ARAB ACADEMY FOR SCIENCE, TECHNOLOGY AND MARITIME TRANSPORT
(AASTMT)
College of Engineering and Technology
Department of Construction and Building Engineering

GEOTECHNICAL ASSESSMENT OF DEWATERING IN CAIRO (CASE STUDY)

By
MAHMOUD MOHAMED KASSEM

A thesis submitted to AASTMT in partial fulfillment of the requirements for the award of the degree of

MASTER of SCIENCE
in
GEOTECHNICAL ENGINEERING

Supervised by

Prof. Fathalla M. El-Nahhas
Geotechnical Engineering & Foundation
Ain-Shams University

Assoc. Prof. Abd El-Moniem Sanad
Structural Engineering
Arab Academy for Science and Technology and Maritime Transport

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____

Prof. Dr. Mohamed Amer
Examiner

____

Prof. Dr. Ahmed Mslam
Examiner

____

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I wish to dedicate this work to my father, my mother, my sisters and my dear wife.

Geotechnical Assessment of Dewatering in Cairo (Case Study)

ABSTRACT

The impact of groundwater on underground construction projects can be enormous. It may affect the design of structure, construction method and cost. A high portion of claims and delays during construction are because of groundwater problems. Its presence may require redesign of structure or even abandoned in some cases.

The objective of this research is to narrow the gap between the design and construction of dewatering process in order to decrease cost and minimize time losses in construction industry. This can be achieved through monitoring the discharge and drawdown during construction from multiple wells and comparing them with the expected values from the empirical calculations and numerical analysis.

In this thesis, the case study is the dewatering system employed during construction of Wady El-Nile Hospital located north-east of Cairo. The site exists at an urban area which forced the contractor to monitor the drawdown and ground settlement during construction due to dewatering process. The project constructed on four stages to make the dewatering process more feasible and to reduce the groundwater table drawdown effect on the surrounding buildings. The dewatering system composed of twenty two deep well and fifteen piezometers.

The numerical analysis was conducted using a finite difference method program called MODFLOW. It is specialized in the analysis of the groundwater flow. This research focuses on feasibility of using such numerical technique for the design of dewatering systems.

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CHAPTER 1

INTRODUCTION

1.1 General

The effect of groundwater on some construction projects is significant. The existence of groundwater can affect the design, method of construction, construction period and overall project cost. A high portion of the encountered problems and construction claims are related to the groundwater effects. Some of these problems have led to redesign of some projects or even the abandoned of some of them.

Construction of buildings, dams, powerhouses and tunnels requires in some cases excavation below the water table. Such excavations require lowering the groundwater table below the slopes and excavation level to prevent raveling and to maintain dry working conditions for construction operation. In some cases, excavation bottom may be underlain by an impervious stratum affected by artesian pressure below it that if not reduced may burst the excavation base Bursting of excavation base may cause severe damage or boiling that destroys the soil bearing capacity.

"It‟s a state-of-art" this quotation said on the dewatering process and that is because of the many variables governing this process and many of them are just estimated with empirical equations. The designing methods also give approximate solutions that neglect some variables. It is important to convert the dewatering process from art to science. The inaccuracy of the design leads to errors in the construction because these designs may over or under estimate the required quantity of groundwater to be removed. This leads to time delays and money waste because of the site trials to reach the optimum dewatering system.

1.2 Objectives of the Research

Management and control of groundwater levels is a multi-discipline process requiring cooperative efforts from a number of specialists in geotechnical engineering, hydrogeology, hydrology, geochemistry, hydrochemistry..etc. Within urban environments, the geotechnical role becomes crucial in order to assess the implications of changing the groundwater levels and to propose and implement additional safety elements or measures. In many cases, control of groundwater levels is essential to prevent, or at least minimize, the expected detrimental effects on existing adjacent buildings and other structures (El-Nahhas et al., 2003).

The rise of groundwater levels in urban regions is becoming an alarming problem in recent years. Every new civil engineering project involving construction at elevations below water table most likely requires some measures of groundwater control. On the other hand, shallow foundations and underground floors of old existing buildings, which were constructed above the water table, may suffer of some detrimental attacks of groundwater and possible flooding due to the unexpected rise of water table. The demand for using deep wells has been increasing to lower the water table at the sites of new developments or within existing urban areas (Mossaad et al., 2000)

The design of a dewatering scheme should provide an estimate of the needed pumping capacity and the expected drawdown within and in the vicinity of the area under consideration. Capacity of the dewatering scheme is reflected on several items; such as the number, depth and spacing of the wells and the type and output of the selected submersible pumps. Prediction of the expected drawdown, taking into account the water-soil characteristics and the complex well-to-well interaction, is also considered one of the difficult tasks during the design stages (El-Nahhas et al., 1999).

Extensive reviews of the recommended guidelines of the analysis, design and construction of controlling groundwater are given by Mansur and Kaufman (1962), Somerville (1986) Preene et al. (2000) and Powers et al. (2007). The role of numerical techniques in back-analysis or predicting the performance of dewatering systems have been expanding during last two decades. Several attempts were made to utilize either the finite element or the finite difference methods for such purpose in Egypt; such as: Abdel-Karim (1992), Hassaneen (1998) and Samieh and Mahmoud (2009).

This research is considered as a step to narrow the gap between design and construction to reduce time and money waste. To achieve such a goal, dewatering project of a well monitored dewatering system was selected and analyzed and the analysis results were compared with the real drawdown readings and recorded discharge quantities of the used deep wells. A numerical analysis was performed using the finite difference program MODFLOW to estimate the water table profile after dewatering. Also, an analytical analysis was made using the empirical equation of Dupuit (1863), which is most commonly used on analyzing dewatering systems. In addition, the equation derived from the drawdown curve of the pumping test was used to estimate another groundwater profile.

The results of the analysis carried out in this study were compared to each other and to the in-situ drawdown values. It was evident that the coefficient of permeability is the most important variable in the analysis process. Therefore, a special part of the thesis was focused on the assessment of using different values of this coefficient on the performance of the dewatering system.

1.3 Organization of Thesis

Chapter 2: Includes brief reviews on the dewatering methods, analytical design methods, the definition of some important parameters and the numerical idealizations of groundwater flow.

Chapter 3: Examines the utilization of the analytical method in a detailed parametric study.

Chapter 4: The numerical analysis using MODFLOW was conducted in a detailed parametric study.

Chapter 5: Details of the experimental work are presented for the selected case study of Wady El-Nile Hospital Extension.

Chapter 6: Application of Finite Difference model to the pumping test and actual pumping of the case study.

Chapter 7: Conclusions of the research and recommendations for future studies.

CHAPTER 2

LITRATURE REVIEW

2.1 General

Construction of buildings, powerhouses, dams, locks, tunnels, and graving docks frequently requires excavation below the water table into water-bearing soils. Such excavations require lowering the groundwater table below the slopes and bottom of the excavation to prevent raveling or sloughing of the slope and to ensure dry and firm working conditions for construction operations.

2.2 Types of Dewatering Systems

Groundwater can be controlled by means of one or more types of dewatering systems. The choice of appropriate system depends on size and depth of the excavation, geological conditions, and characteristics of the soil. The following paragraphs describe the different types of dewatering systems.

2.2.1 Sumps and ditches

Sumps and ditches are performed for small projects in dense, well-graded or cemented soil. This method allow water to seep from side slopes and excavation bottom to be collected into the sumps and ditches then pumped out as shown in Figure (2.1).

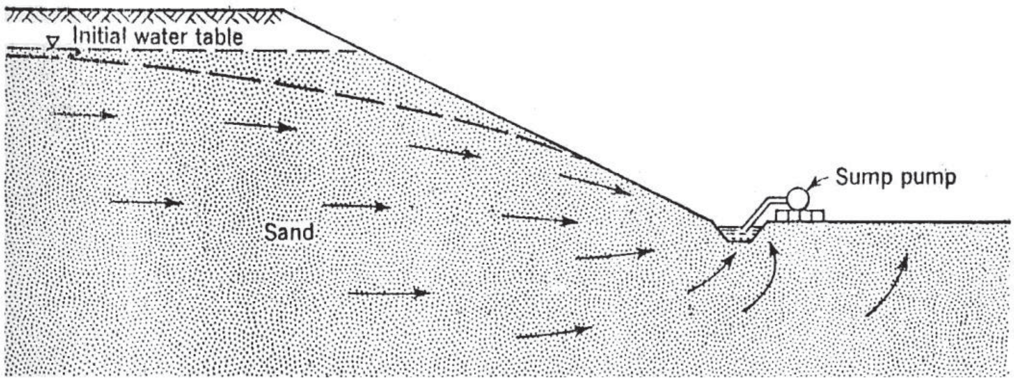


Fig. (2.1) *Sumps and Ditches (Leonards, 1963)*

Collection of seepage in sumps may result in softening and raveling of the lower part of the slope. Springs may also develop that causes erosion and settlement of the adjacent ground surface. Soil and slopes have to be drained before start of excavation; which may reduce the excavation rate. If the slope becomes too steep, the seepage will heavily increase that will raise the need to large ditches and more pumping capacity. Some of these problems might be decreased by stabilizing the excavation bottom and slopes with well-graded sand and gravel.

2.2.2 Sheeting and open pumping

Sheeting and open pumping is one of the first employed dewatering systems before introduction of sophisticated equipments. It is constructed by driving wood or steel sheet piling in the excavation and then pumping the water out of the excavation as shown in Figure (2.2).

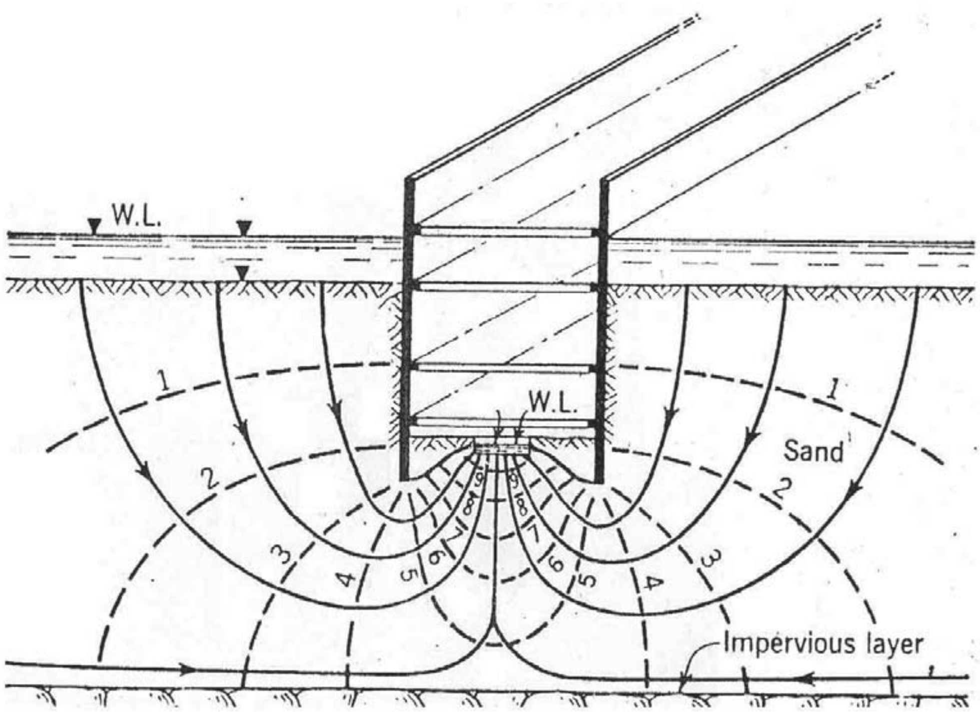


Fig. (2.2) *Sheeting and Open Pumping (Leonards, 1963)*

The system is designed based on seepage analysis into the excavation from the bottom. However, due to the difference of water levels between inside and outside of the excavation high pressure is applied on the sheeting and high seepage pressure will act on the soil causing loss of its shear strength and may cause bracing collapse. Another disadvantage is that the excavated soil is wet and hard to handle.

This method could be successful if the design of the sheeting and bracing take into consideration the resulting seepage forces and the expected loss of soil strength. Increasing the depth of sheets under the bottom of excavation and covering the bottom of the excavation with well-graded soil will decrease and facilitate the construction.

2.2.3 Wellpoints system

A wellpoint is a small well screen 50 to 80 mm diameter, 0.3 to 1 m long. manufactured from brass or stainless steel screen, its end is either closed or self jetting tips. It is commonly installed in lines or rings on one side of narrow excavation or on the two sides of large excavation. Spacing between each two well points is 1 to 4 m center to center. Each wellpoint is attached to a header pipe 0.15 to 0.30 m diameter that is connected to a wellpoint pump (combined vacuum or centrifugal pump). This setup is called wellpoint system and shown in Figure (2.3).

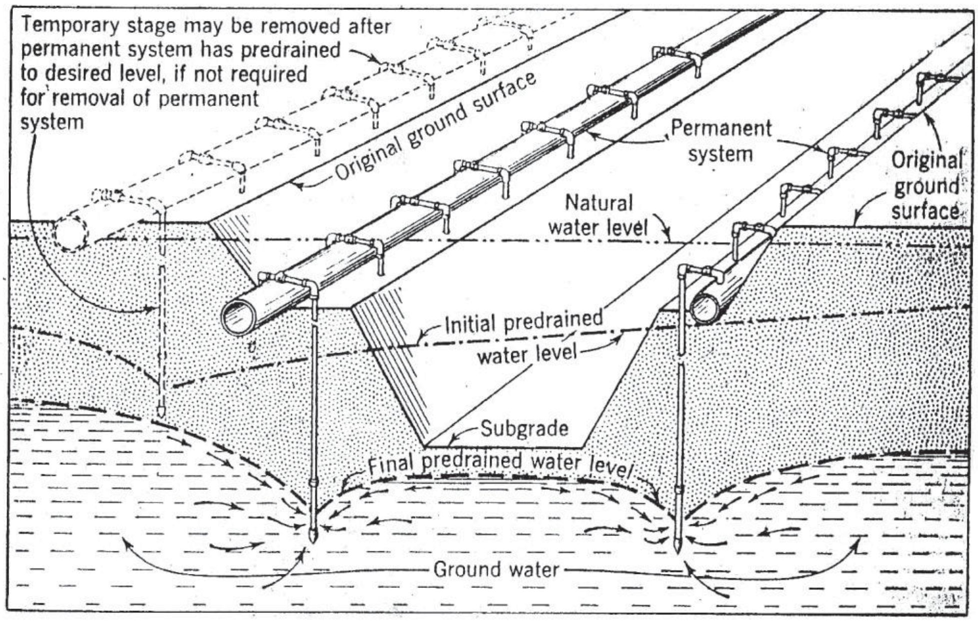


Fig. (2.3) *Wellpoint system configuration (Leonards, 1963)*

This system is one of the most common methods of dewatering for lowering water table for construction purposes. It is economic in dewatering shallow excavations not exceeding 6 to 7 m deep and where the water table to be lowered is near the bottom (Peurifoy, 1996).

Wellpoint systems might be used in deep excavations by installing rows of wellpoints on stages each about 3 to 5 m depth as illustrated in Figure (2.4).

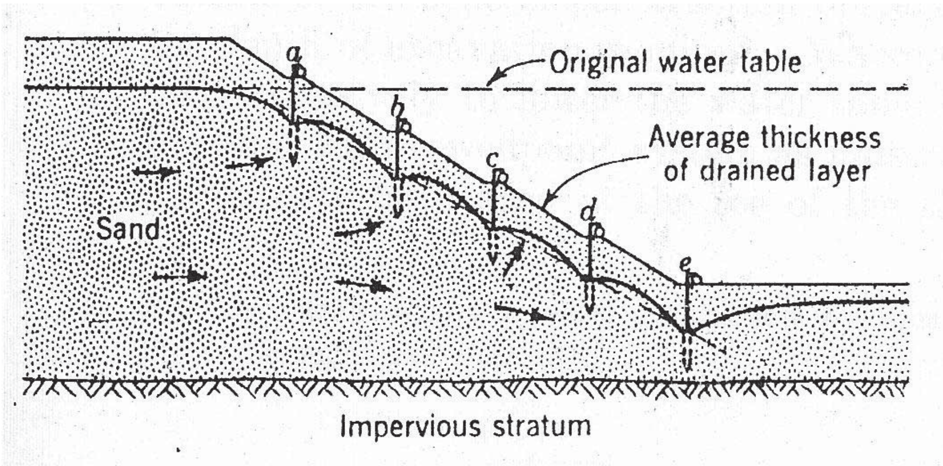


Fig. (2.4) *Multistage wellpoint system (Leonards, 1963)*

2.2.4 Deep wells drainage

This type of dewatering system is suitable for lowering groundwater table where the soil formation becomes more pervious with depth and when the groundwater table is to be lowered more than 7.5 m and successfully operated at depths greater than 100 m (Harris, 1994). A deep well dewatering system consists of deep wells with submersible pumps installed inside the wells. It may be used in combination with a well point system at the toe to intercept any minor seepage between the wells or at the surface in the stratified soil to drain the soil layer above any impermeable one as shown in Figure (2.5). By pumping from the deep wells at the top of the excavation, Figure (2.5), the stability of the slope will not be affected because the seepage is intercepted before affecting the stability of the slopes.

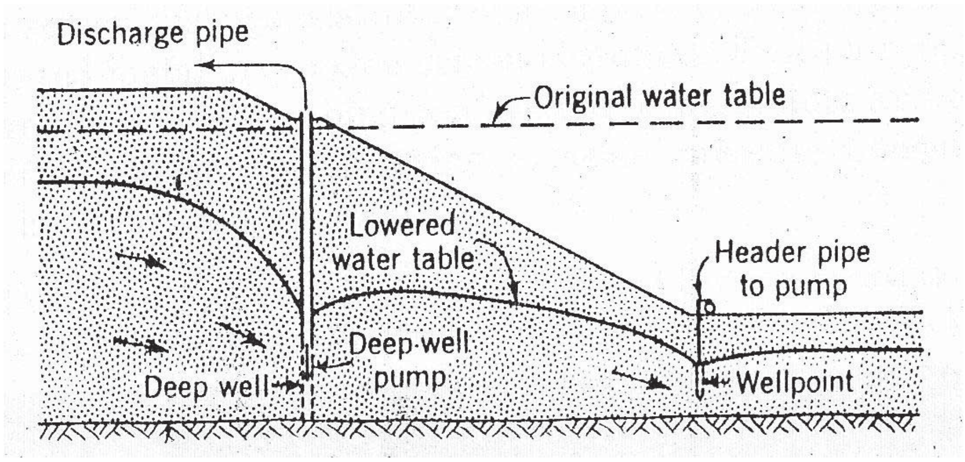


Fig. (2.5) *Combination of Deep well with Wellpoint systems (Leonards, 1963)*

If the bottom of large excavation is underlain by impervious strata that in turn exposed to excessive artesian pressure, then its stability has to be checked and the effective stress at the bottom of the impervious layer must be greater than the artesian pressure. If it is not safe, it may lead to heave or blow of the impervious layer accompanied by soil boiling. Such situation could lead to costly delay of the project, weakness of the soil and may lead to reconsideration of the type of foundation suitable for the structure. Relief of the artesian pressure can best be done by means of deep wells installed out of the excavation area as illustrated in Figure (2.6).

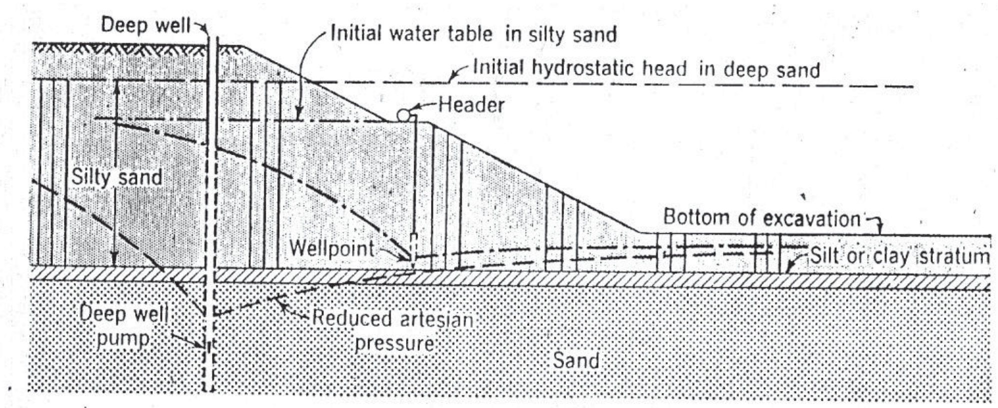


Fig. (2.6) *Reducing artesian pressure using Deep well (Leonards, 1963)*

Deep wells are usually spaced depending on perviousness of the soil layer, source of seepage and amount of submergence available. Such wells commonly have a diameter of 0.3 to 0.50 m. with screen from 6 to 25 m. The screen may consists of a commercial type of water well screen or perforated metal surrounded by geotextile and properly graded sand-gravel filter.

2.2.5 Horizontal drainage

This system is used to avoid open cut work and if submergence is inadequate for deep wells (Leonards, 1963). It consists of number of horizontal perforated pipes projected from one or more reinforced concrete shafts or wells. Sometimes well points are utilized and installed inclined or horizontally. These pipes may be extended 60 m or more in any direction. Groundwater flowing into the well is usually pumped out by means of a turbine pump. This type of system is not suitable for lowering water table in stratified soil.

2.2.6 Ejector systems

Ejector system is designed such that a single pumping station powers multiple wells instead of individual jet pumps at each well. The ejector system, therefore, requires a significance amount of supply and return piping that is not necessary in the individual jet pump arrangement.

The ejector, self illustrated in Figure (2.7), is simply a nozzle and venture device arrangement that is used to lift or suck water from deep well casing, a wellpoint, or even a sump. Ejectors are typically used where the groundwater must be lowered more than 4.5 m and the hydraulic conductivity of the soil is relatively low.

Ejectors have advantages over the other predrainage methods. Unlike wellpoints they are not limited to the 4-5m of suction lift, i.e. multiple stages are unnecessary. The unit cost of ejectors is slightly less than that of deep wells so that they can be used economically on close spacing when the soil condition requires. Also, the operation of manning requirements are less demanding for a system powered by one centrally located pumping station. However, ejectors have also some disadvantages. For instance, they are inefficient in dewatering and maintenances intensive.

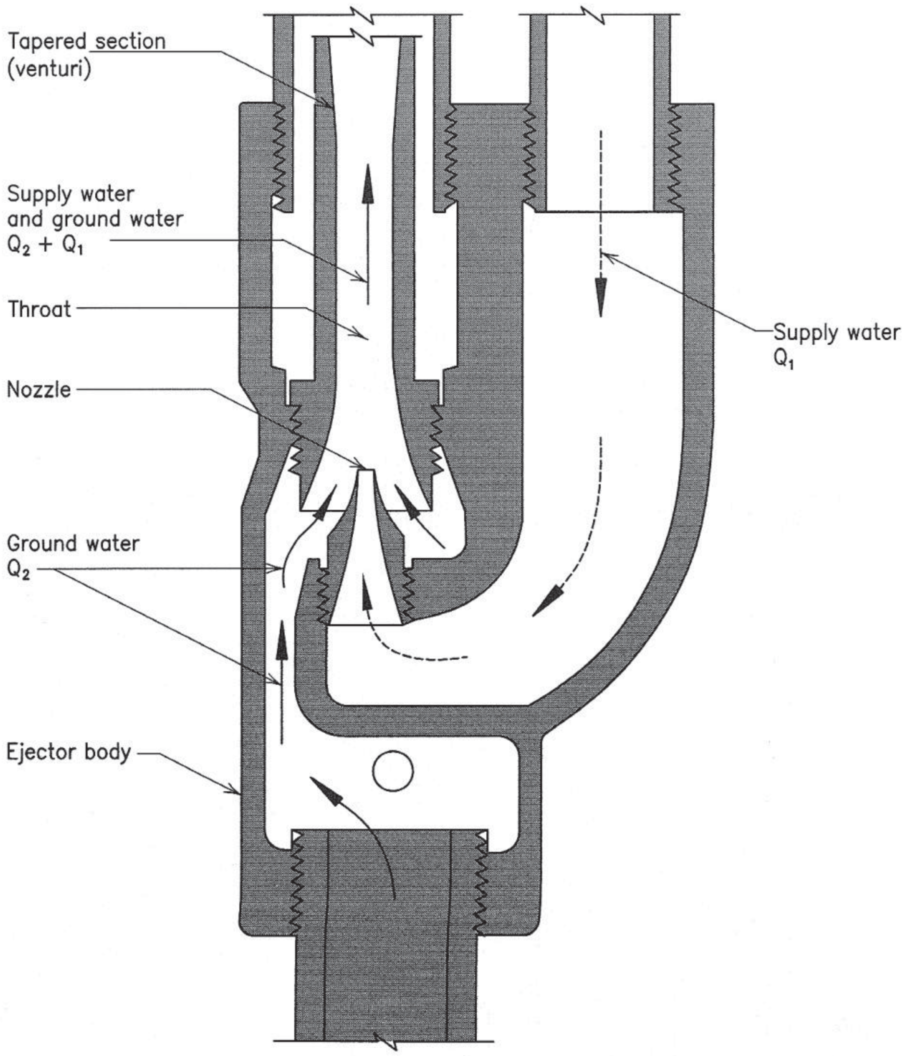


Fig. (2.7) *Ejector pump (Powers et al., 2007)*

2.2.7 Drainage by electroosmosis

Conventional dewatering processes are probably not efficient in dewatering types of soils such as silts, clayey silts, and fine clayey silty sands. However, these soils can be drained by wells or wellpoints combined with the flow of electricity through the soil to the wills. This method of dewatering is called "Electroosmosis" or electrical drainage method. The application of electroosmosis to dewatering of soils was developed by Casagrand (Casagrand, Leo. 1952).

If two electrodes are driven into saturated soil and a direct electric current is passed between them, water contained in the soil will migrate through the soil from the positive electrode (anode) to the negative electrode (cathode). Figure (2.8a) shows the groundwater behavior using Electroosmosis and Figure (2.8b) shows the groundwater behavior in the natural conditions. By making the cathode a well, the water can be removed by pumping. In this manner, water in the soil that otherwise would tend to seep toward the excavated slope and reduce the stability of the soil mass will flow instead toward the wells, thereby increase the shear strength of the soil and stability of slope.

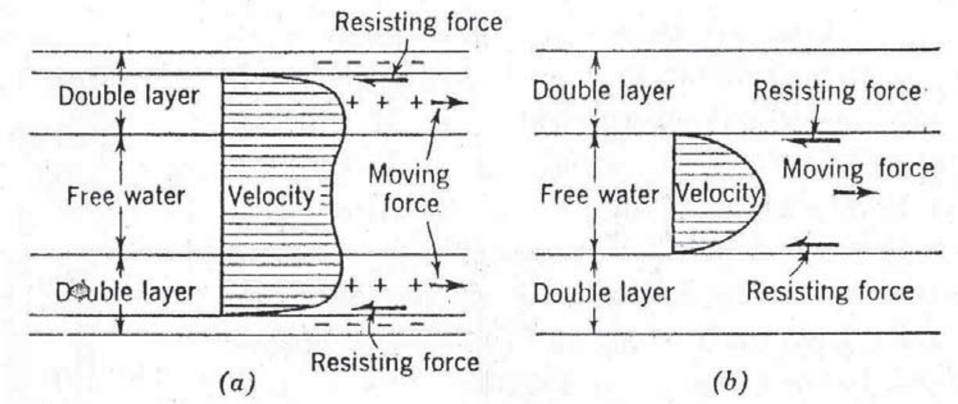


Fig. (2.8) *Groundwater behavior using Electroosmosis (Leonards, 1963)*

2.3 Analytical Methods

The estimation of the quantity of flow Q for any dewatering system can be calculated using closed-form formulas. These formulas could give an acceptable estimation for the dewatering system if they have been applied with judgment and when the values assumed for the variables are appropriate. These formulas are called analytical models and the aquifers conditions should be simplified. In complex aquifer situations, solution by numerical groundwater models will be more reliable.

Use of the analytical formulas is based on the assunption that the steady state has been reached, i.e. that the pumping has been continued until its zone of influence has expanded to where it has intercepted sufficient recharge from other source.

2.3.1 Flow to a drainage trench from a line source:

In some cases, it is required to calculate the flow from line source to a parallel drainage trench. Figure (2.9) illustrates a trench of infinite length fed from a line source on one side. Equation (2.1) can be used to calculate the flow from one side of a trench per unit length.

$$\frac{Q}{x} = \frac{kB(H-h)}{L} \qquad (2.1)$$

Where:

Q = Quantity of flow (m^3/hr)

x = Unit length of trench (m)

k = Coefficient of permeability (m/hr)

B = Aquifer thickness (m)

H = Water head at the source (m)

h = Water head at the trench (m)

L = Distance from the line source (m)

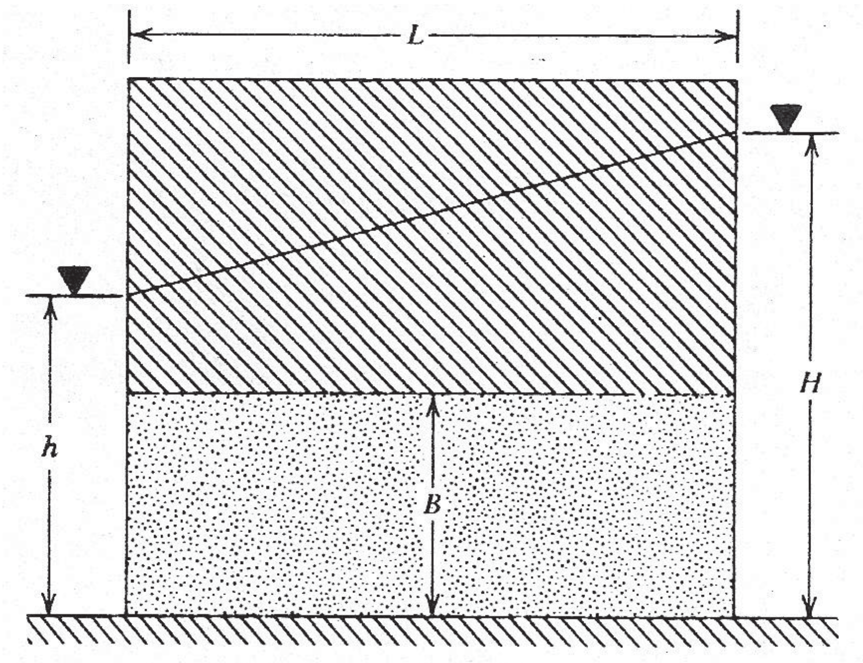


Fig. (2.9) *Flow from a single line source to a drainage trench of finite length in confined aquifer (Powers et al., 2007)*

For water table aquifer Figure (2.10).

$$\frac{Q}{x} = \frac{k(H^2 - h^2)}{2L} \qquad (2.2)$$

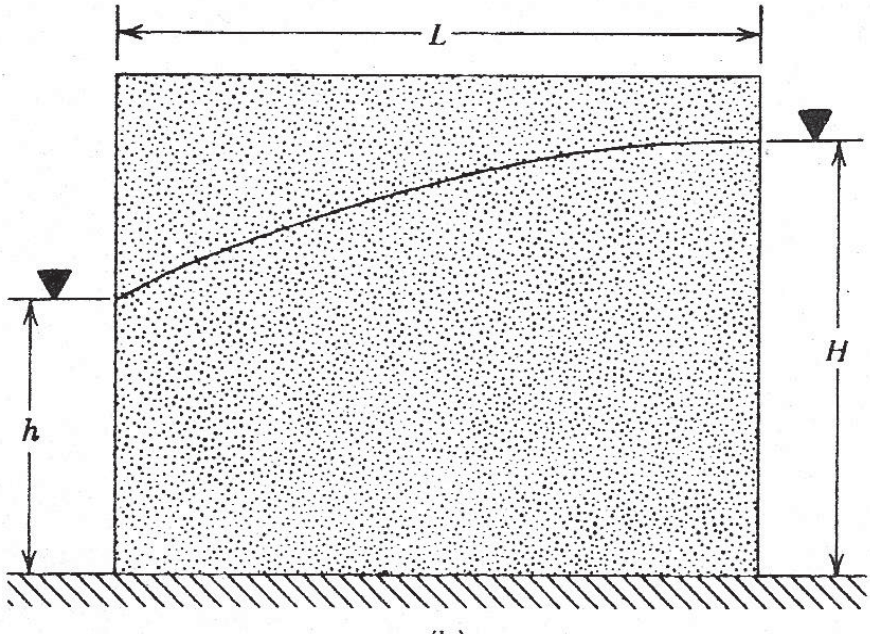


Fig. (2.10) *Flow from a single line source to a drainage trench of finite length in water table aquifer (Powers et al., 2007)*

2.3.2 Radial flow to a well in a water table aquifer:

Predicting the flow in water table aquifer, Figure (2.11), is more difficult than that of the confined aquifer since the saturated thickness and the transmisivity decreases as we approach the well. However, with the simplifying assumptions of Dupuit, a good approximated results is obtained. The flow-draw down relationship is given by (Misstear et al., 2006):

$$Q = \frac{\pi k(H^2 - {h_w}^2)}{ln\, ^{R_0}/_{r_w}} \qquad (2.3)$$

Where:

Q = Well discharge (m^3/hr)

R_o = Radius of influence (m) to be calculated from eqn. (2.10)

r_w = Well radius (m)

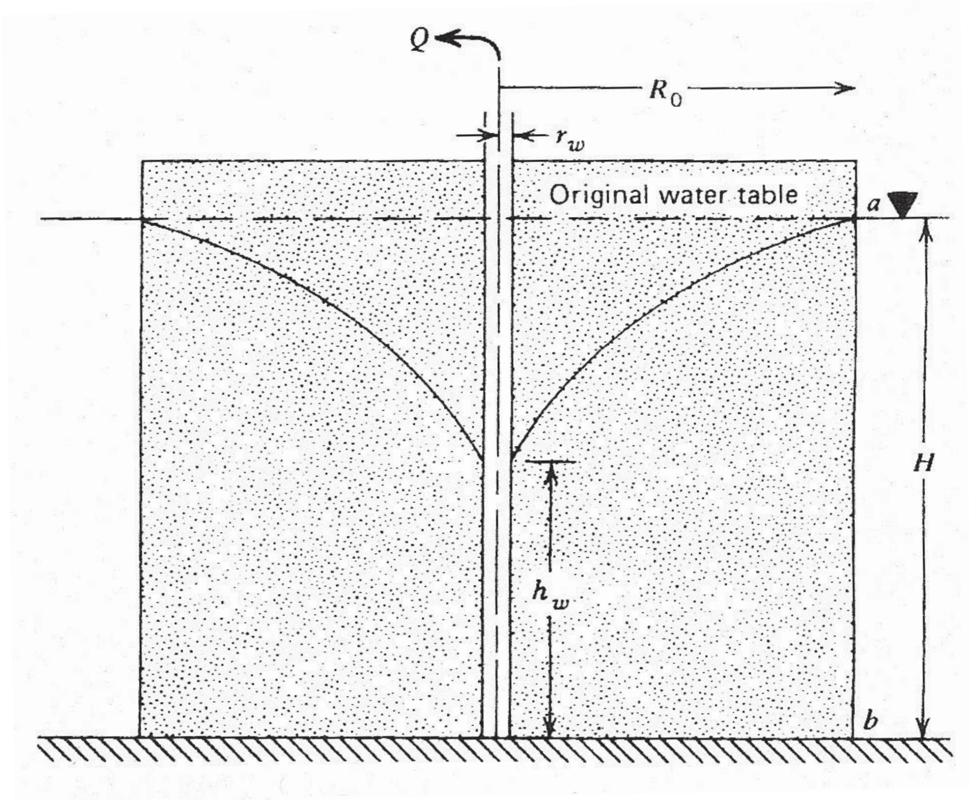


Fig. (2.11) *Equilibrium radial flow to a frictionless well in a water table aquifer (Powers et al., 2007)*

2.3.3 Radial flow to a well in a mixed aquifer:

Sometimes partial dewatering and pressure relief of a confined aquifer may be necessary as illustrated in Figure (2.12). For this conditions, the equation (2.4) can be used in analyzing groundwater flow:

$$Q = \frac{\pi k(2BH - B^2 - h_w^2)}{ln\, {}^{R_0}/_{r_w}} \qquad (2.4)$$

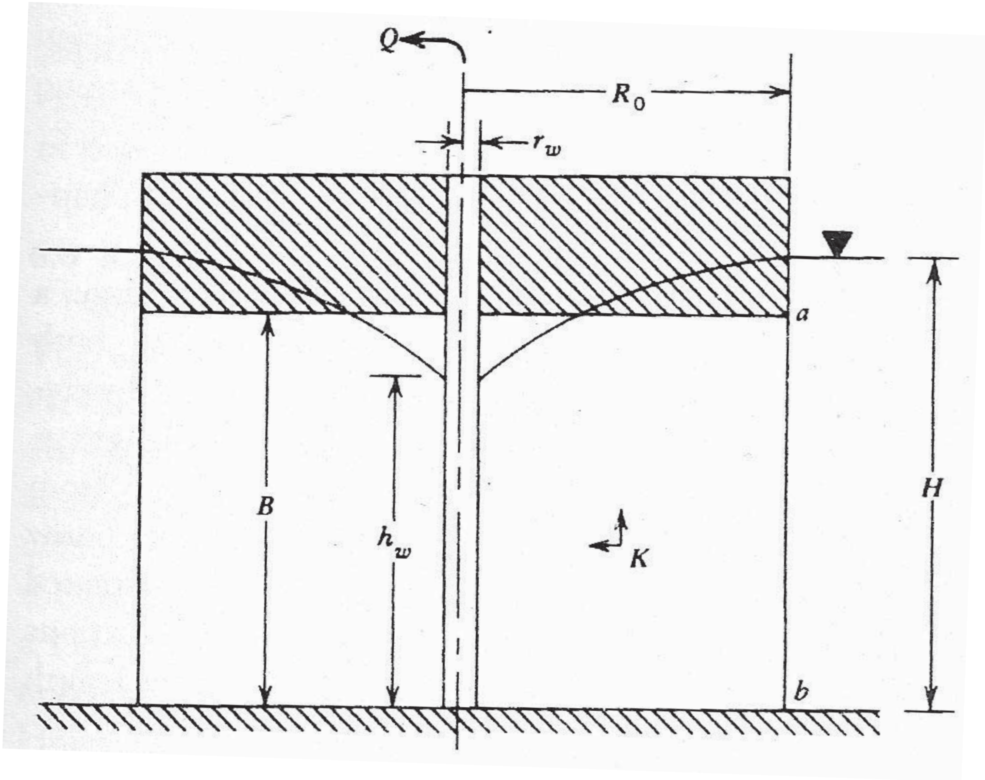


Fig. (2.12) *Equilibrium radial flow to a frictionless well in a mixed aquifer (Powers et al., 2007)*

2.3.4 Radial flow to a well in a confined aquifer:

Figure (2.13) illustrates pumping from a confined aquifer. The initial and final groundwater table is within the impermeable layer. This case is called radial flow from confined aquifer. The quantity of flow at this case can be calculated by the following Equation (2.5);

$$Q = \frac{2\pi kB(H - h_w)}{ln\, {}^{R_0}/_{r_w}} \qquad (2.5)$$

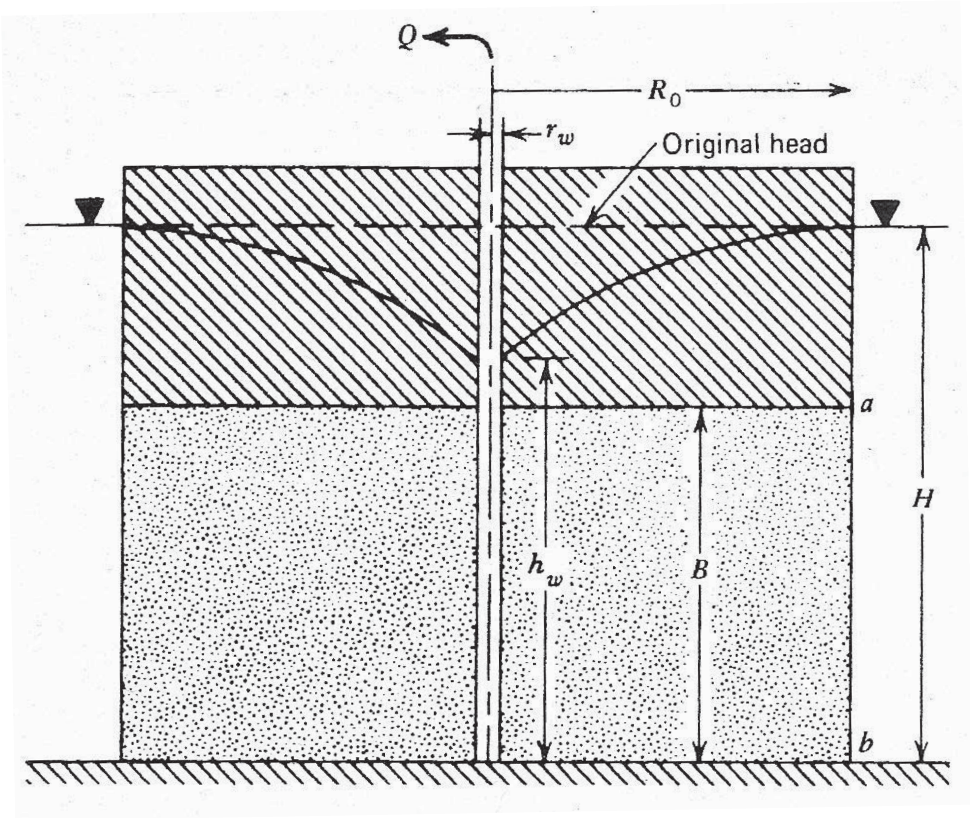


Fig. (2.13) *Equilibrium radial flow to a frictionless well in a confined aquifer (Powers et al., 2007)*

2.3.5 The system as a well: Equivalent radius r_s:

When a set of wells is arranged in a rectangular or circular shape as shown in the Figure (2.14). Equivalence can be made with one single well having the same effect on drawdown. The well equivalent radius can be calculated by the Equation (2.6):

$$r_s = \sqrt{\frac{ab}{\pi}} \qquad (2.6)$$

If the wells are spaced closely and R_0 is great with respect to r_s and the ratio between *a/b* is less then 1.5, the actual *Q* will be larger than the estimated for the equivalent well *(Powers et al., 2007).*

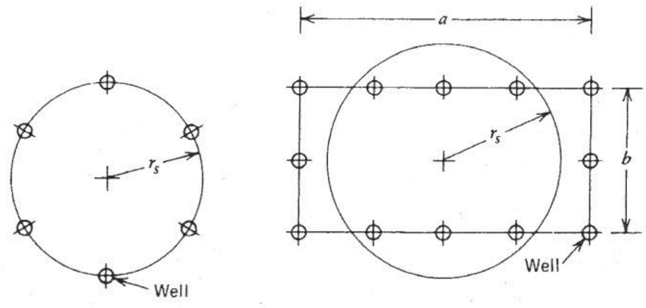


Fig. (2.14) *Approximation of equivalent radius r_s for circular and rectangular system (Powers et al., 2007)*

For long narrow systems, where the ratio a/b is large like trenches (Figure 2.15) the flow from line source in the confined aquifer can be calculated from equation:

$$Q = \frac{2\pi kB(H-h)}{ln\, {}^{R_0}/_{r_s}} + 2\left[\frac{xkB(H-h)}{L}\right] \qquad (2.7)$$

For water table aquifer:

$$Q = \frac{\pi k(H^2-h^2)}{ln\, {}^{R_0}/_{r_s}} + 2\left[\frac{xk(H^2-h^2)}{2L}\right] \qquad (2.8)$$

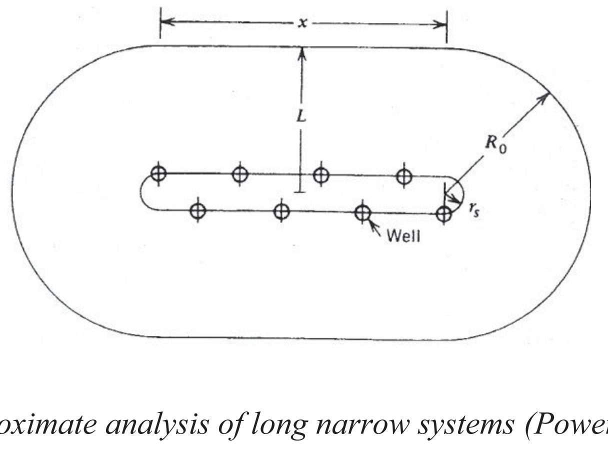


Fig. (2.15) *Approximate analysis of long narrow systems (Powers et al., 2007)*

2.3.6 Radius of influence R_0:

The radius of influence R_o is defined as the maximum distance of influence of a system of wells measured from the center of the system. The radius of influence could be estimated by adapting Jacob formula (Cooper and Jacob, 1946):

$$R_0 = \sqrt{\frac{Tt}{CC_s}} \qquad (2.9)$$

Where:

T = Transmisivity (see section 2.4.3) (m^2/sec)

t = Time since pumping started (min)

C_s = Storage Coefficient (see section 2.4.3)

$C = \frac{3.05\times10^{-6}Q}{\Delta\delta}$

$\Delta\delta$= drawdown difference per log cycle (m)

Empirical relationship developed by Sichardt and Kyrieleis (1930):

$$R_0 = 3000(H-h)\sqrt{k} \qquad (2.10)$$

In many cases, the source of water is estimated to be a line source at a distance L from the center of the system. A line source will produce the same flow to a well as a circular source at twice the distance *(Powers et al., 2007)*:

$$R_0 = 2L \qquad (2.11)$$

2.4 Factors Affecting the Groundwater Flow

Water flows from the source of supply to the well following a non-linear path. The definition of this path depends on the soil properties; depth of aquifer; location of the impermeable layer. The main soil properties needed for this calculation are the coefficient of Permeability (k), Transmisivity (T) and Storage Coefficient (C_s).

2.4.1 Coefficient of Permeability (*k*)

The hydraulic conductivity is defined as the flow through a unit area of aquifer. It can be estimated at the laboratory and have a units of m/sec.

The coefficient of permeability is the most important variable in the design of dewatering process for any project. There are three methods to estimate the coefficient of permeability. Empirical formula, analytical analysis of pumping tests and numerical analysis of pumping tests. In the next paragraphs some details are given for these methods.

2.4.1.1 Empirical formula

Permeability of granular soils can be estimated from grain size distribution curves by empirical methods. Hazen (1892) relates the permeability to the D_{10} size of the soil from the grain size distribution curve (Hazen, 1892).

$$k = CD_{10}^2 \qquad (2.12)$$

Where C is a calibration factor that may vary between 0.007 and 0.017.

D_{10} is the particles diameter corresponding to ten percent finer in (mm).

k is the coefficient of permeability in (cm/sec).

$$k_{average} = \frac{C\sum_{i=1}^{n} D_{10}^2}{n} \qquad (2.13)$$

n is the number of samples

The results obtained from Hazen relationship may be a reasonable approximation if the samples are representative. There are some limitations of Hazen equation. For instance, it does not consider the uniformity and density of the soil. Hazen used very uniform soils to work as a filter, which was his field of interest.

2.4.1.2 Pumping test

When the groundwater has a major effect on any project, the pumping test becomes the best approach for obtaining the aquifer parameters such as Transmisivity, Storativity, Coefficient of Permeability, capacity of wells and other factors that will determine the scope and cost of the dewatering work (*Powers et al., 2007*). If the aquifer at the site is confined, the field Coefficient of Permeability for can be determined using the Dupuit Equation, considering fully penetration conditions (Dupuit, 1863):

$$k = \frac{Q}{2\pi H(h_2 - h_1)} ln\frac{r_2}{r_1} \qquad (2.14)$$

Where Q = Well discharge

H = Aquifer thickness

h_1 = Drawdown in the first piezometer

h_2 = Drawdown in the second piezometer

r_1 = Distance from well to first piezometer

r_2 = Distance from well to second piezometer

In case of using the production well as a piezometer:

$$k = \frac{Q}{2\pi H(h_1 - h_w)} ln\frac{r_1}{r_w} \qquad (2.15)$$

Where h_w = Draw down at the well

r_w = radius of well

A correction factor "G" for the surcharge "Q" is applied in case of partially penetrated aquifer. The factor G is given by:

$$G = \left[1 + 7\sqrt{\frac{r_w}{2H_1}} \cos\frac{\pi\alpha}{2}\right]\alpha \qquad (2.16)$$

Where H = Full depth of confined aquifer

H_1 = Depth up to which the well penetrates.

$\alpha = H_1/H$

In such a case equation (2.15) becomes

$$k = \frac{Q}{2\pi HG(h_1 - h_0)} ln\frac{r_1}{R_0} \qquad (2.17)$$

2.4.1.3 Numerical analysis

Determining the Coefficient of Permeability using a numerical model is carried out by a process called the calibration of on the data collected through the pump test or back analysis of these data. By entering all the test constants collected through the soil exploration and pump test parameters and changing the Coefficient of Permeability till reaching the observed groundwater levels of the test.

2.4.2 Transmisivity (*T*)

The Transmisivity is defined as the flow of water through a unit width of the aquifer Figure (2.16). Transmisivity is a very important factor in determining the quantity of flow to be pumped out in a dewatering project. It could be calculated by knowing the Hydraulic Conductivity (or Coefficient of Permeability) k from the equation $T = kB\ m^2/sec$, where B is the aquifer depth.

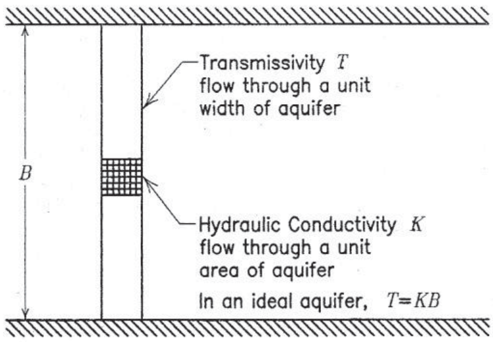


Fig. (2.16) *Transmisivity and hydraulic conductivity (Powers et al., 2007)*

2.4.3 Storage Coefficient (C_s)

The Storage Coefficient is defined as the volume of water released from storage, per unit area per unit reduction in head. Sometimes in water table aquifers, the Storage Coefficient is referred to as Specific Yield. Fine sand and silty sand will have Specific Yield much lower than that for coarse clean sand and will take much more time to reach these yields by gravity drainage. Figure (2.17) shows the relation between Porosity, Specific Yield (Storage Coefficient) and Specific Retention.

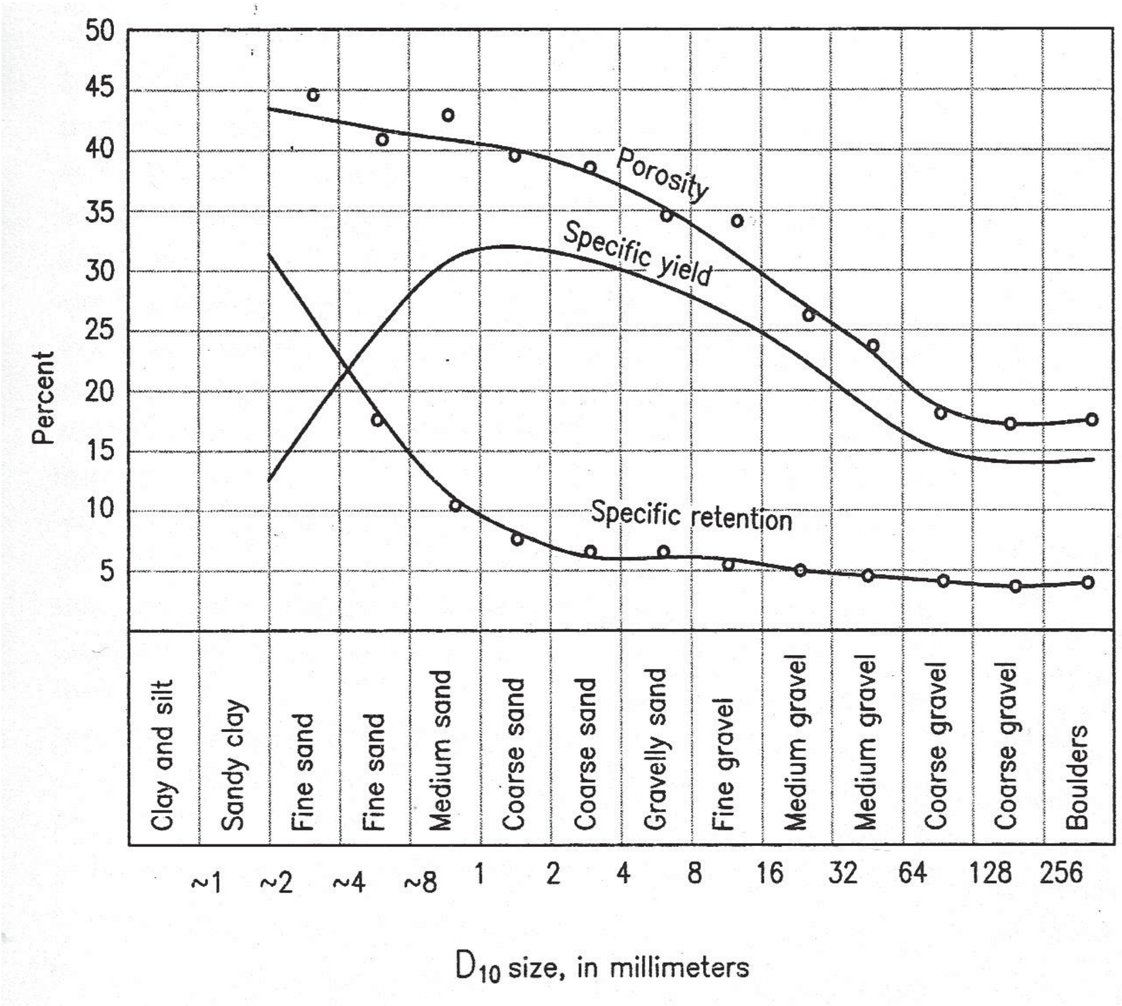


Fig. (2.17) *Relation between porosity, specific retention and Specific Yields (Storage Coefficient) for various soils (Powers et al., 2007)*

The pumping test results can be used to calculate the Transmisivity and Storage Coefficient (Storativity). By plotting the drawdown versus time for each piezometer, the Transmisivity and Storativity can be estimated by Equations (2.18) and (2.19) *(Jacob, 1950).*

$$T = \frac{3.05\times10^{-6}\times Q}{\Delta\delta} \qquad (2.18)$$

$$C_S = \frac{135\ T\ t_0}{r^2} \qquad (2.19)$$

Where T = Transmisivity (m^2/sec)

Q = Quantity of flow (L/min)

$\Delta\delta$ = change in draw down per log cycle (m)

C_S = Storativity Coefficient

t_0 = zero draw down intercept (min)

r = radius of the piezometer from the centre of the pumping well (m)

By plotting the drawdown versus distance at time t for each piezometer, we can get the Radius of Influence (R_0) (Equation 2.9) as well as the Transmisivity and Storativity by Eqs. (2.20) and (2.21).

$$T = \frac{6.1\times10^{-6}\times Q}{\Delta\delta} \qquad (2.20)$$

$$C_S = \frac{135\ T\ t_0}{{R_0}^2} \qquad (2.21)$$

The results from the two graphs should be, theoretically, equal.

2.5 Definition of the ideal aquifer

The main characteristics of an ideal aquifer can be summarized as follows:

1. It is homogeneous and extends horizontally in all directions without encountering recharging or barrier boundaries.
2. Thickness is uniform through out the entire aquifer.
3. It is isotropic; hydraulic conductivity has the same value in the horizontal and vertical directions.
4. Water is instantaneously released from storage and steady state is reached when the head is reduced.
5. The pumping well is frictionless, very small in diameter and fully penetrates the aquifer.

In confined aquifer (typically a sand layer between two clay layers), storage water is released rapidly enough to approximate the required “instantaneous release”.

2.6 Groundwater Modeling Using Numerical Methods

Numerical analysis was recognized at mid-1960s but due to mathematical complexity its use was limited. Nowadays, due to presence of powerful personal computers, numerical models has became invaluable tools. Well documented and extensively tested software like MODFLOW developed by U.S. Geological survey are available. (*Powers et al., 2007*).

Two main approaches are actually used to model the groundwater flow, finite element and finite difference methods. The two methods are different in their principles and each one has its advantages and its disadvantages. A brief description of both methods is given in the following paragraph.

2.6.1 Basic equations of steady flow

Flow in a porous medium can be described by Darcy's law. Considering flow in a vertical *x*-*y*-plane the following equations apply:

$$q_x = -k_x \frac{\partial \emptyset}{\partial x} \qquad q_y = -k_y \frac{\partial \emptyset}{\partial y} \qquad (2.22)$$

The equations express that the specific discharge, q, follows from the permeability, k, and the gradient of the groundwater head. The potential head, ϕ, is defined as follows:

$$\emptyset = y - \frac{P}{\gamma_w} \qquad (2.23)$$

Where y is the vertical position, p is the stress in the pore fluid (negative for pressure) and γ_w is the unit weight of the pore fluid. For steady flow the continuity condition applies:

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \qquad (2.24)$$

Eq. (2.24) expresses that there is no net inflow or outflow in an elementary area, as illustrated in Figure 2.18.

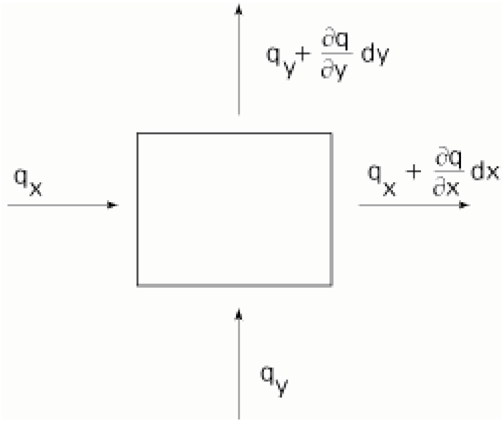


Fig. (2.18) *Illustration of continuity condition*

2.6.2 Finite element for seepage analysis

It is known that when applying the finite element method in the structural problems we have:

$$\{P\} = [K_s] \,.\, \{\Delta\} \tag{2.25}$$

Where: P is the nodal load vector, Δ is the nodal displacement vector and K_s is the overall structure stiffness matrix modified to allow for the restraints of some nodes.

To apply the same concept on the groundwater problems we will have:

$$\{Q\} = [K] \,.\, \{H\} \tag{2.26}$$

Where: Q is the discharge vector at nodes, H is the head vector at the nodes and K is the permeability matrix in the region of flow.

The permeability matrix and the discharge vector are then modified to satisfy the boundary conditions.

2.6.2.1 Formulation of element permeability matrix

The simplest way of approximating the variation of the potential head within a triangular element is by assuming that the head varies linearly within each element. The groundwater head at a point inside an element is defined by linear interpolation between the values at the mesh points (the nodes).

Considering the three-node linear-head triangle element, the potential head with in the element may vary according to the following general equation:

$$h = \alpha 1 + \alpha 2 + \alpha 3\ y \tag{2.27}$$

$$i.e \quad h = [1xy] \begin{Bmatrix} \alpha 1 \\ \alpha 2 \\ \alpha 3 \end{Bmatrix} \tag{2.28}$$

where: α1, α2, α3 are constants to be determined in terms of the head at nodes *i.e* h_1, h_2 & h_3 as follows:

$$\begin{Bmatrix} h1 \\ h2 \\ h3 \end{Bmatrix} = \begin{Bmatrix} 1 & X_1 & Y_1 \\ 1 & X_2 & Y_2 \\ 1 & X_3 & Y_3 \end{Bmatrix} \begin{Bmatrix} \alpha 1 \\ \alpha 2 \\ \alpha 3 \end{Bmatrix} \tag{2.29}$$

Eq. 2.29 may be written as follows:

$$\{h\} = [A]\ .\ \{\alpha\}$$

or $\quad \{\alpha\} = [A]^{-1}\ .\ \{h\}$

Where:

$$[A]^{-1} = \frac{1}{2\Delta} \begin{bmatrix} \theta 1 & \theta 2 & \theta 3 \\ \beta 1 & \beta 2 & \beta 3 \\ \gamma 1 & \gamma 2 & \gamma 3 \end{bmatrix}$$

And Δ is the area of the triangle.

$$2\Delta = \begin{bmatrix} 1 & X1 & Y1 \\ 1 & X2 & Y2 \\ 1 & X3 & Y3 \end{bmatrix} = (X2\, Y3 - X3\, Y2) + (X3\, Y1 - X1\, Y3) + (X1\, Y2 - X2\, Y1)$$

Also we have:

$$\theta i = X_j Y_k - X_k Y_j$$

$$\beta i = Y_j - Y_k$$

$$\gamma i = X_k - X_j$$

From the above, the potential head (h) through out the entire element can be expressed in terms of the head at nodes as follows:

$$h = \frac{1}{2\Delta}[1XY]\begin{bmatrix}\theta1 & \theta2 & \theta3\\ \beta1 & \beta2 & \beta3\\ \gamma1 & \gamma2 & \gamma3\end{bmatrix}\begin{bmatrix}h1\\ h2\\ h3\end{bmatrix} \tag{2.30}$$

or:

$$h = [N_1 N_2 N_3]\begin{Bmatrix}h1\\ h2\\ h3\end{Bmatrix} \tag{2.31}$$

Where:

N_i are the linear interpolation functions for the triangular element

$$N_i = \frac{1}{2\Delta}(\theta i + \beta i\, X + \gamma i\, y) \tag{2.32}$$

The plane representing potential heads at nodes 1, 2 & 3 *i.e* through h_1, h_2 & h_3 is determined by Eq. (2.31)

If i_x , i_y are the gradient in the X and Y directions.

$$\text{Then } \begin{Bmatrix}i_x\\ i_y\end{Bmatrix} = \begin{Bmatrix}\frac{\delta h}{\delta x}\\ \frac{\delta h}{\delta y}\end{Bmatrix} \tag{2.33}$$

And by substituting for (h) from Eq. (2.31)

$$\begin{Bmatrix}ix\\ iy\end{Bmatrix} = \frac{1}{2\Delta}\begin{bmatrix}\beta1 & \beta2 & \beta3\\ \gamma1 & \gamma2 & \gamma3\end{bmatrix}\begin{Bmatrix}h1\\ h2\\ h3\end{Bmatrix}$$

Now for anisotropic soil υx, υy are given by

$$\begin{Bmatrix}\upsilon x\\ \upsilon y\end{Bmatrix} = \begin{bmatrix}kx & ix\\ ky & iy\end{bmatrix} = \begin{bmatrix}kx & 0\\ 0 & ky\end{bmatrix}\begin{bmatrix}ix\\ iy\end{bmatrix}$$

Accordingly, the element permeability matrix [k] can be evaluated as follows:

$$[k] = \frac{1}{2\Delta}\begin{bmatrix}\beta1 & \gamma1\\ \beta2 & \gamma2\\ \beta3 & \gamma3\end{bmatrix}\begin{bmatrix}kx & 0\\ 0 & ky\end{bmatrix}\frac{1}{2\Delta}\begin{bmatrix}\beta1 & \beta2 & \beta3\\ \gamma1 & \gamma2 & \gamma3\end{bmatrix}\Delta$$

$$= \frac{1}{4\Delta}\begin{bmatrix}kx\,\beta1\,\beta1 + ky\,\gamma1\,\gamma1 & kx\,\beta1\,\beta2 + ky\,\gamma1\,\gamma2 & kx\,\beta1\,\beta3 + ky\,\gamma1\,\gamma3\\ & kx\,\beta2\,\beta2 + ky\,\gamma2\,\gamma2 & kx\,\beta2\,\beta3 + ky\,\gamma2\,\gamma3\\ symmetric & & kx\,\beta3\,\beta3 + ky\,\gamma3\,\gamma3\end{bmatrix}$$

2.6.3 Finite difference method

The finite difference solution of groundwater flow is based on Laplace"s equation which can be simplified for two dimensional seepageas follows (Verruijt, 1970):

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \tag{2.34}$$

Figure (2.20) shows, a part of a region in which flow is taking place. For flow in the horizontal direction, using Taylor"s series, we can write

$$h_1 = h_o + \Delta x(\frac{\partial h}{\partial x})_0 + \frac{(\Delta x)^2}{2!}(\frac{\partial^2 h}{\partial x^2})_0 + \frac{(\Delta x)^3}{3!}(\frac{\partial^3 h}{\partial x^3})_0 + \cdots \tag{2.35}$$

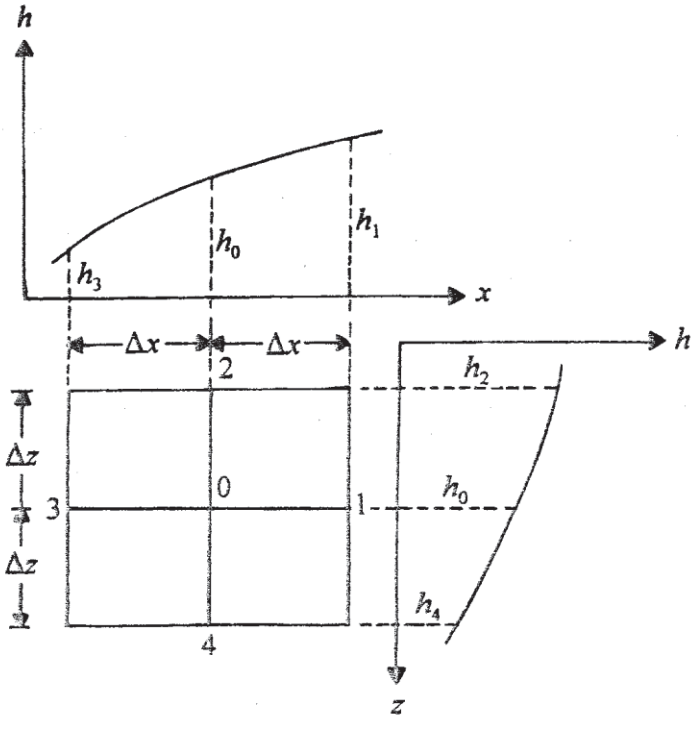


Fig. (2.19) *Hydraulic heads for flow in a region (Das, 2008)* and

$$h_3 = h_o + \Delta x(\frac{\partial h}{\partial x})_0 + \frac{(\Delta x)^2}{2!}(\frac{\partial^2 h}{\partial x^2})_0 - \frac{(\Delta x)^3}{3!}(\frac{\partial^3 h}{\partial x^3})_0 + \cdots \quad (2.36)$$

Subtracting Eqs. (35) and (36), we obtain

$$h_1 - h_3 = 2h_o + \frac{2(\Delta x)^2}{2!}(\frac{\partial^2 h}{\partial x^2})_0 + \frac{2(\Delta x)^4}{4!}(\frac{\partial^4 h}{\partial x^4})_0 + \cdots \quad (2.37)$$

Assuming Δx to be small, we can neglect the third and subsequent terms on the right-hand side of Eq. (36). Thus

$$(\frac{\partial^2 h}{\partial x^2})_0 = \frac{h_1 + h_3 - 2h_0}{(\Delta x)^2} \quad (2.38)$$

Similarly, for flow in the z direction we can obtain

$$(\frac{\partial^2 h}{\partial z^2})_0 = \frac{h_2 + h_4 - 2h_0}{(\Delta z)^2} \quad (2.39)$$

Substitution of Eqs. (38) and (39) into Eq. (34) gives

$$k_x \frac{h_1 + h_3 - 2h_0}{(\Delta x)^2} + k_z \frac{h_2 + h_4 - 2h_0}{(\Delta z)^2} = 0 \quad (2.40)$$

If $k_x = k_z = k$ and Δx = Δz, Eq. (40) simplifies to

$$h_1 + h_2 + h_3 + h_4 - 4h_0 = 0 \quad (2.41)$$

Or

$$h_0 = \frac{1}{4}(h_1 + h_2 + h_3 + h_4) \quad (2.42)$$

Equation (42) can also be derived by considering Darcy‟s law, $q = kiA$. For the rate of flow from point 1 to point 0 through the channel shown in Figure (2.21a), we have

$$q_{1-0} = k\frac{h_1 - h_0}{\Delta x}\Delta z \quad (2.43)$$

Similarly,

$$q_{0-3} = k\frac{h_0 - h_3}{\Delta x}\Delta z \quad (2.44)$$

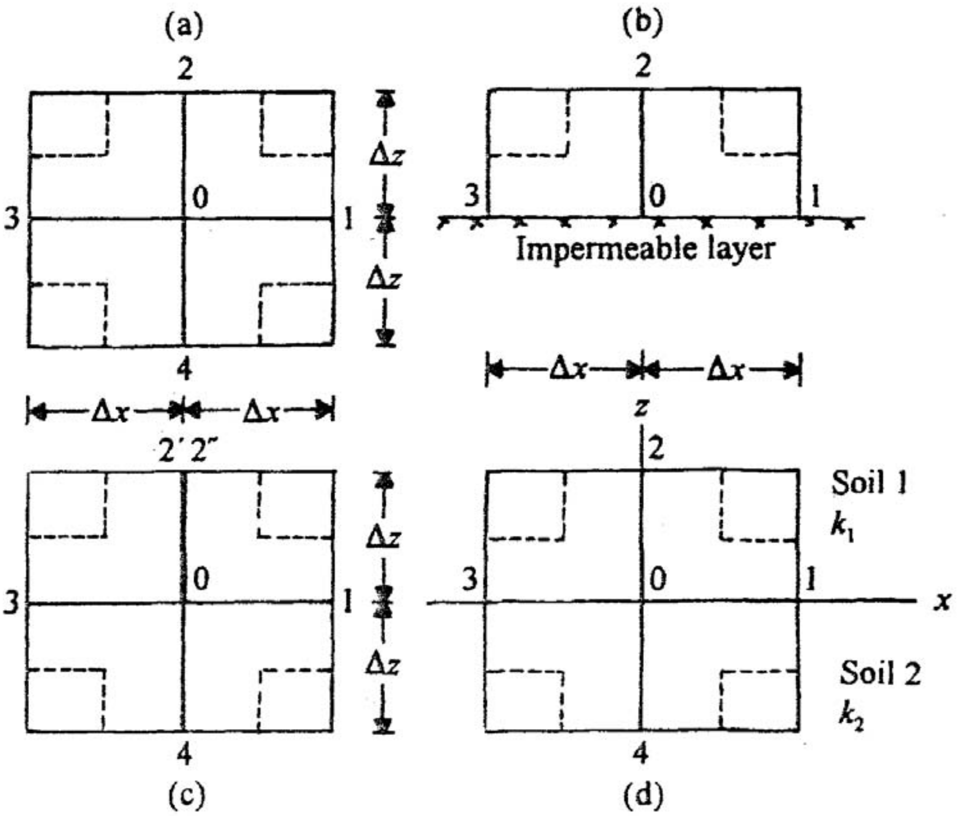


Fig. (2.20) *Numerical analysis of seepage (Das, 2008)*

$$q_{2-0} = k\frac{h_2 - h_0}{\Delta x}\Delta z \qquad (2.45)$$

$$q_{0-4} = k\frac{h_0 - h_4}{\Delta x}\Delta z \qquad (2.46)$$

Since the total rate of flow into point 0 is equal to the total rate of flow out of point 0, q_{in}-$q_{out} = 0$. Hence

$$(q_{1-0} + q_{2-0}) - (q_{0-3} + q_{0-4}) = 0 \qquad (2.47)$$

Taking $\Delta x = \Delta z$ and substituting Eqs. (44-47) into Eq. (42) we get

$$h_0 = \frac{1}{4}(h_1 + h_2 + h_3 + h_4)$$

If the point 0 is located on the boundary of a pervious and impervious layer, as shown in Figure (2.21b), the previous equations must be modified as follows:

$$q_{1-0} = k\frac{h_1 - h_0}{\Delta x}\frac{\Delta z}{2} \tag{2.48}$$

$$q_{0-3} = k\frac{h_0 - h_3}{\Delta x}\frac{\Delta z}{2} \tag{2.49}$$

$$q_{0-2} = k\frac{h_0 - h_2}{\Delta x}\Delta x \tag{2.50}$$

For continuity of flow,

$$q_{1-0} - q_{0-3} - q_{0-2} = 0 \tag{2.51}$$

With $\Delta x = \Delta z$, combining Eqs. (48-50) gives

$$\frac{h_1 - h_0}{\Delta x} - \frac{h_1 - h_0}{\Delta x} - (h_1 - h_0) = 0$$

$$\frac{h_1}{2} + \frac{h_3}{2} + h_2 - 2h_0 = 0$$

Or

$$h_0 = \frac{1}{4}(h_1 + 2h_2 + h_3) \tag{2.52}$$

CHAPTER 3

PARAMETRIC STUDY – Mathematical Solution

3.1 Introduction

In general, the main controlling factors for any dewatering system are the quantity of water to be pumped to accomplish the stated purpose and the associated drawdown depending on this total quantity of water pumped. There are many parameters affecting the drawdown and the dewatering process.

Using the Dupuit equation, Section 2.4.1.2, to model the radial flow of a deep well in a confined aquifer and considering the partial penetration effect (Equation 2.17), the results of a parametric study, to exhibit the influence of the different analysis parameters, are presented and discussed in the following.

3.2 Determination of the Soil-Water Parameters

During the analysis and design of dewatering systems, representative values of the soil-water parameters is crucial. Several parameters of soil and well specifications can affect the groundwater lowering in term of the drawdown. The effect of some of those parameters may be in opposite directions and when added together may lead to no global variation of the drawdown. Therefore, the influence of each parameter has to be investigated individually before integrating them in any analysis. Thus, a parametric study was conducted to exhibit the sensitivity of the results to the variation of each soil-water parameters in order to choose a reasonable value for each one.

The parametric study investigates the effect of the well depth, well radius (r_w), thickness of the confined permeable layer (H), quantity of flow discharge (Q) and the coefficient of permeability (k) on the groundwater drawdown. Variation of these parameters may also have some effect on the magnitude of the radius of influence (R_o) that is needed on analytically analyzing dewatering systems. Therefore, the parametric study was extended to examine the effect of R_o variation on the drawdown.

The soil profile at the site consists of, a layer of stiff to very stiff silty clay with thickness 7.5 m, followed by dense to very dense medium sand. The well is 22 m deep, the first 7.5 m are solid then 12.5 m filtered and the last 2 m works as a sump.

3.2.1 Well depth within the permeable layer (H_1)

Figure (3.1) shows the effect of varying the well depth on the drawdown at variable distance from the pumping well. The parameters used in the analysis were k_v=k_h = 0.031 cm/sec, r_w = 20 cm, H = 50 m, R = 100 m, Q = 65 m^3/hr. The well penetration in the confined aquifer was changed from 10 to 50 m. The permeability value was assessed based on the pumping test results. Such assessment is included in the calculation sheet at the end of the chapter.

Figure (3.1), exhibits that the drawdown decreases when the well penetration of the aquifer increases. This behavior is non-linear for distances (up to about 20 m); then tends to be linear when distance is beyond 20 m, for the site under consideration.

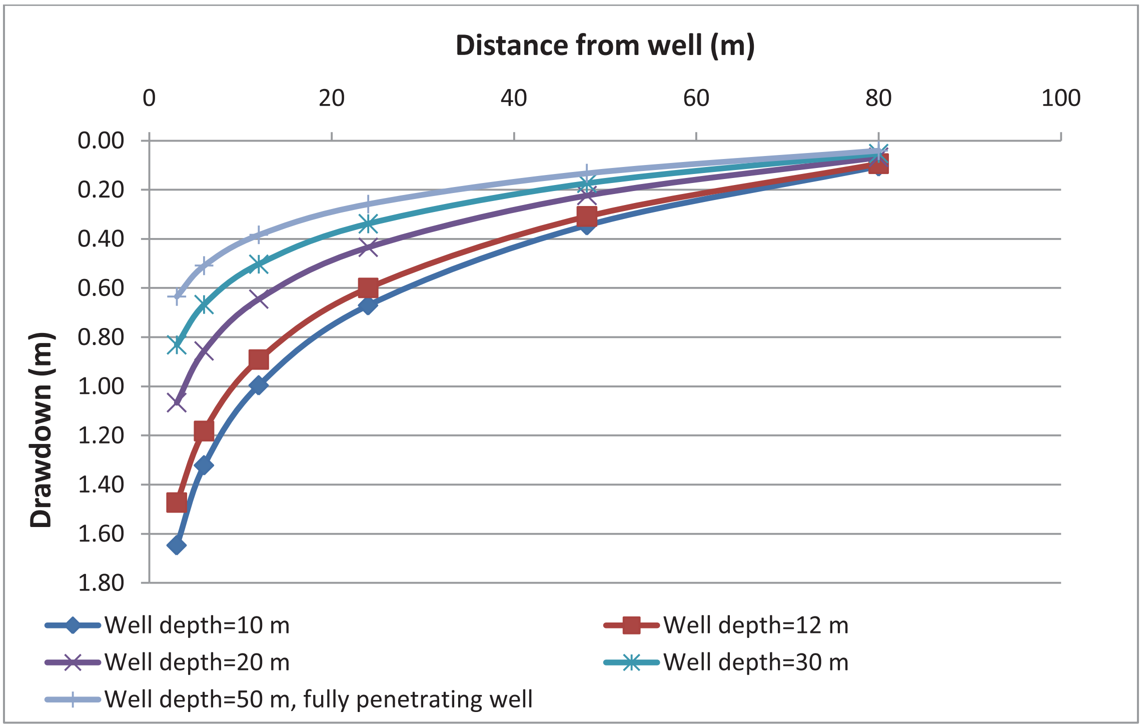


Fig. (3.1) *Drawdown vs. Well depth*

It is obvious according to Fig. (3.1) that the relation between the well penetration in the aquifer and drawdown is inversely proportional.

3.2.2 Radius of well r_w

Figure (3.2) shows the effect of variable well radius on the drawdown calculated at several distances from the well. In this study, the following variables were kept constant k_v=k_h = 0.031 cm/sec, well depth = 22 m, H = 50 m, R = 100 m, Q = 65 m^3/hr. The well radius was varied from 6" to 20" (0.15 to 0.50 m). It was noticed that increasing r_w leads to a slight decrease of drawdown.

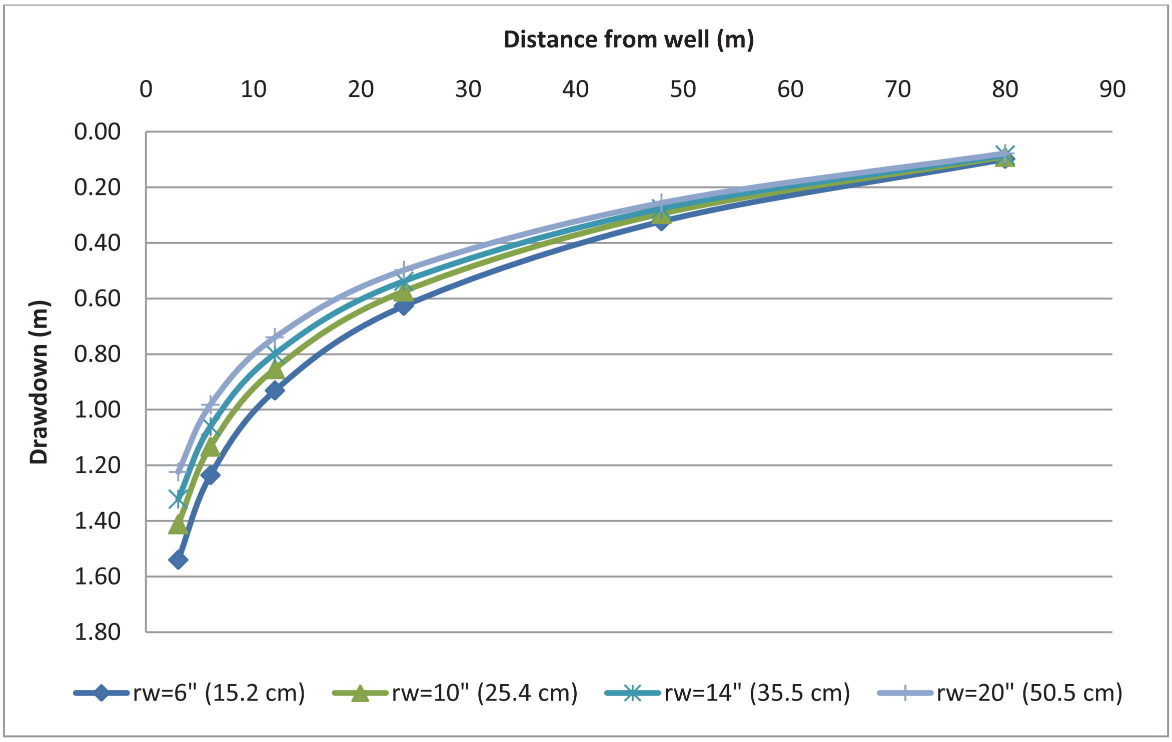


Fig. (3.2) *Drawdown vs. Well radius r_w*

3.2.3 Thickness of the confined aquifer H

The effect of the confined aquifer thickness „H" on the estimated drawdown values at several distances from the well was assessed. Thickness was varied from 14.5 to 100 m, while keeping other parameters constant (r_w = 20 cm, total well length = 22 m, k = 0.031 cm/sec, R = 100 m, Q = 65 m^3/hr).

Figure (3.3) shows that increasing the confined aquifer height decreases the drawdown for range 14.5 to 30 m; beyond this thickness no significant influence of H on the drawdown is noticed, for the considered site and well conditions. Increasing H by 100% (from 15 to 30 m) decreases drawdown by 30%.

In the following sections of this study H was assumed equal to 50 m, which was considered adequate based on the above result.

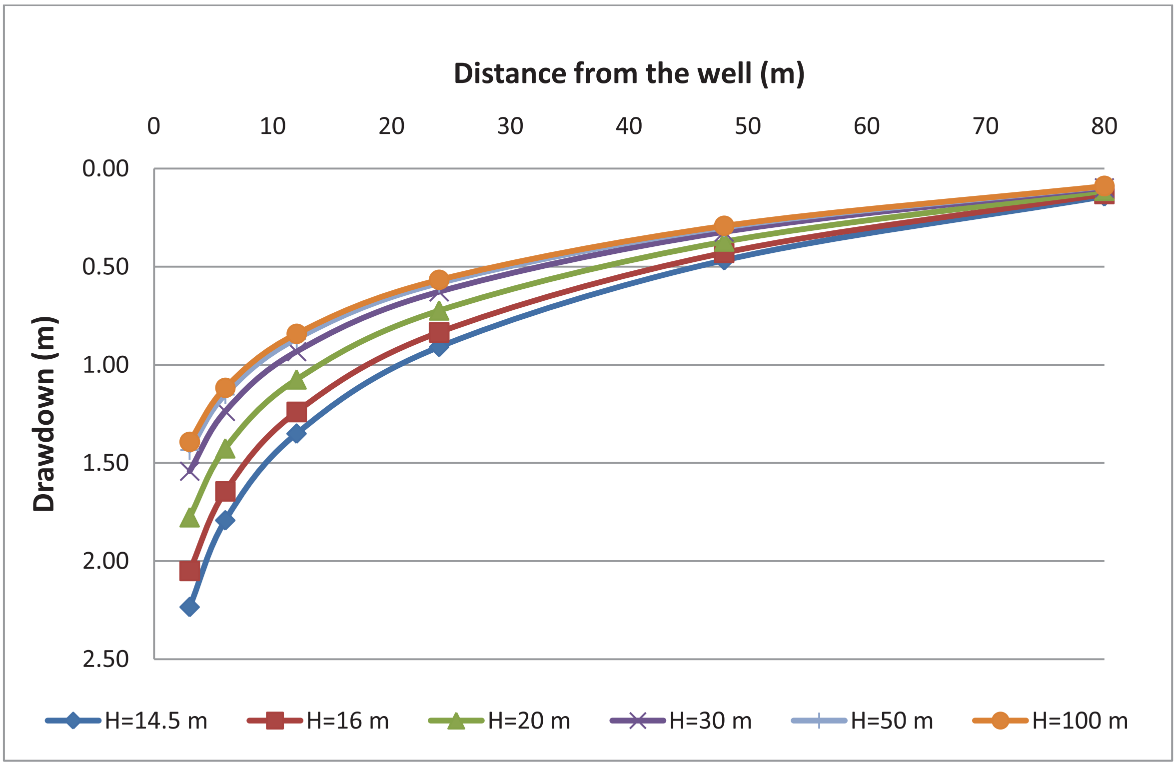


Fig. (3.3) *Drawdown vs. Sand layer height H*

3.2.4 Quantity of flow Q

The effect of the well discharge on the groundwater drawdown is exhibited in Fig. (3.4). The figure exhibits that increasing Q_w, increases the drawdown.

It should be noted that these results represent the case of : r_w = 20 cm, well depth = 22 m, H = 50 m, k = 0.031 cm/sec, R_o = 150 m.

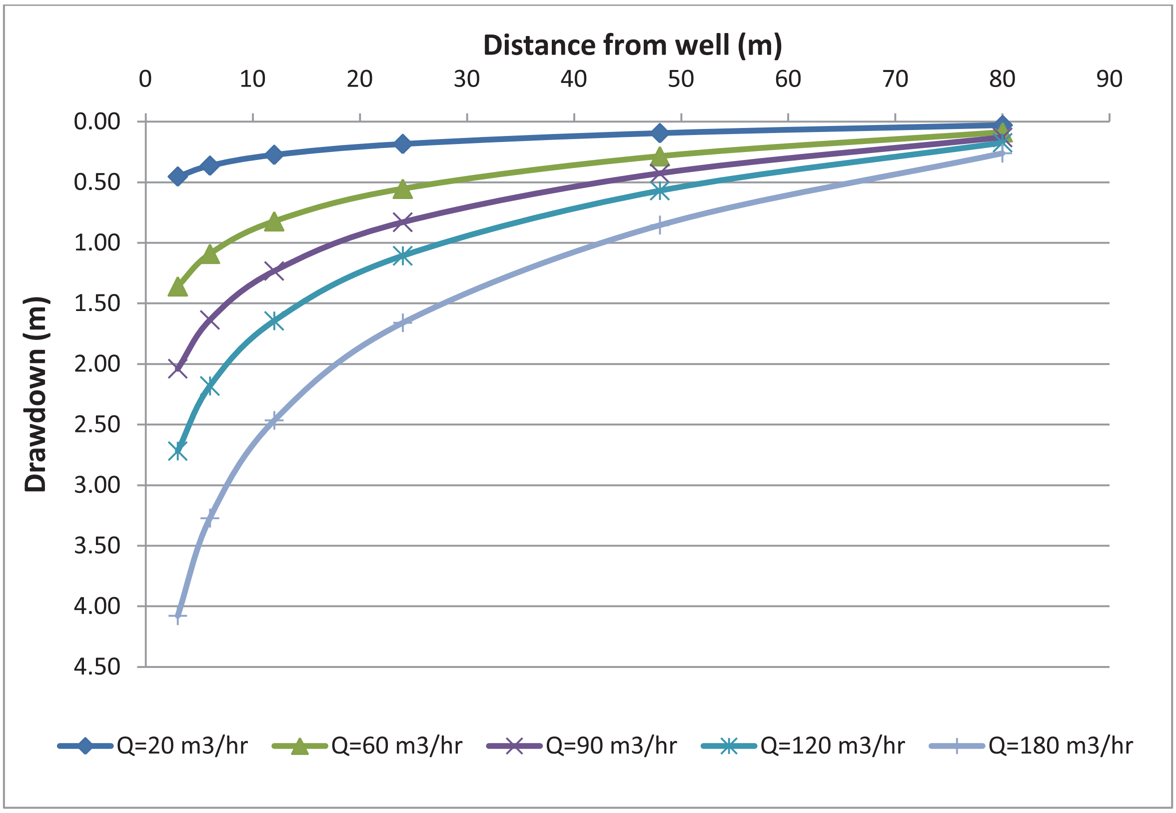


Fig. (3.4) *Drawdown vs. Quantity of flow Q*

3.2.5 Radius of influence (R)

Figure (3.5) shows the effect of varying the radius of influence on the predicted drawdown at variable distance from the pumping well; in these calculations we have considered k_h=k_v = 0.031 cm/sec, r_w = 20 cm, H = 50 m, well depth = 22 m, Q = 65 m^3/hr. The radius of influence was changed from 100 to 2000 m. The relation between the radius of influence and the drawdown is directly proportional. If the radius of influence is increased, the drawdown is increased but the ratio of increasing varies, Increasing the radius of influence from 100 to 400 m increases the drawdown by about 40 %, for the site and well conditions that are under consideration.

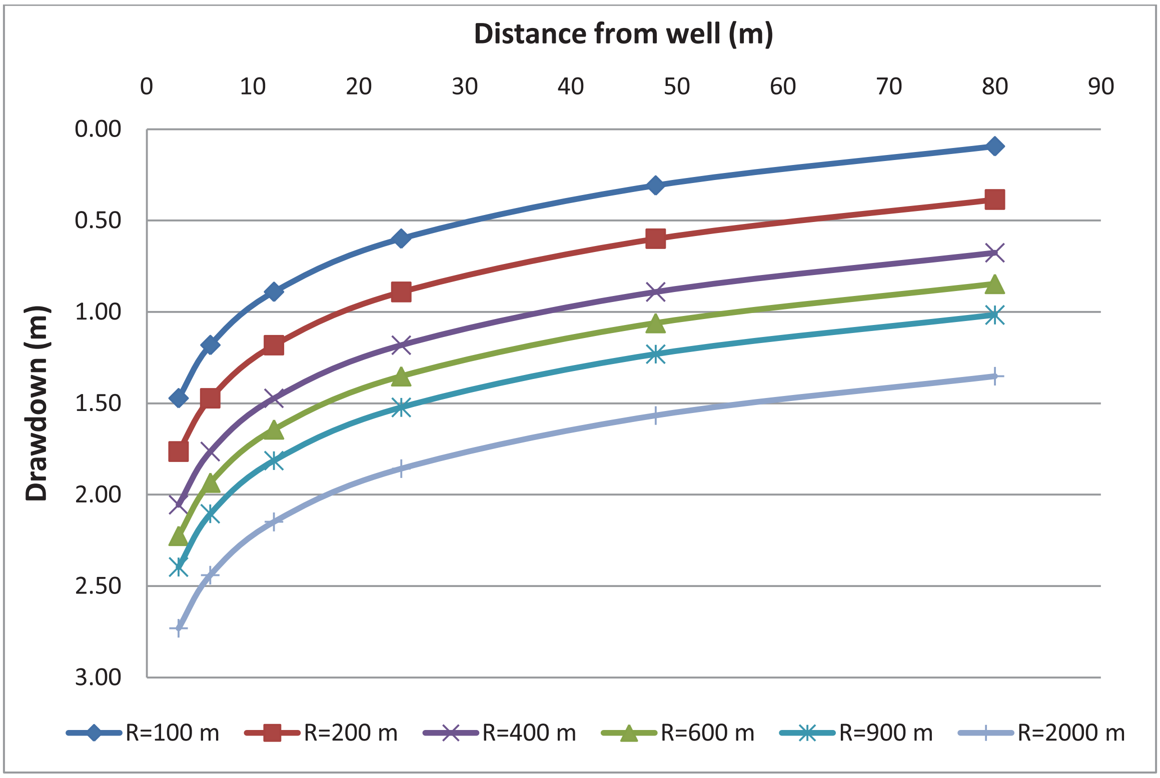


Fig. (3.5) *Drawdown vs. Radius of influence R*

3.2.6 Coefficient of permeability k

In this section the parametric study, the coefficient of permeability „k" was varied from (0.01 to 0.08 cm/sec) and the corresponding drawdown at different distances from the well were estimated. Increasing the permeability from 0.01 to 0.05 cm/sec decreases the drawdown by 80%. Beyond this value the drawdown decreases by lower rate.

Figure (3.5) exhibits the influence of the k-value on the model results. Consequently, assessment of a k-value needs a thorough investigation / considerations.

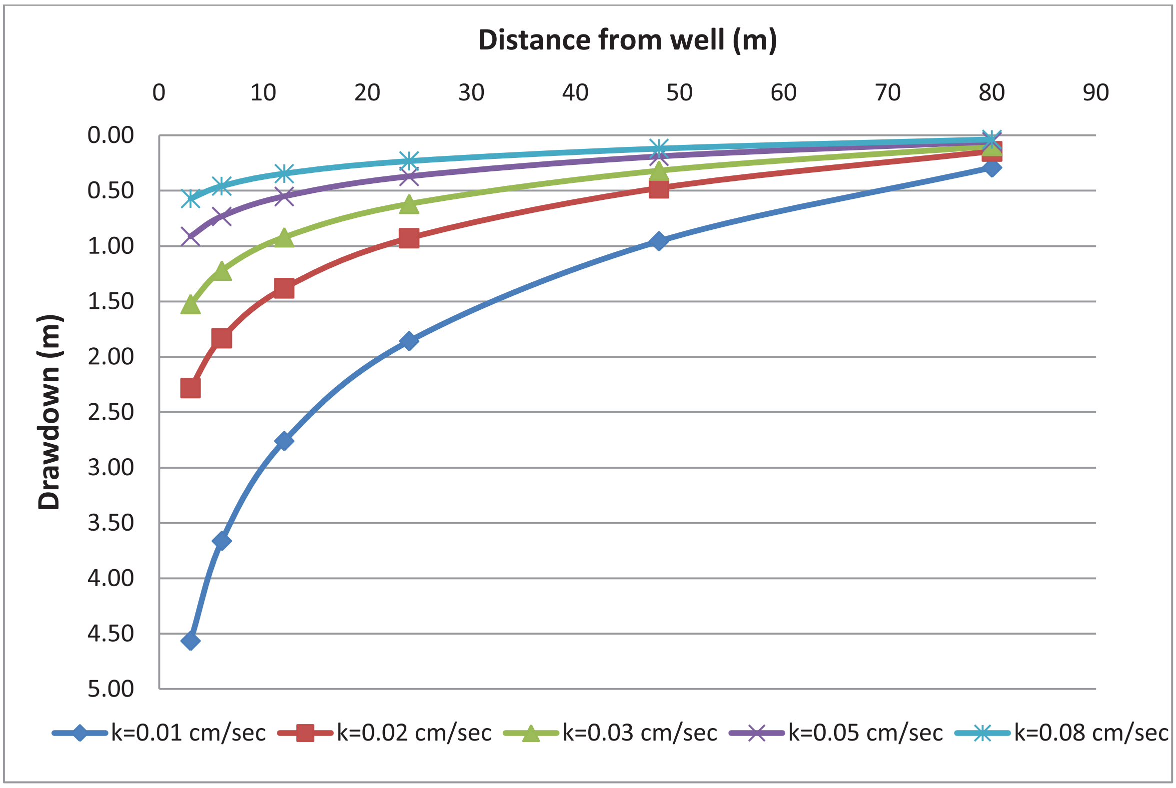
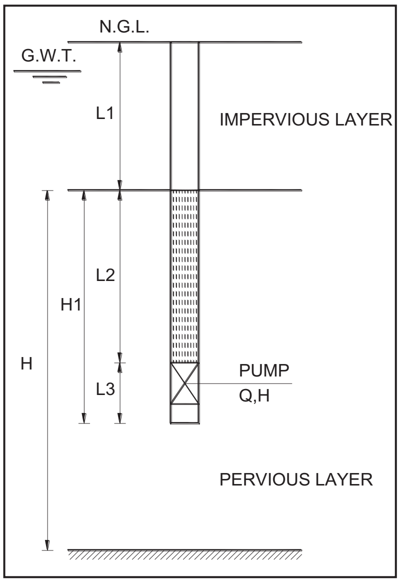


Fig. (3.6) *Drawdown vs. Coefficient of permeability k*

CALCULATION OF COEFFICIENT OF PERMEABILITY (K) USING THE PUMPING TEST

$$K = \frac{Q}{2H\pi.G(h_2 - h_1)} \ln(r_2 / r_1)$$



Well radius r_w =	0.20	m
Well discharge Q =	65	m^3/hr

h_n = Drawdown in piezometer No. n (m)

r_n = Radial distance from the well & piezometer No. n (m)

H= Confined aquifer thickness (m)

G = Well penetration coefficient

r_1 = The radial distance outside the well

$$G = \frac{H1}{H}\left[1 + 7\sqrt{(\frac{r_w}{2H1})} \times \cos(\frac{\pi H1}{2H})\right]$$

<u>Calculation of coefficient of permeability (k) between well and piezometer 4</u>

$h_4 - h_w$ =	1.55	m
r_w =	0.2	m
r_4 =	32.83	m
Ln (r_4/r_w) =	4.54	
kHG =	30.31	m^2/hr
kHG =	84.26	cm^2/sec.

<u>Calculation of coefficient of permeability (k) between well and piezometer 7</u>

$h_7 - h_w$ =	1.48	m
r_7 =	0.2	m
R_w =	31.24	m
Ln (r_w/r_7) =	4.49	
kHG =	31.40	m^2/hr
kHG =	87.28	cm^2/sec.

<u>Calculation of coefficient of permeability (k) between well and piezometer 8</u>

$h_8 - h_w$ =	1.79	m
r_8 =	0.2	m
r_w =	75.44	m
Ln (r_w/r_8) =	5.37	

kHG =	31.05	m^2/hr
kHG =	86.33	$cm^2/sec.$

Calculation of coefficient of permeability (k) between well and piezometer 1

$h_1 - h_w$ =	1.71	m
r_w =	0.2	m
r_1 =	71.96	m
Ln (r_w/r_1) =	5.33	
kHG =	32.22	m^2/hr
kHG =	89.57	$cm^2/sec.$

Calculation of coefficient of permeability (k) between Piezometers 7 & 8

$h_7 - h_8$ =	0.31	m
r_7 =	31.24	m
r_8 =	75.44	m
Ln (r_8/r_7) =	0.88	
kHG =	29.42	m^2/hr
kHG =	81.79	$cm^2/sec.$

FINAL CALCULATIONS :

G	kHG cm2/sec	k cm/sec	Piezometer
0.056	84.259	0.030	W & 4
0.056	87.279	0.031	W & 7
0.056	86.329	0.031	W & 8
0.056	89.573	0.032	W & 1
0.056	81.791	0.029	7 & 8
Average		0.0307	

CHAPTER 4

NUMERICAL ANALYSIS

4.1 Introduction

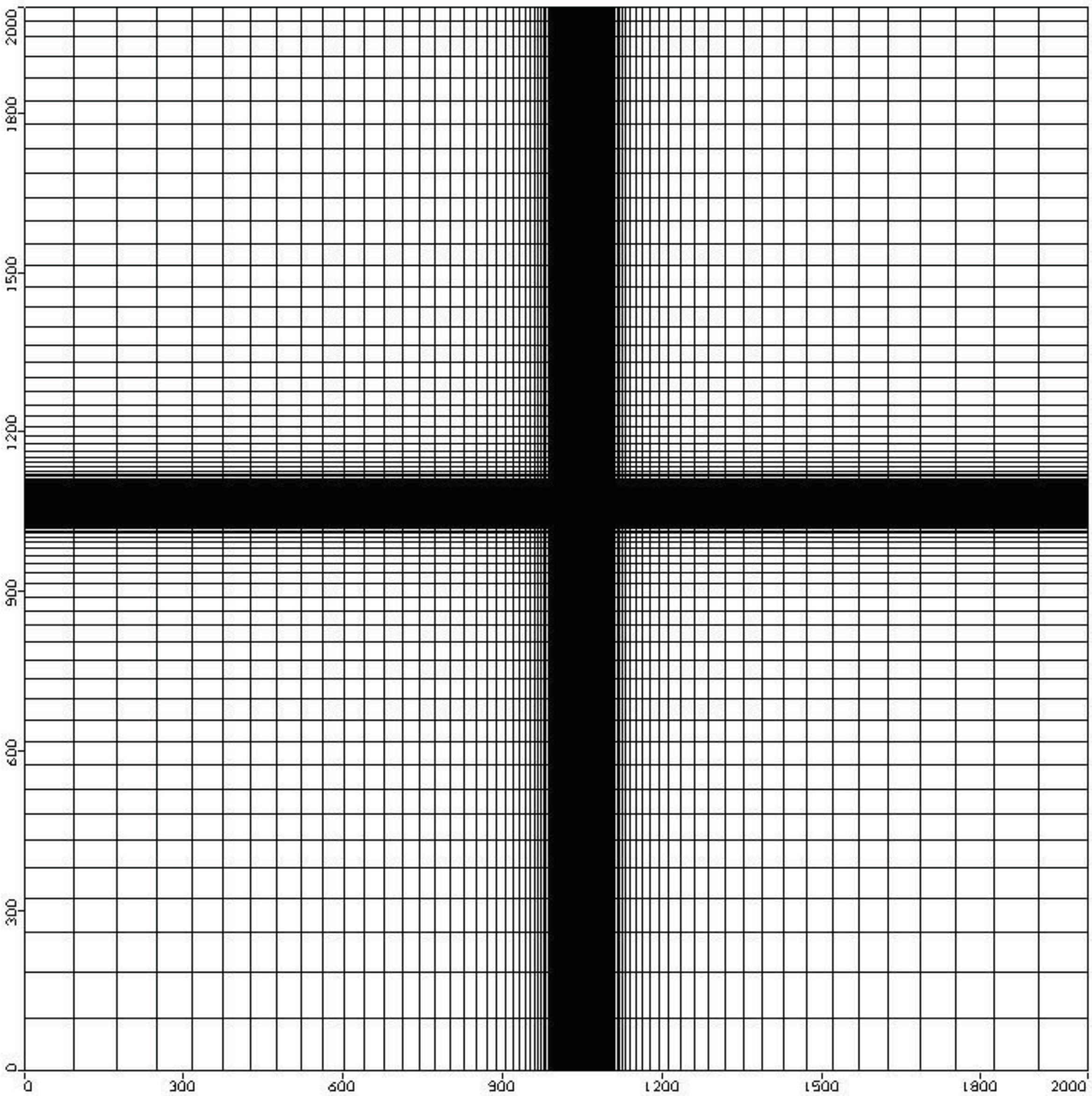
A groundwater numerical model using the finite difference method is a mathematical approximation of a real case that exist in-situ. The numerical modeling is performed in this research using the MODFLOW finite difference program. The main variable of any dewatering system is the drawdown. There are many parameters affecting this variable in a dewatering system, (number of wells, wells length, wells arrangements,… etc.).

This chapter presents the results of a numerical parametric study that was conducted as a part of this research using the numerically method. In this study, the pumping test described in the previous chapter was reanalyzed numerically. The results of this study is presented in the following.

4.2 Parametric Study

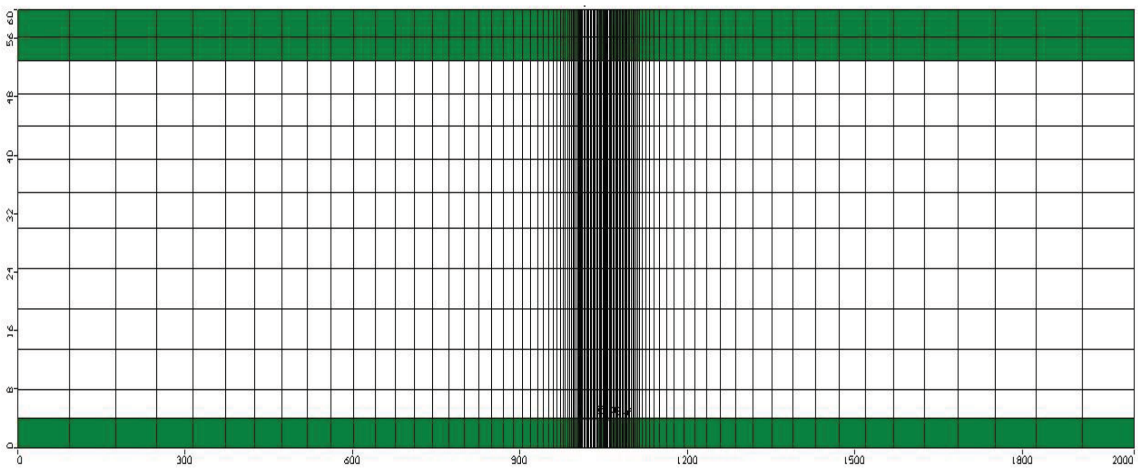
During the analysis and design of dewatering systems, assessment of the soil-water parameters is crucial. Several parameters of the soil and well configuration can affect the groundwater lowering in terms of drawdown. The effect of those parameters can be in opposite directions and when added together they may alternate mutually leading to no global variation of the drawdown as discussed in sec.3.2. Therefore, the influence of each parameter has to be investigated individually before integrating them in any analysis. In this study, a parametric study was carried out to assess the sensitivity of the drawdown to the variation of these parameters in order to guide the selection of a reasonable value for each parameter. The parametric study takes into consideration the well depth, well discharge, coefficient of permeability (k) and aquifer thickness.

The discretization for numerical analysis will be constant through all the considered analysis scenarios. Figure (4.1-a) shows a plan view of the whole mesh. Figure (4.1-b) Is a close view of the mesh showing the well location (red point) and the location of the observation wells (green points). Figure (4.1-c) Shows a side view of the mesh.

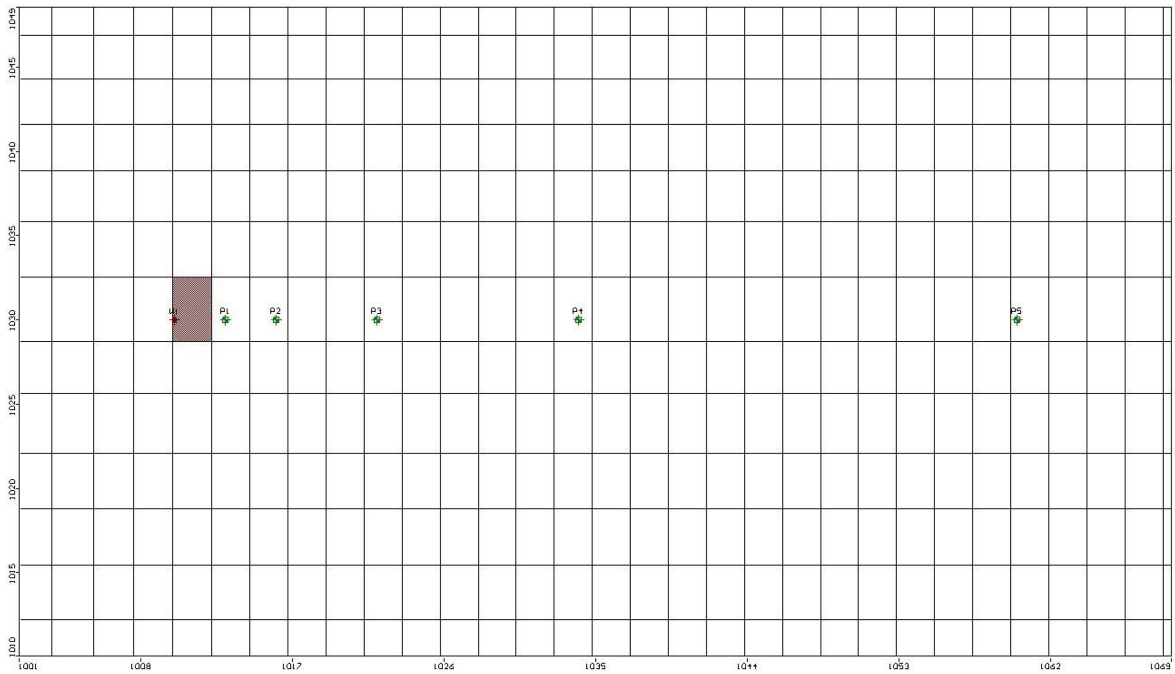


(a) Plan view of the whole mesh

Fig. (4.1) *(a) Plan view of the whole mesh. (b) Side view of the mesh. (c) Close view of the mesh showing the pump (red point) and the observation wells (green points).*



(b) Side view of the mesh



(c) Close plan view of the mesh showing the pump (red point) and the observation wells (green points)

4.2.1 Well depth (H_1)

Figure (4.2) shows the effect of changing the well depth on the drawdown measured at different distances from the well. In this analysis, the well discharge was taken equal to 65 m^3/hr, aquifer thickness was taken equal to 50 m, radius of influence was taken equal to 150 m, and the coefficient of permeability was taken equal to 0.0307 cm/sec.

The total well length values used in the analysis were 21.5, 27.5, 37.5 and 57.5 m, i.e. the penetrated well length in the aquifer were 14, 20, 30 and 50 m. The drawdown was calculated using the numerical model and plotted at distances 3, 6, 12, 24 and 50 m from the well.

The figure shows that the drawdown tends to increase by decreasing the well depth. It is also noticed that the calculated drawdown at a distance of about 40 m is not significantly affected by the well depth, for the conditions under consideration.

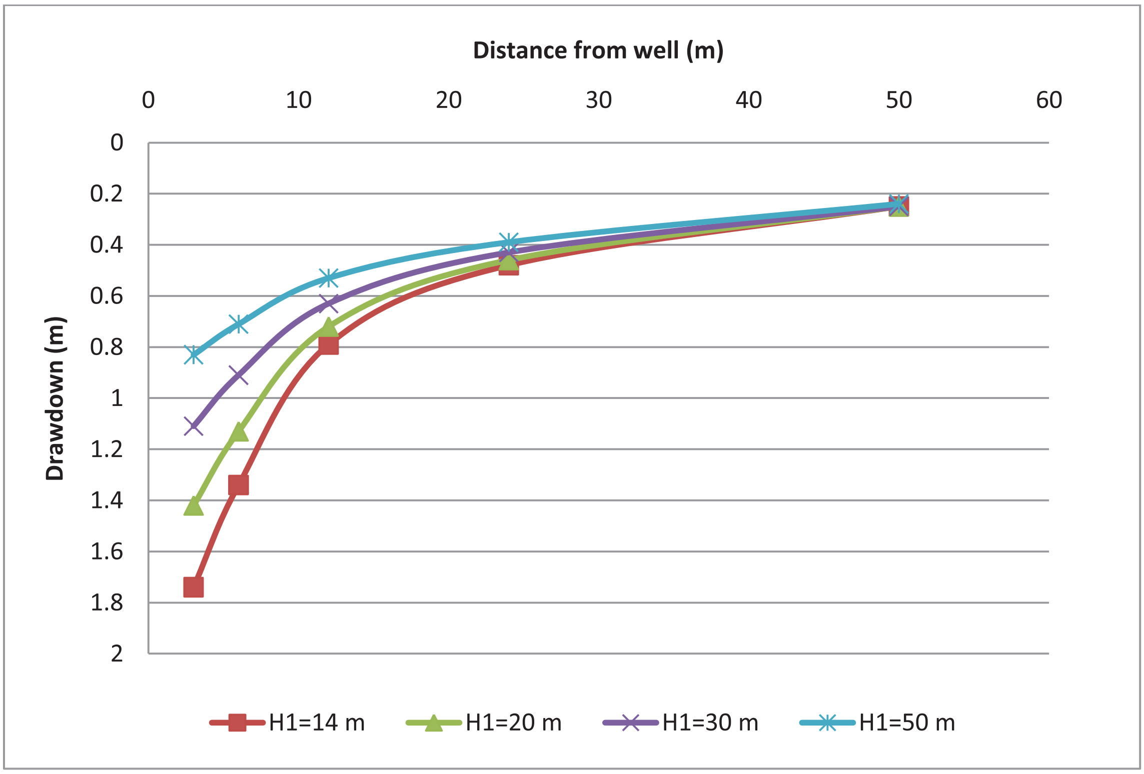


Fig. (4.2) *Effect of well depth on the drawdown at different distances from the well*

4.2.2 Well discharge (Q)

Figure (4.3) shows the effect of changing the well discharge on the drawdown at different distances from the well. In this study, the considered total well length was equal to 22 m, aquifer thickness was equal to 50 m, radius of influence was equal to 150 m, and the coefficient of permeability was equal to 0.0307 cm/sec. The well discharge values used in the analysis were 20, 60, 90, 120 and 180 m^3/hr. The drawdown variation with distances from the well is shown in Fig. (4.3). The figure illustrates that the drawdown increases by increasing the well discharge. This effect extends to a radial distance from the well larger than about 50 m, for the considered well and site conditions.

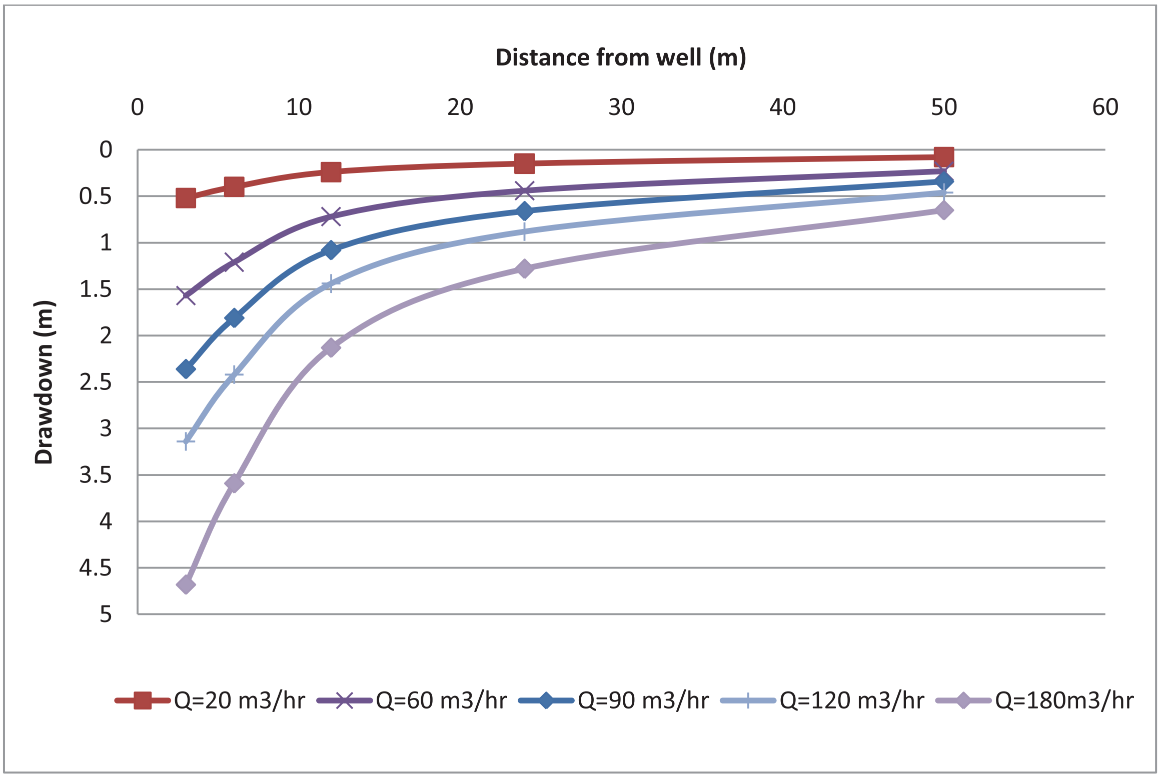


Fig. (4.3) *Effect of quantity of well discharge on the drawdown at different distances from the well*

4.2.3 Coefficient of permeability (k)

Figure (4.4) shows the effect changing of the value of the coefficient of permeability on the drawdown at different distances from the well. In this study, the considered well length thickness was 22 m, aquifer thickness was 50 m, well discharge was 65 m^3/sec, radius of influence = 150 m. The values of the coefficient of permeability used in the calculation were 0.01, 0.02, 0.03, 0.05, and 0.08 cm/sec, considering isotropic condition (k_h=k_v), The drawdown is plotted at distances of 3, 6, 12, 24 and 50 m from the well as shown in Fig. (4.4).

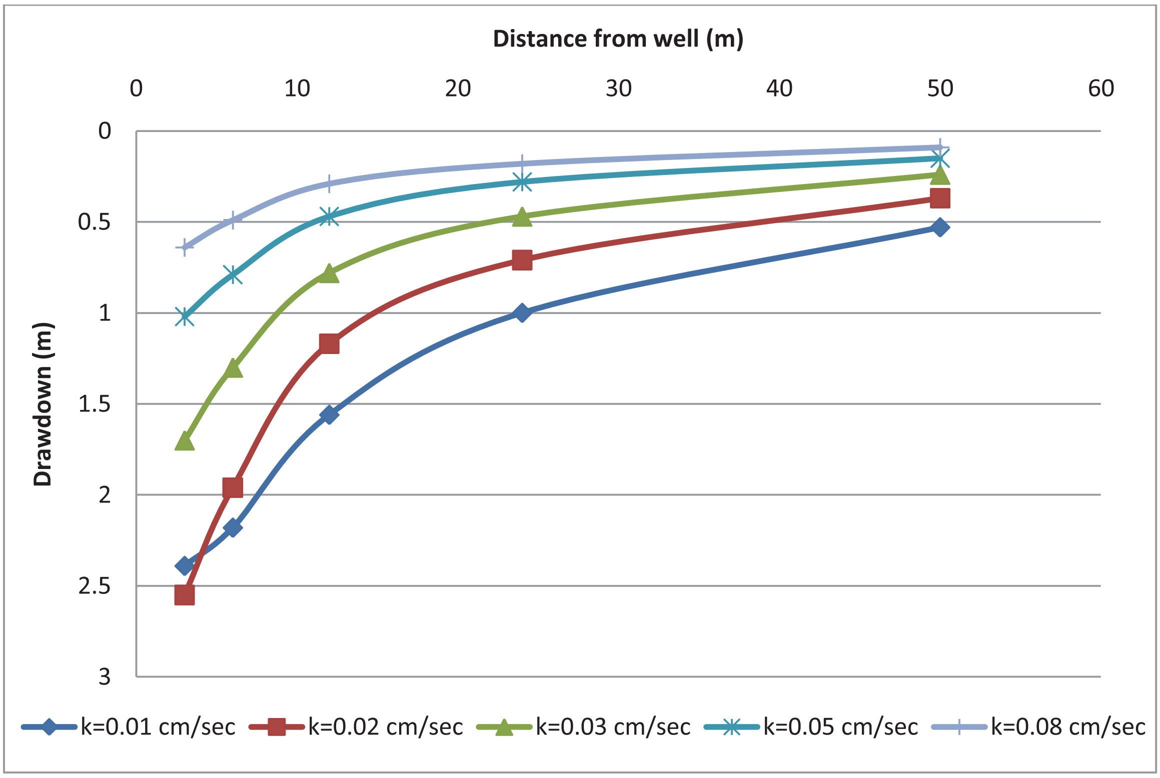


Fig. (4.4) *Effect of permeability change on drawdown (Isotropic condition)*

In figure (4.5) every line represents a constant value of the horizontal coefficient of permeability and the x-axis shows the change in vertical coefficient of permeability, and how the change of each one affect the drawdown at distance 12 m from the well.

It can be noted that the variation of k_h to k_v do not make a significant change in drawdown so it is acceptable to consider k_h equals k_v, i.e. isotropic condition.

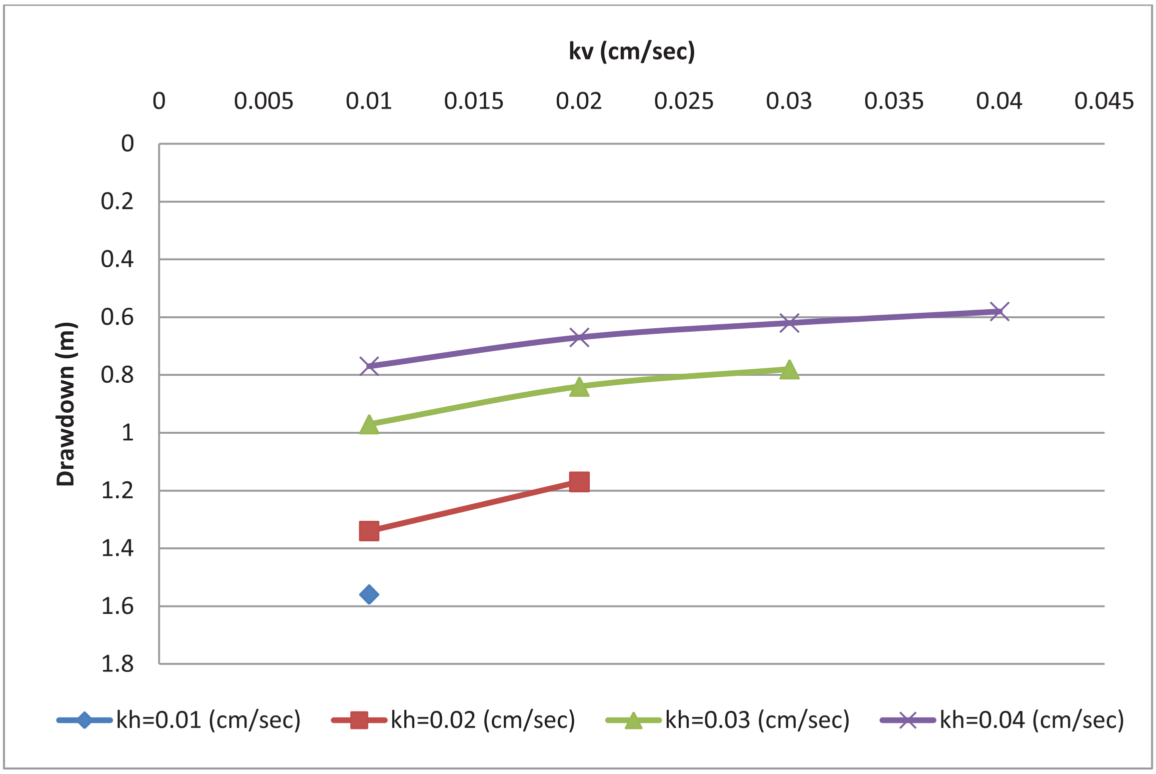


Fig. (4.5) *Variation of the Drawdown values at a distance of 12 m from the well with different kh/kv values*

4.2.4 Aquifer thickness (H)

Figure (4.6) shows the effect of variation of the aquifer thickness on the estimated drawdown at different distances from the well. In this analysis, the following data was considered: the total well length was 22 m, coefficient of permeability was 0.0307 cm/sec, well discharge was 65 m^3/sec, and the radius of influence was 150 m.

The aquifer thickness considered in the analysis were 14.5, 20, 30 and 50 m. The drawdown was calculated at distances of 3, 6, 12, 24 and 50 m from the well as shown in Fig. (4.6). The figure shows that the drawdown increases with increase of aquifer thickness.

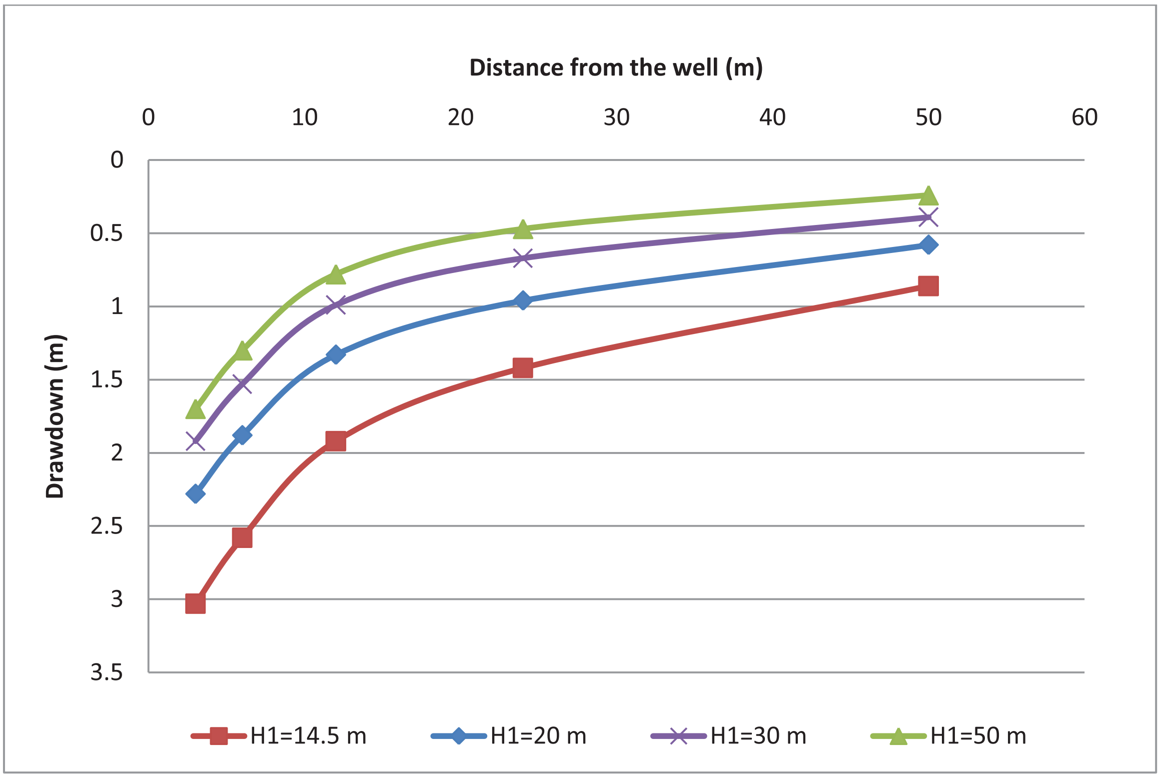


Fig. (4.6) *The effect of aquifer height change on drawdown*

4.2.5 Radius of influence (R_o)

It should be noted that changing the radius of influence requires changing the location of vertical boundaries of the selected mesh. Figure (4.7) shows the effect of changing the radius of influence on the drawdown measured at different distances from the well. In this calculation the following data was considered: the well discharge was 65 m^3/hr, aquifer thickness was 50 m, the total well length was 22 m, and the coefficient of permeability was 0.0307 cm/sec.

The values of the radius of influence used in the analysis were 100, 200, 300, 400 and 500 m. The drawdown was calculated using the numerical model and plotted at distances of 3, 6, 12, 24 and 50 m from the well.

Figure (4.7) shows that the drawdown tends to increase by increasing the radius of influence. It is obvious that the R_o values from 300 to 500 do not make a significance change in the drawdown but it matches the drawdown measured during the pump test illustrated in section 5.4.1.

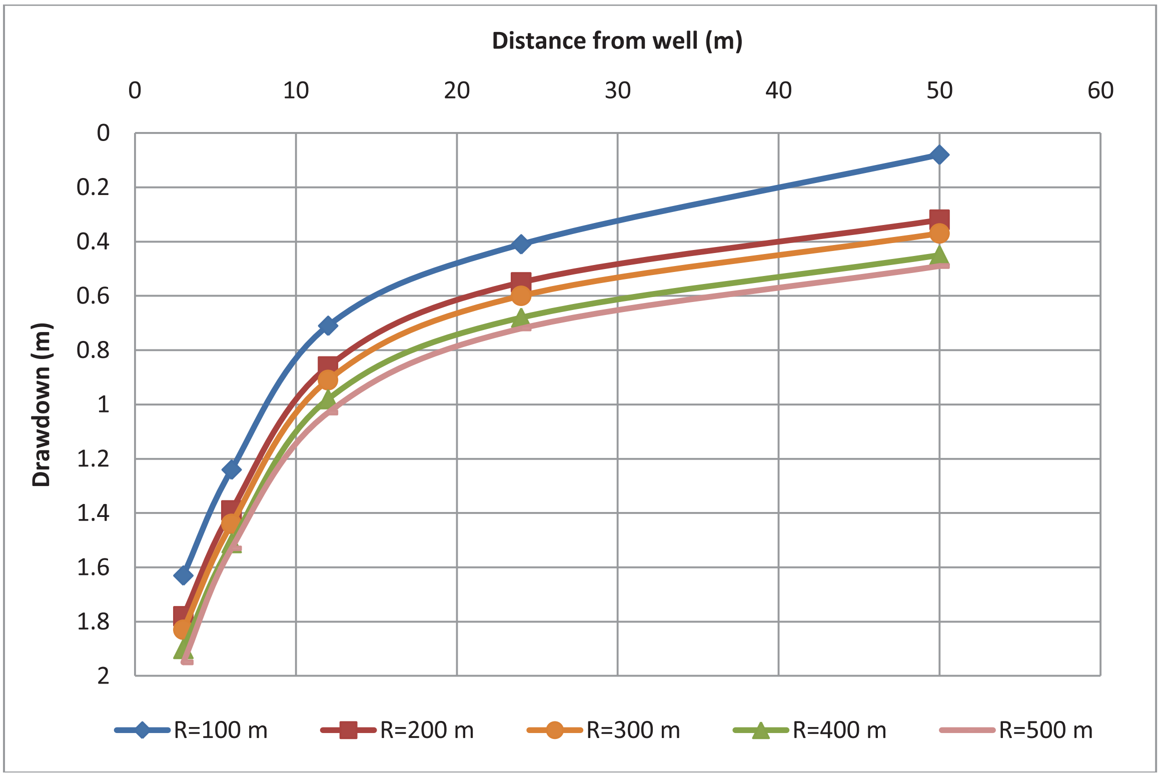


Fig. (4.7) *Effect of Radius of influence change on the drawdown*

CHAPTER 5

CASE STUDY OF GROUNDWATER LOWERING FOR THE EXTENTION OF WADI EL-NILE HOSPITAL, CAIRO