichlet conditions)

(b) Flow is known across surfaces bounding the region (Neumann conditions)

(c) Some combination of (a) and (b) is known for surfaces bounding the region(mixed conditions)

The groundwater hydrologist must sometimes approximate boundary conditions to limit the region of the problem domain. If inconsistent or incomplete boundary conditions are specified, the problem itself is ill defined. (Wang and Anderson, 1982)

3.3 NUMERICAL SOLUTION

The solution can be obtained by using experimental, analytical or numerical methods. Analytical methods give exact solutions. However in real problems, often the boundaries of the flow domain have irregular shapes, or are too complex to describe, the domain is inhomogeneous or the assumptions to obtain an analytical solution are not realistic. Therefore numerical methods are used to overcome the difficulties. Some of the numerical methods used are:

- Finite element method
- Finite difference method
- Boundary element method

The numerical solutions necessitate the use of computer programs. There are many programs that utilize finite difference technique to simulate groundwater problems. MODFLOW is one of the leading three dimensional groundwater problems worldwide (Mc Donald and Harbaugh, 1988). Finite difference method is the method that the MODFLOW program uses in solving complex groundwater problems. The MODFLOW program is divided into a main program and a series of independent subroutines called modules. The modules are grouped into ‘packages’, each of which is a group of modules that deals with a single aspect of the simulation (Charbeneau, 2000). The Well Package, Boundaries Package and Properties Package are the main packages used in this thesis.

CHAPTER 4

HYPOTHETICAL CASE STUDY

CASE 1

In this part, as Case 1-a hypothetical rectangular ideal aquifer will be considered aiming to analyze the effects of groundwater dam on mainly water storage. The case will be handled step by step. In the first step the hypothetical aquifer will be simulated in natural conditions, without dam, without wells. This situation will be called as Case 1-a. In the next case, Case 1-b, wells will be added to the scenario. The next case, Case 1-c, will be with dam wall and without wells, and Case 1-d will be both with wells and with dam wall. The content of Cases 1-a, 1-b, 1-c, and 1-d are shown in Table 4.1 for simplicity.

Table 4.1 Content of Cases 1-a, 1-b, 1-c, and 1-d

Case 1-a	Without wells without dam
Case 1-b	With wells without dam
Case 1-c	Without wells with dam
Case 1-d	With wells with dam

Figure 4.1 shows the plan view of idealized rectangular aquifer and the location of the wells and underground dam.

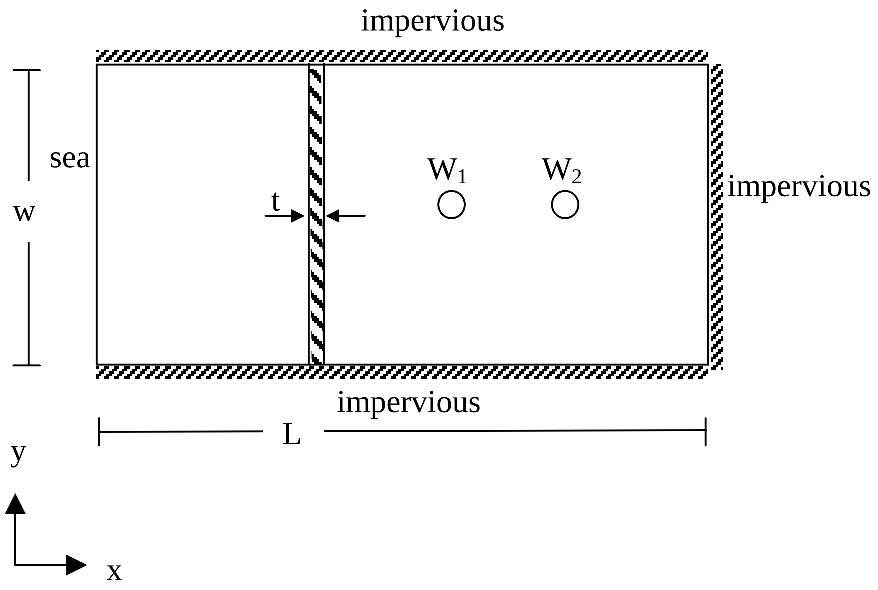


Figure 4.1 Plan view of idealized rectangular aquifer with dam wall and wells

4.1 Case 1-a

In Case 1-a, natural conditions of the hypothetical ideal aquifer is considered. The groundwater flow in this aquifer is simulated using MODFLOW. The values of L is the length of aquifer along the flow direction namely x direction in MODFLOW, w is the width of the aquifer in y direction and also used as the length of the dam wall in this hypothetical case, b is the thickness of the soil in z direction, K_s is the hydraulic conductivity of the soil, K_w is the hydraulic conductivity of the dam wall, t is the thickness of the dam wall, h_{sea} presents the sea level above impervious bottom, h_{gw} presents groundwater level above impervious bottom, R is the recharge value, Q_1 and Q_2 are the discharges from the wells W_1 and W_2, n_{ef} and n_t are the effective and total porosity values, S_s and S_y are the specific storage and specific yield values. These are all used as inputs in MODFLOW. This is done to form a basis to make comparison between different scenarios. The inputs of Case 1-a, which is analyzed in steady state as other cases, are given as in the Table 4.2.

Table 4.2 Inputs of Case 1-a

Aquifer length	L	4000 m
Aquifer width	w	800 m
Aquifer thickness	b	10 m
Dam wall thickness	t	no dam
Groundwater level	h_{gw}	2 m
Mean sea level	h_{sea}	1 m
Conductivity of soil	K_s	0.02 m/s
Conductivity of dam	K_w	no dam
Recharge	R	0.007 m/day
Discharge from W_1	Q_1	no well
Discharge from W_2	Q_2	no well
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.02 (-)
Effective porosity	n_{ef}	0.02 (-)
Total porosity	n_t	0.02 (-)

The effective porosity can be thought of as the volume of pore space that will drain in a reasonable period of time under the influence of gravity. Sometimes the effective porosity is much less than total porosity, but since this case is hypothetical they are taken equal. When the inputs given in Table 4.2 are used in MODFLOW, the variation of water table elevation along x-axis is as given in Figure 4.2. On this figure, the distances from the constant head boundary (sea) in meters are shown on the horizontal axis, whereas the elevations of water table in meters are given in the vertical axis. The head values increase from constant head boundary to the impermeable boundary with the effect of recharge.

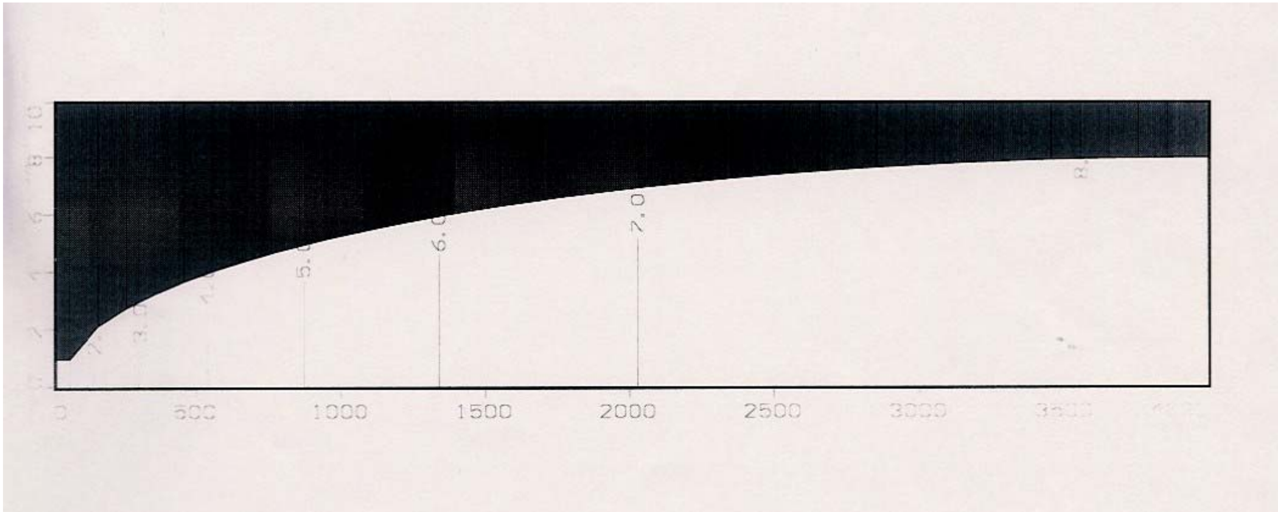


Figure 4.2 Case 1-a, cross-section of water table without wells without dam

4.2 Case 1-b

When wells are added to Case 1-a Case 1-b can be established. In MODFLOW the discharge values Q_1 and Q_2 for wells W_1 and W_2 are added. The inputs are as given in Table 4.3.

Table 4.3 Inputs of Case 1-b

Aquifer length	L	4000 m
Aquifer width	w	800 m
Aquifer thickness	b	10 m
Dam wall thickness	t	no dam
Groundwater level	h_{gw}	2 m
Mean sea level	h_{sea}	1 m
Conductivity of soil	K_s	0.02 m/s
Conductivity of dam	K_w	no dam
Recharge	R	0.007 m/day
Discharge from W_1	Q_1	–5000 m^3/day
Discharge from W_2	Q_2	–5000 m^3/day
Specific storage	S_s	0,001 (1/m)
Specific yield	S_y	0.02 (-)
Effective porosity	n_{ef}	0.02 (-)
Total porosity	n_t	0.02 (-)

The minus values are just because the water is extracted through the discharging wells and it is in accordance with the convention used in MODFLOW. Figure 4.3 shows the variation of water table elevation along x-axis in existence of two wells for Case 1-b. On this figure also, the distances from the constant head boundary (sea) in meters are shown on the horizontal axis, whereas the elevations of water table in meters are given in the vertical axis. The definition of the axes in the following figures should be understood in the same manner as it is stated for Figures 4.2 and 4.3.

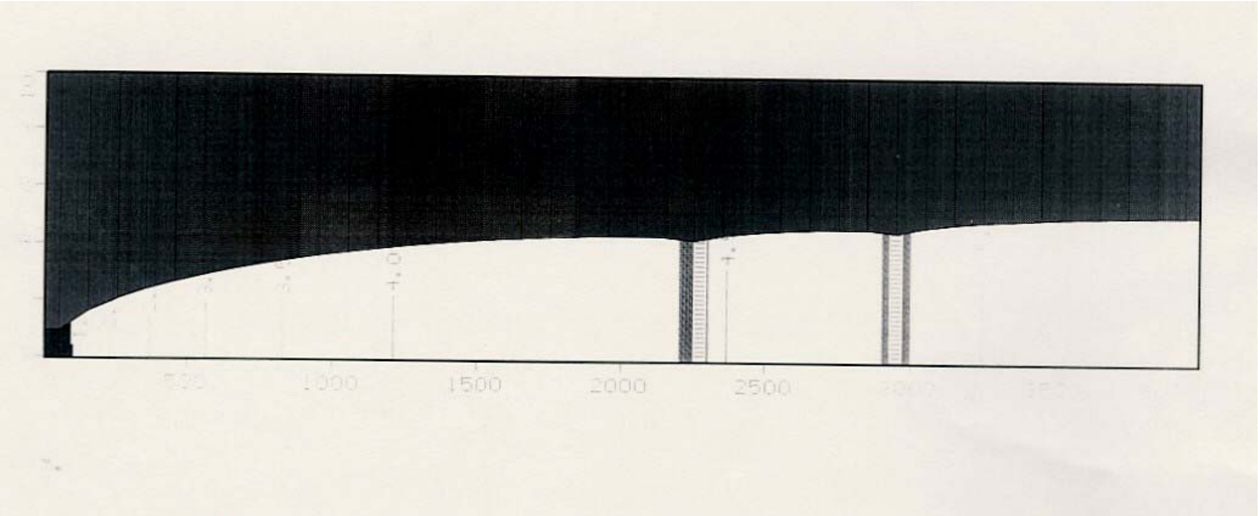


Figure 4.3 Case 1-b, cross-section of water table with two discharging wells

Q_1=Q_2=5000 m^3/day

4.3 Case 1-c

When wells are removed from Case 1-b and dam wall is added to Case 1-b then Case 1-c is established. The thickness of the dam wall, b and the conductivity value of the wall, K_w that is much less than K_s are additional inputs used in MODFLOW. The inputs of Case 1-c are in the Table 4.4.

Table 4.4 Inputs of Case 1-c

Aquifer length	L	4000 m
Aquifer width	w	800 m
Aquifer thickness	b	10 m
Dam wall thickness	t	8 m
Groundwater level	h_{gw}	2 m
Mean sea level	h_{sea}	1 m
Conductivity of soil	K_s	0.02 m/s
Conductivity of dam	K_w	0.0001 m/s
Recharge	R	0.007 m/day
Discharge from W_1	Q_1	no well
Discharge from W_2	Q_2	no well
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.02 (-)
Effective porosity	n_{ef}	0.02 (-)
Total porosity	n_t	0.02 (-)

Increase in head values in the reservoir behind dam wall is seen in Figure 4.4. The location of the dam wall can be changed. If the dam wall is replaced 500 m distance from constant head boundary, the water table rises to the surface as it is seen in Figure 4.5. The figure shows the existence of wetland conditions. The recharge value is a very high value so that the effect of moving the dam wall can be easily seen.

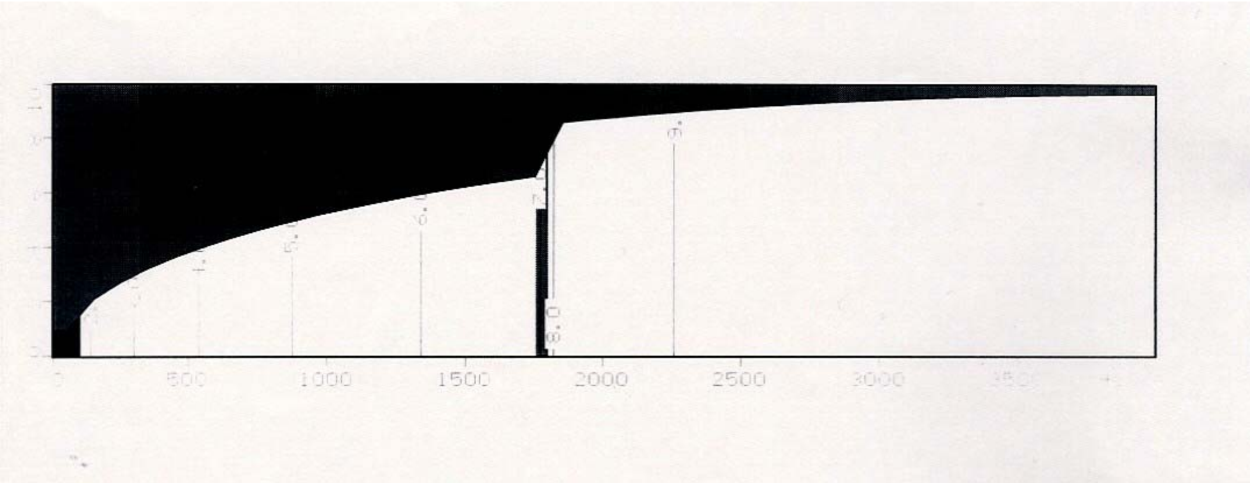


Figure 4.4 Case 1-c, cross-section of water table with dam wall

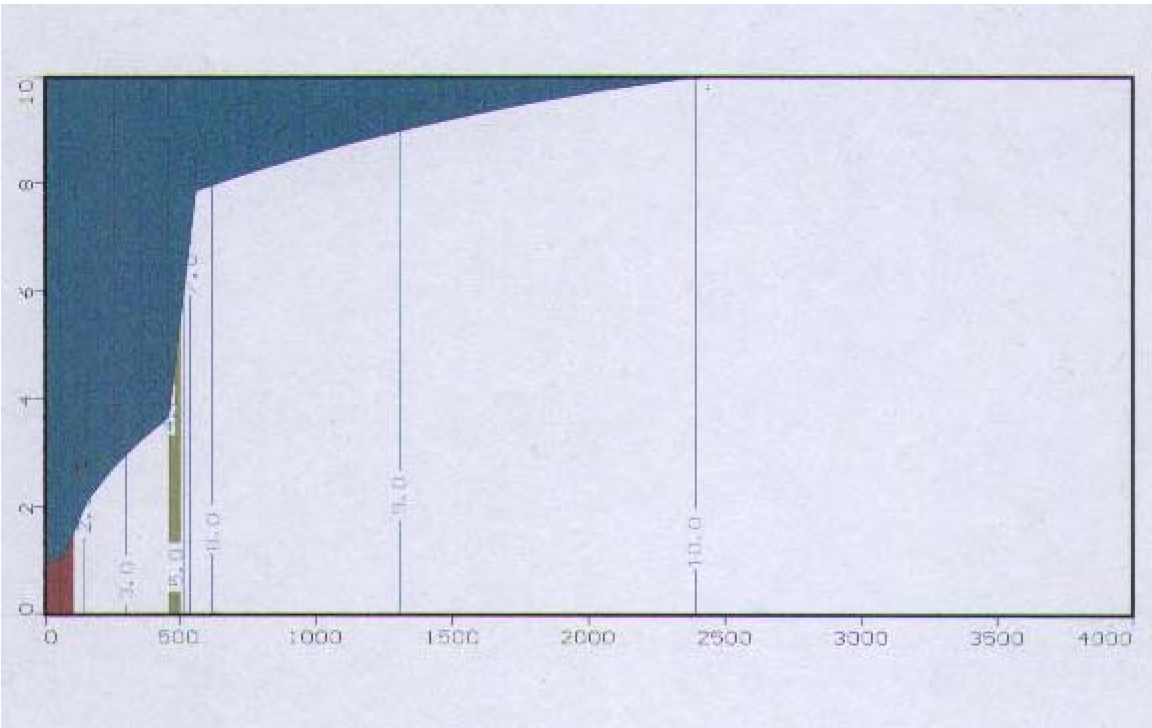


Figure 4.5 Case 1-c, cross-section of water table with dam wall relocated seaward at x=500 m

4.4 Case 1-d

Case 1-d is the hypothetical ideal case with wells and with dam. This case gives us to see the combined effect of cases 1-b and 1-c in MODFLOW.

Table 4.5 Inputs of Case 1-d

Aquifer length	L	4000 m
Aquifer width	w	800 m
Aquifer thickness	b	10 m
Dam wall thickness	t	8 m
Groundwater level	h_{gw}	2 m
Mean sea level	h_{sea}	1 m
Conductivity of soil	K_s	0,02 m/s
Conductivity of dam	K_w	0.0001 m/s
Recharge	R	0,007 m/day
Discharge from W_1	Q_1	–5000 m^3/day
Discharge from W_2	Q_2	–5000 m^3/day
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.02 (-)
Effective porosity	n_{ef}	0.02 (-)
Total porosity	n_t	0.02 (-)

After reaching Case 1-d by increasing the Q values the diminishing of the water head levels from upstream to the dam is observed. The upper limit value of Q_1 and Q_2 are found by trial and error in MODFLOW. If this value is exceeded than the region around the wells will be dried. Figure 4.6 shows the combined effects of wells and dam wall on the aquifer.

By making trial and error solution; by changing the Q values, the maximum discharge extracted without drying the aquifer for this case is found as 7005 m^3/day. If this value is exceeded for this case the wells are dried. The section consisting wells extracting with 7005 m^3/day showing head values is in Figure 4.7.

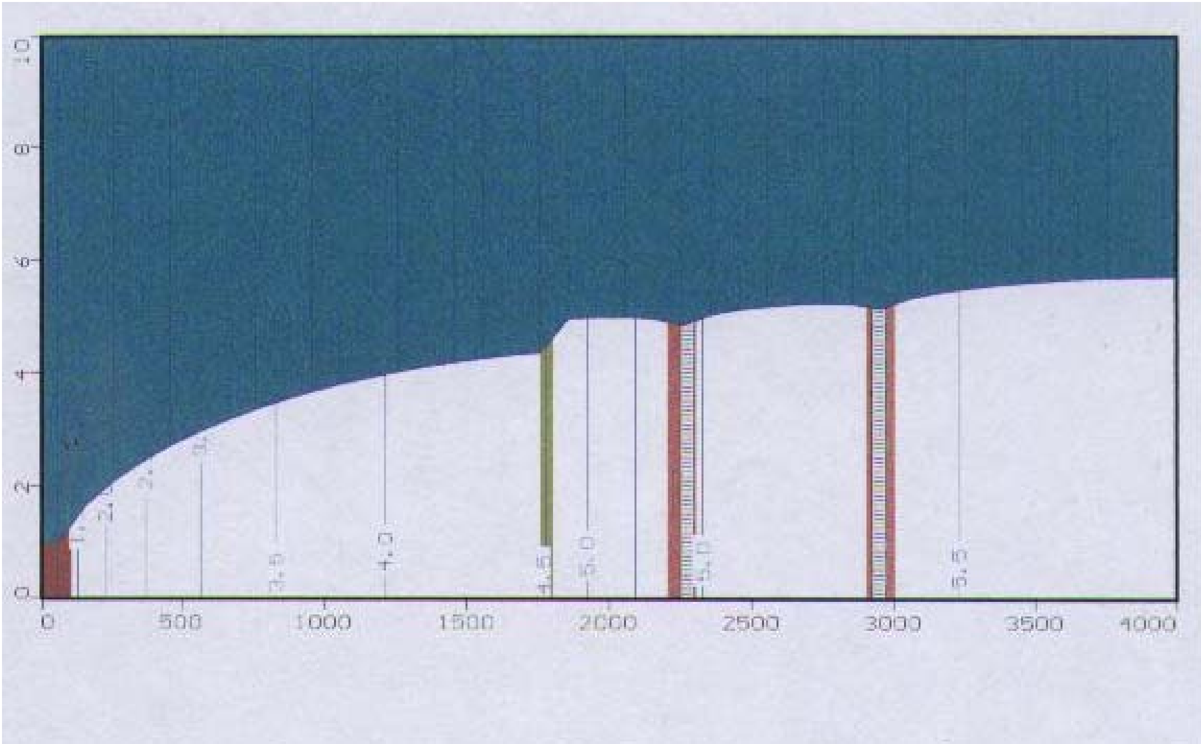


Figure 4.6 Case 1-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=5000 m^3/day

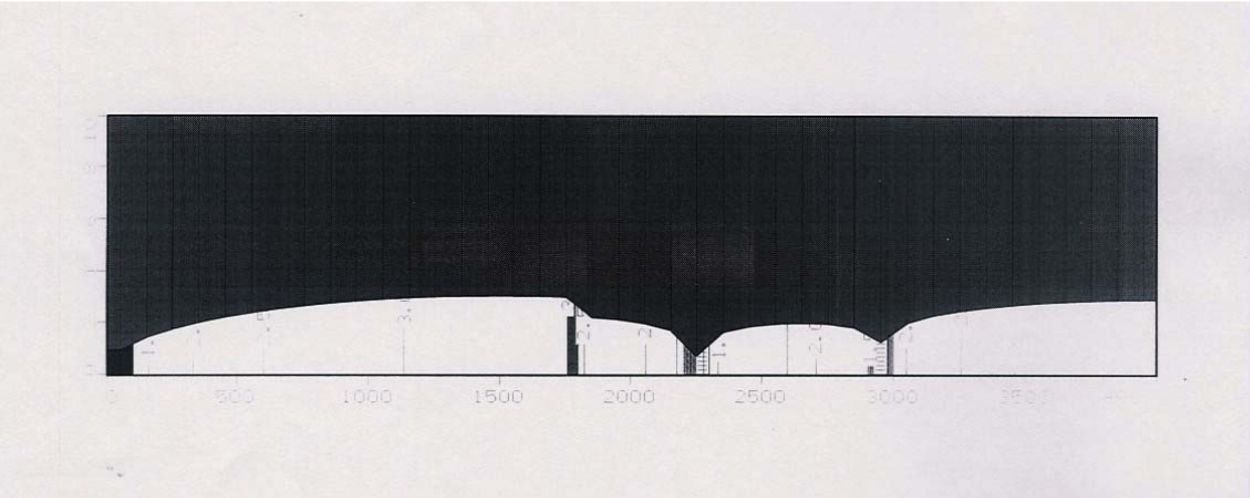


Figure 4.7 Case 1-d, cross-section of water table with dam wall and two discharging wells $Q_1=Q_2=7005$ m^3/day

4.5 Unsteady solution of Case 1

The results of the unsteady simulation can answer the questions like how many days the aquifer is filled or emptied.

In steady solution the limit maximum value of Q_1 and Q_2 was found. In unsteady solution the period that can be benefited from the aquifer is calculated.

The methodology used for the calculations in MODFLOW is based on the output of steady solution. The output of steady solution of Case 1-c, with dam without wells case, is used as initial head for the unsteady simulation of Case 1-d which was the case with dam with wells and with the limit maximum value of Q_1 and Q_2. Q_1 and Q_2 were found as 7005m^3/day as limit value for the hypothetical scenario. The results of the unsteady solution give opportunity to calculate:

a) The number of days it takes the reservoir behind dam to be emptied up to steady state (not dried) when $Q_1=Q_2=7005$ m^3/day

b) The number of days it takes the reservoir behind dam to be filled when $Q_1=Q_2=7005$m^3/day

c) The number of days at the end of which the wells get dried in case the recharge value drops down to R=0.003 m/day and still $Q_1=Q_2=7005$ m^3/day

d) By iteration the appropriate Q_1 and Q_2 values, in case these discharges can be extracted for 90 days (the wells should not be dried till that time) and still R=0.003 m/day

4.5.1 Duration in which steady state is reached for Q=7005 m^3/day

The output of Case 1-c, with dam without wells, is used as initial head value and extraction from wells as Q_1=Q_2=7005 m^3/day is applied. The Q_1 and Q_2 values are the limit maximum values that can be extracted in steady solution. By unsteady solution it is obtained that it takes approximately 200 days the steady state is reached. This means the decrease in head values of the control points becomes a negligible amount after 200 days.

Three control points are selected to check the water table elevation. Two of the control points are on W_1 and W_2 and the other, which is point P (2700, 500), is between the two wells. The head values on these control points are taken from the output file of MODFLOW and transferred to MS Excel. Figures 4.8, 4.9 and 4.10 show the decreasing head values.

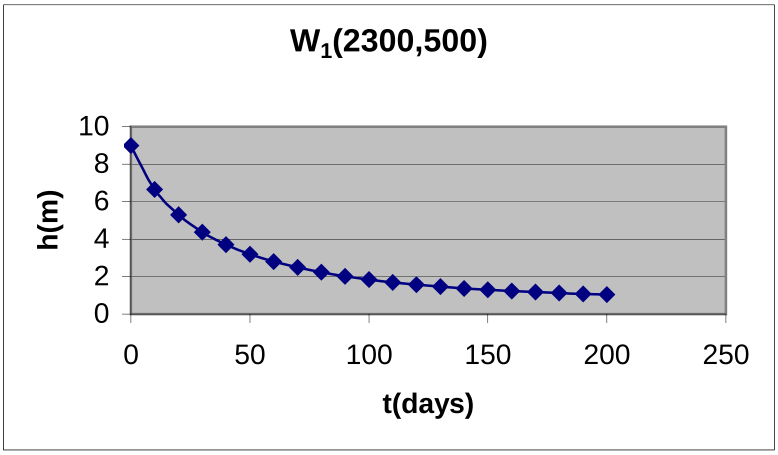


Figure 4.8 Head vs. time for control point W_1 (2300,500)

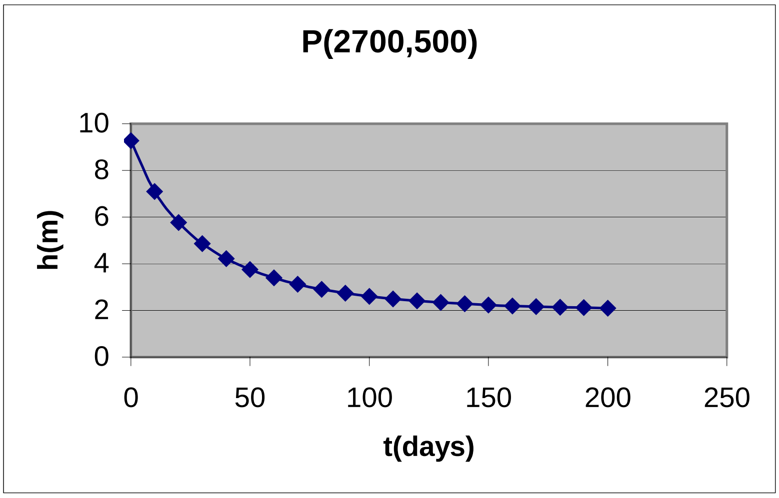


Figure 4.9 Head vs. time for control point P (2700,500)

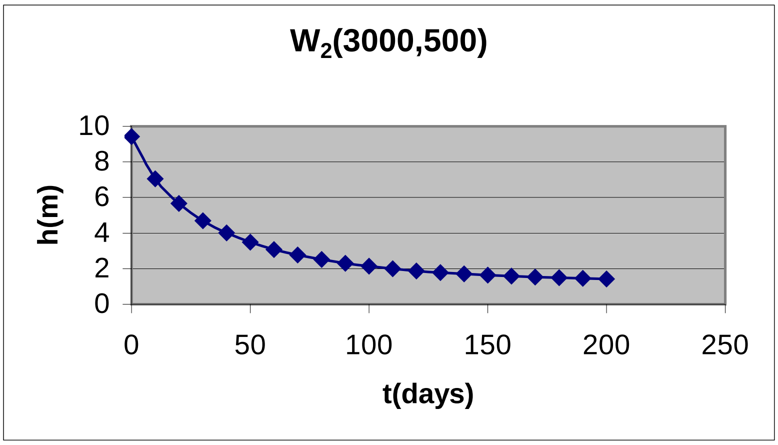


Figure 4.10 Head vs. time for control point W_2 (3000,500)

4.5.2 Duration in which steady state is reached for Q=0

The output of Case 1-d, with dam with wells, $Q_1=Q_2=7005\ m^3/day$, is used as initial head value for this case. For the unsteady solution of this case, the wells are deactivated. Since recharge comes to the system and no extraction is made, the reservoir behind dam is filled. It is obtained that in 110 days, the change of increase in head values become negligible and steady state is reached. The changes in head values for the same points are in the following figures. The head values at these points are taken from the output file of MODFLOW and transferred to MS Excel. Results are in the Figures 4.11, 4.12 and 4.13.

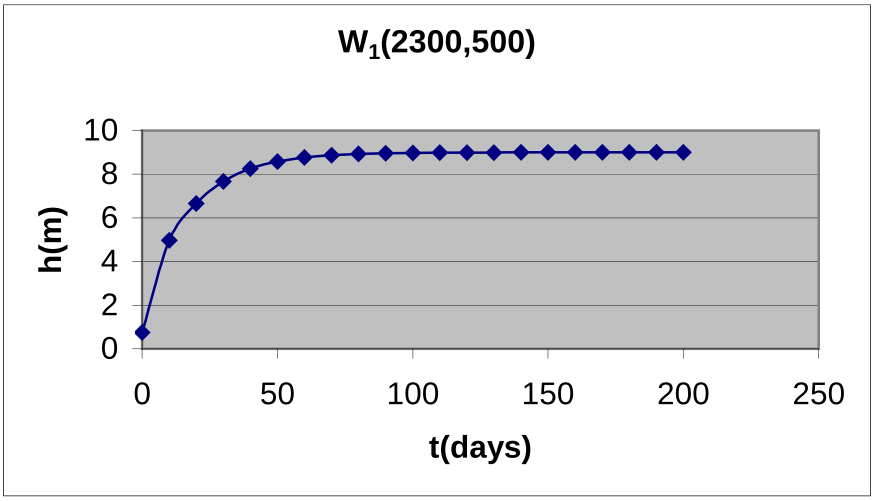


Figure 4.11 Head vs. time for control point W_1 (2300,500)

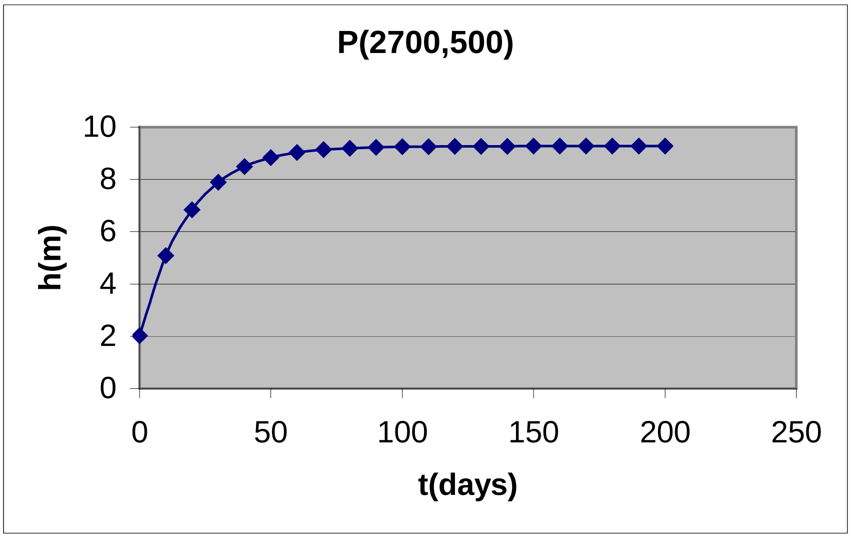


Figure 4.12 Head vs. time for control point P (2700,500)

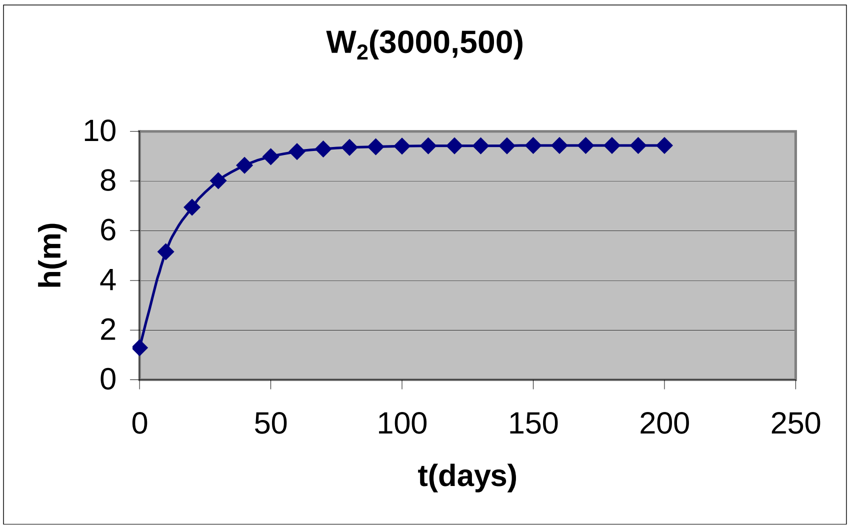


Figure 4.13 Head vs. time for control point W_2 (3000,500)

4.5.3 Limit Extraction Duration when R=0.003 m/day Q=7005 m^3/day

The calculations in MODFLOW for the part 4.5.a are made in the same manner except the recharge value. The recharge value drops down normally in the season water is extracted by wells from the dam. So in the hypothetical case R=0.003 m/day is taken as recharge value. When output of the unsteady solution for these outputs is taken from MODFLOW it is seen that after 20 days period the wells get dried for Q1=Q2=7005 m^3/day discharge and R=0.003 m/day recharge value.

4.5.4 Limit discharge for 90 days extraction when R=0.003 m/day

For R=0.003 m/day, the limit maximum extraction from wells for 90 day is found as 3500 m^3/day by running MODFLOW several times using different discharge values. Duration is chosen due preference. After 90 days for Q_1=Q_2=3500 m^3/day the wells get dried.

In the Case 1 the inputs used are exaggerated values to make visible effects on groundwater behavior. For example, parameters like recharge and storage area of the aquifer are very high. This makes the effects of building dam wall quite visible. However in real life the site conditions will be completely different from Case 1.

CHAPTER 5

REAL CASE STUDY

CASE 2

5.1 Description of the study area

The site that the study is made on is near Kocaalan Creek in Çamlı Köyü, Marmaris, Muğla. The investigation area is located between 36° 57’ to 37° 00’ latitude north, and between 28° 15’ and 28° 18’ longitude east. The area is approximately 25 km^2. The economy in the region depends upon agriculture, tourism and fishing. The average mean annual precipitation is 1193.4 mm and average annual temperature is 18.56 degrees Celsius.

The most important river in the region is Kocaalan Creek that the study is made on. The discharge values around Kocaalan Creek are measured on 4 different points for 18 months. Due to these measurements the annual average discharge of Kocaalan Creek is estimated as 2.27 m^3/s. The sources at around the area of study flow seasonally and diminish by summer.

The hydraulic conductivity (K) value of the aquifer was found as 0.000424 m/s. The thickness of alluvium is taken as 68 m in this study (Akdeniz, 2003). The location of the site is given in Figure 5.1. The boundaries of the flow domain have to be represented on grid system to get a solution in MODFLOW. The stages of digitization of the aquifer are shown in stages in the following pages in Figures 5.2, 5.3 and 5.4. Table 5.1 shows the content of the sub-cases of Case 2.

Table 5.1 Content of Cases 2-a, 2-b, 2-c, and 2-d

Case 2-a	Without wells without dam
Case 2-b	With wells without dam
Case 2-c	Without wells with dam
Case 2-d	With wells with dam

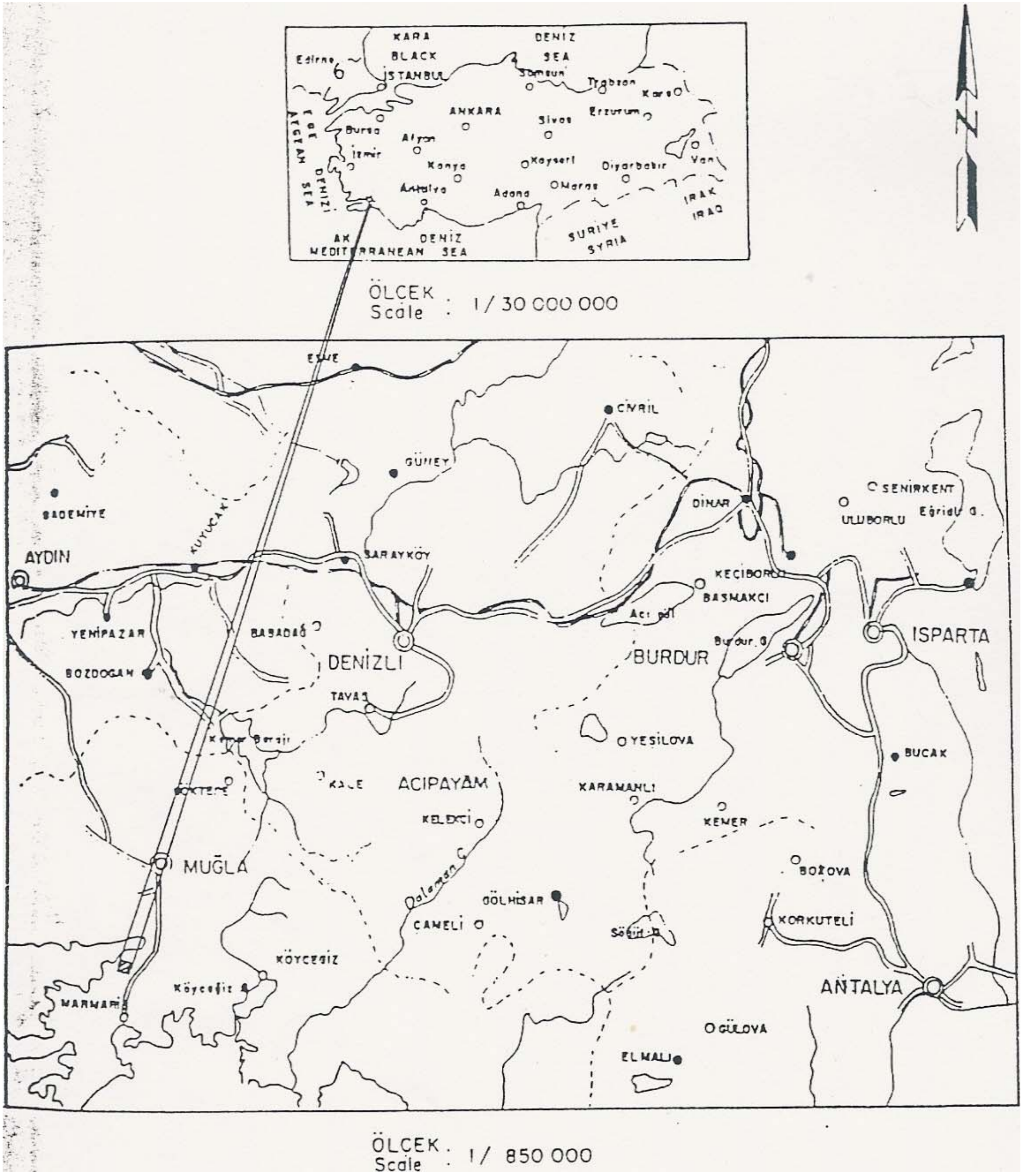


Figure 5.1 The study area and its location in Turkey

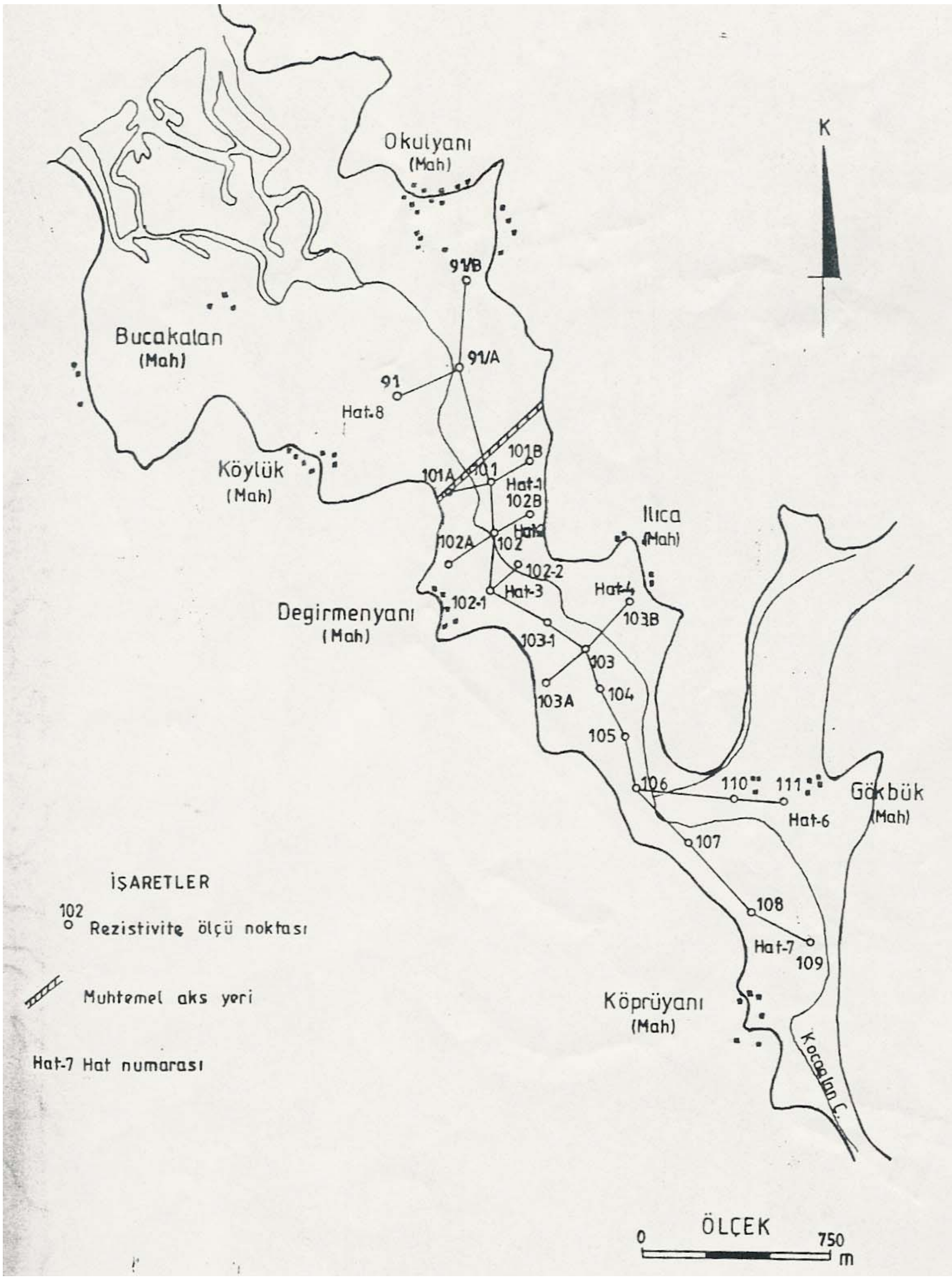


Figure 5.2 Map of the aquifer and the potential dam construction site

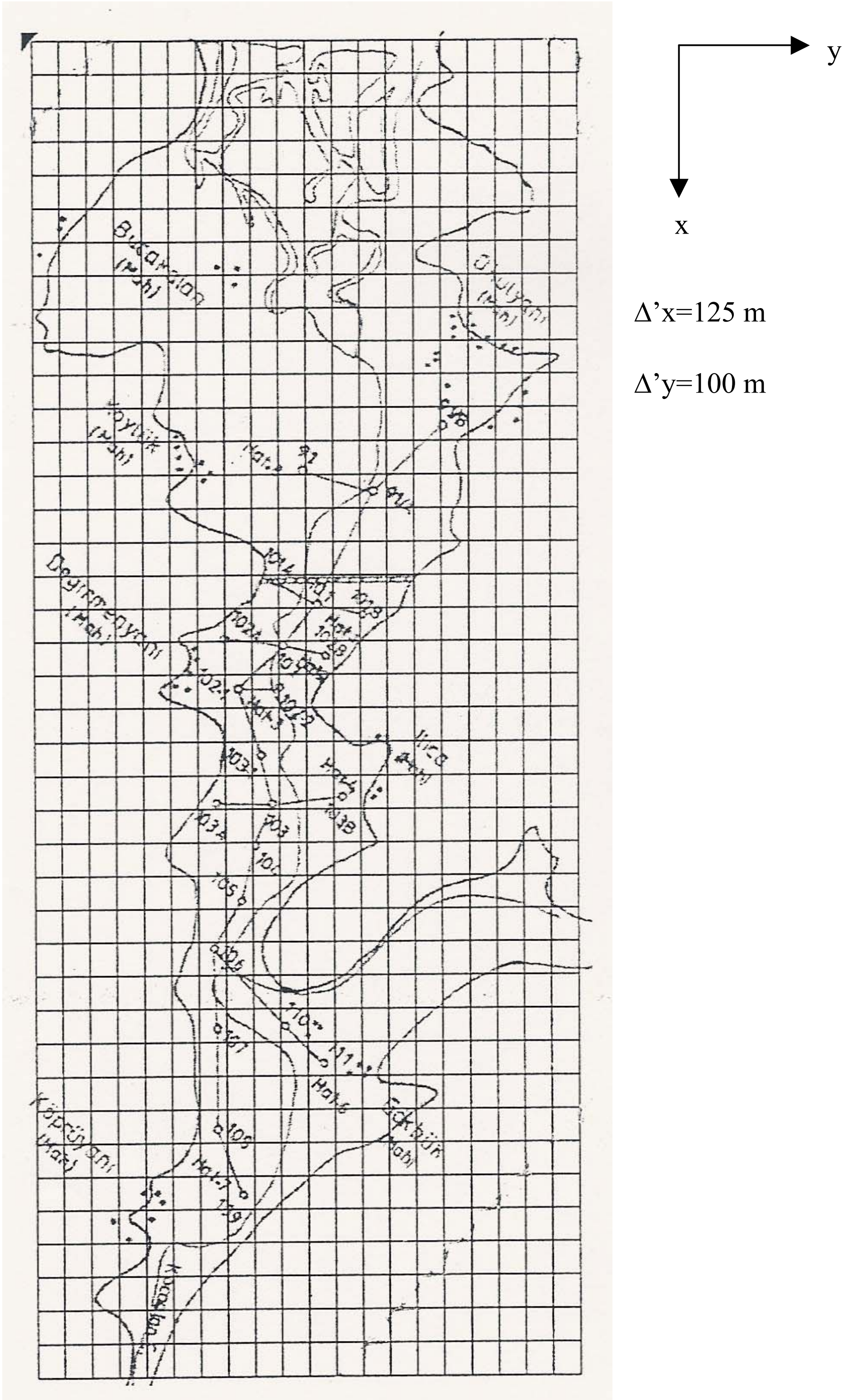


Figure 5.3 Finite-difference grid used to model study area

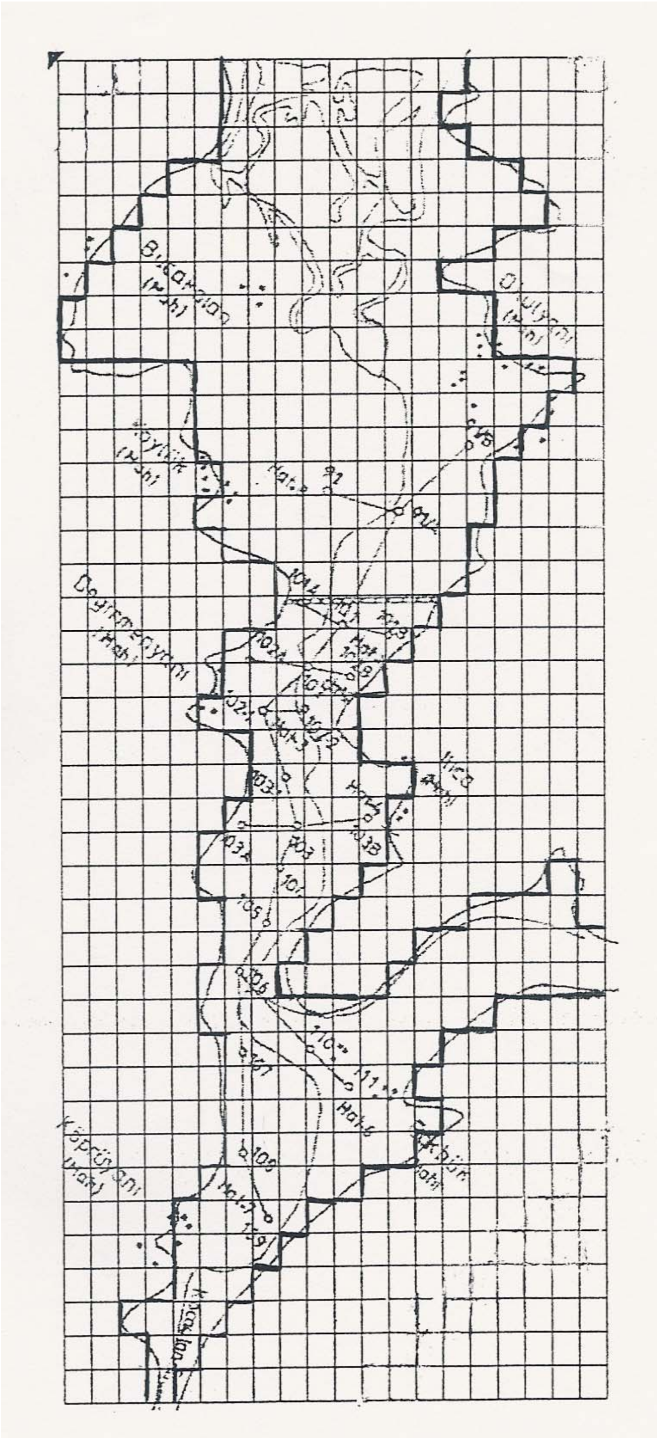


Figure 5.4 Approximation of aquifer boundaries

5.2 Case 2-a

In Case 2-a, the groundwater flow in the aquifer under natural conditions is analyzed. Neither water extraction is done nor there exists dam. The only external factor influencing the system is recharge. With the same definitions given in Case 1-a in Chapter 4, the inputs of Case 2-a are given in Table 5.2.

Table 5.2 Inputs of Case 2-a

Aquifer length	L	5000 m
Aquifer width	w	2000 m
Aquifer thickness	b	68 m
Dam wall thickness	t	No dam
Groundwater level	h_{gw}	66 m
Mean sea level	h_{sea}	60.5 m
Conductivity of soil	K_s	0.000424 m/s
Conductivity of dam	K_w	No dam
Recharge	R	0.0008175 m/day
Discharge from W_2	Q_1	No well
Discharge from W_2	Q_2	No well
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.15 (-)
Effective porosity	n_{ef}	0.15 (-)
Total porosity	n_t	0.30 (-)

Some of the inputs that should be used in MODFLOW were lacking in the report. Therefore values consistent with the site area are selected. The boundaries other than constant head boundary are assumed as impermeable. The annual average recharge cannot be accepted as net recharge because of the effects of evaporation, capillarity and other losses. Net recharge value is estimated as 25% of annual average recharge. The variation of water table elevation obtained from the output file of MODFLOW is shown in Figure 5.5 from top view and in Figure 5.6 in cross-section. The units used on the axes are in meters as it is also valid for the remaining figures.

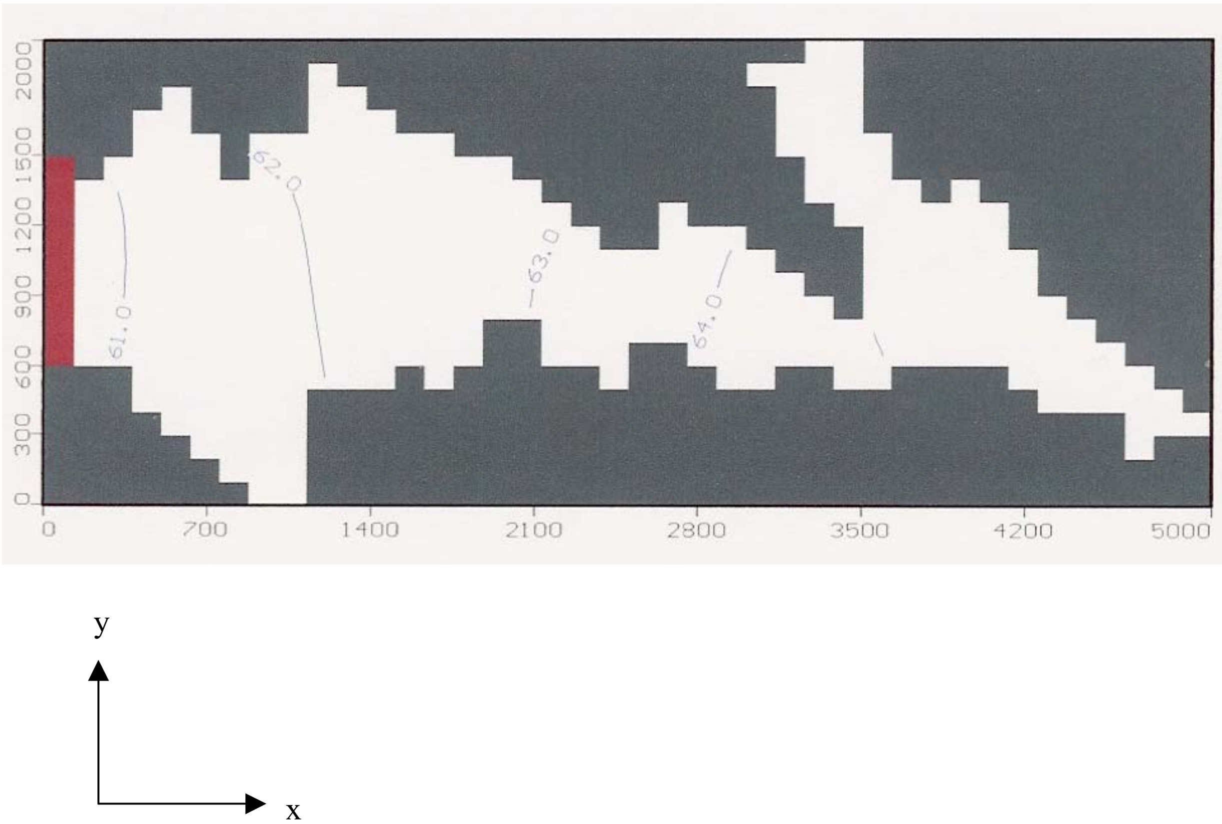


Figure 5.5 Case 2-a, top view of water table without wells without dam

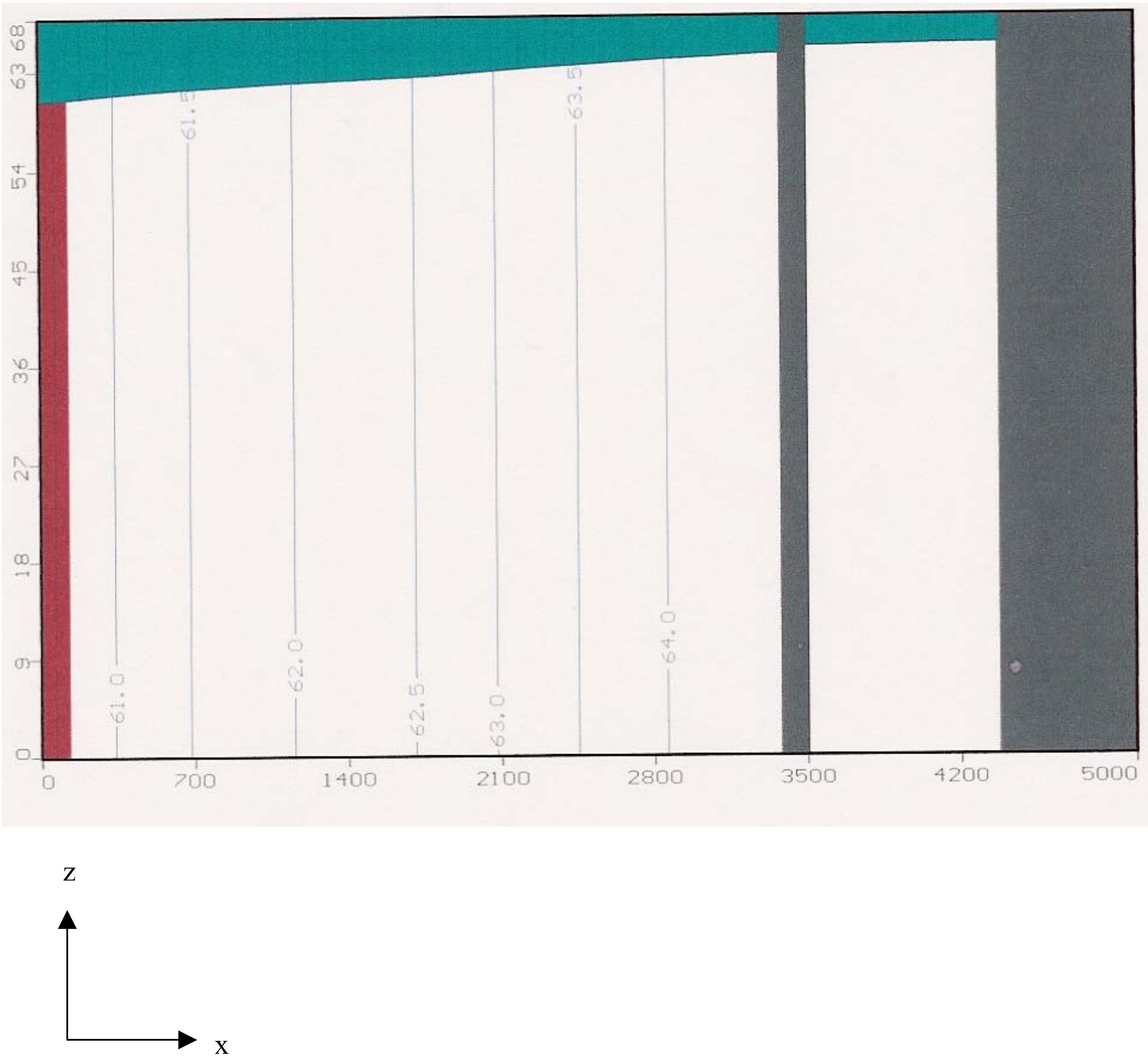


Figure 5.6 Case 2-a, cross-section of water table without wells without dam

In Figure 5.6 the rectangular boundary obtained at about x=3500 represents inactive cells. That should not be mixed with dam wall. The output shows increasing water level from constant head boundary, which is sea to the impermeable boundary.

5.3 Case 2-b

In this case wells are added to the system. The inputs of Case 2-b are given in Table 5.3.

Table 5.3 Inputs of Case 2-b

Aquifer length	L	5000 m
Aquifer width	w	2000 m
Aquifer thickness	b	68 m
Dam wall thickness	t	No dam
Groundwater level	h_{gw}	66 m
Mean sea level	h_{sea}	60.5 m
Conductivity of soil	K_s	0.000424 m/s
Conductivity of dam	K_w	No dam
Recharge	R	0.0008175 m/day
Discharge from W_1	Q_1	-900 m^3/day
Discharge from W_2	Q_2	-900 m^3/day
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.15 (-)
Effective porosity	n_{ef}	0.15 (-)
Total porosity	n_t	0.30 (-)

The location of wells in x-y coordinate is W_1 (3000, 850) and W_2 (4000, 850). The location of the wells is arranged after several iterations to reach the most appropriate coordinates for this site. Since there is no groundwater dam in this stage, seawater can enter the aquifer due to extraction from wells. The wells are not located near sea level because of seawater intrusion problem.

Seawater intrusion is the movement of seawater into fresh water aquifers due to natural processes or human activities. When a change occurs in one part of the hydrologic system it affects the others. Extracting water from wells causes local declines in ground water levels in the vicinity of the pumped wells and may cause localized seawater intrusion. Intrusion can affect the quality of water not only at the pumping well site, but also at other well sites, and undeveloped portions of the aquifer. As a result, subsequent wells completed in the aquifer may encounter salty water in the once fresh aquifer.

The interface between salt water and fresh water shown in Figure 5.7 is abrupt. The actual interface may be somewhat diffused due to diffusion processes and lateral migration of the interface over time. However the assumption of abrupt interface simplifies the problem in the cases of practical interest.

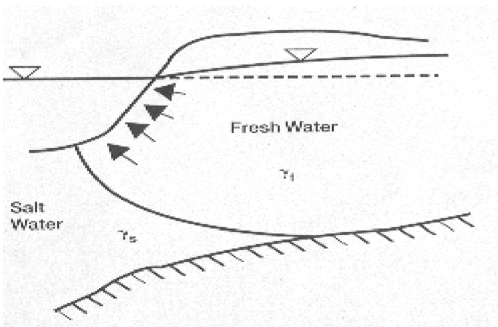


Figure 5.7 Typical cross-section of a coastal aquifer

In Case 2-b, extracting maximum water is aimed. However seawater intrusion limits the amount that can be extracted from the aquifer. The criterion used to determine the maximum possible extraction is that the head values in the cells inside the model domain near to the outflow boundary should not be lower than that of the assumed constant head along the outflow boundary. This ensured that salt-water intrusion does not take place (Das, 1998).

Using this criterion in the study, the constant head boundary level, which is 60.5 m, can be accepted as a control for Case 2-b. The discharge value is established by trial and error solution so that the water level in the aquifer does not fall below 60.5 m. The numerical value of maximum discharge without causing seawater intrusion is 900 m^3/day. Since there exists two wells the total discharge is 1800 m^3/day. The outputs of Case 2-b are given in Figure 5.8 and 5.9 showing the variation of water table elevation after steady state is reached.

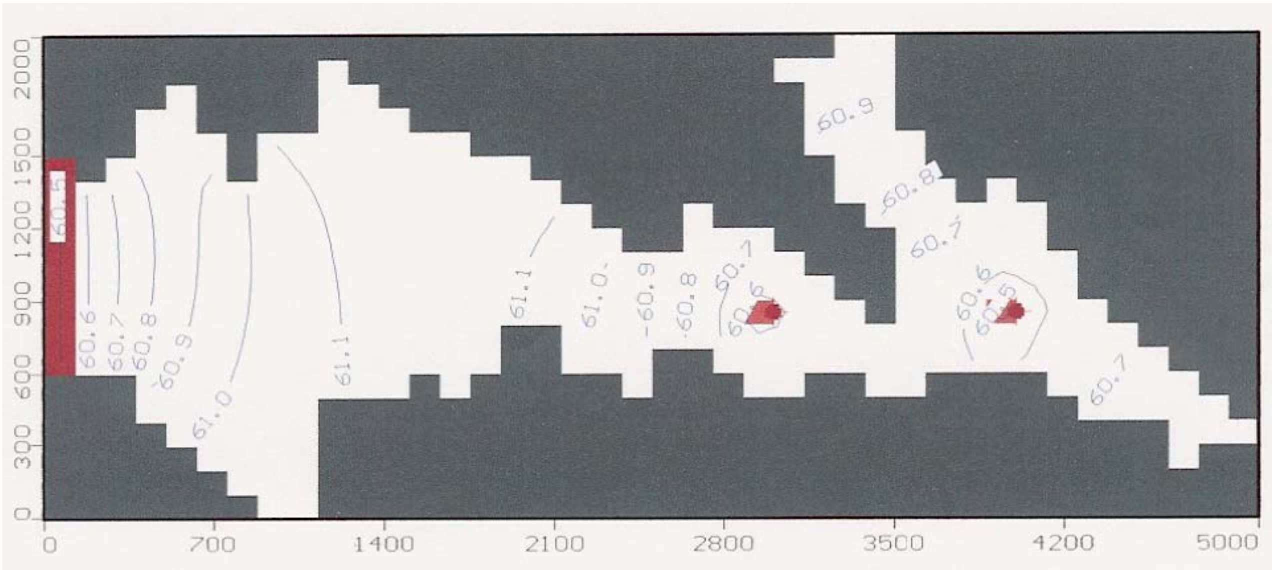


Figure 5.8 Case 2-b, top view of water table with two discharging wells

$Q_1=Q_2=900\ m^3/day$

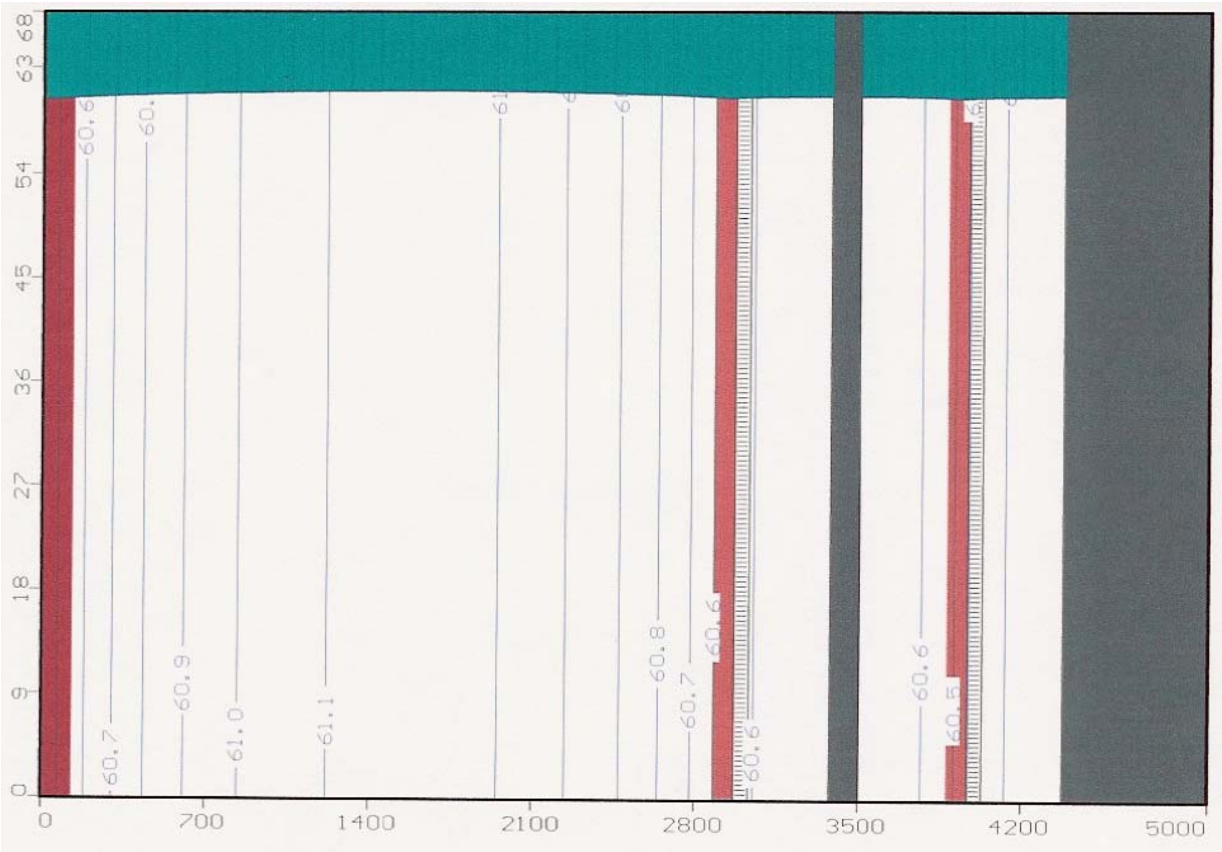


Figure 5.9 Case 2-b, cross-section of water table with two discharging wells

$Q_1 = Q_2 = 900\ m^3/day$

5.4 Case 2-c

In this case dam wall is added to the system in natural condition. All inputs are given in Table 5.4.

Table 5.4 Inputs of Case 2-c

Aquifer length	L	5000 m
Aquifer width	w	2000 m
Aquifer thickness	b	68 m
Dam wall thickness	t	10 m
Groundwater level	h_{gw}	66 m
Mean sea level	h_{sea}	60.5 m
Conductivity of soil	K_s	0.000424 m/s
Conductivity of dam	K_w	0.0000025 m/s
Recharge	R	0.0008175 m/day
Discharge from W_2	Q_1	No well
Discharge from W_2	Q_2	No well
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.15 (-)
Effective porosity	n_{ef}	0.15 (-)
Total porosity	n_t	0.30 (-)

There are mainly two parameters, related with dam wall, affecting the storage of water in the aquifer in Case 2-c. The first one is the hydraulic conductivity value of the dam wall. By decreasing the conductivity values for the dam wall the storage of water can be increased. Using less permeable materials can decrease the conductivity. In Chapter 2 different types of dam walls were explained. The choices for the material should be given considering economical factors, labor conditions, level of maintenance etc. for the site specific conditions.

The second parameter related with dam wall is the location of the dam wall. The location of the dam wall is used as given in the original report. Figure 5.10 shows the variation of water table elevation with the existence of dam wall in Case 2-c.

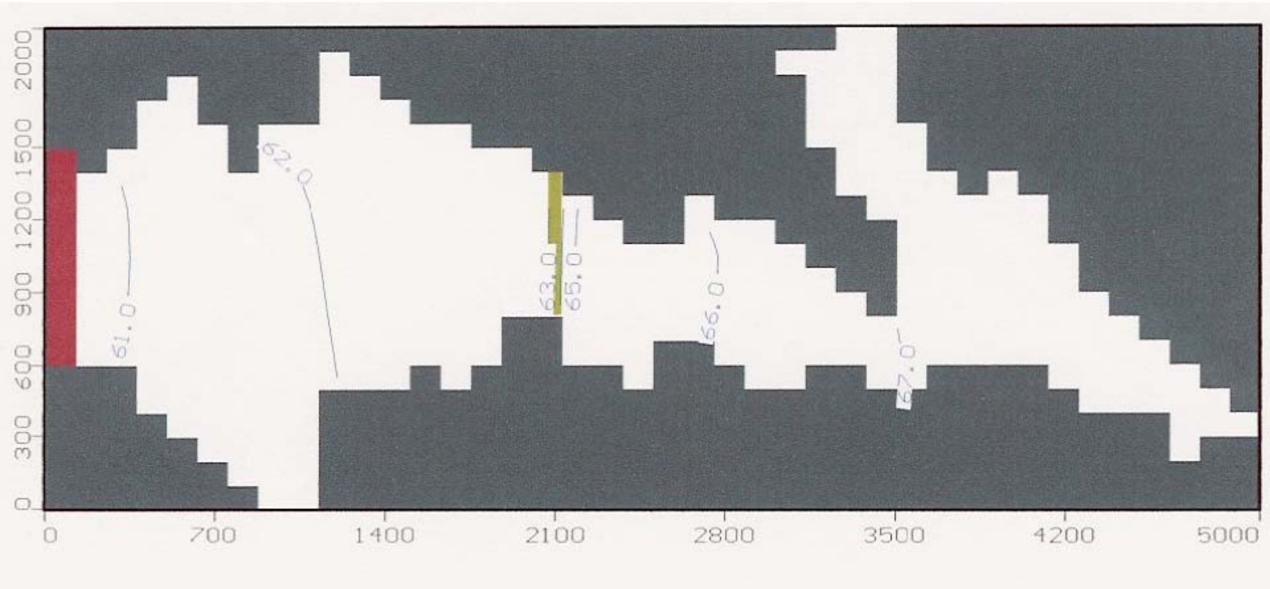


Figure 5.10 Case 2-c, top view of water table with dam wall

By moving the dam wall seaward the storage of water can be increased, but for the site-specific conditions the material used for the dam wall increases. Therefore this may not be economically feasible. However the value of water changes due to the need for water. Finally, for this case; the cross-section of the output after reaching steady state showing the variation of water table elevation in existence of dam wall is given in Figure 5.11. The sudden increase in head values due to the existence of dam wall can easily be seen in the figure.

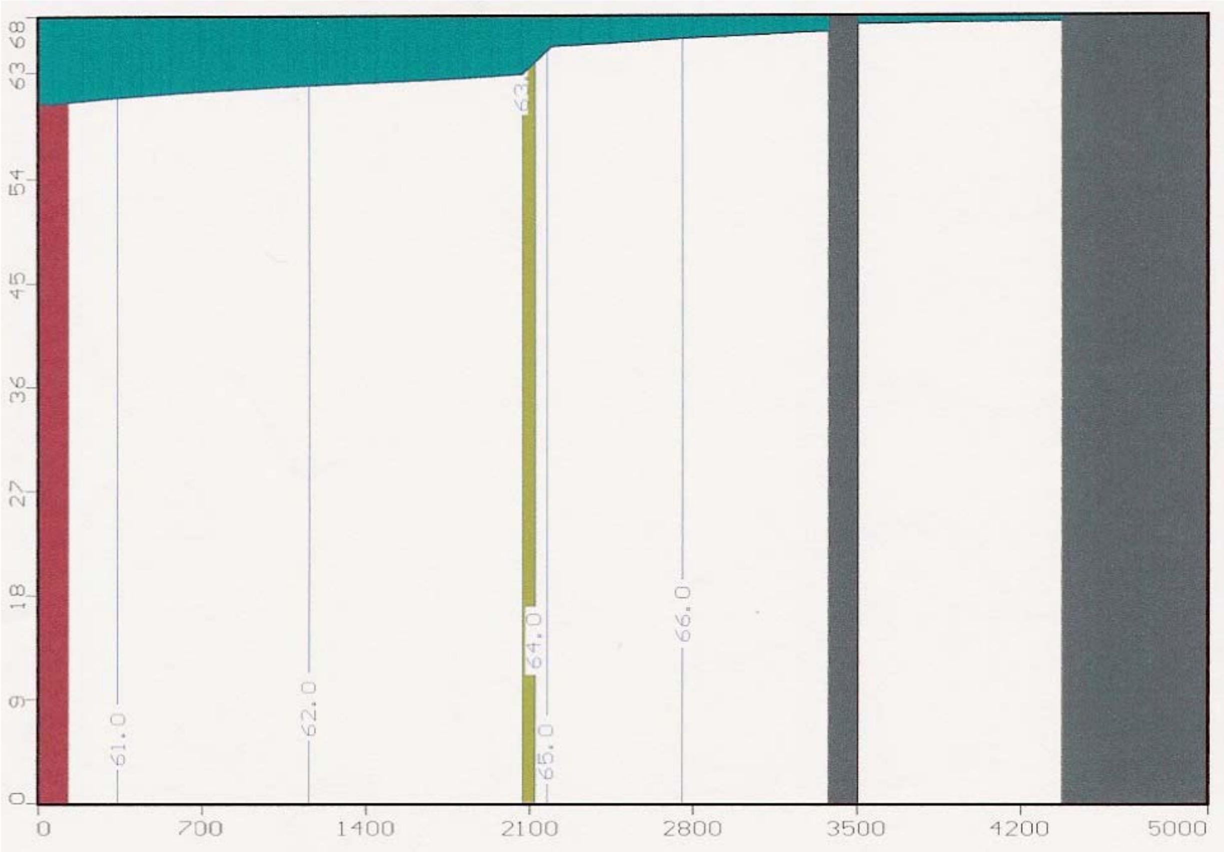


Figure 5.11 Case 2-c, cross-section of water table with dam wall

5.5 Case 2-d

Both dam wall and wells exist in this case and their combined effect can be analyzed. The inputs of Case 2-d are given in Table 5.5.

Table 5.5 Inputs of Case 2-d

Aquifer length	L	5000 m
Aquifer width	w	2000 m
Aquifer thickness	b	68 m
Dam wall thickness	t	10 m
Groundwater level	h_{gw}	66 m
Mean sea level	h_{sea}	60.5 m
Conductivity of soil	K_s	0.000424 m/s
Conductivity of dam	K_w	0.0000025 m/s
Recharge	R	0.0008175 m/day
Discharge from W_1	Q_1	-4302 m3/day
Discharge from W_2	Q_2	-4302 m^3/day
Specific storage	S_s	0.001 (1/m)
Specific yield	S_y	0.15 (-)
Effective porosity	n_{ef}	0.15 (-)
Total porosity	n_t	0.30 (-)

Q1=Q2=4302 m^3/day value is the maximum discharge that can be extracted in steady solution without controlling whether the water is fresh or not. If the wells are pumped at higher rates than 4302 m^3/day, drying occurs in the vicinity of the wells. When Q1=Q2=4302 m^3/day discharge is extracted from the wells, the variation of water table elevation in the aquifer, with the existence of dam wall and wells, occurs as shown in Figures 5.12 and 5.13.

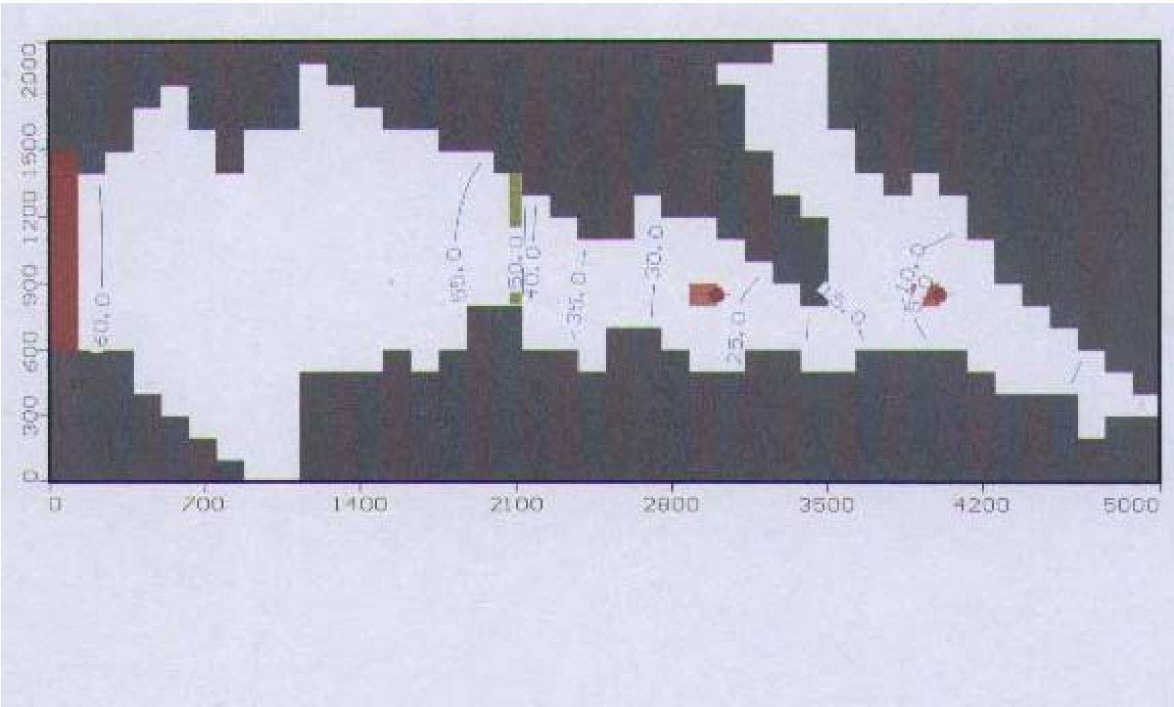


Figure 5.12 Case 2-d, top view of water table with dam wall and two discharging wells Q_1=Q_2=4302 m^3/day

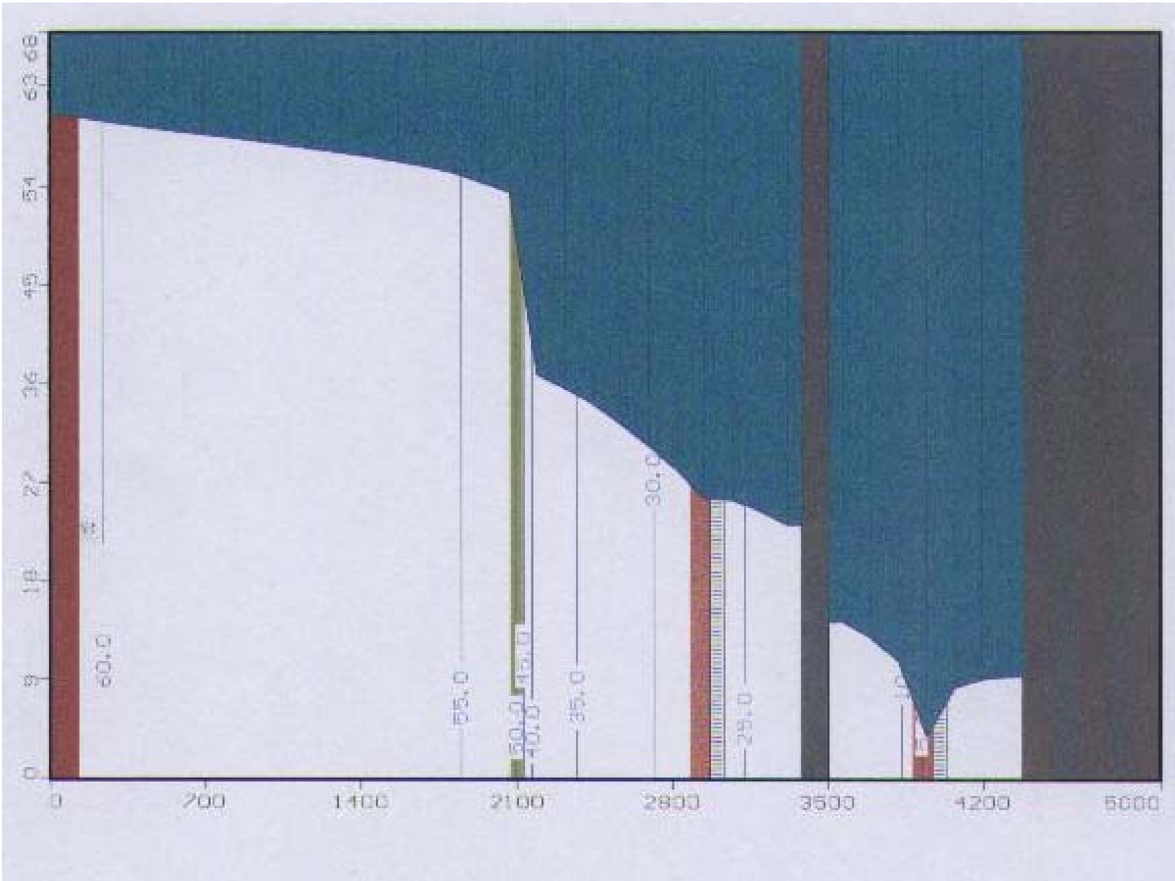


Figure 5.13 Case 2-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=4302 m^3/day

As it is seen in Figure 5.13, the head value on the dam wall is less than the constant head, which means there is a flow from sea to land. Therefore the 4302 m^3/day discharge value cannot be accepted as fresh water value.

The aim of this part is finding the maximum freshwater value that can be drawn by wells. By making trial and error solution, changing the discharge value the maximum freshwater is found as 1200 m^3/day for each well, 2400 m^3/day in total. This value is found by using the criterion that the head values on dam wall should not fall below constant head value. The variation of water table elevation in Case 2-d, with dam and with wells, when 1200 m^3/day extraction is made from each well till steady state is reached, is shown in Figure 5.14 and 5.15.

The maximum freshwater discharge with dam is 2400 m^3/day, and without dam is 1800 m^3/day, so there is 600 m^3/day increase in total discharge from two wells. This value is a very small value but consistent with sustainability. Decreasing the hydraulic conductivity, K_w of dam wall, can increase the freshwater storage.

Figure 5.14 Case 2-d, top view of water table with dam wall and two discharging wells Q_1=Q_2=1200 m^3/day

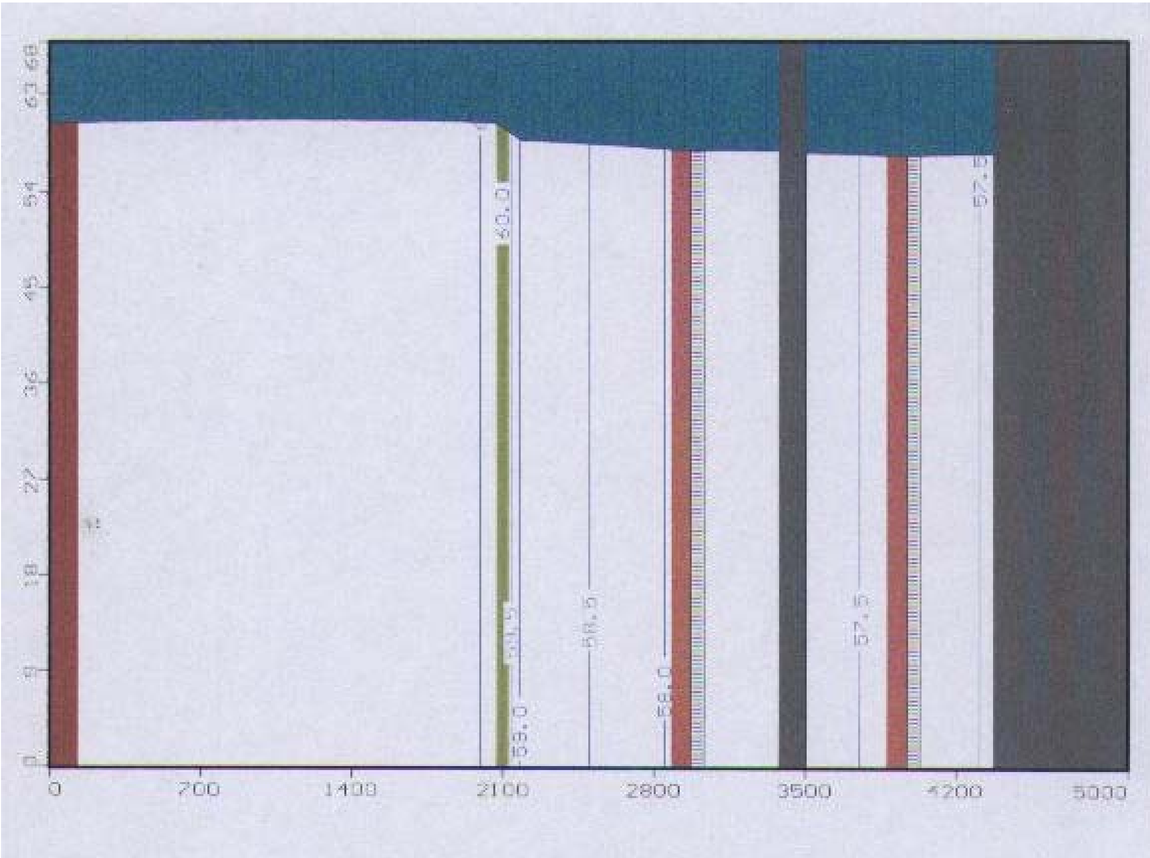


Figure 5.15 Case 2-d, cross-section of water table with dam wall and two discharging wells Q_1=Q_2=1200 m^3/day

5.6 Unsteady solution of Case 2

In unsteady solution the period that steady solution is reached, and the period that the aquifer can be benefited is found. The durations that will be found by unsteady solution are as follows:

5.6.1 Duration in which steady state is reached for Q=900 m^3/day in Case 2-b

Case 2-b is the case with wells and without dam. The maximum fresh water that could be drawn from the aquifer without allowing the head on wells fall below constant head is found 900 m^3/day for each well in steady solution. Since there are two wells total discharge is 1800 m^3/day. In unsteady solution the duration that the steady state is reached with this discharge value is found as 5100 days, which is nearly 14 years in MODFLOW. This means total fresh water volume that can be supplied from the aquifer without building dam wall is equal to 9.18×10^6 m^3.

5.6.2 Duration in which steady state is reached for Q=1200 m^3/day in Case 2-d

Case 2-d is the case with dam and with wells. $Q1=Q_2=1200$ m^3/day is the maximum fresh water that could be drawn from the aquifer with the given conditions. Total discharge is 2400 m^3/day for two wells. The period that the extraction can be made with $Q1=Q_2=1200$ m^3/day without falling below constant head on dam wall is found as 7000 days equal to 19.17 years. Total fresh water volume that can be supplied from the aquifer building dam wall is equal to 16.8×10^6 m^3. This means fresh water storage with given assumptions is increased by 7.62×10^6 m^3. In percentage there is 83 % increase in the storage of fresh water in the aquifer.

5.6.3 Duration in which steady state is reached for Q=0 in Case 2-d (preceded by Q=1200 m^3/day)

After using $Q_1=Q_2=1200$ m^3/day, extraction is stopped. The reservoir behind the dam is refilled by recharge in 5360 days.

To make a comparison between two steady state reaching situations in Case 2-d an arbitrary discharge value $Q_1=Q_2=3831$ m^3/day is selected. The period that steady state is reached using this discharge value is 9900 days and the filling period is 9000 days. For $Q1=Q_2=1200$ m^3/day the periods are 7000 and 5360 days respectively. When the ratios 9900/9000, which is 1,1 and 7000/5360,which is 1,3 are considered, using the less discharge value it is seen that aquifer is used more efficiently.

In Case 2 numerical computations are made on a more realistic basis than Case 1. The solutions are not expected to correspond one to one to the site because lots of assumed values are used as inputs.

CHAPTER 6

SUMMARY AND CONCLUSION

In this study, utilization of groundwater dams in the management of groundwater resources is analyzed using the computer code, MODFLOW. Two different case studies are provided. In the first case, an idealized rectangular aquifer is considered. In the second, the map of the potential groundwater dam construction site in Çamlı Köyü, Marmaris, Muğla is used. In this later case, some of the input data necessary for modeling were lacking. In order to overcome this shortage, a set of assumptions and appropriately estimated values are used. The approach and the conclusions are summarized as follows;

If the groundwater dam were not built, the recharged water would flow towards the constant head boundary, which is the sea. By preventing this seaward flow, additional water supply is provided. This is a contribution to the sustainable development.

The discharge values, which may be considered as the potential yield of the aquifer, found in Case 1 and Case 2 are small in amount compared to conventional methods, such as yield of surface reservoirs. Consequently, groundwater dams are to be considered as alternative or complementary solution to development of water resources.

Maximum discharge that can be extracted through the wells without causing drying in the vicinity of the wells can be found using MODFLOW. In addition to that, as it is applied in Case 2, MODFLOW can be used as a tool to find the increase in fresh water storage, with the given assumptions, by building the dam wall.

All of these approaches, which are applied in Case 1 and Case 2 for steady and unsteady solutions, are useful for the planning and design of groundwater dams.

LIST OF REFERENCES

Ahnfors, O., 1980, 'Groundwater arresting sub-surface structures'. Govt. of India, SIDA-assisted Groundwater Project in Noyil, Ponnan and Amavarati River Basins, Tamil Nadu and Kerala, Report 1:16, 22 p.

Akdeniz, U., 2003, 'Personal communication'. DSİ Geotechnical Services and Groundwater Department, Ankara

Archibugi F.; Nijkamp P., 1989, 'Economy and Ecology: Towards Sustainable Development'. Kluwe Academic Publishers, Dordrecht.

Beaumont, R.D.; Kluger J.W., 1973, 'Sedimentation in reservoirs as a means of water conservation'. IAHR Congress, Istanbul, 3-7 September 1973, pp. A28-1-A28-6

(BCEOM), 'Bureau Central d'Etudes pour les Equipements d'Outre-Mer 1978'Les barrages souterrains'. Ministerede la Cooperation. 135 pp.

Burger, S.W.; Beaumont, R.D., 1970, 'Sand storage dams for water conservation.' Convention: water for the future. Water Year 1970. Republic of South Africa, 9 pp.

Charbeneau, R. J., 2000, 'Groundwater Hydraulics and Pollutant Transport'. Prentice-Hall Inc., New Jersey .

Das R.K., 1998, 'Study of Groundwater flow with underground dam: Phuket Island'. Thailand. M. Eng. Thesis, Asian Institute of Technology, Bangkok, Thailand, No. WM97-2, 91 p.

Garagunis, C.N., 1981, 'Construction of an impervious diaphragm for improvement of a sub-surface water-reservoir and simultaneous protection from migrating salt water'. Bulletin of the International Association of Engineering Geology, No.24, pp 169-172

Hansson, G.; Nilsson, A., 1986, 'Groundwater dams for rural water supplies in developing countries'. Ground Water, Vol.24, No.4, July-August 1986, pp. 497-506

Kocabaş, İ., 2003, 'Personal communication'. DSİ Geotechnical Services and Groundwater Department, Ankara

Lowdermilk, W.C., 1953, 'Some problems of hydrology and geology in artificial recharge of underground aquifers'. In: Ankara Symposium on Arid Zone Hydrology Proceedings, UNESCO, pp. 158-161

Matsuo, S., 1975, 'Underground dams for control groundwater'. Publication No.117 de l'Association Internationale des Sciences Hydrologiuqes, Symposium de Tokyo (décembre, 1975)

Matsuo, S., 1977, 'Environmental control with underground dams'. Proceedings of the Speciality Session on Geotechnical Engineering and Environmental Control. Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, July 1977.pp. 169-182

McDonald, M.G. and A.W. Harbaugh, 1988. 'MODFLOW-A Modular Three Dimensional Finite-Difference Groundwater Flow Model'. USDI, U.S. Geological Survey, National Center, Reston, Virginia

Nilsson, A., 1988, 'Groundwater dams for small-scale water supply'. IT Publications, London. 64 pp.

Nissen-Petersen, E., 1982. 'Rain Catchment and water supply in rural Africa: a manual'. Hodder and Stoughton, Great Britain. 83 pp.

Pavlin, B., 1973, 'Establishment of sub-surface dams and utilization of natural sub-surface barriers for realization of underground storages in the coastal karst spring zones and their protection against sea water intrusion'. In. Trans. 11 th Int. Congress on Large Dams, Vol. 1, Madrid, pp. 487-501

Sargın, A. H., 2003, 'Personal communication'. DSİ Geotechnical Services and Groundwater Department, Ankara

Turfan, M., 2001. 'Benefits and Concerns about Dams'. ICOLD 69th Annual Meeting, Dresden Symposium

Wang, H.F., Anderson, 1982. M. P., 'Introduction to Groundwater Modeling'. W.H. Freeman and Company, San Francisco.

Wipplinger, O., 1958. 'The storage of water in sand'. South-West Africa Adminstration, Water Affairs Branch, 1958. 107 pp.