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EFFECTIVENESS OF IMPERMEABLE BARRIERS FOR RETARDATION OF POLLUTANT MIGRATION

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THE UNIVERSITY OF OKLAHOMA

GRADUATE COLLEGE

EFFECTIVENESS OF IMPERMEABLE BARRIERS FOR RETARDATION OF POLLUTANT MIGRATION

A DISSERTATION

SUBMITTED TO THE GRADUATE FACULTY

in partial fulfillment of the requirements for the

degree of

DOCTOR OF PHILOSOPHY

BY

ROBERT CHARLES KNOX

Norman, Oklahoma

1983

EFFECTIVENESS OF IMPERMEABLE BARRIERS FOR

RETARDATION OF POLLUTANT MIGRATION

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ABSTRACT

Currently one of the most popular methods of containing contaminated ground water is through use of subsurface impermeable barriers. These barriers can take one of three forms; slurry walls, grout curtains, or steel sheet-piles. Successful operation of these barrier systems is dependent upon three basic criteria. First, the barrier must be truly impermeable and remain so over time even upon exposure to the contaminated ground water. Second, there must exist an underlying impermeable formation, at a reasonable depth, to which the barrier can be connected. Third, an adequate connection between the barrier and the underlying formation must be assured.

This paper presents the results of the analysis of the movement of contaminated ground water under or through an imperfect barrier. The first phase of the analysis consists of the development of an analytical solution for the flow of ground water under a barrier and a simple numerical integration technique for developing concentration breakthrough curves. This simple solution algorithm was applied to the cases of variable recharge rates and lengths, variable depths of penetration of the barrier, and anisotropic soils. The second phase of the analysis involves applying a numerical solute transport model to analyze the performance of a barrier with and without the effects of hydrodynamic dispersion, and in the presence of a layered soil, and

finally the performance of a fully penetrating but partially permeable barrier.

ACKNOWLEDGEMENTS

The author would like to note those who have contributed their talents, time and effort to this work.

Words cannot express the gratitude due Dr. Larry Canter, Chairman of the Dissertation Committee, for providing educational advisement, technical assistance and much needed employment opportunities throughout my career at the University of Oklahoma.

Deepest thanks go to Drs. Mike Devine, Edwin Klehr, Steve McLin, and George Tauxe for their encouragement, technical comments, suggestions and advice.

Appreciation is expressed to Mrs. Leslie Rard and Ms. Valerie Wattenbarger for performing the arduous task of typing the manuscript.

Sincere thanks go to my friends and family for their support. Special recognition goes to my mother, Mrs. Ruth Knox, for being a totally positive influence in my life. Fond memories and undieing gratitude are held for C.D. and Florence Baker who made a college education possible.

Special mention needs to be made of Dr. Johnny Griggs, Dr. Jodie Edge and Ms. Linda Greene whose help and friendship provided incentive during trying times.

This work is dedicated to the memory of my father, the late Robert William Knox; University of Oklahoma, 1944-1945.

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CHAPTER I

INTRODUCTION

A growing awareness of the importance and potential threat to this nation's ground water resources resulted in the Environmental Protection Agency (EPA) studying the feasibility of a national ground water protection strategy. Preliminary information indicated that the strategy should be forward-looking and emphasize preventive approaches rather than only concentrating on cleanup of contaminated aquifers. Existing legislation and the Superfund program give emphasis to aquifer cleanup and restoration (U.S. EPA, 1980).

One of the popular new methods of confining the movement of leachates and/or contaminated ground water is the impermeable barrier. The impermeable barrier is an impermeable, vertical wall which is usually placed downgradient of a polluted formation to restrict movement of the polluted ground water. Barriers usually take one of three different forms: slurry walls, grout cutoffs, or steel sheet-piles.

Slurry walls represent one type of impermeable barrier, and employing them for pollution control is a relatively new application of this technology. Construction involves pumping a slurry made of water and bentonite clay into a trench while excavation of the trench is proceeding. The slurry maintains wall stability through hydrostatic pressure, thus decreasing the required width of excavation. As excavation proceeds, backfilling with a soil-bentonite or cement-bentonite mixture follows. The backfill combines with the slurry to form an impermeable membrane (Knox, et al., 1982).

Grout cutoffs are another type of impermeable barrier and are constructed by injecting a liquid, slurry, or emulsion under pressure into the soil. The fluid injected will move away from the point of injection to occupy the available pore spaces. As time passes, the injected fluid will solidify resulting in a decrease in the original soil permeability and an increase in the soil bearing capacity. Like slurry walls, grouting techniques have been used for years in the construction industry, but are only now finding applications for ground water pollution control.

Sheet piling involves driving lengths of steel that connect together into the ground to form a thin impermeable permanent barrier to flow. Sheet piling materials also include timber and concrete, however, their application to polluted ground water cases is in doubt due to corrosive actions and costs, respectively. Sheet-piling is also a relatively old technology just now finding applications for ground water pollution control.

One important factor common to all three technologies is the need for the barrier to be able to key into an underlying impermeable zone. If a continuous connection of the barrier to the impermeable formation cannot be assured there will exist a window or passage for pollutants to move through.

Objective

The objective of this research will be to make an assessment of the effectiveness of barriers in retarding pollutant migration assuming a continuous connection is not made with an underlying impermeable formation. The effectiveness of the barrier for various conditions will be evaluated by comparing the dimensionless "pollutant breakthrough curves" developed for each situation with the curve developed for the no barrier situation.

Scope

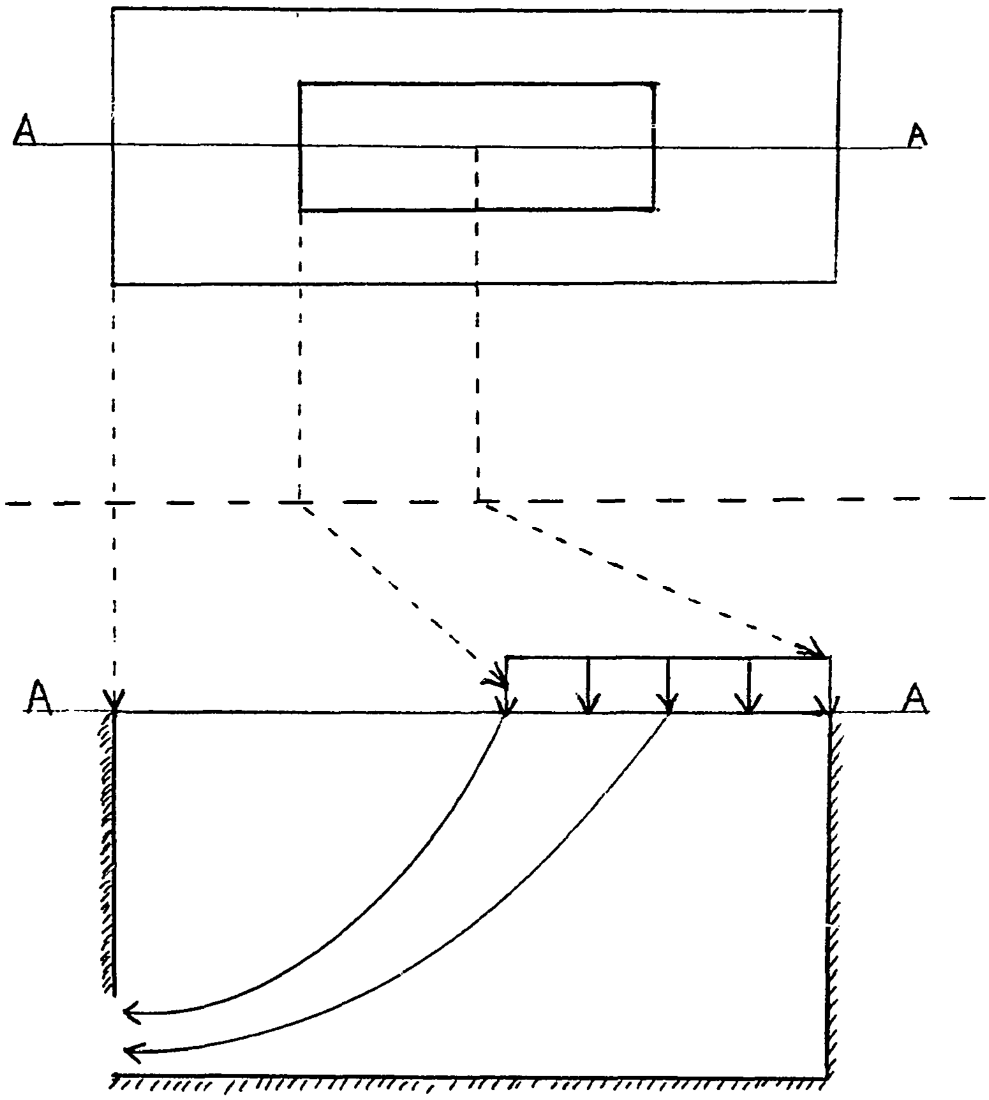
The general configuration of the problem addressed can be found in Figure I-1. The figure assumes a rectangular waste source leaching pollutants at a constant rate exactly equal to the maximum rate at which the underlying formation can accept them, i.e., steady-state conditions. The waste source is completely surrounded by a vertical impermeable barrier not adequately keyed into an underlying impermeable formation. Although hypothetical, this configuration represents an idealized application of barrier technology. Ideally, one would want to surround the waste source in order to confine or trap the pollutants within a specified area.

The scope of the analysis involved five phases as outlined in Figure I-2. These phases are discussed briefly below.

a. Phase I -- The preliminary investigation and literature reviews are discussed in detail in Chapters II and III.

b. Phase II -- The second phase of the analysis involved developing an analytical solution for the dimensionless streamline ($\overline{\Psi}$) distribution in the dimensionless aquifer and under the barrier imposed on the aquifer. This solution was developed utilizing Fourier analysis and the

Plan



Profile

Figure I-1: Schematic of Proposed Problem.

Phase I
Preliminary Investigation and Literature Review

Phase II
Analytical Solution Development

Phase III
Development of Plug Flow Numerical Model and
Analysis of Variations in Problem Input

Phase IV
Utilization of Documented Solute Transport Model
for Analysis of Variations in Problem Input

Phase V
Assessment of the Effectiveness of Impermeable
Barriers

Figure I-2: Flow Chart For Study

separation of variables technique. The analytical solution was to serve mainly as a means of verifying the simplified numerical model developed in Phase III. The details of the solution development are given in Chapter IV.

c. Phase III -- The third phase of the analysis consisted of developing a numerical model for determining the dimensionless streamline ($\overline{\Psi}$) distribution. Additionally, the model was programmed to integrate the area under given values of $\overline{\Psi}$ and calculate X and Y direction velocities. From the relationship of area versus streamline, pollutant breakthrough curves for a variety of simple conditions were developed assuming plug flow in the aquifer. The different situations examined by this simple technique are outlined in Table I-1. The development of the numerical model and the theory behind plug flow analysis are outlined in Chapter IV.

d. Phase IV -- The fourth phase of the analysis involved utilizing a solute transport numerical model from the U.S. Geological Survey. The model utilized was titled, "Computer Model of Two-Dimensional Solute Transport and Dispersion in Ground Water" (Konikow and Bredehoeft, 1978). The characteristics and required manipulations of the Konikow-Bredehoeft (K-B) model are outlined in Chapter IV. A discussion of the reasons for utilizing the K-B model in addition to the plug flow model are outlined below.

e. Phase V -- The results of the analyses in Phases III and IV will be the basis for assessing the effectiveness of impermeable barriers for retarding pollutant migration. A comparison will be made between the performances of an imperfect barrier and no barrier for a wide variety of input conditions. The results generated by the two models are analyzed in Chapter V. A summary of these results is found in Chapter VI along with a series of recommendations for future research.

The purposes of utilizing the solute transport model in addition to the previously described plug flow model were five-fold. First, the solute transport model could be programmed to examine convective transport only. These results could then be utilized to confirm the accuracy of the plug flow model. Second, the solute transport model could also be programmed to examine the effects of

Table I-1: Problems Examined By Plug Flow Analysis.

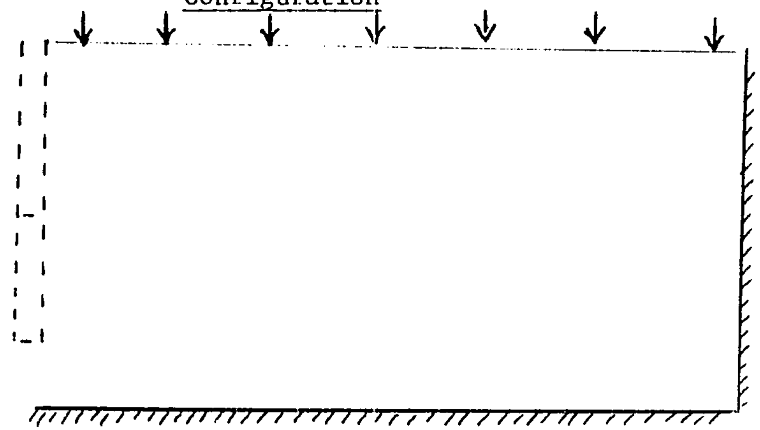
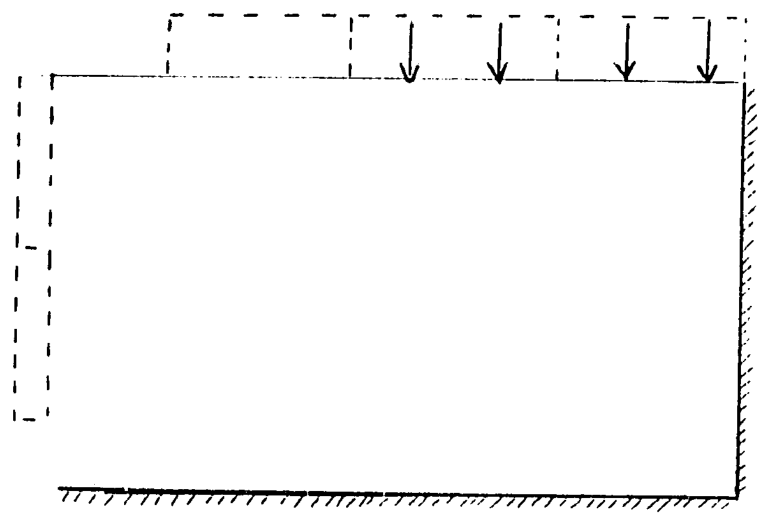
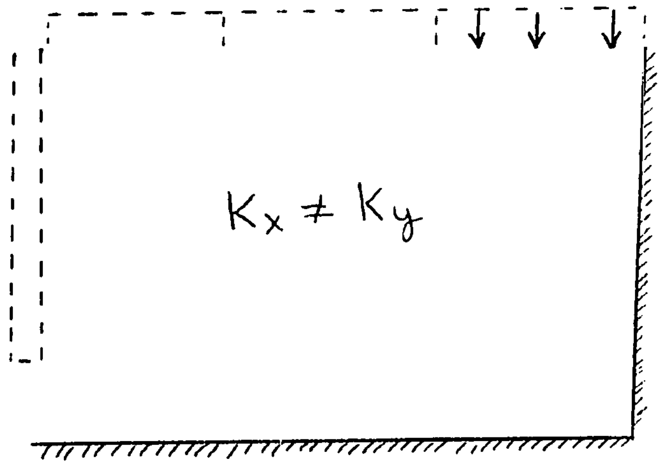
Problem	Configuration
I. Full length recharge and varying depths of penetration in a homogeneous, isotropic aquifer.	
II. Variable lengths of recharge and varying depths of penetration in a homogeneous, isotropic aquifer.	

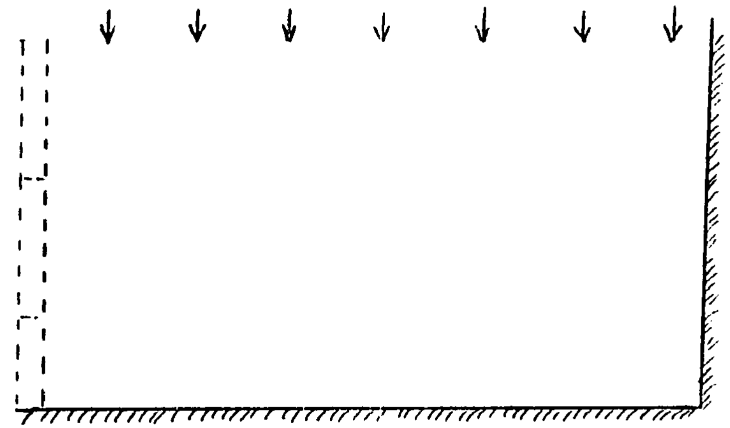
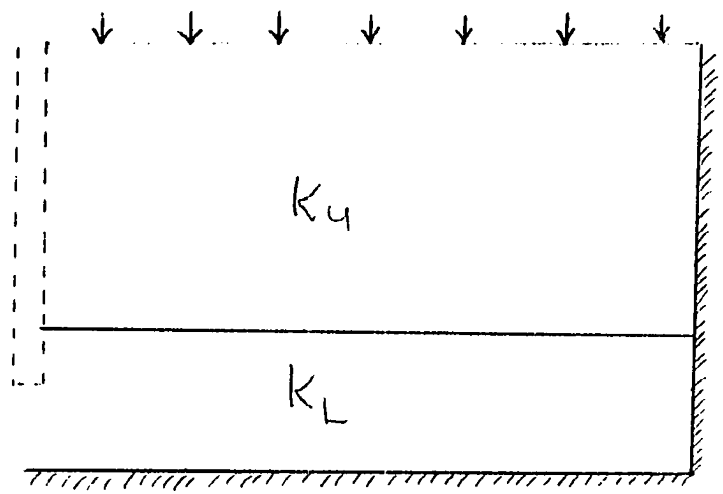
Table I-1 (continued)

Problem	Configuration
III. Variable recharge over aquifer which is dominantly conductive in X or Y direction.	

hydrodynamic dispersion on the pollutant breakthrough curves. Third, the solute transport model can produce actual numbers for some of the parameters of concern, whereas the plug flow model produces dimensionless ratios. The dimensionless ratios give good indications of the relative behavior of impermeable barriers; however, in dealing with ground water pollution control, one would like to have at least a feel for the magnitude of the changes in certain parameters. One of the most important parameters discussed in this report is that of a "time lag". The dimensionless curves give no feeling for the magnitude of this time lag, i.e., minutes, days, or years. The solute transport model, on the other hand, generates real numbers for this time lag. The fourth purpose of the solute transport model was to examine two situations for which the simple model was not suited; these situations are listed in Table I-2. One situation is that of a partially permeable barrier. The second situation is one where the barrier has been keyed into a relatively impermeable soil, but not into the underlying impermeable formation. The fifth reason for utilizing the solute transport model was to provide credibility to the results reported. Numerical models available from the U.S. Geological Survey have been calibrated, tested and verified. Utilizing a "proven" model lends some credence to the analysis. Additionally, the use of a previously developed model eliminates the unnecessary step of developing a model to solve the solute transport equation.

Reasons for selecting the K-B model were varied. First, although this model was originally developed for areal problems it is applicable to cross-section problems with minor adjustments. Second,

Table I-2: Additional Problems Examined By Solute Transport Model.

Problem	Configuration
I. Relatively but not completely impermeable barrier placed at varying depths in a homogeneous, isotropic aquifer subject to full length recharge.	
II. Variable recharge over aquifer in which barrier has been keyed into a relatively impermeable soil, but not the impermeable layer.	

the K-B model is extremely well documented and easily obtainable. Third, the K-B model uses finite-difference and method-of-characteristics techniques to solve the governing equations; these techniques are more easily understood than the more complex finite-element techniques.

The end result of the analyses outlined above is an assessment of the effectiveness of impermeable barriers not adequately keyed into an underlying impermeable formation. This assessment was accomplished utilizing two different techniques. First, a simple model based on the assumption of plug flow in the aquifer was utilized for a variety of different geometries. Second, a full scale solute transport model was utilized to examine the effects of hydrodynamic dispersion, to generate some preliminary numbers for certain parameters, and to examine some unique problems.

The preliminary investigation and literature reviews are discussed in Chapters II and III. The development of the simple model is discussed in Chapter IV. The results of both the analysis by the simple model and analysis by the K-B model are described in Chapter V. Chapter VI is a summary of results of the assessment of the effectiveness of barrier technology. Appendix I includes a program listing for the analytical solution and Appendix II includes a program listing for the simple plug flow model.

CHAPTER II

GROUND WATER MODELING

This research includes the modeling of a particular ground water system; therefore, this chapter presents a review of the literature on ground water modeling. The objective of this chapter is to provide basic information on ground water modeling and summarize the status of work in this field. This will be accomplished by an introduction to the types of models available and their basic theory, and a review and categorization of over 260 references from current literature.

Introduction to Modeling

Ground water modeling is a general term that encompasses several different aspects of the behavior of subterranean water systems. Four processes of potential relevance in any ground water modeling study include ground water flow, solute transport, heat transport, and deformation. Ground water flow modeling studies are usually undertaken to determine the responses of an aquifer to pumping, injection, or recharge stresses. Responses would include flow velocities and drawdowns (or upconing). Mass transport modeling studies are usually concerned with the movement within an aquifer system of a solute. These studies have become increasingly important with the current interest on ground water pollution. Heat transport models are usually focused on developing geothermal energy resources, and deformation studies are employed to analyze the effects of ground water removal on land subsidence.

The need for ground water models stems from the fact that the equations governing the behavior of the four processes above are all complex second order partial differential equations which are not amenable to analytical solution. Ground water models try to circumvent these difficulties by either; 1) simulating the behavior of the aquifer system on a small scale; or 2) using simplifying assumptions or numerical approximations to the governing equations.

Ground water models can be grouped into four broad categories; physical models, analytical models, stochastic models, and mathematical or numerical models. This chapter will briefly review the first three categories and then concentrate on numerical techniques used in ground water modeling studies.

Physical Models

The earliest attempts at modeling ground water were of the physical type. Physical models can be divided into two categories; scale models and analogs.

Scale Models

Scale models are actual physical replicas of an aquifer that have been "scaled down" for study in the laboratory. The most common scale models are the soil column (one-dimensional models), the Hele-Shaw apparatus (two-dimensional model), and the sand-tank (three-dimensional model). In these models, media is placed in such a way that it parallels the soil structure of the aquifer of concern. The models are then subjected to certain stresses such as water removal or injection, or contaminated recharge. The response of the models is obtained through direct measurements. The behavior of the prototype aquifer to real life stresses can then be predicted by using scale relationships.

Because of size restrictions, the scale factors will not be the same in all directions or for all soil parameters. For example, the equation for unsteady flow in a confined aquifer (written in polar coordinates) is

$$\frac{\delta^2 h}{\delta r^2} + \left(\frac{1}{r}\right)\frac{\delta h}{\delta r} + \frac{1}{r^2}\left(\frac{\delta^2 h}{\delta \theta^2}\right) = \frac{S}{T}\frac{\delta h}{\delta t} \qquad \text{(II-1)}$$

where

h = hydraulic head -- (length or L dimensions)

r = radius from well -- (L)

θ = angle from a given plane

S = storage coefficient of the aquifer -- (dimensionless)

T = transmissivity of the aquifer -- (L^2 divided by time or T)

The same equation applied to a scale model is

$$\frac{\delta^2 h_m}{\delta r_m{}^2} + \left(\frac{1}{r_m}\right)\left(\frac{\delta h_m}{\delta r_m}\right) + \frac{1}{r_m{}^2}\left(\frac{\delta^2 h_m}{\delta \theta_m{}^2}\right) = \frac{S_m}{T_m}\left(\frac{\delta h_m}{\delta t_m}\right) \qquad \text{(II-2)}$$

Letting U be defined as the ratio of the model parameter divided by the actual aquifer parameter, five different scale relationships can be identified:

$$U_r = \frac{r_m}{r},\ U_z = \frac{h_m}{h},\ U_t = \frac{t_m}{t},\ U_s = \frac{S_m}{S},\ U_T = \frac{T_m}{T}$$

Through substitution and algebraic manipulation, the equation governing the behavior of the model can be expressed in terms of its own measureable parameters and the various scale ratios. The equation is

$$\frac{\delta^2 h_m}{\delta r_m^2} + \frac{1}{r_m}\left(\frac{\delta h_m}{\delta r_m}\right) + \frac{1}{r_m^2}\left(\frac{\delta^2 h_m}{\delta \theta_m^2}\right) = \frac{S_m}{T_m}\left(\frac{\delta h_m}{\delta t_m}\right)\left(\frac{U_T \cdot U_t}{U_S \cdot U_r^2}\right) \qquad \text{(II-3)}$$

The above relationship holds only if

$$\left(\frac{U_T \cdot U_t}{U_S \cdot U_r^2}\right) = 1 \qquad \text{(II-4)}$$

Therefore, in the use of a physical scale model three of the scale ratios can be manipulated with the other ratio being dictated by Equation II-4 (DeWiest, 1965). The behavior of the model to stresses can then be measured and predictions can be made on the behavior of actual aquifers through use of the scale factors.

Analogs

Analog models are based on the fact that a direct analogy can be made between ground water flow and some easily measureable phenomena from a different field of study. One type of analog is the electric analog. In this type of model, the properties of an aquifer (permeability, storage coefficient, etc.) are simulated by various electronic components (resistors, capacitors, etc.), and the voltage across these components is analogous to the potential (or head) of water in the aquifer. Three of the more common electric analogs are discussed briefly below. Listed in Table II-1 are the relationships between aquifer parameters and their electrical analogs.

Table II-1: Analog Parameter Relationships (DeWiest, 1965)*.

Ground Water vs Electrical	Equation Form
R-C NETWORK	
Total flow (gallons) = C_1 x charge (coulombs)	$q = C_1Q$
Head (feet) = C_2 x potential loss (volts)	$h = C_2E$
Flowrate (gallons/day) = C_3 x current (amperes)	$Q = C_3I$
Time (days) = C_4 x time (seconds)	$t_d = C_4t_s$
Transmissivity (gal/day/ft) = C_2/C_3 x resistance (ohms)	$T = \frac{C_2}{C_3} R$
Storativity (dimensionless) = $\frac{C_1}{C_2}$ x capacitance (farads)	$S = \frac{C_1}{7.48a^2C_2} C$
	a = spacing of components (feet)
CONDUCTIVE-LIQUID/CONDUCTIVE-SOLID	
Head = C_1 x potential	
Hydraulic Conductivity = C_2 x electrical conductivity	
Specific Discharge = C_3 x current per unit area	
ELECTRICAL RESISTANCE	
Porosity = C_1 x resistance	

*C_1, C_2, C_3, C_4 all represent constants.

1. Resistor-Capacitance Network Analog

   The resistor-capacitance (R-C) network dissipates electrical energy in somewhat the same way a porous medium consumes ground water energy to let water travel through its voids.

2. Conductive-Liquid/Conductive-Solid Analogs

   These analogs utilize the difference in conductance of different materials to create a voltage potential. This voltage potential is analogous to the velocity potential or head in ground water flow.

3. Electrical Resistance Analog

   This is really a specialized case of the resistance-capacitance network analog. In this model variable resistors (rheostats) are used to simulate the fillable/drainable porosity of a soil. Hence these analogs will be most useful in studying time-varying recharge phenomena.

Another model analog for ground water flow can be constructed with a stretched thin rubber membrane. Small slopes of the membrane surface can be expressed in polar coordinates as follows (Todd, 1980):

$$\frac{d^2z}{dr^2} + \frac{1}{r}\frac{dz}{dr} = \frac{-W_m}{T_m} \qquad \text{(II-5)}$$

where

dz = deflection at a radial distance dr from a central deflecting point

W_m = weight of membrane/area

T_m = uniform membrane tension

If the right side of Equation II-5 can be set equal to zero, by using lightweight (small W_m) material or turning the model vertical, it becomes analogous to the steady-state ground water flow equation in polar coordinates.

$$\frac{\delta^2 h}{\delta r^2} + \frac{1}{r}\frac{\delta h}{\delta r} = 0 \qquad \text{(II-6)}$$

Therefore, measurements of the deflections of the membrane from protrusions become analogous to drawdowns from a well.

The final example of an analog is the thermal analog. It is known that the flow of heat in a uniform body and steady-state ground water flow both obey the Laplace equation as follows:

$$\nabla^2 Q = 0 \qquad \text{or} \qquad \nabla^2 h = 0$$

where

∇ = Laplacean Operator ($\nabla = \delta/\delta x + \delta/\delta y + \delta/\delta z$)

By adding a heat source or sink to a given material and measuring temperatures an analogy to ground water flow can be developed. Analogous parameters for thermal analogs are listed in Table II-2.

Summary of Physical Models

With the advent of high-speed computational capabilities, the use of physical models for predicting ground water flows has decreased. Physical models are also limited by space, time, cost, and accuracy deficiencies. However, scale models are currently being used in laboratory studies of the transport and fate of certain ground water contaminants. Analog models are not applicable to the movement of contaminants in ground water.

Analytical Models

Analytical models are usually developed by considering highly idealized conditions or using significant simplifying assumptions to obtain a solution to the governing equations. The most famous examples of analytical solutions are for the steady state flow of ground water

Table II-2: Thermal Analog Parameters

Ground Water	"is analogous to"	Thermal
Hydraulic conductivity	$\propto$	Thermal conductivity
Storativity	$\propto$	Model thickness x density x specific heat
Flow rate	$\propto$	Heat flow rate
Head	$\propto$	Temperature

under both confined or unconfined conditions using the Dupuit-Forchheimer assumptions. The Dupuit-Forchheimer assumptions are: (1) purely horizontal flow; and (2) flow uniformly distributed with depth. Utilizing these assumptions and employing Darcy's Law gives rise to two relationships describing the flow of ground water.

$$Q = \frac{2\pi KD(h_2-h_1)}{\ln(r_2/r_1)} \qquad \text{(II-7a) confined}$$

$$Q = \frac{\pi K(h_2^2-h_1^2)}{\ln(r_2/r_1)} \qquad \text{(II-7b) unconfined}$$

where

Q = volumetric flow rate -- (L^3/T)

h = hydraulic head -- (L)

K = hydraulic conductivity -- (L/T)

D = depth of aquifer -- (L)

r = radius from well -- (L)

π = 3.1415

ln = natural logarithm

Equations II-7a and II-7b are overly simplistic for all but the most general studies. Conversely, most other analytical solutions have been developed for extremely idealized or specific situations.

There have been several analytical technniques developed for analyzing ground water flow problems. The use of the "separation of variables" technique for solving the governing second order partial differential equation has been widespread. Freeze and Cherry (1979) used the technique to solve the equation for steady-state flow through a confined aquifer. Van Der Kamp (1973) used the technique to develop an equation describing periodic flow and wave propagation in ground water. An early application of the technique was done by Toth (1962) to the problem of subsurface flow to parallel drains. Kirkham and Powers (1972) have developed a general solution using the separation of variables technique. Kirkham and Powers have also outlined a general procedure for applying their solution to a variety of ground water flow problems. This procedure will be outlined in Chapter IV.

The use of analytical solutions to delineate flow paths has been applied extensively to the problem of subsurface flow to parallel drains. Jury (1975) has used the solution generated by Kirkham and a numerical integration technique to develop contaminant travel times for parallel drain situations. Reilly (1978) has used a dimensionless solution to develop a technique for determining contaminant travel times from the top of an aquifer to a pumping well. McLin (1980) has expanded the work of Jury (1975) by using a dimensionless analytical solution to verify and calibrate a numerical model. The model is then used to make multiple iterations for different geologic situations. McLin (1980) also used a numerical integration scheme to calculate pollutant breakthrough curves.

Bear (1979) discusses several analytical solutions to relatively simple, one-dimensional solute transport problems. However, even simple solutions tend to get bogged down with advanced mathematics.

Summary of Analytical Models

Analytical models represent an attractive alternative to both physical and numerical models in terms of decreased complexity and input requirements. Analytical models are often only feasible when based on significant simplifying assumptions. These assumptions may not allow the model to accurately reflect the conditions of interest. Additionally, even some of the simplest analytical models tend to involve complex mathematics.

Stochastic Models

Stochastic modeling involves using statistical methods applied to large amounts of aquifer data to generate empirical relationships between the various properties of the aquifer and its behavior. The objective of a stochastic model is that given a specific input, the model will generate as an output the "expected value" of a certain characteristic with a specified variability. Stochastic models are generally used for regional management purposes. Stochastic models are sometimes limited by their inherent large data requirements.

Bakr, et al. (1978) used statistical methods to examine the spatial variability in subsurface flows. Bathals, Ramachandra and Spooner (1977) have developed a statistical procedure for analyzing regional aquifer systems. Carlsson and Carlstedt (1977) use statistical analysis of pump-test data from wells to calculate average values of transmissivity and permeability. Other stochastic modeling examples are described by Delleur, et al., 1979; Gelhar, 1977; Newman and Yakowitz, 1979; Sagar, 1979; Smith and Freez, 1979; Ross, Koplik and Crawford, 1978; and Yakowitz, 1976.

Summary of Stochastic Models

The use of stochastic methods to analyze ground water systems has been minor and usually limited to analysis of regional problems or problems with significant amounts of input data. Stochastic models are limited by their large input data requirements.

Numerical Models

The most popular approach for modeling ground water behavior is through the use of numerical techniques. Numerical modeling techniques are really analytical models that are so large that they require use of digital computers, capable of multiple iterations, to converge on a solution. Numerical models can be grouped into two broad classifications based on their spatial approach to the aquifer: (1) lumped models; and (2) distributed models.

Lumped Models

The lumped models attempt to predict the behavior of the aquifer as a whole unit. The approach used is to estimate the total change in a given parameter as the difference between total input and output. For example, a simple water balance equation for a stream-connected phreatic aquifer system can be represented by (McLin and Gelhar, 1979):

$$n\frac{dh}{dt} = q_n + q_a - q_o - q_p \pm q_l \qquad \text{(II-8)}$$

where

h = average thickness of the ground water zone,

t = time

n = average effective porosity,

q_n = natural recharge rate per unit surface area,

q_a = artificial recharge rate per unit surface area,

q_o = natural aquifer outflow rate per unit surface area,

q_p = aquifer pumping rate per unit surface area,

q_l = river leakage rate per unit surface area.

The corresponding mass balance equation for the stream-connected phreatic aquifer system would be (McLin and Gelhar, 1979):

$$n \frac{d(h_c)}{dt} = q_n c_n + q_a c_a - q_o c - q_p c \pm q_e c_e + nhr \qquad \text{(II-9)}$$

where

r = volumetric source-sink term, that accounts for contaminant additions or degradation within the flow zone,

c = average aquifer concentration,

c_n = concentration of natural recharge,

c_a = concentration of artificial recharge.

Hence, an aquifer-wide accounting procedure for the parameters listed in Equations II-8 and II-9 will provide data necessary for a predictive model. This type of modeling is just now gaining popularity and, as such, the literature on the subject is limited.

Distributed Models

The second classification of numerical models are the distributed models. Distributed models use numerical techniques to describe the behavior of an aquifer at selected points (nodes). Several different numerical techniques exist for solving partial differential equations. However, the three most popular techniques for ground water studies are finite difference methods, finite element methods, and the method of characteristics. The theory behind the methods and their specific applications are outlined below.

1. Finite Difference Methods

Probably the simplest and most popular numerical technique for solving the partial differential equations governing ground water behavior is the use of finite differences. The technique is based on approximating the partial differential terms by their truncated Taylor's series expansion. Consider for example, the distribution of head in one dimension (Figure II-1). The finite difference approximation for the rate of change of head at some point i could be written

$$\frac{\delta h}{\delta x} \approx \frac{h(i) - h(i+1)}{\Delta x} \tag{II-10}$$

This is called the first forward approximation in that it involves the i+1 term. Similarly, the first backward approximation would be written

$$\frac{\delta h}{\delta x} \approx \frac{h(i-1) - h(i)}{\Delta x} \tag{II-11}$$

The central difference approximation is independent of the head at node i

$$\frac{\delta h}{\delta x} \approx \frac{h(i+1) - h(i-1)}{2\Delta x} \tag{II-12}$$

Similarly, the approximations for second order or higher derivatives can be derived. For example, the second difference at node i could be written

$$\frac{\delta^2 h}{\delta x^2} \approx \frac{h(i-1)-2h(i)+h(i+1)}{(\Delta x)^2} \tag{II-13}$$

Consider the equation for transient flow

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = \frac{S}{T}\frac{\delta h}{\delta t} \tag{II-14}$$

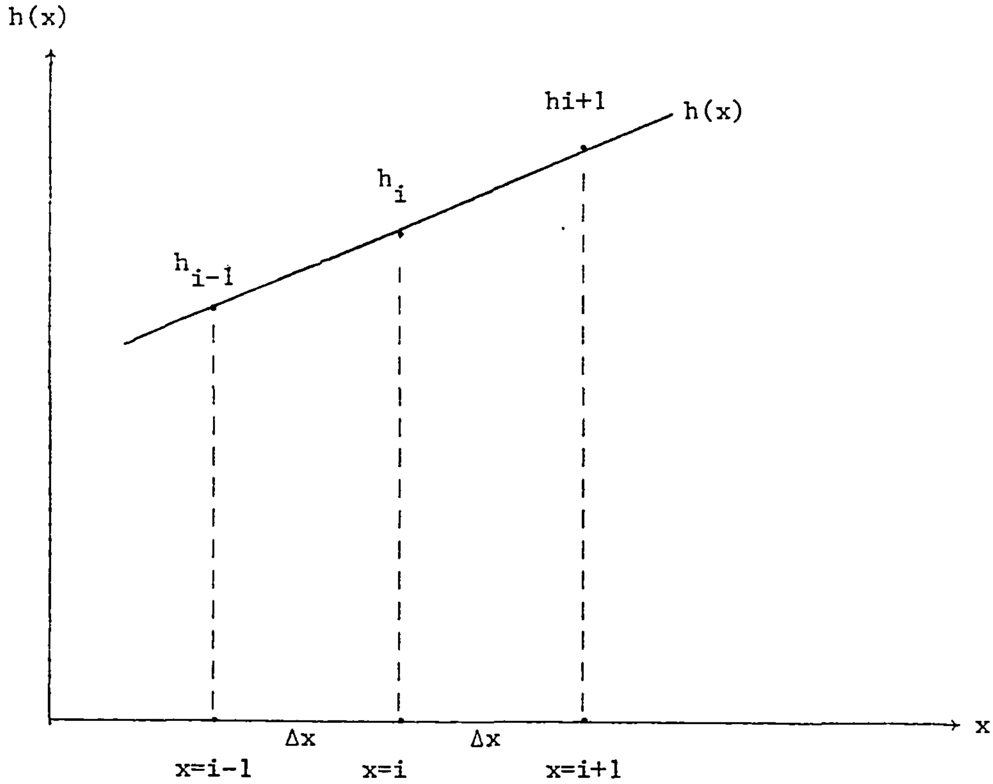


Figure II-1: Distribution of Head in One Dimension.

Using the forward difference approximation to both the spatial and temporal derivatives yields

$$\frac{h^{k}_{i-1,j}-2h^{k}_{i,j}+h^{k}_{i+1,j}}{(\Delta x)^2}+\frac{h^{k}_{i,j-1}-2h^{k}_{i,j}+h^{k}_{i,j+1}}{(\Delta y)^2}=\frac{S}{T}\left(\frac{h^{k+1}_{i,j}-h^{k}_{i,j}}{\Delta t}\right) \tag{II-15}$$

where

i = denotes position in x direction

j = denotes position in y direction

k = elapsed time

In Equation II-15 all values of h are known at the kth time step and the value of $h_{i,j}$ at the k+1 time step can be solved for directly. This is called an explicit scheme.

If Equation II-14 is written with the backward difference approximation for the time derivative the following is obtained:

$$\frac{h^{k+1}_{i-1,j}-2h^{k+1}_{i,j}+h^{k+1}_{i+1,j}}{(\Delta x)^2}+\frac{h^{k+1}_{i,j-1}-2h^{k+1}_{i,j}+h^{k+1}_{i,j+1}}{(\Delta y)^2}=\frac{S}{T}\left(\frac{h^{k+1}_{i,j}-h^{k}_{i,j}}{\Delta t}\right) \tag{II-16}$$

Equation II-16 has 5 unknowns. However, Equation II-16 can be written for each node in the grid resulting in a set of simultaneous equations. These equations can be solved to give a new value of h at each node for the k+1 time increment. This is called an implicit scheme.

Explicit schemes are simple to formulate, but have severe restrictions on the grid spacing and time increments. Implicit methods are more complicated, but more versatile. They require greater computer storage capacity, but use less running time than explicit methods. Implicit models are superior in that they also permit the use of larger time values.

There are quite a large number of models that have been developed using finite difference techniques. The majority of these models have been developed with a specific application in mind. Two models that have been developed for general application are outlined in Table II-3. These models have been used extensively with applications to a variety of problems.

2. Finite Element Methods

Whereas the finite difference techniques approximate the partial differential equations by a differential approach, the finite element method approximates the equations by the integral approach. The finite-element technique involves solving a differential equation for ground water behavior by means of a variational calculus. Consider the equation for two-dimensional non-steady ground water flow in a non-homogeneous aquifer

$$\frac{\delta}{\delta x}\left(K_x b \frac{\delta h}{\delta x}\right) + \frac{\delta}{\delta y}\left(K_y b \frac{\delta h}{\delta y}\right) + Q_s = S\frac{\delta h}{\delta t} \qquad \text{(II-17)}$$

The solution to this equation is equivalent to finding a solution for h that minimizes the variational function

$$F = \iint \left\{\frac{Kx}{2}\left(\frac{\delta h}{\delta x}\right)^2 + \frac{Ky}{2}\left(\frac{\delta h}{\delta y}\right)^2 + \left(S\frac{\delta h}{\delta t} - Q_s\right)h\right\}dxdy \qquad \text{(II-18)}$$

To obtain a numerical solution for this equation, the aquifer is sub-divided into "finite elements". The parameters Kx, Ky, S and Qs are kept constant for a given element, but they may vary from element to element. The differential $\delta F/\delta h$ is then evaluated for each node and equated to zero. The resulting system of simultaneous equations can then be readily solved by a digital computer (Todd, 1980).

Table II-3: Selected Available Ground Water Models Using Finite Difference Techniques

Prickett, T.A. and Lonnquist, C.G., "Selected Digital Computer Techniques for Groundwater Resource Evaluation", Illinois State Water Survey Bulletin, 55, 1971, Urbana, Illinois.

Generalized digital computer program listings are given that can simulate one-, two-, and three-dimensional nonsteady flow of ground water in heterogeneous aquifers under water table, nonleaky, and leaky artesian conditions. Programming techniques involving time varying pumpage from wells, natural or artificial recharge rates, the relationships of water exchange between surface waters and the ground water reservoir, the process of ground water evapotranspiration, and the mechanism of converting from artesian to water table conditions are also included. The discussion of the digital techniques includes the necessary mathematical background, documented program listings, theoretical versus computer comparisons, and field examples. Also presented are sample computer input data and explanations of job setup procedures. A finite difference approach is used to formulate the equations of ground water flow. A modified alternating direction implicit method is used to solve the set of resulting finite difference equations. The programs included are written in FORTRAN IV and will operate with any consistent set of units.

Trescott, P.C., Pinder, G.F. and Larson, S.P., "Finite Difference Model for Aquifer Simulation in Two Dimensions with Results of Numerical Experiments", U.S. Geological Survey Techniques of Water Resources Investigations, Book 7, Chapter C1, 1976.

The model will simulate ground-water flow in an artesian aquifer, a water-table aquifer, or a combined artesian and water-table aquifer. The aquifer may be heterogeneous and anisotropic and have irregular boundaries. The source term in the flow equation may include well discharge, constant recharge, leakage from confining beds in which the effects of storage are considered, and evapotranspiration as a linear function of depth to water. The theoretical development includes presentation of the appropriate flow equations and derivation of the finite-difference approximations (written for a variable grid). The documentation emphasizes the numerical techniques that can be used for solving the simultaneous equations and describes the results of numerical experiments using these techniques. Of the three numerical techniques available in the model, the strongly implicit procedure, in general, requires less computer time and has fewer numerical difficulties than do the iterative alternating direction implicit procedure and line successive overrelaxation (which includes a two-dimensional

Table II-3 (continued)

correction procedure to accelerate convergence). The documentation includes a flow chart, program listing, an example simulation, and sections on designing an aquifer model and requirements for data input. It illustrates how model results can be presented on the line printer and pen plotters with a program that utilizes the graphical display software available from the Geological Survey Computer Center Division. In addition the model includes options for reading input data from a disk and writing intermediate results on a disk.

Because of the complexity of the mathematical description, a finite-element method model would not be as easy to manipulate and apply to a unique problem. As such, most finite-element models have been developed for specific types of problems. The use of finite-element techniques in ground water modeling is increasing as knowledge of the numerical technique continues to spread.

3. Method of Characteristics

The equation used to describe the two-dimensional areal transport and dispersion of a given non-reactive dissolved chemical species in flowing ground water can be written as follows (Konikow and Bredehoeft, 1978):

$$\frac{\delta(Cb)}{\delta t} = \frac{\delta}{\delta x_i}\left(bD_{i,j}\frac{\delta C}{\delta x_j}\right) - \frac{\delta}{\delta x_i}(bCV_i) - \frac{C'W}{\varepsilon} \qquad i,j = 1,2 \qquad \text{(II-19)}$$

where

C = the concentration of the dissolved chemical species -- (mass or M divided by L^3)

D_{ij} = the coefficient of hydrodynamic dispersion (a second-order tensor) -- (L^2/T)

b = the saturated thickness of the aquifer -- (L)

C' = the concentration of the dissolved chemical in a source or sink fluid -- (M/L^3)

V_i = the seepage velocity in the direction of x_i -- (L/T)

K_{ij} = the hydraulic conductivity tensor -- (L/T)

ε = the effective porosity of the aquifer, (dimensionless)

W = the volume flux per unit area -- (L/T)

The first term on the right side of Equation II-19 represents the change in concentration due to hydrodynamic dispersion. The second term describes the effects of convective transport, while the third term represents a fluid source or sink (Konikow and Bredehoeft, 1978).

The finite difference and finite element techniques have been applied to both ground water flow and solute transport studies. The method of characteristics is a technique used specifically for solute transport problems, especially those situations where convective transport dominates.

The approach is not to solve the transport equation directly, but rather to solve an equivalent system of ordinary differential equations. The ordinary differential equations are obtained by rewriting the transport equation using the fluid particles as the point of reference. That is, instead of observing how the concentration changes with time at a fixed position in space, changing concentrations associated with fluid movement are noted. Therefore, the velocity distribution represents necessary information. In two dimensions, the end result is three equations for x-velocity, y-velocity and concentration; the solution of which are called the characteristic curves, hence the name, method of characteristics (Geo Trans, Inc., no date).

This is accomplished numerically by introducing a set of moving points (or reference particles) that can be traced within the stationary coordinates of a finite-difference grid. Points are placed in each finite-difference block and then allowed to move a distance proportional to the length of the time increment and the velocity at that point (see Figure II-2). The moving points effectively simulate convective transport because the concentration at each node varies as x finite-difference node

• reference particle

o new location

flow line

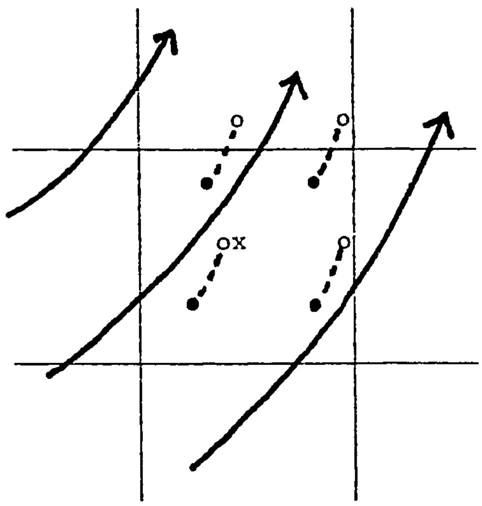


Figure II-2: Finite-Difference Grid Showing Reference Particles.

different points having different concentrations enter and leave the area of that block. Once the convective effect is determined, the remaining parts of the transport equation are solved using finite-difference approximations and matrix methods (Geo Trans, Inc., no date). Such a procedure is used in a two dimensional solute transport model developed by Konikow and Bredehoeft (1978).

Summary of Numerical Models

Numerical modeling techniques are not a cureall and can meet with considerable difficulties. Both the finite difference and finite element techniques can be plagued by the problems of numerical dispersion and oscillations when applied to solute transport problems. Numerical dispersion tends to smear or flatten out the effects of solute transport while numerical oscillation shows an inconsistent pattern in the results (Figure II-3). This is especially true in cases where convection dominates the transport process. Techniques have been devised to minimize the effects of these numerical difficulties. Upstream weighting can reduce oscillations and the use of small grid spacings can reduce the effects of numerical dispersion.

Numerical models can be of the lumped or distributed type. Lumped models are simple in theory but can require significant amounts of data. The use of lumped models is just now gaining popularity. Distributed models are based on approximating the governing equations and utilizing computer techniques to iteratively converge on a solution. The three major numerical methods utilized in distributed models are finite difference, method of characteristics, and finite

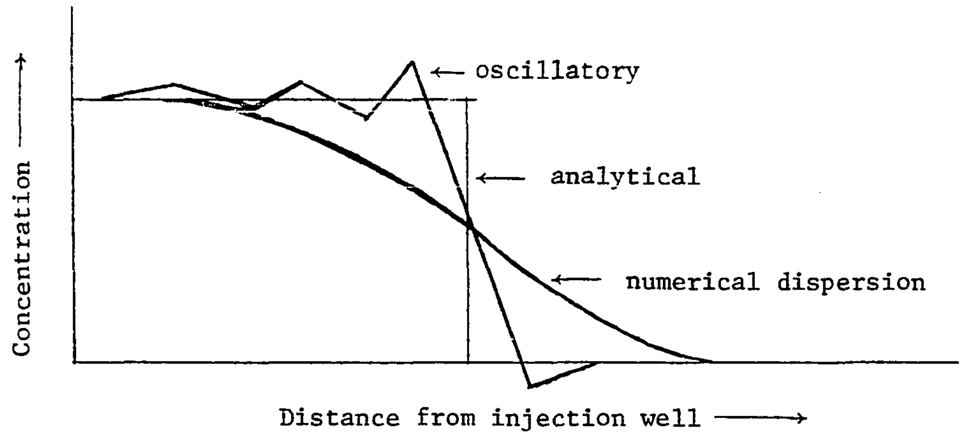


Figure II-3: Typical Numerical Solutions for Problem Having Convection Only.

element techniques. Finite difference and method of characteristic techniques are generally regarded as simpler in theory and formulation, but less accurate than finite-element techniques. All three methods are subject to certain numerical errors. Distributed models can require moderate amounts of input data just in crder to be calibrated.

Status of Ground Water Modeling

The state-of-the-art in ground water modeling in the United States is covered by Appel and Bredehoeft (1976); Bachmat, et al. (1978); Brown (1979); Dracup, et al. (1972); Evanson, et al. (1974); Lehman (1975); Lehman (1977); Prickett (1979); and Vahsteenkiste (1974). McLaughlin (1979) presents a critical review of uncertainty in model development and application at a specific site in the U.S., while Massing (1976) reviews the status of mathematical modeling in the Federal Republic of Germany. Water Research Center (1976) describes the formulation, calibration, and use in resource development and management of numerical models. Anderson (1979) provides an excellent review of the formulation of contaminant transport models, their application to field problems, difficulties in obtaining input data, and their curent status of usage. Gundlach (1978) describes the potential use of ground water models by a U.S. Army Corps of Engineers District Office. This study is of particular value in providing comparisons of models and practical information relating to model selection and usage. Moore (1980) illustrates, through case studies, how predictive ground water models have provided the information needed for sound management and planning of water resources in the United States.

Flow Models

The most basic task of any ground water model is to solve the flow equation. As mentioned previously, all models must solve the flow equation before any other tasks can be initiated. Several authors have described the development of models to solve the flow equation and they are listed in Table II-4.

In order to obtain accurate results with a flow model, the user must have values for the physical parameters of the aquifer, i.e., transmissivity, dispersivity, storage coefficient, and others. Many times these parameters are not available and are too expensive (in terms of money, time, or work required) to measure. In these instances, ground water models can be utilized to obtain these parameters by solving what is called the Inverse Problem through use of sensitivity analysis. By knowing drawdowns (or concentrations in the case of solute transport) at certain wells (obtained through pump tests), the parameters within the model can be changed until the prediction of drawdowns/concentrations by the model matches those actually observed. Having obtained the parameters, the model can then be applied to predict the long-term effects on the aquifer. Guvanson and Volker (1978), McElwee and Yukler (1978), Narasimhan (1979), Rushton (1978), and Tang and Pinder (1979) all apply the above method for determining transmissivity and storage coefficient. Gelhar (1977) and Newman and Wilson (1978) have looked at hydraulic conductivity. Tagamets (1973) and Umari, Willis and Liv (1979) have examined dispersivities. Other considerations have been: effective surface area, Claassen and White (1978); viscosity, Kimbler (1972); and

Table II-4: References on Flow Model Development

Bathals, Ramachandra and Spooner, 1977
Chan, et al., 1978
Cooley, 1973
Demarsily, et al., 1978
Fogg, et al., 1979
Gibbs, 1974
Khan, 1979
King, 1980
Langhette, 1974
Liggett, et al., 1979
Liu and Liggett, 1978
Mallory, 1979
Narashimhan, 1979
Prickett and Lonquist, 1971
Reed, Bodinger and Terry, 1976
Reisenauer, Cearlock and Bryan, 1975
Seckel, 1978
Texas Water Development Board, 1973

vertical layering, DeVries and Kent (1973). Newman and Yakowitz (1979) have used a stochastic model in parameter determination, and Warner and Yow (1979) developed parameter determination programs for hand calculators. Other references related to parameter determination are Aguado, Sitar and Remson (1977); Evanson (1973); Fields and Watson (1975); Lefebre (1977); Murty and Scott (1977); Navarro (1977); Narasimhan and Witherspoon (1977); and Williams and Liu, (1975).

These are numerous cases in which a model has been applied to a specific aquifer to determine if the aquifer can supply a given amount of water for a specified period of time and if so, what are the hydrological consequences, usually drawdown of the water table (unconfined aquifers) or the piezometric surface (confined aquifers). This is the most common application of ground water models. Authors who have developed flow models and made specific application of their models for this purpose are listed in Table II-5.

Another popular use of ground water models is to apply a flow model to an aquifer and another model to a surface water source and determine the consequences of conjunctive use of surface and ground water. By changing certain variables in each model and examining the tradeoffs associated with these changes, the optimum use of ground and surface water can be obtained. Brutsaert (no date); Haimes (1975 and 1976); Hays, Harp and Laguros (1975); and Knapp (1975) have all developed conjunctive use models. Cohn (1978), Boster and Martin (no date), and Brown and Deacon (1972) have included economic analyses in their conjunctive use models, while Bureau of Reclamation (1977a), Konikow and Bredehoeft (1974), and Perez, et al. (1974) have added

Table II-5: References on Specific Applications of Ground Water Flow Models for Aquifer Production

Bakr, et al., 1978
Birtles and Reeves, 1977
Cosner, 1975
Crist, 1975
Durbin, Kapple and Freckleton, 1978
Feldman, Whittlesey and Butcher, 1976
Haeni and Handman, 1977
Halepaska, 1974
Hermann, 1976
Hodgson, 1978
Houdaille, 1978
Kipp, et al., 1976
Klent, et al., 1976
Krishnamurthi, 1977
Land, 1975
Luzier, 1979
Maddox, 1975
Marie, 1976
McAvey, 1978
Morgan, 1974
Nichols, 1977
Planert, 1976
Robertson and Mallory, 1977
Sechrist, et al., 1970
Skrivan, 1977
Westphal, 1978
Wilson and Gerhart, 1979

ground water quality considerations to their conjunctive use models. Cunningham and Sinclair (1979) have developed a model which couples two-dimensional, transient, saturated subsurface flow and one-dimensional gradually-varied, unsteady, open-channel flow.

Mass Transport Models

The most widely studied aspect of mass transport modeling is solute transport which deals with modeling the fate and transport of pollutants. As with flow models, there are numerous different solute transport models, but these models all solve the same basic equation and usually only differ in their mathematical approach. A number of references related to solute transport models are listed in Table II-6.

Specific application of solute transport models for general water quality prediction has been done by Bredehoeft and Pinder (1973); Gorelick, Remson and Cottle (1979); Konikow (1977); Konikow and Bredehoeft (1978); Labodie, Kahn and Helwey (1976); Mercado (1977); Pimental (1977); Robson (1978 and 1974); Westphal, et al. (1976); and Young, et al. (1976). Gelhar and Wilson (1975) have applied a solute transport model to the problem of highway de-icing; and Brunch (1976) and Nishi, Brich and Lewis (1975) have studied ground water pollution control by electro-osmosis through the use of solute transport models. Warner (1979) discusses a digital transport model for diisopropylmethylphosphonate (DIMP) at the Rocky Mountain Arsenal, Colorado.

The largest specific applications of solute transport models have been to the problem of saltwater intrusion in coastal aquifers.

Table II-6: References on Solute Transport Models

Bredehoeft, 1976
Cabrera and Marino, 1976
Charbeneau, 1981
Dracup and Kaylus, 1974
Duquid and Reeves, 1976
Friedrichs, Cole and Arnett, 1977
Grove, 1977
Guymon, et al., 1970
Hunt, 1978
Jennings, Male and Adrian, 1980
Karplus, 1973
Kashef, 1975
Khaleel and Redell, 1977
Knapp and Padio, 1976
Konikow, 1976
Konikow and Bredehoeft, 1978
Land and Mountcastle, 1976
Landon and Metry, 1979
Marine, 1975
Pickens and Lennox, 1976
Prickett, Naymik and Lonnquist, 1981
Reddell and Sunada, 1970
Reddell and Sunada, 1969
Saleem, 1973
Sauty, 1980
Scott, 1969
Tyagi and Todd, 1971
Van der Veer, 1978

The objective of these models is to predict the movement of the saltwater-freshwater interface, resulting from the pumping of inland freshwater aquifers. A list of references on this topic is found in Table II-7.

A number of authors have applied mathematical modeling to the subject of wastewater disposal by injection. Development and application of models for municipal and industrial wastewater injection can be found in Grove (1976); Heidar and Cartwright (1974); McDonald and Fleck (1978); Peterson and Lau (1974); Thomas (1973); and Williams (1978). A highly studied aspect of wastewater injection is disposal in deep saline aquifers. Because of density differences between wastewater and brackish (saline) water, the injected wastewater will form a plume and rise and disperse. Models have been developed and applied to tracking the movement of these plumes by Henry, et al. (1972); Intercomp (1976); Larson, et al. (1977); Wheatcraft (1977); Wheatcraft, Peterson and Heutmaker (1976); and Williams (1977). Christensen (1978) addressed a unique problem in that he used a solute transport model to predict the movement of the saltwater-freshwater interface out of a freshwater aquifer as freshwater is injected.

Although the predominant application of the mass transport equation has been to conservative pollutants as outlined in solute transport, the equation can be modified to include a term that accounts for concentration changes of non-conservative pollutants due to chemical precipitation, decay, die-off, etc. Arnett, et al. (1976; Duguid and Reeves (1976); Grove (1970); Gureghian, Beskid and Marmer (1978); Holland and Wilson (1978); Tierney, Lusso and Shaw (1978); and

Table II-7: References on Saltwater Intrusion Models

Cheng, 1975
Christensen, 1978
Christensen and Rubin, 1978
Gardner, 1978
Helwog and Labodie, 1977
Heutmaker, Peterson and Wheatcroft, 1977
Kashef, 1968
Kashef, 1970
Kawatani, 1976
Kono, 1974
McWharter, 1972
Meyer, 1973
Michael, Gelhar and Wilson, 1972
Muller, 1974
Pagenkopf, 1978
Pinder, 1975
Pinder and Cooper, 1970
Pinder and Page, 1976
Rofail, 1977
Segol, Pinder and Gray, 1975
Shrivastava, 1978
Simundich, 1978
Stephenson, 1978
Van der Veer, 1977

Ward, Huff and Eyman (1979) have all applied the mass transport equation to the problem of underground radioactive waste disposal and subsequent transport through the ground water.

Recharge, Infiltration, and Leachate Models

Flow models and mass transport models are used in studies concerning recharge, infiltration, and leachates, and the amount of work done in this field dictates its being classified by itself. A popular application of the flow model has been to the problem of artificial recharge of aquifers. The proposed scheme is to recharge freshwater aquifers, through injection wells or recharge basins, in order to replenish dwindling supplies. The authors listed in Table II-8 have all applied the flow model to ground water recharge problems. The mass transport model has been applied by Edworthy, Stott and Wilkinson (1978); Swain (1978); Westphal, et al. (1976); and Wilson, Rasmussen and O'Donnel (1976) to predict the quality of water returning to an aquifer by artificial recharge.

The Bureau of Reclamation (1977c, 1977d, 1977e) has developed a return flow quality simulation model for modeling the plant-soil-aquifer system. Blanchar and Palmer (1979), Stolinski (1975), and Walker (1976) have also applied models to simulate irrigation runoff effects on ground water. Young, et al. (1976) have examined nitrate content of ground water on fertilized farmland.

A number of studies have been undertaken to determine the feasibility of land application to treat municipal and industrial wastewaters and their effects on underlying ground water. Weeter

Table II-8: References on Artificial Recharge Models

Bagdadipour, Harp and Laguros, 1971
Bianchi and Haskell, 1968
Brunch, 1973
Bureau of Reclamation, 1977
Chowdhury and Shakeja, 1978
Coleman, et al., 1972
Durbin, Kapple and Freckleton, 1978
Durbin and Morgan, 1978
Glugla, 1974
Harp and Laguros, 1972
Krashin, Peresunjko, 1974
Krishnamurthi, 1977
Lin, 1978
Panika and Mathus, 1969
Steinberg and Ragan, 1977

(1979) used mathematical models to determine evapotranspiration and infiltration rates for a proposed system using evapotranspiration as a means of treating septic tank and aerobically treated effluents. Orcutt (1976) developed an engineering-economic model of systems utilizing aquifer storage for the irrigation of public parks and golf courses with reclaimed municipal wastewater. Steinberg and Ragan (1977) used a 3-dimensional model to describe water table rise under a wastewater land application site. Ostrowski (1976) has applied a 2-dimensional model to the same problem. Willis (1976) used mathematical models to predict the time and spatial variations of conservative and non-conservative pollutant concentrations within the soil profile due to artificial recharge of an aquifer with municipal wastewater. Bouwer (1970) has used the analog technique to determine the horizontal and vertical hydraulic conductivity of the aquifer underlying the ground water recharge and sludge treatment system at Flushing Meadows, Arizona. Bausam and Schaub (1978) are using a mathematical model to predict enteric virus treatability of soil systems used for treatment of domestic wastewater. Konikow (1977) has examined chloride movement from an industrial waste disposal pond in Colorado. Welby (1981) has developed a graphical approach for predicting ground water pollutant movement.

Fuller (1977) has used a landfill simulation model for predicting solute concentration changes for 12 different chemical species found in municipal solid waste. Fungaroli (1971) has developed a model for predicting the quantity and time variation of leachates. Oztunali and Aikens (1980) have developed a mass transport model for shallow-buried hazardous wastes, and Lentz (1981) has examined seepage through a soil cover into a landfill.

Heat Transport Models

Heat transport models are similar to mass transport models in that they must first solve the ground water flow equation before solving the heat transport equation. Cheng, Yeung and Lau (1975); Cheng and Chau (1976); and Moore, et al. (1977) represent heat transport model development references, while Willhit and Wagner (1974) give examples of applying the heat transport model to the problem of waste heat disposal. Grubaugh and Reddell (1980) have developed a model to predict the response of a multiple ground water aquifer to the operat on of injection and pumping wells transporting hot water (heated by solar collectors) for long-term aquifer storage of solar energy.

Evaluation of Models

The dramatic increase in the use of ground water modeling has been accompanied by studies on the capabilities and limitations of both models and modelers. Mercer and Faust (1980a, b) have stressed the need for understanding, by the modeler, of the mathematical equations and initial and boundary conditions before a modeling problem can be formulated. Nelson and Schur (1978) have called for a preliminary evaluation of the capability of models. Anderson (1979) and Newman (1973) review problems associated with using inadequate field data concerning aquifer characteristics as input to models. Brissaud, Lambert and Couchet (1978); Gershon and Nir (1969); Gray and Pinder (1976); Nakano (1978); and Robson (1978) have examined the limitations of models due to inaccuracy of assumptions in the basic underlying equations and/or the initial and boundary conditions. Gates and Kisiel (1974) have examined the economic costs of various modeling strategies. Gureghian (1978) presents a comparison of the use of finite element versus finite difference numerical solution techniques. Karplus and Cardenas (1974) evaluate the advantages of using a unique computer language and Naymik (1978) examines the superiority of two-dimensional models over one-dimensional models. Dettinger and Wilson (1981) discuss the nature and sources of uncertainty encountered in ground water flow models. Prickett (1981) points out that educational materials are needed to help managers understand the purposes, capabilities, and results of mathematical models, because few existing models are sufficiently documented. It should be noted that despite all the references and applications, modeling techniques are not exact and there is no perfect model. A good number of the models now being promoted in the literature are application specific. Modeling studies can be costly and require trained personnel and adequate computer facilities. Modeling studies can also be time-consuming. All these factors should be considered prior to the initiation of any study.

Although somewhat simpler than transport models, ground water flow models are not without their limitations. The ability to adequately characterize the aquifer, both spatially and geologically, is necessary to the success of any flow studies. Flow models often are subject to input of rather narrow initial and boundary conditions which can limit their applicability to a given situation. As with any model, flow models will require at least moderate amounts of data in order to be calibrated.

Solute transport models are generally larger and more complex than flow models and as such, are subject to still more limitations. Data requirements are more extensive for solute transport models. This must be weighed with the fact that ground water quality monitoring has only recently received much attention. A problem unique to solute transport models relates to the input of aquifer dispersivities. Freeze and Cherry (1979) have addressed the problems associated with dispersivities and they state:

> Longitudinal dispersivity values determined by column tests are generally viewed as providing little indication of the in situ dispersivity of the geologic materials. Dispersivity has the distinction of being a parameter for which values determined on borehole-size samples are commonly regarded as having little relevance in the analysis of problems at the field scale.
>
> It is generally accepted that longitudinal and transverse dispersivities under field conditions are larger than those indicated by tests on borehole samples. In other words, tracer or contaminant spreading in the field as a result of dispersion is greater than is indicated by laboratory measurements. This difference is normally attributed to the effects of heterogeneities on the macroscopic flow field. Since most heterogeneities in geological materials occur at a larger scale than can be included in borehole samples, dispersivity values from tests on small samples can be viewed as representing a property of the medium but at a scale of insufficient size for general use in prediction of dispersion in the field.
>
> Studies of contaminant migration under field conditions require dispersivity measurements in the field. Although this premise is generally accepted, there is little agreement on the types of field dispersivity tests or methods for test analysis that are most appropriate. This state of affairs may be the result of the fact that relatively few detailed field dispersivity tests have been conducted, rather than a result of excessive difficulties of the task. It has only been in recent years that dispersivity at the field scale has received much attention. In comparison to the many thousands of field

hydraulic conductivity and transmissivity tests that have been conducted in the common types of geologic materials, only a few tens of field dispersivity tests are reported in the literature.

Another problem common to most all ground water models is complexity. In general, heat and mass transport models are more complex than flow models and finite-element techniques are more complicated than method of characteristics and finite-difference techniques. Since the application of any model will almost certainly require data input and possible model modifications, the complexity of a model can be a significant limiting factor.

Listed in Table II-9 are the modeling references grouped according to the type of model used and the phenomena investigated. Listed in Table II-10 are the modeling references grouped according to their specific application.

Table II-9: Modeling References Grouped According to Type

Phenomena	Model Type	References*
Ground Water Flow	Analog	Heisel, Moulton, Panika, Patten
	Analytical	Barends, Brown, Chan, Choudhury, Cohn, Faulkner, McLaughlin, Prickett, Vanderberg
	Stochastic	Bagdadipour, Bathals, Carlsson, Delleur, Gelhar, Newman, Rao, Sagar, Smith, Yakowitz
	Lumped Numerical	Birtles, Gelhar, McAvey
	Distributed Numerical	Aquado, Bianchi, Boster, Brown, Brunch, Brutsaert, Coleman, Cooley, Corapcioglu, Cosner, Crist, Cunningham, Daniels, DeMarsily, DeVries, Durbin, Evanson, Feldman, Fields, Fogg, Gates, Gibbs, Glugla, Gureghian, Guvanson, Haeni, Haimes, Halepaska, Harp, Hays, Helm, Hermann, Houdaille, Karplus, Khan, Kimbler, King, Kipp, Klent, Knapp, Kraeger, Krashin, Krishnamurthi, Land, Langhettee, Lentz, Liggett, Lin, Liu, Luzier, Maddox, Mallory, Marie, McElwee, Nakano, Narasimhan, Navarro, Naymik, Newman, Nichols, Planert, Prickett, Reed, Reisenauer, Robertson, Rushton, Sechrist, Seckel, Skrivan, Tang, USGS, Van der Veer, Williams, Wilson
	Systems Analysis, Linear Programming, Other	Dracup, Helwog, Hodgson

Table II-9 (continued)

Phenomena	Model Type	References*
Mass Transport	Analog	Bouwer, Michael, Orcutt, Wheatcraft, Williams
	Analytical	Tyagi, Walton
	Stochastic	Gelhar, Kaufman, Ross
	Lumped Numerical	None
	Distributed Numerical	Anderson, Arnett, Bausam, Blanchar, Bredehoeft, Brissaud, Brunch, Bureau of Reclamation, Cabrera, Charbeneau, Cheng, Christensen, Classen, Crouch, Dracup, Duguid, Edworthy, Fuller, Fungaroli, Gardner, Gershon, Gorelick, Gray, Grove, Gureghian, Guymon, Heidari, Heutmaker, Holland, Hsieh, Hunt, INTERCOMP, Jennings, Karplus, Kashef, Kawatoni, Khaleel, Kimbler, Knapp, Konikow, Kono, Labodie, Land, Landon, Larson, Lefebre, Marine, McDonald, McWhorter, Mercado, Meyer, Muller, Murty, Nelson, Nishi, Ostrowski, Oztunali, Pagenkopf, Perez, Peterson, Pickens, Pinder, Redell, Robson, Rochinskii, Rofail, Saleem, Sauty, Scott, Segol, Shrivastava, Simundich, Steinberg, STephenson, Stolinski, Swain, Tagaments, Theis, Thomas, Tierney, Umari, US EPA, USGS, Walker, Waller, Ward, Warner, Weeter, Westphal, Wheatcraft, Williams, Willis, Wilson, Young
	Systems Analysis, Linear Programming, Other	None

Table II-9 (continued)

Phenomena	Model Type	References*
Heat Transport	Analog	None
	Analytical	None
	Stochastic	None
	Lumped Numerical	None
	Distributed Numerical	Cheng, Dominico, Grubaugh, Henry, Heilman, Morre, Pimental, Wilhit
	Systems Analysis, Linear Programming, Other	None

*References are listed by the last name of only the first author in the reference citation.

Table II-10: Modeling References Grouped According to Application

Area	Authors
Aquifer Production	Bathals, Chan, Cooley, Cosner, Daniels, Delleur, DeMarsily, Durbin, Gibbs, Haimes, Halepaska, Helm, Hermann, Hodgson, Houdaille, Klent, Land, Liggett, Liu, Luzier, Maddox, Marie, McAvey, Mongan, Moulton, Narasimhan, Nichols, Patten, Planert, Prickett, Rao, Robertson, Seckel, Smith, Westphal, Wilson, Yakowitz
Conjunctive Use	Birtles, Boster, Brutsaert, Bureau of Reclamation, Cohn, Cunningham, Dracup, Feldman, Haimes, Hays, Knapp, Kraeger, Perez
Parameter Determination	Aquado, Bakr, Carlsson, Claasen, DeVries, Evanson, Fields, Gelhar, Guvanason, Haimes, Lefebre, McElwee, Murty, Narasimhan, Navarro, Nelson, Newman, Rushton, Tagaments, Tang, Umari, Willis
Recharge	Bagdadipour, Bianchi, Bouwer, Brunch, Bureau of Reclamation, Chowdhury, Coleman, Crist, Edworthy, Gershon, Glugla, Harp, Krishnamurthi, Langhettee, Lin, Orcutt, Panika, Sagar, Stolinski, Swain, Van der Veer, Willis, Wilson
Saltwater Intrusion	Christensen, Gardner, Heutmaker, Karplus, Kashef, Kawatuni, Kono, McWhorter, Meyer, Michael, Muller, Pinder, Rofail, Segol, Shrivastava, Stephenson, USGS, Wheatcraft
Wastewater Disposal and Injection	Anderson, Bausam, Bouwer, Brown, Brunch, Grove, Heidari, Henry, Hsieh, INTERCOMP, Jennings, Kaufmann, Kimbler, Larson, Orcutt, Perez, Peterson, Reddell, Steinberg, Thomas, Weeter, Wheatcraft, Williams, Willis
Irrigation/Stream-Aquifer	Blanchar, Bureau of Reclamation, Cabrera, Gelhar, Heluog, Konikow, Kraeger, Krashin, Labodie, Mercado, Stolinski

Table II-10 (continued)

Area	Authors
Leachates and Transport	Bredehoeft, Bureau of Reclamation, Crouch, Dracup, Duguid, Edworthy, Gershon, Grove, Hunt, Khaleel, Knapp, Konikow, Land, Nelson, Pickens, Redell, Robson, Rochinskii, Ross, Saleem, Scott, Simūndich, Swain, Theis, Tyagi, Van der Veer
Landfills	Fuller, Fungaroli, Lentz, Oztunali, USGS
Radioactive	Arnett, Duguid, Grove, Gureghian, Holland, Marine, Tierney, Ward

CHAPTER III

LITERATURE REVIEW ON IMPERMEABLE BARRIERS

This chapter presents a review of the current literature pertinent to the research topic. More specifically, this chapter will review the literature available on the different types of impermeable subsurface barriers and then discuss the work done on modeling subsurface pollutant transport as related to subsurface barriers. The three types of barriers (sheet piles, grout curtains, and slurry walls) will be discussed in terms of their construction, limitations, design, costs, and previous applications. The discussion on modeling will focus mainly on the work applied directly to subsurface barriers or the development of simplified analytical solutions.

Sheet Piling

Sheet piling involves driving lengths of steel[1] that connect together into the ground to form a thin impermeable permanent barrier to flow. Sheet piling requires that the steel sections be assembled prior to being driven into the ground. The lengths of steel have connections along both edges. The connections may be either slotted or

[1]Sheet piling materials also include timber and concrete, however, their application to polluted ground water cases is in doubt due to corrosive actions and costs, respectively.

ball and socket types. The sections are then driven individually into the ground by use of a pile hammer. The types of pile hammers include: drop, single-acting steam, double-acting steam, diesel, vibratory and hydraulic. For each type of hammer listed the driving energy is supplied by a falling mass, which strikes the top of the pile (Bowles, 1977). After the piles have been driven to their desired depth, the remaining above-ground portions are cut off. Initially, sheet piles are not totally impermeable because of small gaps in the connections. As time passes, these gaps are closed as ground water flow carries fine particles into the gaps and closes them by clogging.

Steel sheet piling can be considered permanent because experience has shown that corrosion is not a factor in causing failures (Bowles, 1977). One strong advantage of sheet piling is that the sections are reuseable and do not have to be left in place permanently. If conditions permit, steel piles may be removed and reused.

Construction of steel sheet piles as a means of ground water control can be effective and economical in some specific cases. In general, however, this is probably an over-elaborate technique to achieve a relatively simple result. As the size of a project increases, sheet piling will become uneconomical because of high material and shipping costs. In addition, pile driving requires a relatively uniform, loose, boulder-free soil for ease of construction. Other advantages/disadvantages are listed in Table III-1.

In situations where the function of the sheet pile is just to restrict ground water flow, i.e. no significant load resistance is required, a light weight steel will be adequate. Construction costs of

Table III-1: Advantages and Disadvantages of Steel Sheetpiles[a].

Advantages	Disadvantages
1. Construction is not difficult; no excavation is necessary.	1. The steel sheet piling initially is not watertight.
2. Contractors, equipment, and materials are available throughout the U.S.	2. Driving piles through ground containing boulders is difficult.
3. Construction can be economical.	3. Certain chemicals may attack the steel.
4. No maintenance required after construction.	
5. Steel can be coated for protection from corrosion to extend its service life.	

[a]Tolman et al., 1978.

a 1700 foot long and 60 foot deep light-weight steel cut-off wall are reported to range from $650,500 to $956,000 (Tolman et al, 1978). To date, sheet piling has been proposed as a means of ground water pollution control, but no specific applications have been reported. Sheet piling has been used for ground water flow control under dams and into deep excavations.

Grouting

Grouting is the process of injecting a liquid, slurry, or emulsion under pressure into the soil. The fluid injected will move away from the point of injection to occupy the available pore spaces. As time passes, the injected fluid will solidify resulting in a decrease in the original soil permeability and an increase in the soil bearing capacity.

Grouts are usually classified as particulate or chemical. Particulate grouts consist of water plus some particulate material which will solidify within the soil matrix. Chemical grouts usually consist of two or more liquids which will gel when they come in contact with each other. Listed in Table III-2 are materials commonly used for grouts. Listed in Table III-3 are the properties of cement grout additives.

Two of the more popular methods of grout installation are the stage and packer methods. In stage grouting, holes are drilled down to the geologic seam closest to the surface and the grouting fluid is injected. The holes are then cleaned, drilling continues down to the next seam and grouting continues. The process is repeated until a

Table III-2: Materials Commonly Used for Grout

cement, water
cement, rock flour, water
cement, clay, water
cement, clay, sand, water
asphalt
clay, water
various chemicals

Table III-3: Properties of Admixtures used with Cement Grouts

Admixture	Property
Calcium chloride Sodium hydroxide Sodium silicate	Accelerates setting time
Gypsum Lime sugar Sodium tannate	Retards setting time
Finely ground bentonite	Increases plasticity Reduces grout shrinkage
Clay Ground shale Rock flour	Reduces cost of grout Reduces strength of grout

sufficient depth has been obtained. In general, the stage method proceeds downward, utilizing increasing injection pressures. In the packer method, holes are drilled down to the maximum planned depth. A zone of specified thickness is then partitioned off by placing packers at the top and bottom of the zone. Grout is then injected into the zone between the two packers. The mechanism is then moved up to the next zone to be grouted and the process is repeated. The packer method moves upward from the bottom utilizing decreasing injection pressures. Advantages of the packer method include: grouting pressures can be adjusted specifically to a particular foundation depth; walls of the borehole remain smooth and an excellent seal with packers can be achieved; and high pressures used in the stage method may cause fracturation. However, these can be offset by increased equipment needs and time for installation (Bowen, 1981).

Another method of grout injection is the driven-rod method. In this method, a perforated rod is driven to a desired depth and grout is injected as the rod is slowly withdrawn. This method is limited to shallow depths and relatively boulder-free soils.

Problems that might arise during construction of grout systems include: leakage of grout around the injection pipe of the hole being grouted; loss of presssure below the water table resulting in sand and water being forced into the pipe; and grout surfacing in outlying areas due to lateral migration (Bowen, 1981). It is also desirable to deposit cement grouts in clean seams from which any clay or unconsolidated materials have been removed, thus adding the burden of washing boreholes (Peurifoy, 1979).

Construction of grout cutoff walls requires certain equipment in addition to normal borehole or drilling equipment. The list includes one or more air compressors; one or two grout mixers; one agitator-type reservoir tank; one or more grout pumps; and grout discharge pipes or hoses, valves, and pressure gauges (Peurifoy, 1979).

The first consideration in the design of a grout cutoff system is the actual composition of the grout. The composition will be a function of several variables including: the soil type to be injected into; the pollutant to be inhibited; and the time since pollution started and the time for installation. In general, chemical grouts must be used in fine-grained soils. However, chemical grouts (usually silicate) are not suitable for highly acidic or alkaline environments because their gel formation is an acid-base reaction. For coarse or gravel soils particulate grouts are suitable. The amount of cement or bentonite in a particulate grout varies widely. The quantity of bentonite which can be incorporated in a grout is dependent upon the following considerations: the workability of the mixture, increased bentonite concentrations increase the stiffness of slurry until it may become unpumpable; adding bentonite to cement slurries decreases their compressive strength; increased bentonite concentrations yield lower specific gravity slurries showing a reduced tendancy to migrate through the soil after placement; and increased bentonite concentrations reduce settlement or sedimentation of the slurries before injection (Jones, 1963).

A general guide for the selection of grouts is shown in Figure III-1. Knowing the soil type of interest, one can enter Figure III-1 at

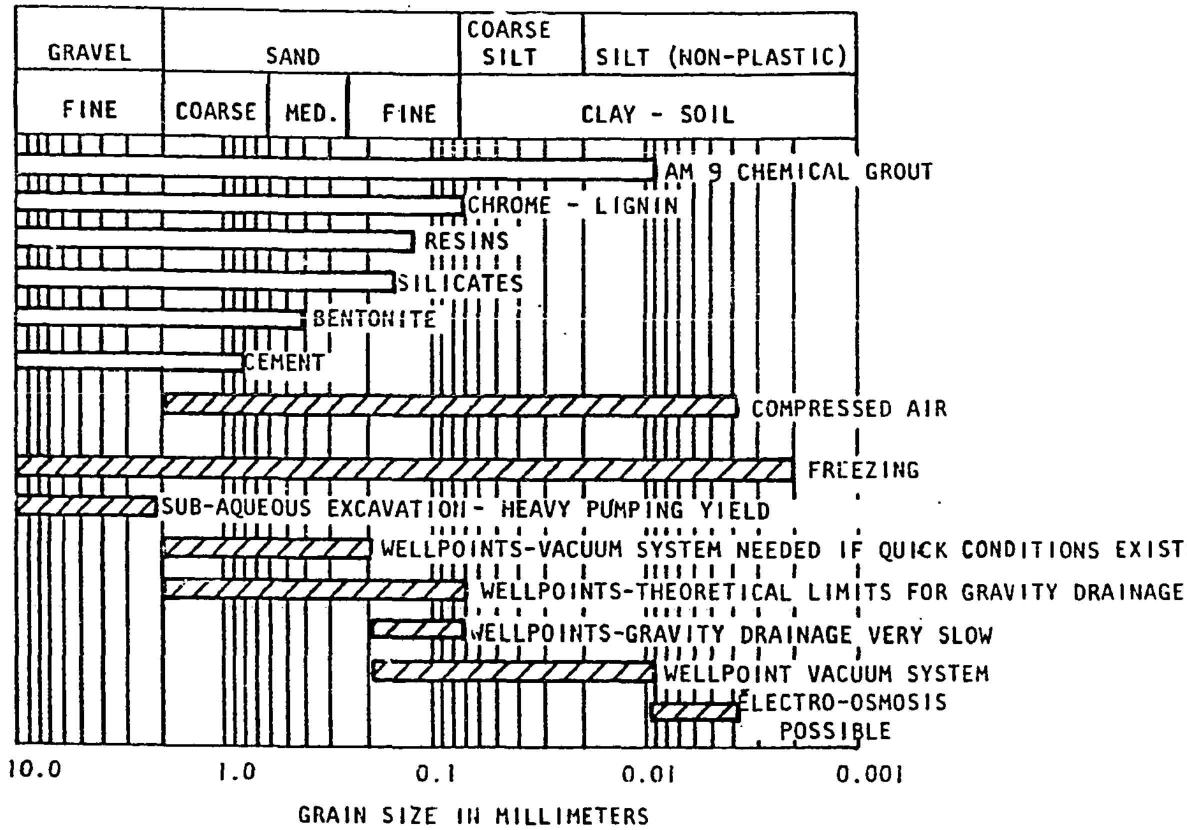


Figure III-1: Soil Gradation Limits for Grout Injection (from American Cyanamid Company).

the top and move down to determine both the applicable types of grouts and the available grouting procedures. The range of soil types to which a particular type of grout is applicable are covered by the solid white horizontal bars. The range of soil types to which a particular grouting procedure is applicable are indicated by the cross-hatched bars.

The second design consideration is the pressure at which the grout is to be injected. The use of excess pressures may weaken the strata by fissuring the rock or by opening fissures in otherwise closely jointed rock. Pressure-induced fissures will result in waste of grout. In contrast, the pressure should be kept high enough to ensure penetration of the grout and decrease the time required for grouting. The allowable grouting pressure is best determined by carrying out hydraulic fracture tests in the strata to be grouted (Morgenstern and Vaughan, 1963).

There are no legal or institutional measures aimed specifically at grouting technologies. However, a number of chemical grouting agents have been banned or their use discontinued because of their toxicity. Huibregtse and Kastman (1981) note that one of the most successful grouts, AM-9, is highly toxic and has been removed from most markets because of its potentially hazardous effects on ground water. Other grouts considered to be toxic and presenting a potential for ground water pollution are the lignin and formaldehyde-based grouts. Although not specifically mentioned, the use of chemical grouts might need to be analyzed for compliance with the Underground Injection Control Program developed by the U.S. Environmental Protection Agency.

Listed below is one of the five classifications of injection wells, covered by this program, which could conceivably include grouting procedures (Federal Register, 1979).

> Class III -- includes all special process injection wells, for example, those involved in the solution mining of minerals, in situ gasification of oil shale, coal, and so forth, and the recovery of geothermal energy.

The implementation of any grouting program should be preceded by an inspection of the regulations governing injection wells particular to the state of interest.

A number of physical/chemical advantages and disadvantages for grouting are listed in Table III-4. In general, grouts are applicable to a wide variety of soil types; grouts have been used for a number of years; and grouts have been proven successful. However, these positive attributes have only been identified for grouting applied to construction and soil stabilization projects. When applied for ground water pollution control, grouts show a more limited range of applicable soil types and conditions. Additionally, the effectiveness of grouts for this type of application has not been proven.

Costs for grout cutoff systems are high, hence they will be applicable only to small localized cases of pollution. Costs have been reported to range from \$142 to \$357 per installed cubic foot (Lu, Morrison and Stearns, 1981). For comparison of different grout types, Table III-5 lists the relative costs of grout.

The technology of grouting has been used in the construction industry for years. To date, most applications of grouting technology have been for increasing a soils' bearing capacity (to aid in tunnel

Table III-4: Advantages and Disadvantages of Grout Systems

Advantages[a]	Disadvantages
1. When designed on basis of thorough preliminary investigations, grouts can be very successful.	1. Grouting limited to granular types of soils that have a pore size large enough to accept grout fluids under pressure yet small enough to prevent significant pollutant migration before implementation of grout program.[b]
2. Grouts have been used for over 100 years in construction and soil stabilization projects.	2. Grouting in a highly layered soil profile may result in incomplete formation of a grout envelope.[b]
3. Many kinds of grout to suit a wide range of soil types are available.	3. Presence of high water table and rapidly flowing ground water limits groutability through; a. extensive transport of contaminants b. rapid dilution of grouts[b]
	4. Some grouting techniques are propietary.[a]
	5. Procedure requires careful planning and pretesting. Methods of ensuring that all voids in the wall have been effectively grouted are not readily available.[a]

(a) Tolman et al, 1978.
(b) Huibregtse and Kastman, 1981.

Table III-5: Relative Costs of Grout*

Type of Grout	Basic Cost Figure
Portland cement	1.0
Silicate base - 15 percent	1.3
Lignin base	1.65
Silicate base - 30 percent	2.2
Silicate base - 40 percent	2.9
Urea formaldehyde resin	6.0
Acrylamide (AM-9)	7.0

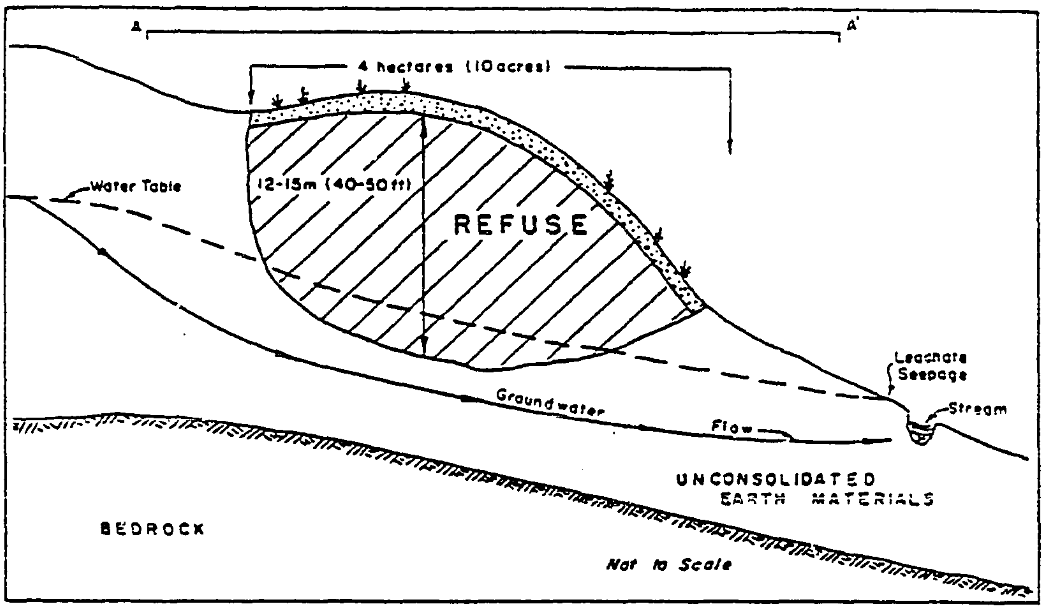
* Base unit = 1.0. Under a given set of conditions, where portland cement grout costs 1.0 times $/unit, other types of grout will cost the given figure times $/unit. (Tolman et al, 1978).

construction for example) or to decrease the permeability of a soil to inhibit water movement (such as a cutoff wall for a dam). The applications of this technology to the problem of ground water pollution are so recent that the data on their performance will not be available for a number of years. Huibregtse and Kastman (1981), however, have analyzed the feasibility of mobile grouting units for protecting ground water threatened by hazardous spills on land.

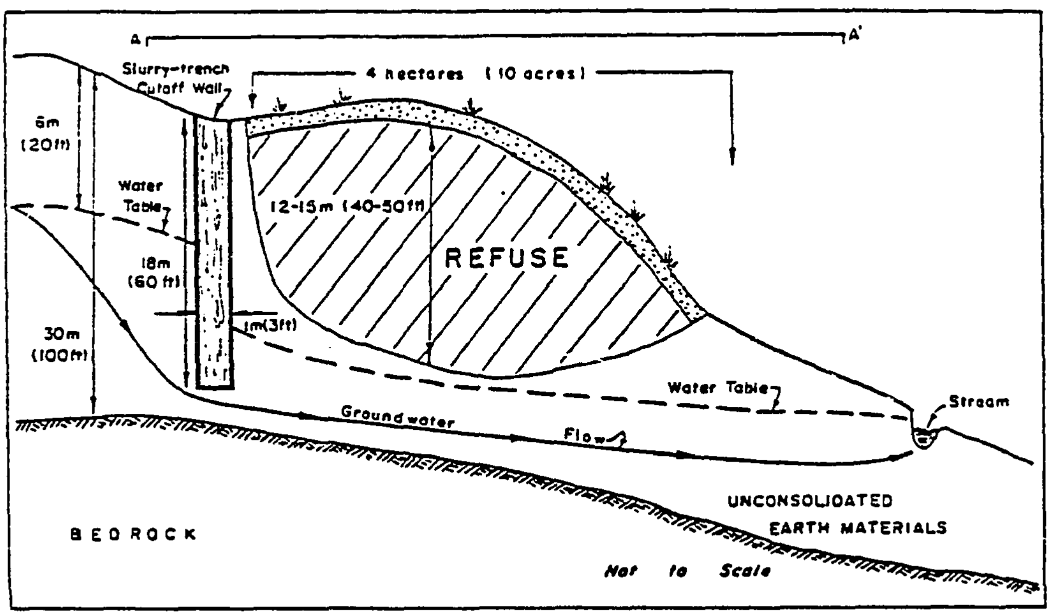
Slurry Walls

Slurry walls represent a technology for encapsulating an area to either prevent ground water pollution or restrict the movement of previously contaminated ground water. The technology involves digging a trench around an area and backfilling with an impermeable material. Slurry walls can either be placed upgradient from a waste site (Figure III-2) to prevent flow of ground water into the site, or placed around a site (Figure III-3) to prevent movement of polluted ground water away from a site. Usually, slurry walls will require a complementary technology such as surface capping or purge wells.

The most common type of slurry wall construction is the trench method. In this method a trench is excavated, in the presence of a bentonite-water slurry, down to a desired depth. After excavation, the trench can be solidified by backfilling with a mixture of bentonite and the excavated material or it can be allowed to solidify itself by incorporating cement in the original slurry. The backfilled trench is usually called a soil-bentonite (SB) trench, and the other method is called a cement-bentonite (CB) trench. Some advantages of each method are outlined in Table III-6.



(a)



(b)

Figure III-2: Cross Section of Landfill Before (a) and After(b) Slurry-Trench Cutoff Wall Installation (Tolman, et al., 1978).

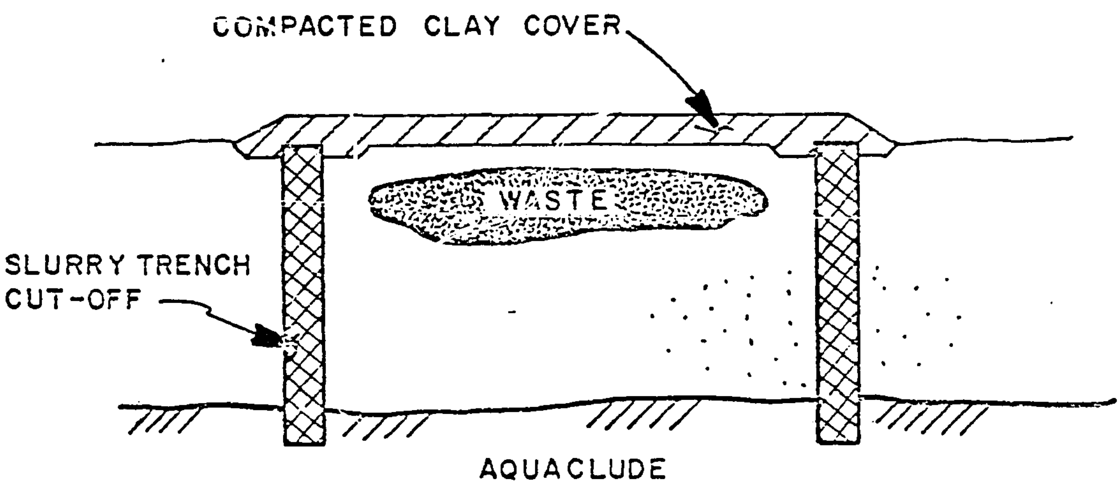


Figure III-3: Isolation of Existing Buried Waste (Ryan, 1980).

Table III-6: Comparison of Cement-Bentonite and Soil-Bentonite Slurry Trenches (Ryan, 1980)

Cement-Bentonite (C-B)	Soil-Bentonite (S-B)
1. Independent of availability or quality of soil for backfill.	1. Lower material costs.
2. More suitable for limited access areas.	2. Can achieve lower permeability than C-B.
3. Cement sets quickly. Can cut trenches or allow traffic over wall in just a few days.	
4. Can be constructed in sections. S-B requires continuous trenching in one direction.	

Another method of slurry wall construction is the vibrating beam method. In this approach, a beam with a pressure hose attached is driven into the ground, then slurry is injected under pressure as the beam is gradually withdrawn. This is similar to grouting and possesses similar advantages/disadvantages. This approach can be more economical and proceed faster than the trench method. However, the continuity of the wall cannot be guaranteed, the in-place thickness of the wall is considerably smaller, and the presence of boulders in the soil hinders this method more than the trench method.

Construction of slurry trenches is generally simple and consists only of excavating, recirculating the slurry, and backfilling. Excavation can be accomplished by any one or combinations of the following: backhoe, draglines, clamshells, bucket scrapers, or rotary drilling equipment. The choice of the specific type of excavation equipment is generally governed by the depth and width of excavation (Xanthakos, 1979). The backhoe is usually desirable when depths required are shallow. For depths of 30 meters or more, draglines are required. Because dragline bucket widths usually exceed 2 meters, they are not economical for C-B trenches due to high material costs. Clamshells can be cable mounted or, like the bucket scraper, mounted to a rigid sliding bar and used for deep trench excavation (Ryan, 1980). Recirculation of the slurry is important for maintaining the integrity of the slurry. During excavation the slurry will be subjected to losses through seepage and changes in density through addition of excavated material. Control of these changes is achieved by continuously recirculating the slurry through a central mixing unit which may have provisions for separating excavated materials from the original slurry. Backfilling operations may require mixing of different soil types prior to placement in the trench. Mixing can be accomplished by discing and blading. The mixed soil is then bulldozed into the trench, partially mixing with and displacing the slurry. Backfilling follows trenching after an interval of time sufficient to prevent interference between the two activities.

An important final aspect of slurry trench construction is keying into an underlying impervious zone. Trenches will require 2 or 3 feet for keying into clay materials and will require a grout connection when keying into impervious bedrock (D'Appolonia, no date).

The important design considerations of any slurry-trench are the composition of the slurry and the ensuing impermeable wall. Design procedures for slurry walls range from general rules of thumb to overly detailed analysis of all aspects of the system. A general guidelines approach is probably most helpful.

D'Appolonia (no date) suggests that viscosities of both C-B and S-B slurries should be such that drain times from a Marsh Funnel range from 40 to 50 seconds. It is also recommended that the specific gravity of the slurry be at least 15 pounds per cubic foot less than the unit weight of the backfill. The permeability of an S-B cut-off is essentially equal to the permeability of the backfill material. The permeability of the S-B backfill will depend on the soil and the amount of bentonite blended in. These characteristics are depicted in Figures III-4 and III-5. The ideal consistency for backfill placement is a paste having a water content slightly above the liquid limit of the

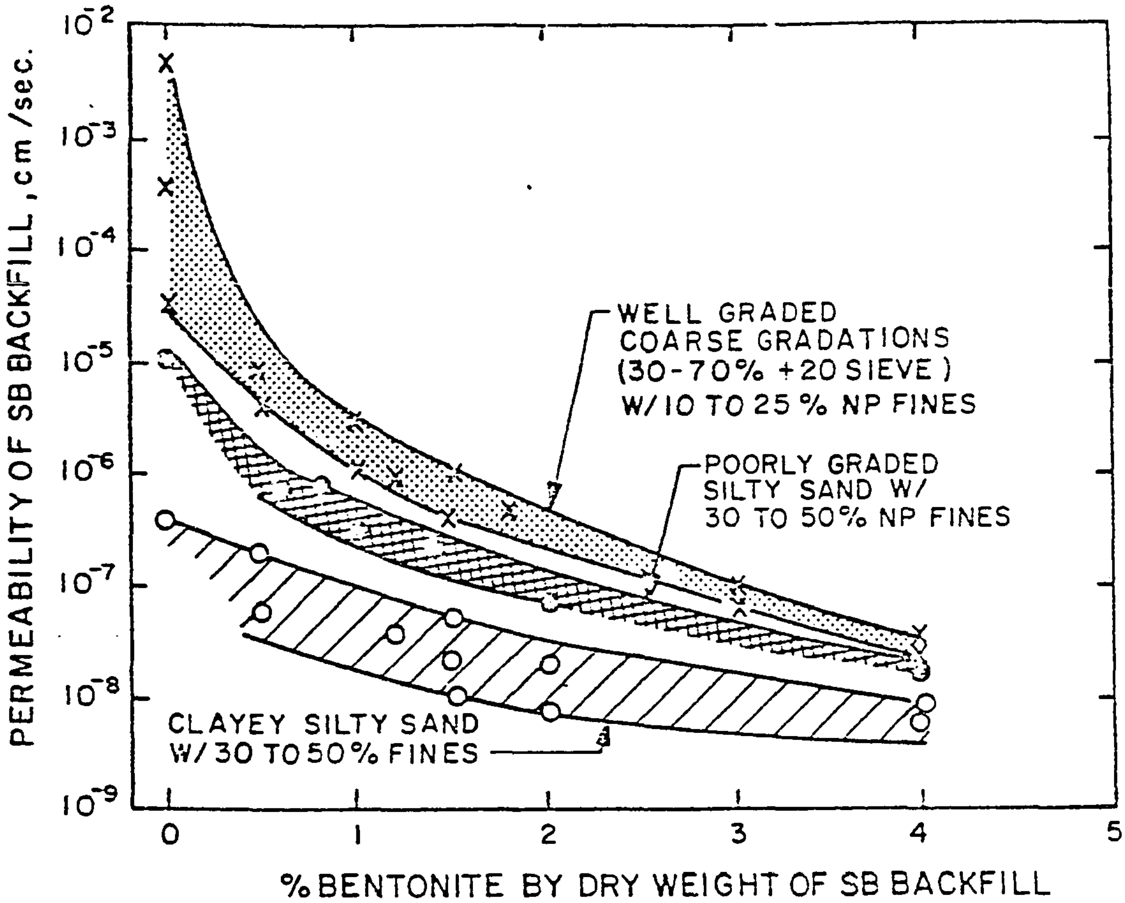


Figure III-4: Relationship Between Permeability and Quantity of Bentonite Added to Soil-Bentonite Backfill (Ryan, 1980).

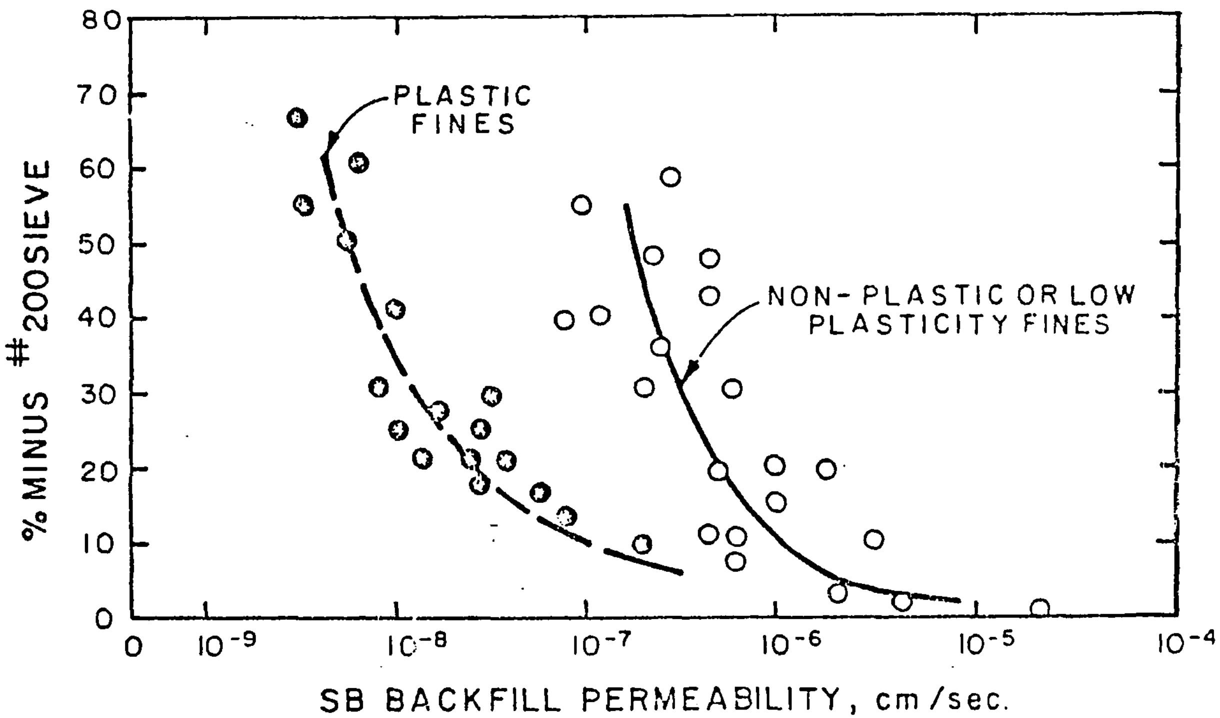


Figure III-5: Permeability of Soil-Bentonite Backfill Related to Fines Content (Ryan, 1980).

sand-clay-bentonite backfill mix. This usually corresponds to a slump cone reading of 2 to 6 inches. Durability of the slurry-trench refers to its resitance to attack from contaminants. In the presence of clean ground water both C-B and S-B trenches show little deterioration and can be considered permanent. However, C-B trenches show poor performance records where acids or sulphates are present. Similarly, exposure of an S-B trench to certain contaminants can lead to increased permeability through 1) pore fluid substitution or 2) the increased solubility of barrier minerals in the contaminant fluid. Pore fluid substitution can be the result of high concentrations of salts, which attract the waters of hydration, or it can result from ion substitution within the clay matrix. S-B trench permeabilities have been shown to increase in the presence of certain organics, calcium, magnesium, heavy metals and solutions of high ionic strength (D'Appolonia, no date).

Xanthakos (1979) suggests that a slurry system should be designed based on the functions of the slurry. These functions are:

1. Support the face of the excavation and also prevent the soil from sloughing and peeling off.

2. Seal the formation and form the filter cake, preventing slurry loss to the ground.

3. Suspend detritus, thereby preventing sludgy unconsolidated layers from accumulating on the bottom of the trench.

4. Carry the cuttings in the slurry volume, thereby preventing sedimentation in the mud circuit (Xanthakos, 1979).

All of the above functions can be controlled by manipulating the physical properties of the slurry, i.e., density, viscosity, etc. Xanthakos (1979) has outlined a series of simple steps and procedures for proportioning the materials that make up a slurry. These are outlined below:

1. Determine the density required for trench stability. The density can be controlled by the presence of colloid and non-colloid solid materials. If the depth to an impermeable formation (H) is known, the required slurry density can be calculated by the following (see Figure III-6).

$$\gamma_f = \frac{\gamma(1-m^2)K\alpha + \gamma^1 m^2 K\alpha + \gamma_w m^2}{n^2}$$

$$K\alpha = \tan^2(45-\theta^1/2)$$

γ_f = required slurry density

where γ = unit weight of soil

γ_w = unit weight of water

γ^1 = effective (buoyant) weight of soil

θ^1 = angle of shear resistance.

n = slurry level as a fraction of total depth of excavation (H)

m = natural ground water level as a fraction of total depth of excavation (H)

2. Select the funnel viscosity by reference to Table III-7.

3. Establish any applicable control limits from Table III-8.

4. Determine whether control agents (peptizers, polyelectrolytes, fluid-loss-control materials, etc.) are necessary and economically justified.

5. Proportion the constituent materials (water, bentonite, control agents, and non-colloid solids). This phase merely consists of a quantitative estimation. The proportioning may be empirical and depend on experience if the properties of the materials selected are known, or it may have a technical basis of tests and estimations.

In general, slurry-trenches are attractive alternatives when an impervious natural barrier exists at a reasonable depth and the waste area is relatively large. In order to obtain a low permeability,

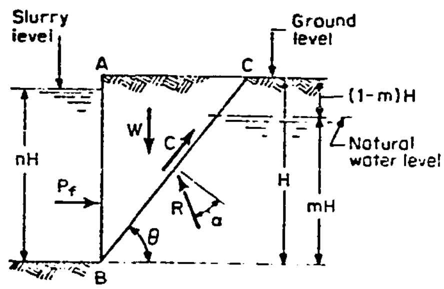


Figure III-6: Stability of a Trench for Arbitrary Slurry and Natural Water Level (Xanthakos, 1979)

Table III-7: Funnel Viscosity for Common Types of Soil (Xanthakos, 1979)

Type of soil	Funnel viscosity s/946 cm^3*	
	Excavation in dry soil	Excavation with groundwater
Clay	27–32	
Silty sand, sandy clay	29–35	
Sand, with silt	32–37	38–43
Fine to coarse	38–43	41–47
And gravel	42–47	55–65
Gravel	46–52	60–70

*Time is that required for 946 cm^3 to flow through funnel of standard dimensions.

Table III-8: Control Limits for the Properties of Slurries (Xanthakos, 1979).

	Property							
Function	Average bentonite concentration,† %	Density, lb/ft³	sp gr	Plastic viscosity, centipoises	Marsh cone viscosity	10-min gel strength (Fann), lb/100 ft²	pH	Sand content, %
Face support	>3-4	>64.3	>1.03	....	Limits	‡	....	>1§
Sealing process	>3-4	......	.....	....	established	....	....	1
Suspension of detritus	>3-4	......	.....	....	by soil type	>12-15		
Displacement by concrete	<15	<78	<1.25	<20		....	<12	<25
Separation of noncolloids	.....	......	.....	....		....	....	<30
Physical cleaning	<15	<78	<1.25	....		....	....	<25
Pumping of slurry	.....	......	.....	....		Variable		
Limits	>3-4	>64.3	>1.03	<20		>12-15	<12	>1
	<15	<78	<1.25	....		....	....	<25

*Controls are not considered necessary for apparent viscosity and yield stress. Whereas fluid loss commonly is judged by standard filtration test and a maximum film thickness of 2 mm, better control limits are established by stagnation-gradient tests.

†Should be expected to vary widely because of different bentonite brands.

‡The shear strength of filter cake is more applicable to peel-off control (also the time required for its formation).

§Optional.

contaminant-resistant backfill, a high percentage of fine plastic material must be used. A clayey sand or sandy clay containing 30 to 60% fines blended with the bentonite slurry is usually satisfactory for most waste isolation applications (D'Appolonia, no date).

A list of the advantages/disadvantages of slurry-trenches compared to grouting, sheet piling, pumping or other techniques is shown in Table III-9.

Costs for a 60 foot deep, 3 foot wide trench have been estimated to range from $294 to $495 per lineal foot (Tolman, et al., 1978).

There are many instances where slurry wall technologies have been employed. Slurry systems have been employed for their impermeability as cutoff and diaphragm walls and slurries have been used to aid in the construction of load bearing elements and foundations. For excellent discussions of each of these types of applications, the reader is referred to Xanthakos (1979).

To date, there has been very little information provided on the performance of slurry walls applied to ground water pollution control problems. This does not mean the technology has not been applied. Slurry walls have been constructed at the Rocky Mountain Arsenal in Colorado and the Gilson Road Hazardous Waste Dump in New Hampshire to name two of the more well-known applications. However, no information has been published on the performance of these or any other slurry walls.

Table III-9: Advantages and Disadvantages of Slurry Trenches

Advantages	Disadvantages[a]
1. Construction methods are simple.[a]	1. Cost of shipping bentonite from west.
2. Adjacent areas not affected by ground water drawdown.[a]	2. Some construction procedures are patented and will require a license.
3. Bentonite (mineral) will not deteriorate with age.[a]	3. In rocky ground, over-excavation is necessary because of boulders.
4. Leachate-resistant bentonites are available.[a]	4. Bentonite deteriorates when exposed to high ionic strength leachates.
5. Low maintenance requirements.[a]	
6. Eliminate risks due to strikes, pump breakdowns, or power failures.[b]	
7. Eliminate headers and other above ground obstructions.[b]	

(a) - Tolman et al., 1978

(b) - Ryan, 1980

Modeling of Impermeable Barriers

A review of the literature on ground water modeling in general is presented in Chapter II. The review presented below concerns itself only with information presented on the behavior of impermeable barriers and modeling studies directed specifically at impermeable barriers.

To date the emphasis of the literature has been toward describing construction techniques and outlining design procedures for impermeable barriers. A minimal amount of work has been done on evaluating the efficiency of such systems for inhibiting ground water movement. Xanthakos (1979) describes a procedure for evaluating the efficiency of slurry cutoff systems. The discussion is based on the work of Ambraseys (1963) who outlines a procedure in which seepage loss is estimated by assuming that there exists openings in the wall, evenly distributed as a group of parallel slits.

Little information is available on the ability of impermeable barriers to retard pollutant migration in a quantitative sense. Several authors have qualitatively described the limitations of the various systems. Tolman (1978) has noted that bentonite deteriorates when exposed to high ionic strength leachates. D'Appolonia (no date) reinforces this point by stating that special design considerations should be taken when a slurry wall will be subject to permeation by contaminated water. D'Appolonia (no date) also presents some results on work done to determine the amount of permeability increase of bentonite filter cakes exposed to various permeants (Tables III-10 and III-11). Hughes (1975) also notes that bentonite slurries can increase in permeability upon exposure to substantial quantities of dissolved

Table III-10: Increase in Permeability of Bentonite Filter Cakes Caused by Leaching with Various Pollutants (D'Appolonia, no date).

Permeant	Final Permeability/Initial Permeability			
	SB 125	NPB	SS 100	M 179
Lignin in Ca^{++} solution	1.9	1.5	2.5	1.4
NaCl based salt solution (conductivity 170,000)	2.7	1.8	2.7	
Amonium Nitrate (10,000 PPM)	1.8		2.8	
Phenol and salt solution (conductivity 30,000)	1.4	1.5		1.4
Acid Mine Drainage (pH ≈ 3)		1.5	1.3	
Calcium and Magnesium salt solution (10,000 PPM)	2.9	3.2	3.2	

Bentonites:

SB 125	Slurry Ben 125
NPB	National Premium Brand
SS 100	Saline Seal 100
M 179	Dowell M 179

Table III-11: SB Permeability Increase Due to Leaching with Various Pollutants (D'Appolonia, no date).

Pollutant	SB Backfill (Silty or clayey sand) 30 to 40% fines
Ca^{++} or Mg^{++} @ 1000 PPM	N
Ca^{++} or Mg^{++} @ 10,000 PPM	M
NH_4NO_3 @ 10,000 PPM	M
Acid (pH>1)	N
Strong Acid (pH<1)	M/H*
Base (pH<11)	N/M
Strong Base (pH>11)	M/H*
Benzene	N
Phenol Solution	N
Sea Water	N/M
Brine (SG=1.2)	M
Acid Mine Drainage ($FeSO_4$ pH≈3)	N
Lignin (in Ca^{++} solution)	N
Organic residues from pesticide manufacture	N
Alcohol	M/H

N - No significant effect; permeability increase by about of factor of 2 or less at steady state.

M - Moderate effect; permeability increase by factor of 2 to 5 at steady state.

H - Permeability increase by factor of 5 to 10.

* - Significant dissolution likely.

inorganic or organic material. However, Hughes (1975) also notes that this phenomena can be reduced by prehydration of the bentonite with fresh water or by using some of the new contaminant-resistant bentonites. Huibregtse and Kastman (1981) have outlined several limitations of the grouting technology which might inhibit their performance in preventing pollutant migration. Additionally, Tolman, et al. (1978) points out that no method exists for ensuring that all the voids in a grout wall have been effectively closed. Tolman, et al. (1978) notes that one limitation of steel sheet-piling is that it is not initially watertight.

The single most important factor of all three systems is that of an adequate connection between the barrier and an underlying formation. D'Appolonia (no date) says that experience has shown that when slurry walls have failed to perform as expected, the "failures have been due either to imperfect connection between the slurry trench cut-off and the underlying aquiclude or failure to completely excavate the slurry trench, thereby leaving zones of unexcavated pervious material above the aquiclude". This point is also noted by Millet and Perez (1981) who state that it is reasonable to specify a 1 or 2 foot penetration into the aquiclude.

Warner (1979) presents the results of a modeling study done to determine the effects on ground water movement and on solute concentrations of a bentonite barrier in the aquifer near the Rocky Mountain Arsenal, Colorado. The model assumed conservative (nonreactive) transient transport of the solute and steady-state ground water flow. However, in the simulations it was assumed that the barrier was impermeable and totally penetrated the entire saturated thickness of the aquifer. The study concluded that the use of this idealized bentonite barrier in conjunction with removal wells could effectively control the movement of the pollutant.

Hammond and Metry (no date) have studied the effectiveness of ion exchange barriers for retarding the migration of radionuclides buried in low-level radioactive waste sites. Their analysis was based on the assumption that the interstitial ground water velocities through the barrier were small (0.1 to 1.0 ft/year) and that transport of the solute through the barrier was dominated by molecular diffusion. Based on this assumption and consideration of one-dimensional flow only, the concentration breakthrough times were calculated by use of an analytical solution. Their results showed that the breakthrough times were strongly affected by the width of the barrier and the assumed distribution coefficient.

Work on describing the flow of ground water under impermeable subsurface barriers has been limited mainly to analyzing seepage under dams with cutoffs. DeWiest (1965) has used flow nets to graphically analyze the flow under a sheet-pile on an infinitely deep stratum. DeWiest's analysis has shown that the pressure distribution depends on the ratio of the depth of water to depth of sheet-pile.

Summary

The literature available on subsurface barriers is dominated by design and construction information. Most of this information comes from past experience of using barriers for purposes other than ground water pollution control. Information of the performance of barriers applied to ground water pollution control problems is sparse. Qualitative descriptions of the limitations of subsurface barriers are available, but no quantitative analysis of these limitations exists. It is generally agreed that the connection between the barrier and an underlying impermeable formation is the key to successful functioning of the barrier. One modeling study on the transport of contaminants through a barrier was identified, but this study assumed a highly impermeable barrier. A second modeling study concerning the ability of a barrier to retard pollutant migration was found, but this study assumed an ideal barrier that was both impermeable and fully penetrating.

CHAPTER IV

METHODS AND PROCEDURES

This chapter presents the methods and procedures used in this study. The first step was to review the available literature and the results are described in Chapters II and III. This chapter presents the development of the analytical solution to the ground water flow equation for the distribution of dimensionless streamlines. A numerical model for calculating the distribution of and integrating the area under these streamlines is also outlined. Finally, the characteristics and required manipulations of a packaged numerical solute transport model are discussed.

Analytical Solution

One of the first steps in this research was to find or develop an analytical solution to the ground water flow equation which could be used to calibrate and confirm the accuracy of the subsequent numerical computer model. Because of the specificity of the problem, it was decided to develop an analytical solution.

The problem to be analyzed is that of flow of contaminated ground water under an impermeable barrier (Figure IV-1). The equation governing the steady state flow of water through an isotropic, homogeneous media (in terms of the stream function) is given by:

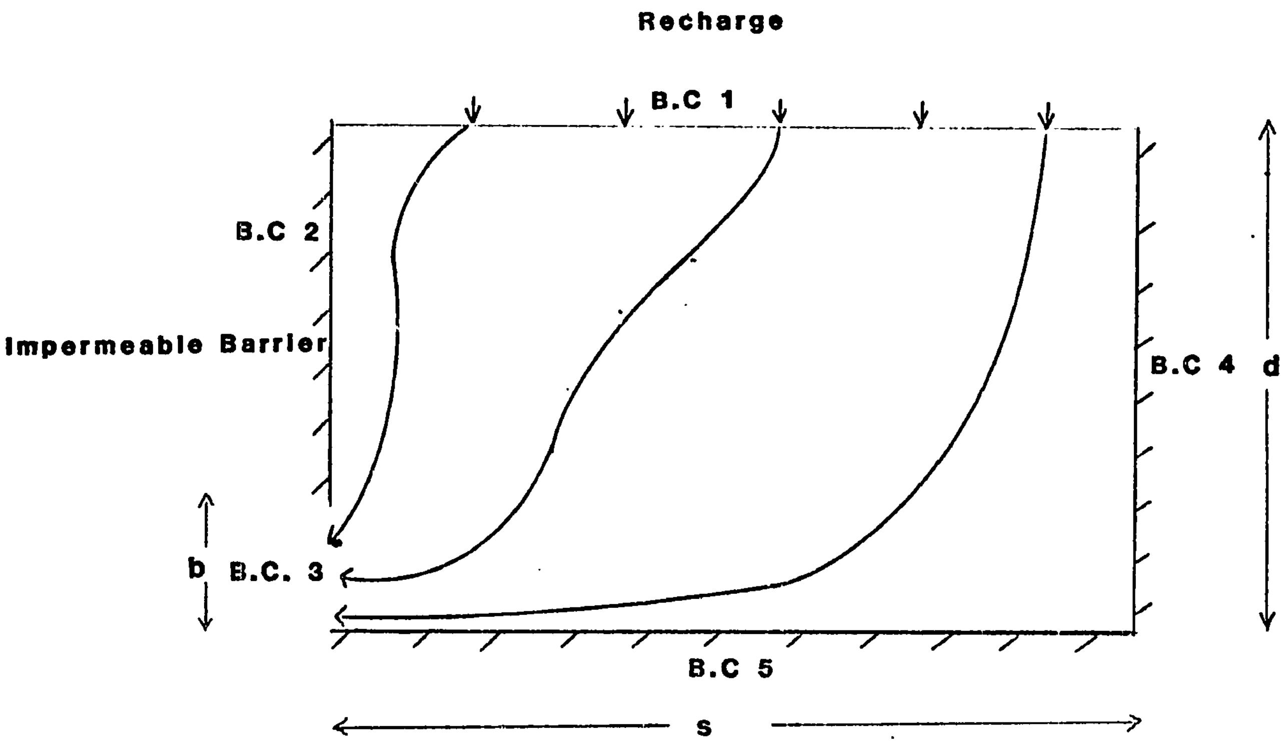


Figure IV-1: Flow of Contaminated Recharge Under an Impermeable Barrier.

$$\frac{\delta^2\Psi}{\delta x^2} + \frac{\delta^2\Psi}{\delta y^2} = 0$$

x, y = horizontal and vertical directions respectively

where Ψ is the stream function that describes the flow paths for steady flow through the aquifer. This is the well known Laplace equation.

One method for solving the equation is called the separation of variables technique. A general solution by this technique has been developed by Kirkham and Powers (1972). The analytical solution particular to the proposed problem will be generated by substituting the appropriate boundary conditions into the general solution below

$$\Psi(x,y) = A + Bx + Cy + Dxy + \sum_n E_n \left\{ \begin{matrix} \sinh \\ \text{or} \\ \cosh \end{matrix} \quad \alpha\, n(b \pm \begin{matrix} x \\ \text{or} \\ y \end{matrix}) \right\} \left\{ \begin{matrix} \sin \\ \text{or} \\ \cos \end{matrix} \quad \alpha\, n(c \pm \begin{matrix} x \\ \text{or} \\ y \end{matrix}) \right\}$$

where

A, B, C, D, E_n, α, b, c = constants, and

n = 1, 2, ...

By substituting the boundary conditions of the specified problem into the above equation, one can generate a specific solution for the distribution of the streamlines in the aquifer. The boundary conditions for the proposed problem are listed below (Figure IV-1).

B.C.1	$\Psi = \Psi_o (1 - y/b)$	$x = 0$	$0<y<b$
B.C.2	$\Psi = 0$	$x = 0$	$b<y<d$
B.C.3	$\Psi = \{\frac{x}{s}\} \Psi_o$	$0<x<s$	$y = d$
B.C.4	$\Psi = \Psi_o$	$x = s$	$0<y<d$
B.C.5	$\Psi = \Psi_o$	$0<x<s$	$y = 0$

where

b = height of opening in barrier,

d = total depth of aquifer, and

s = total width of aquifer.

The particular combination of sinh (cosh) and sin (cos) for this problem must meet two requirements. First, the summation term must be periodic on the boundary of question ($x=0$, $0<y<d$). Second, the summation term must reduce to zero at the other boundaries. The single combination that satisfies these requirements is

$$\Psi = A + Bx + Cy + Dxy + \sum_n I_n \{\sinh(n\pi(s-x)/d)\sin(n\pi y/d)\} \quad \text{(IV-1)}$$

The first four coefficients of the above equation can be determined by substituting boundary conditions 3, 4, and 5, for which the summation term is zero.

B.C.4 $\quad \Psi = \Psi_o$ or $\frac{d\Psi}{dy} = 0 \quad x = s \quad 0<y<d$

$$\frac{d\Psi}{dy} = C + Dx = 0$$

$$C + Ds = 0$$

$$D = -C/s$$

B.C.5 $\quad \Psi = \Psi_o$ or $\frac{d\Psi}{dx} = 0 \quad 0<x<s \quad y = 0$

$$\frac{d\Psi}{dx} = B + Dy = 0$$

$$B + 0 = 0$$

$$B = 0$$

B.C.3 $\quad \Psi = \{x/s\}\ \Psi_o \quad 0<x<s \quad y = d$

$$\Psi = \{x/s\}\ \Psi_o = A + Cy + Dxy$$

$$\frac{\Psi_o x}{s} = A + Cd + (-C/s)\ xd$$

$$\frac{\Psi_o x}{s} = A + Cd\ \{1 - x/s\}$$

$$A = \frac{\Psi_o x}{s} - Cd\ \{1 - x/s\}$$

$$A = \frac{\Psi_o x - Cd\{s-x\}}{s}$$

Substituting for A, B, and D in equation IV-1 yields:

$$\Psi = \frac{\Psi_o x - Cd\{s-x\}}{s} + Cy + (-C/s)xy + \Sigma(\text{terms})$$

$$\Psi = \frac{\Psi_o x - Cd\{s-x\} - Cxy}{s} + Cy$$

In order to determine C, the above expression is resubstituted into the boundary conditions:

B.C.4 $\quad \Psi = \Psi_o \quad x = s \quad 0<y<d$

$$\Psi_o = \frac{\Psi_o s - Cd\{s-s\} - Csy}{s} + Cy$$

$$\Psi_o = \frac{\Psi_o s}{s} - Cy + Cy$$

$$\Psi_o = \Psi_o \text{ O.K.}$$

B.C.3 $\quad \Psi = \{x/s\}\ \Psi_o \quad 0<x<s \quad y = d$

$$\{x/s\}\ \Psi_o = \frac{\Psi_o x - Cd\{s-x\} - Cxd}{s} + Cd$$

$$\{x/s\}\ \Psi_o = \frac{\Psi_o x}{s} - \frac{Cds}{s} + \frac{Cxd}{s} - \frac{Cxd}{s} + Cd$$

$$\{x/s\}\ \Psi_o = \frac{\Psi_o x}{s} \text{ O.K.}$$

B.C.5 $\quad \Psi = \Psi_o \quad 0<x<s \quad y = 0$

$$\Psi_o = \frac{\Psi_o x - Cd\{s-x\} - Cx(0)}{s} + C(0)$$

$$\Psi_o\, s = \Psi_o\, x + Cdx - Cds$$

$$\Psi_o\, s - \Psi_o\, x = -Cd\,\{s - x\}$$

$$\Psi_o\,\{s - x\} = -Cd\,\{s - x\}$$

$$C = \frac{-\Psi_o}{d}$$

So $A = \frac{\Psi_o x}{s} - Cd\ \{1 - x/s\}$

$$A = \frac{\Psi_o x}{s} + \Psi_o\,\{1 - x/s\}$$

$$A = \frac{\Psi_o x}{s} + \Psi_o - \frac{\Psi_o x}{s}$$

$$A = \Psi_o$$

and $D = -C/s = \frac{\Psi_o}{ds}$.

The equation now takes the form

$$\Psi(x,y) = \Psi_o + \left(\frac{-\Psi_o}{d}\right)y + \left(\frac{\Psi_o}{ds}\right)xy + \sum_n I_n \sinh\,[n\pi(s-x)/d]\,\sin\,(n\pi y/d)$$

One simplifying step is to divide the summation terms by sinh $(n\pi s/d)$. This step insures that the summation term will range from 0 to 1 as is desirable in a dimensionless solution.

The boundary conditions of interest can now be substituted to determine I_n:

B.C.2 $\quad \Psi = 0 \quad x = 0 \quad b<y<d$

$$0 = \Psi_o + \left(\frac{-\Psi_o y}{d}\right) + \Sigma\ I_n \sin\,(n\pi y/d)$$

$$\Psi_o\,[y/d-1] = \Sigma I_n \sin\,(n\pi y/d)$$

B.C.1 $\quad \Psi = \Psi_o\,[1 - y/b] \quad x = 0 \quad 0<y<b$

$$\Psi_o\,[1 - y/b] = \Psi_o + [-\Psi_o y/d] + \Sigma\ I_n \sin\,(n\pi y/d)$$

$$\Psi_o - \Psi_o\, y/b - \Psi_o + \Psi_o y/d = \Sigma\ I_n \sin\,(n\pi y/d)$$

$$\Psi_o\ y\ [1/d - 1/b] = \Sigma\ I_n \sin (n\pi y/d)$$

$$\Psi_o\ y\ [\frac{b-d}{bd}] = \Sigma\ I_n \sin (n\pi y/d)$$

I_n is now the Fourier coefficient defined by

$$I_n = \frac{2}{d}\int f(y) \sin (n\pi y/d)\, dy$$

where

$$f(y) = \begin{cases} \Psi_o y\ [\frac{b-d}{bd}] & 0<y<b \\ \Psi_o\ [y/d-1] & b<y<d \end{cases}$$

So I_n now becomes

$$I_n = \frac{2}{d}\ [\int_d^b \Psi_o y\ [\frac{b-d}{bd}] \sin(n\pi y/d)dy + \int_b^d \Psi_o\ [y/d-1] \sin(n\pi y/d)dy]$$

Breaking the integration into two steps yields

1. $$\frac{2\Psi_o}{d}\ [\frac{b-d}{bd}]\ \int_o^b y\ \sin\ (\frac{n\pi y}{d})dy =$$

$$\frac{2\Psi_o}{bd^2}\ (b-d)\ (\frac{-y\ \cos(n\pi y/d)}{(n\pi/d)} + \frac{\sin(n\pi y/d)}{(n\pi/d)^2}\Big|_o^b\ ) =$$

$$\frac{2\Psi_o}{bd^2}\ (b-d)\ (\frac{-b\ \cos(n\pi b/d)}{(n\pi/d)} + \frac{\sin(n\pi b/d)}{(n\pi/d)^2}) =$$

$$\frac{2\Psi_o}{d^2}\ (\frac{-\cos(n\pi b/d)}{(n\pi/d)}) + \frac{2\Psi_o(b-d)}{bd^2}\ (\frac{\sin(n\pi b/d)}{(n\pi/d)^2})$$

2. $$\frac{2\Psi_o}{d^2}\ [\int_b^d y\ \sin(n\pi y/d)dy] - \frac{2\Psi_o}{d}\ [\int_b^d \sin(n\pi y/d)dy]$$

$$\frac{2\Psi_o}{d^2}\ [\frac{-y\ \cos(n\pi y/d)}{(n\pi/d)} + \frac{\sin(n\pi y/d)}{(n\pi/d)^2}\Big|_b^d\ ] + \frac{2\Psi_o}{d}\ [\frac{\cos(n\pi y/d)}{n\pi/d}\Big|_b^d\ ]$$

$$\frac{2\Psi_o}{d^2}\ [\frac{-d\ \cos(n\pi)}{(n\pi/d)} + \frac{b\ \cos(n\pi b/d)}{(n\pi/d)} - \frac{\sin(n\pi b/d)}{(n\pi/d)^2}] + \frac{2\Psi_o}{d}\ [\frac{\cos(n\pi)}{(n\pi/d)} - \frac{\cos(n\pi b/d)}{(n\pi/d)}]$$

$$\frac{-2\Psi_o\cos(n\pi)}{n\pi} + \frac{2\Psi_o b}{d}\,\frac{\cos(n\pi b/d)}{n\pi} - \frac{2\Psi_o}{(n\pi)^2}\sin(\frac{n\pi b}{d}) + \frac{2\Psi_o\cos(n\pi)}{n\pi}$$

$$- \frac{2\Psi_o\cos(n\pi b/d)}{n\pi} =$$

$$\cos(n\pi b/d)\ [\frac{2\Psi_o b}{dn\pi} - \frac{2\Psi_o}{n\pi}] + \sin(n\pi b/d)\ [\frac{-2\Psi_o}{n\pi^2}] =$$

$$\cos(n\pi b/d)\ [\frac{2\Psi_o(d-b)}{dn\Psi}] + \sin\ (n\pi b/d)\ [\frac{-2\Psi_o}{n\pi^2}]$$

Adding the results of 1. and 2. results with

$$\cos(n\pi b/d)\ [\ \frac{2\Psi_o(d-b)}{dn\pi} + \frac{2\Psi_o b}{dn\pi} - \frac{2\Psi_o}{n\pi}\ ] + \sin(\frac{n\pi b}{d})\ [\frac{-2\Psi_o}{(n\pi)^2} + \frac{2\Psi_o(b-d)}{b(n\pi)^2}]$$

$$= \sin\ (\frac{n\pi b}{d})\ [\frac{-2\Psi_o b}{d(n\pi)^2} + \frac{2\Psi_o(b-d)}{b(n\pi)^2}]$$

$$I_n = \frac{-2\Psi_o d}{b(n\pi)^2}\sin(n\pi b/d)$$

So the specific solution is

$$\Psi(x,y) = \Psi_o + (-y/d)\Psi_o + \frac{xy}{ds}\Psi_o + \sum_n \frac{-2\Psi_o d}{b(n\pi)^2}\,\frac{\sinh(n\pi(s-x)/d)\sin(n\pi y/d)\sin(n\pi b/d)}{\sinh(n\pi s/d)}$$

To make the equation totally dimensionless, each variable must be divided by some constant with the same dimensions. More specifically, each variable can be divided by the maximum value that variable can attain. The following substitutions are made:

$X = x/s$

$Y = y/d$

$B = b/d$

$N = s/d$

$\overline{\Psi} = \Psi/\Psi_o$

These substitutions yield the following dimensionless equation

$$\overline{\Psi} = 1 - Y + XY + \sum_n \frac{-2}{B(n\pi)^2} \frac{\sinh(n\pi N(1-X)\sin(n\pi Y)\sin(n\pi B)}{\sinh(n\pi N)}$$

The above equation has been programmed on the IBM computer at the University of Oklahoma. The program includes a step to calculate the coordinates of the constant $\overline{\Psi}$ values on 0.05 intervals between 0 and 1. The source deck for the program is found in Appendix I.

Numerical Flow Model

The next step in the research was to develop a numerical model for calculating the dimensionless streamline distribution and incorporating flexibility to handle situations other than that depicted in Figure IV-1. The model was calibrated with the analytical solution developed above.

The governing equation for ground water flow through an anisotropic, homogeneous media is

$$K_x \frac{\delta^2\psi}{\delta x^2} + K_y \frac{\delta^2\psi}{\delta y^2} = 0$$

where

K_x = hydraulic conductivity in the x-direction

K_y = hydraulic conductivity in the y-direction

Dividing both sides of the equation by K_y and remembering that $\overline{\Psi} = \psi/\psi_0$, $X = x/s$ and $Y = y/d$, yields

$$\frac{K_x}{K_y} \frac{d^2}{s^2} \frac{\delta^2\overline{\Psi}}{\delta X^2} + \frac{\delta^2\overline{\Psi}}{\delta Y^2} = 0$$

Now letting $K = K_x/K_y\ (d/s)^2$ be the aspect ratio yields

$$K \frac{\delta^2 \overline{\Psi}}{\delta X^2} + \frac{\delta^2 \overline{\Psi}}{\delta Y^2} = 0$$

The continuous derivatives of the above equation can be replaced by their finite difference approximations which yields

$$K\left(\frac{\overline{\Psi}_{i,j+1} - 2\overline{\Psi}_{i,j} + \overline{\Psi}_{i,j-1}}{\Delta X^2}\right) + \left(\frac{\overline{\Psi}_{i+1,j} - 2\overline{\Psi}_{i,j} + \overline{\Psi}_{i-1,j}}{\Delta Y^2}\right) = 0$$

Solving for $\overline{\Psi}_{i,j}$ yields

$$\overline{\Psi}_{i,j} = \frac{K\Delta Y^2}{2(K\Delta y^2 + \Delta x^2)} (\overline{\Psi}_{i,j+1} + \overline{\Psi}_{i,j-1}) + \frac{\Delta X^2}{2(K\Delta Y^2 + \Delta X^2)} (\overline{\Psi}_{i+1,j} + \overline{\Psi}_{i-1,j})$$

This equation has also been programmed on the IBM computer using an iterative alternating direction solution procedure to converge upon the solution for $\overline{\Psi}_{i,j}$ at the nodes of an artificial grid representing the 2-dimensional aquifer. The source deck for this program is found in Appendix II. The model calculates a value of $\overline{\Psi}^{calc}_{i,j}$ for node (i,j) by using its four surrounding nodes. A new value of $\overline{\Psi}$ for node (i,j) is calculated by a relaxation equation of the form

$$\overline{\Psi}^{new}_{i,j} = \overline{\Psi}^{old}_{i,j} + 1.5\, (\overline{\Psi}^{calc}_{i,j} - \overline{\Psi}^{old}_{i,j})$$

The relaxation factor accelerates the convergence of the method. A convergence criterion of 0 was input to the model to increase accuracy. So, the process is controlled by an iteration limit (ITER = 30). The end result is a dimensionless $\overline{\Psi}$ distribution for all the nodes in the grid.

Solute Travel Time Utilizing Plug Flow Analysis

Development of pollutant breakthrough curves can be accomplished by considering the kinematics of the flow field in Figure IV-1 as was done by McLin (1980). This type of analysis ignores the effects of dispersion and inherently assumes that vertical flow effects can be ignored for the elongated (width/depth ratio $\geq$ 4) aquifer. In essence, the assumption is that the contaminated recharge travels as "plugs" (Figure IV-2). By definition the flow between two streamlines must remain constant. The time for a particle to flow from one point (S_1) to another (S_2) is equal to the volume of water in the flow field divided by the volumetric flow rate. Using a unit dimension in the third direction, the time becomes equal to the area of flow divided by the areal flow rate. From Figure IV-2 this time can be expressed by

$$t = \frac{n dA(\Psi)}{d\Psi}$$

where $Q = d\Psi$

Using the following substitutions to non-dimensionalyze the equation

A_o = SD, the cross section of the aquifer,

$\Psi_o = \varepsilon S$,

ε = recharge rate,

$\overline{A}(\overline{\Psi}) = A(\overline{\Psi})/A_o$, a dimensionless area, and

$\overline{\Psi} = \Psi/\Psi_o$, a dimensionless streamline

yields

$$t = \frac{nSDd\overline{A}(\overline{\Psi})}{\varepsilon Sd\overline{\Psi}} \qquad \text{or} \qquad t/t_c = \frac{d\overline{A}(\overline{\Psi})}{d\overline{\Psi}}$$

where $t_c = n\ D/\varepsilon$ is defined as the solute response time for the aquifer.

Hence, if the numerical model can generate the distribution of the dimensionless $\overline{\Psi}$, we can numerically integrate to determine the

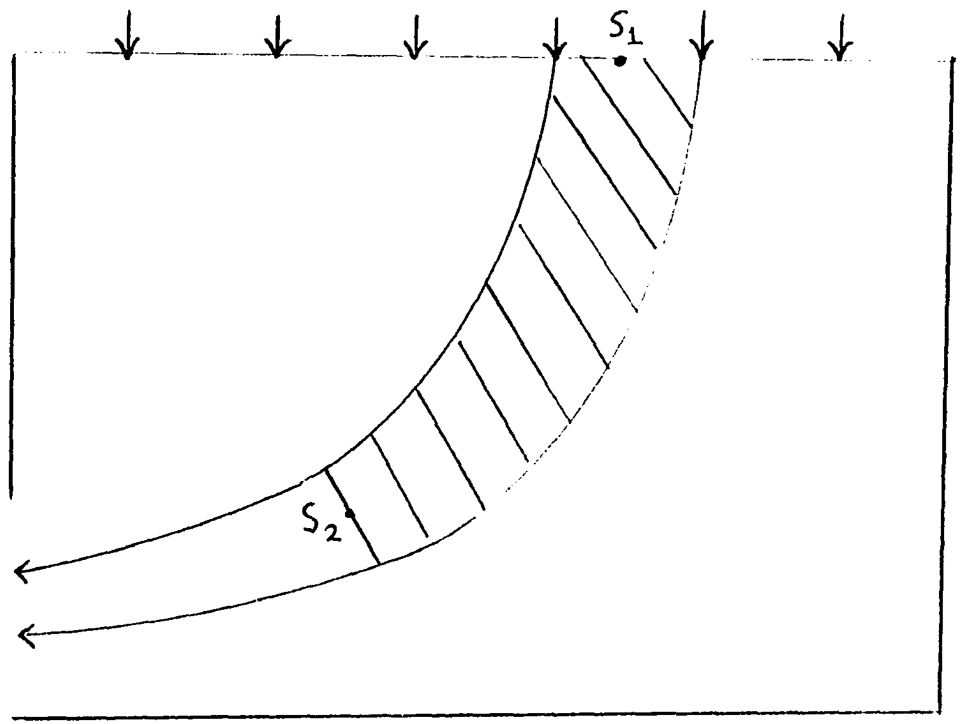


Figure IV-2: Illustration of Plug Flow.

relationship of $\overline{\Psi}$ vs. $A(\overline{\Psi})$. The slope of this curve can then be determined to generate the $d\overline{A(\Psi)}/d\overline{\Psi}$ term needed (see Figure IV-3).

If one considers the flow of the contaminant to be "plug flow" as outlined in Figure IV-2, the outflow concentration can be generated. If each of the stream tubes in Figure IV-2 carries one tenth of the contaminated recharge (ε) at a concentration C_o, then the average concentration of the outflow at any time will depend on the number of stream tubes that have arrived at the point of outflow. More specifically, if two of the stream tubes have arrived, the average concentration C, at the point of outflow, will be equal to $0.2C_o$. The relationship in terms of an equation is

$$C/C_o = \overline{\Psi}$$

Hence, pollutant breakthrough curves (C/C_o vs t/t_c) can be generated from the known distribution of $\overline{\Psi}$ in the aquifer.

The model developed for this study calculates both the lines of constant $\overline{\Psi}$ and the area under each line. The model calculates lines of constant $\overline{\Psi}$ on increments of 0.05 by moving horizontally through the dimensionless aquifer. Moving horizontally is more accurate than vertical movement because there are four times as many nodes in the horizontal direction. The area under each curve is calculated by summing the areas between two adjacent node crossings. The area between two adjacent node crossings is calculated by assuming the shape of a trapezoid (Figure IV-4).

The last function of the model is to calculate the velocities or specific discharge vectors in the x and y directions. The model

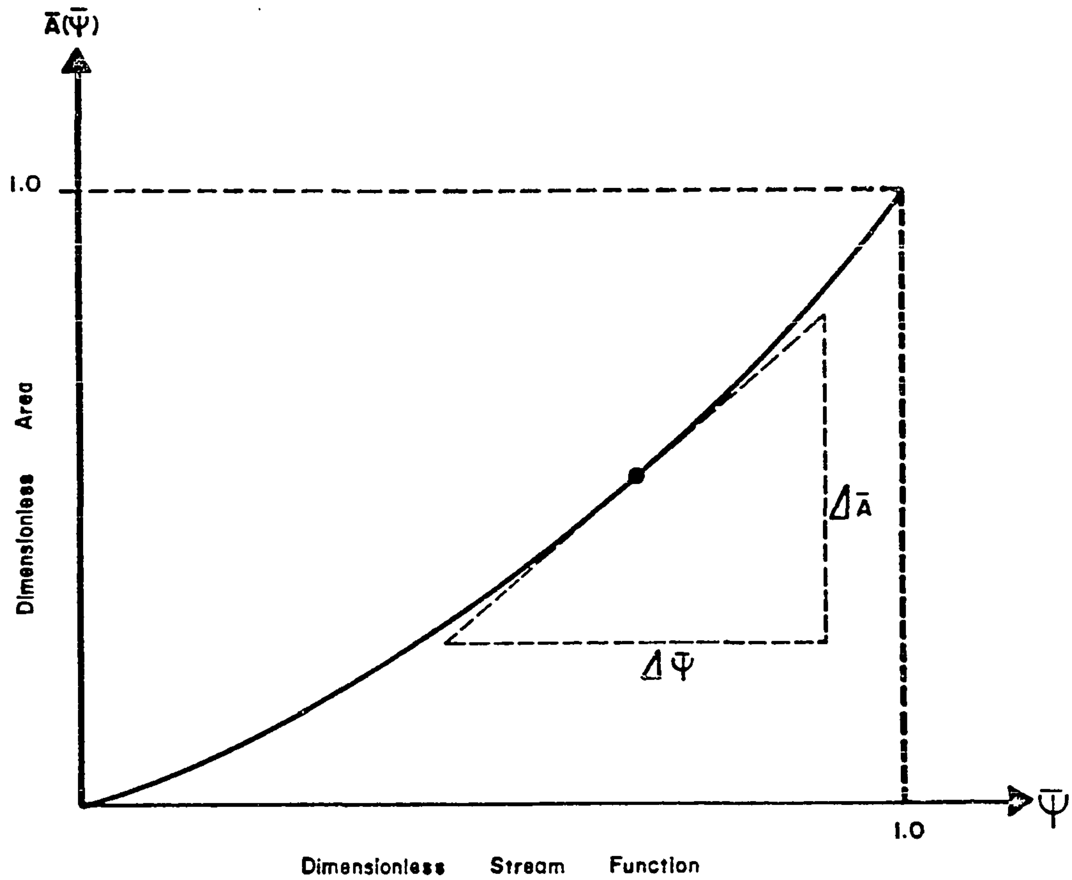


Figure IV-3: Idealized relationship indicating the total dimensionless area swept out by a given dimensionless streamline (McLin, 1980).

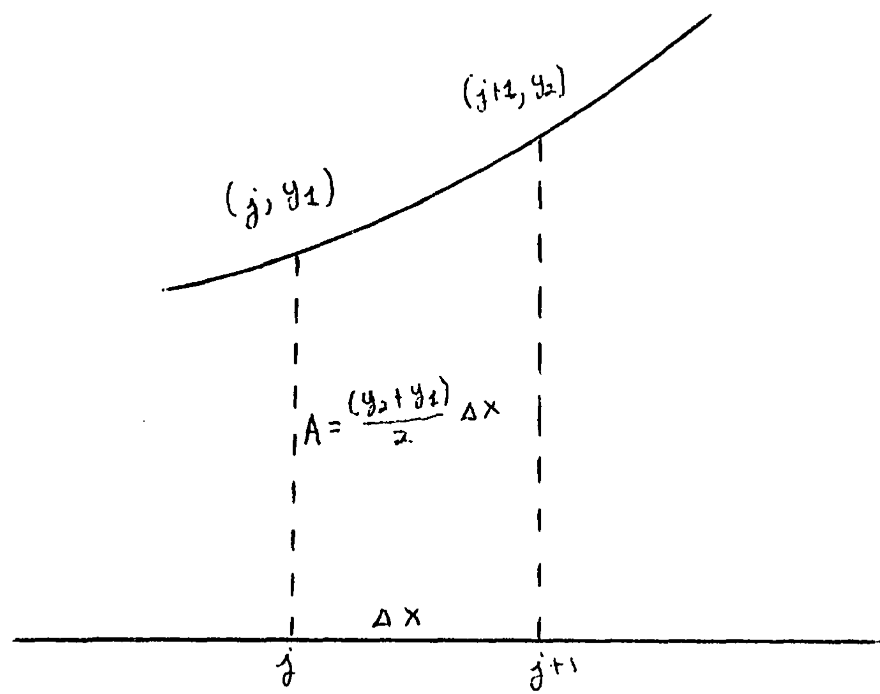


Figure IV-4: Trapezoidal Approximation of Area Under Streamlines.

uses the $\overline{\Psi}$ distribution to calculate these quantities. By definition, the Darcy flow velocities are (Bear, 1979):

$$V_x = K_x \frac{\delta\Psi}{\delta y} \quad \text{and} \quad V_y = K_y \frac{\delta\Psi}{\delta x}$$

However, the distribution developed is of dimensionless $\overline{\Psi}$ for a dimensionless aquifer. It must now be converted back to a dimensional aquifer. This can be done by utilizing the following relationships

$$\frac{\delta\Psi}{\delta y} = \frac{\delta\overline{\Psi}}{\delta Y}\left(\frac{\Psi_o}{D}\right), \quad \frac{\delta\Psi}{\delta x} = \frac{\delta\overline{\Psi}}{\delta X}\left(\frac{\Psi_o}{S}\right), \quad \Psi_o = \varepsilon S$$

so

$$V_x = K_x \frac{\delta\Psi}{\delta y} = \frac{\delta\overline{\Psi}}{\delta Y}\left(\frac{\Psi_o}{D}\right) K_x = \frac{\delta\overline{\Psi}}{\delta Y}(\varepsilon K_x)\ (S/D)$$

For the model S/D = 4, and assuming $\varepsilon = 1.0$ yields

$$V_x = 4K_x \quad \frac{\delta\overline{\Psi}}{\delta Y}$$

Similarly

$$V_y = K_y \frac{\delta\Psi}{\delta x} = \frac{\delta\overline{\Psi}}{\delta X}\left(\frac{\Psi_o}{S}\right) K_y = \frac{\delta\overline{\Psi}}{\delta X}\left(\frac{\varepsilon S}{S}\right) K_y = K_y \frac{\delta\overline{\Psi}}{\delta X}$$

Therefore, both x and y velocities can be calculated at any node by examining the rate of change of $\overline{\Psi}$ in orthogonal directions.

Analysis Through Use of Solute Transport Model

The final phase of the analysis was to utilize a numerical solute transport model to examine further variations of the aquifer. The model chosen was a two-dimensional model of solute transport and dispersion developed by Konikow and Bredehoeft (1978) for the U.S. Geological Survey.

There are four reasons for utilizing the Konikow-Bredehoeft (K-B) model. First, the K-B model will be used to verify the conceptual results produced by the previous simplified model. Second, the K-B model will allow for the examination of the effects of hydrodynamic dispersion on the pollutant breakthrough curves. Third, the K-B model will enable the analysis of two cases of interest not amenable to analysis by the previously discussed simplified model. Fourth, the K-B model will allow for the generation of some actual numbers for the time factors (t/t_c) in the dimensionless breakthrough curves.

Like all solute transport models, the K-B model must solve both the ground water flow equation and the mass transport equation. The structure of the K-B model is such that the flow equation is solved by employing a finite-difference approximation to the partial differential equation and an alternating direction implicit procedure for solving the resulting simultaneous equations. The mass transport equation is solved in two parts: (1) first, the effects of convective transport are evaluated using the method of characteristics; and (2) the effects of hydrodynamic dispersion are evaluated using a finite-difference scheme.

The K-B model was designed for application to two-dimensional areal flow problems but is easily applied directly to two-dimensional cross sections. The model can examine transient flow problems or steady-state flow problems. The structure of the model is such that the outermost nodes of the grid approximating the aquifer are designated as no-flow boundaries. However, nodes within the boundaries can be designated as "constant head" or "no-flow". Sources of constant recharge and constant solute concentration can also be designated. The model also allows the two directional hydraulic conductivities (K_x and K_y) to be specified for each node. The model has a number of other characteristics but those listed above are the ones of interest for this study.

The K-B model seemed ideal for application to the problem proposed; however, it did require a number of manipulations. As delivered, the K-B model is capable of handling a 32 x 32 grid. The major manipulation of the model was to set the arrays to fit the elongated (width/depth = 4) aquifer. To examine a cross-sectional rather than an areal problem, hydraulic conductivities had to be substituted for transmissivities and a unit width in the third direction (normal to the cross section) had to be specified. The constant head conditions are handled by specifying a very low leakage at a node. Steady-state conditions are generated by specifying the storage coefficient as zero.

Because the K-B model works with hydraulic head rather than stream functions, the boundary conditions depicted in Figure IV-1 had to be converted from functions of streamlines to functions of head. The no-flow boundary conditions (B.C.2, B.C.4, B.C.5) are easily incorporated into the model as discussed above. Boundary condition 1 can be converted to a head condition by use of the Cauchy-Riemann conditions as follows:

B.C.1 $\Psi = \Psi_o\ (1 - y/b),\ \frac{\delta\Psi}{\delta y} = \frac{-\Psi_o}{b} = \text{constant}$

By Cauchy-Riemann $\frac{\delta\Psi}{\delta y} = \frac{-\delta h}{\delta x}$

So $\frac{\delta h}{\delta x} = \frac{\Psi_o}{b}$

or $\delta h = \frac{\Psi_o}{b}\ \delta x$

Integrating $h = \frac{\Psi_o}{b}\ x + C$

But examining Figure IV-1 shows that $x = 0$ and h can only equal a constant; therefore:

$h = C$ -- a constant

Boundary condition 3 can be converted in a similar manner.

B.C.3 $\quad \psi = \psi_o\,(x/s) \quad \frac{\delta\Psi}{\delta x} = \frac{\Psi_o}{S}$

By Cauchy-Riemann $\frac{\delta\Psi}{\delta x} = \frac{\delta h}{\delta y}$

So $\frac{\delta h}{\delta y} = \frac{\Psi_o}{s}$

or $\delta h = \Psi_o/s\ (\delta y)$

Integrating $h = \frac{\Psi_o}{s}\, y + C$

But $\psi_o = \varepsilon s$ and $y = d$ on the boundary, so

$h = \varepsilon d + C$

Taking $\varepsilon = 1$ and noting that $C = 0$ yields

$h = d$ -- a constant

So there are two constant head boundary conditions and three no-flow boundary conditions as input into the K-B model. The only other manipulations required are changes in the hydraulic conductivity matrix to suit the problem of interest.

CHAPTER V

PRESENTATION AND ANALYSIS OF RESULTS

This chapter presents the results of the analysis of contaminated recharge flowing under a partially penetrating impermeable barrier. The first part of the analysis utilizes the analytical solution developed in Chapter IV to verify the simple numerical model. The second part of the analysis involves utilizing the simple numerical model to examine the effects of variable depths of penetration, rates and lengths of recharge, and anisotropic soils. The third part of the analysis involves the use of the selected numerical solute transport model to examine the effects of hydrodynamic dispersion, layered soils, and fully-penetrating but partially permeable barriers.

Phase I -- Verification of Simple Numerical Model

The first phase in the analysis of the problem was to verify the accuracy of the simple numerical model. To verify the numerical model, the results were compared to those generated by the analytical solution developed previously. Depicted in Figure V-1 is the dimensionless $\overline{\Psi}$ distribution for the dimensionless aquifer subject to full recharge and 50% barrier penetration as predicted by the simple numerical model. In Figure V-2 is the $\overline{\Psi}$ distribution for the same situation as predicted by the analytical solution. These results are

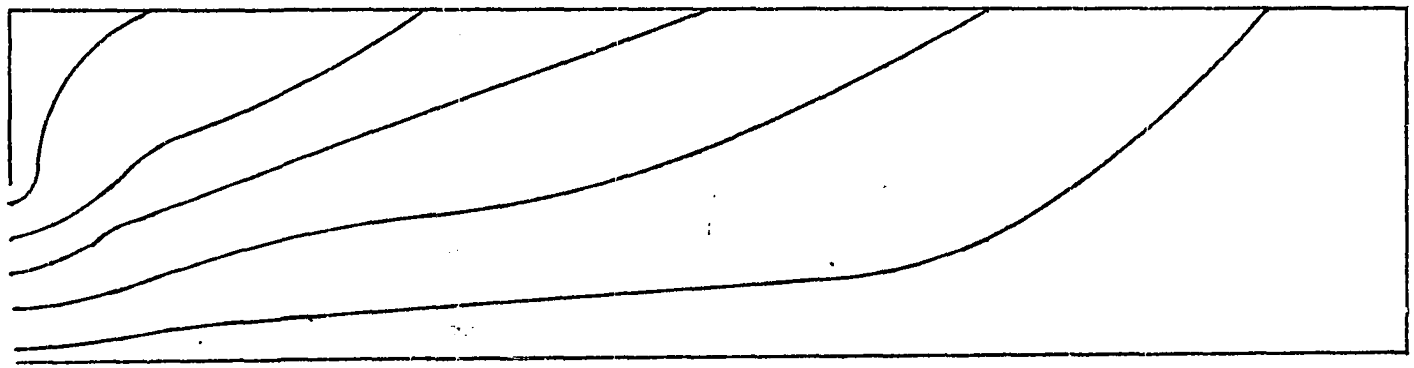


Figure V-1: Dimensionless Streamline Distribution Generated by Numerical Model.

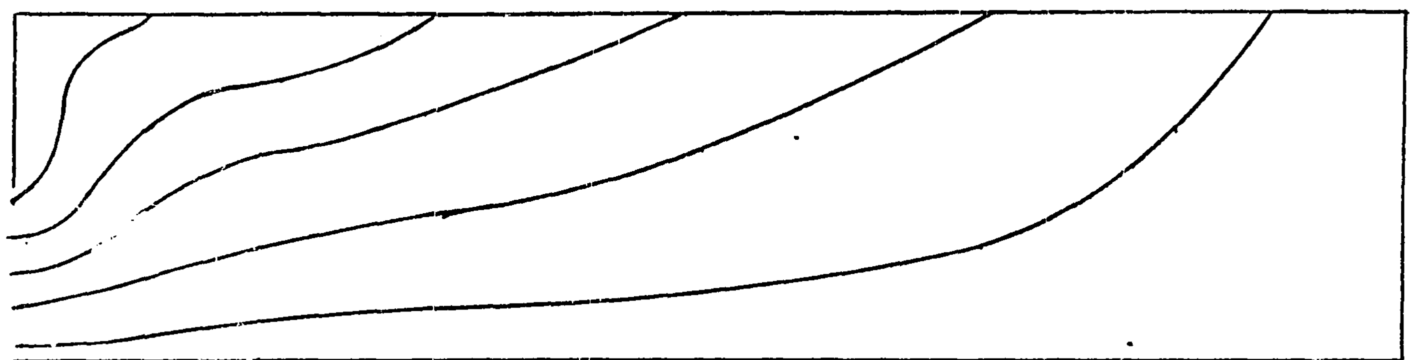


Figure V-2: Dimensionless Streamline Distribution Generated by Analytical Model.

presented in tabular form in Table V-1. Comparison of the two figures and the values in Table V-1 shows an almost identical $\overline{\Psi}$ distribution. Minor variations between the two models can be attributed to one of three possible sources. First, the summation term in the analytical solution was truncated to include only the first ten terms rather than the totally comprehensive infinite sum. Second, the convergence criteria of zero on the numerical model is superseded by an iteration limit of thirty. Both of these procedures have a very small but definite contribution to the accuracy of the output. The third possible source of error is the inherent truncation error associated with the use of any digital computer. However, the magnitude of the error in Figures V-1 and V-2 is so small as to qualify as negligible. Therefore, it can be concluded that the simple numerical model does accurately predict the dimensionless $\overline{\Psi}$ distribution in the dimensionless aquifer. Hence, the simple numerical model can then be used for analysis of variations on the aquifer.

Phase II -- Plug Flow Analysis by Simple Numerical Model

The second phase of the analysis was to examine the behavior of the aquifer under a variety of recharge lengths, barrier depths, and soil conductivity variations. This phase of the analysis used the simplifying assumptions and numerical integration procedure outlined in Chapter IV. The results of each variation are discussed individually below and are concluded with a summary.

Each analysis was performed for the cases of no barrier, 50% penetrating barrier, and 90% penetrating barrier. The 90% penetrating

Table V-1: Dimensionless Streamline Distribution Produced By the Analytical Solution and Simple Numerical Model.

	PSI=0.1			PSI=0.5			PSI=0.9	
X	Analytical Y	Numerical Y	X	Analytical Y	Numerical Y	X	Analytical Y	Numerical Y
0.0	0.455	0.450	0.0	0.250	0.250	0.0	0.050	0.050
0.025	0.665	0.671	0.050	0.332	0.330	0.100	0.080	0.081
0.050	0.853	0.848	0.100	0.431	0.426	0.200	0.111	0.110
0.075	0.943	0.942	0.150	0.516	0.513	0.300	0.137	0.139
0.100	1.000	1.000	0.200	0.586	0.580	0.400	0.165	0.170
			0.250	0.646	0.642	0.500	0.199	0.197
			0.300	0.704	0.701	0.600	0.250	0.250
			0.350	0.764	0.760	0.700	0.333	0.331
			0.400	0.831	0.830	0.800	0.499	0.500
			0.450	0.908	0.911	0.900	1.000	1.000
			0.500	1.000	1.000			

barrier simulates the case of a barrier that has been inadequately keyed into an underlying formation. Although arbitrarily selected for purposes of comparison, the 50% penetrating barrier could be representative of a case where a barrier has been used for ground water flow control or minor plume management. The no barrier case is used as a means of assessing the effects of the with-barrier cases.

The majority of the figures in this phase of the analysis will show a relationship of relative concentration (C/C_o) versus dimensionless time (t/t_c). The development of the relationship of C/C_o versus t/t_c was outlined in Chapter IV. The C/C_o term, in each case, represents the average concentration under the barrier (C) divided by the input concentration (C_o) at a particular dimensionless time (t/t_c). The t/t_c term represents the time required for that particular concentration (C) to develop under the barrier, divided by a constant time parameter (t_c) specific to the problem.

Problem 1: Full Recharge, Varying Depths of Penetration, Isotropic Soil

The first and probably the simplest problem to be examined is that of full length recharge and varying depths of penetration of the barrier in a homogeneous, isotropic aquifer. Conceptually, this situation is equivalent to placing a barrier right next to a source of contaminated recharge.

Plotted in Figure V-3 are the pollutant breakthrough curves for no barrier and two different depths of penetration of the barrier. All three of the curves show a fictitious immediate response in the form of a relative concentration (C/C_o) increase. The curves also show that as

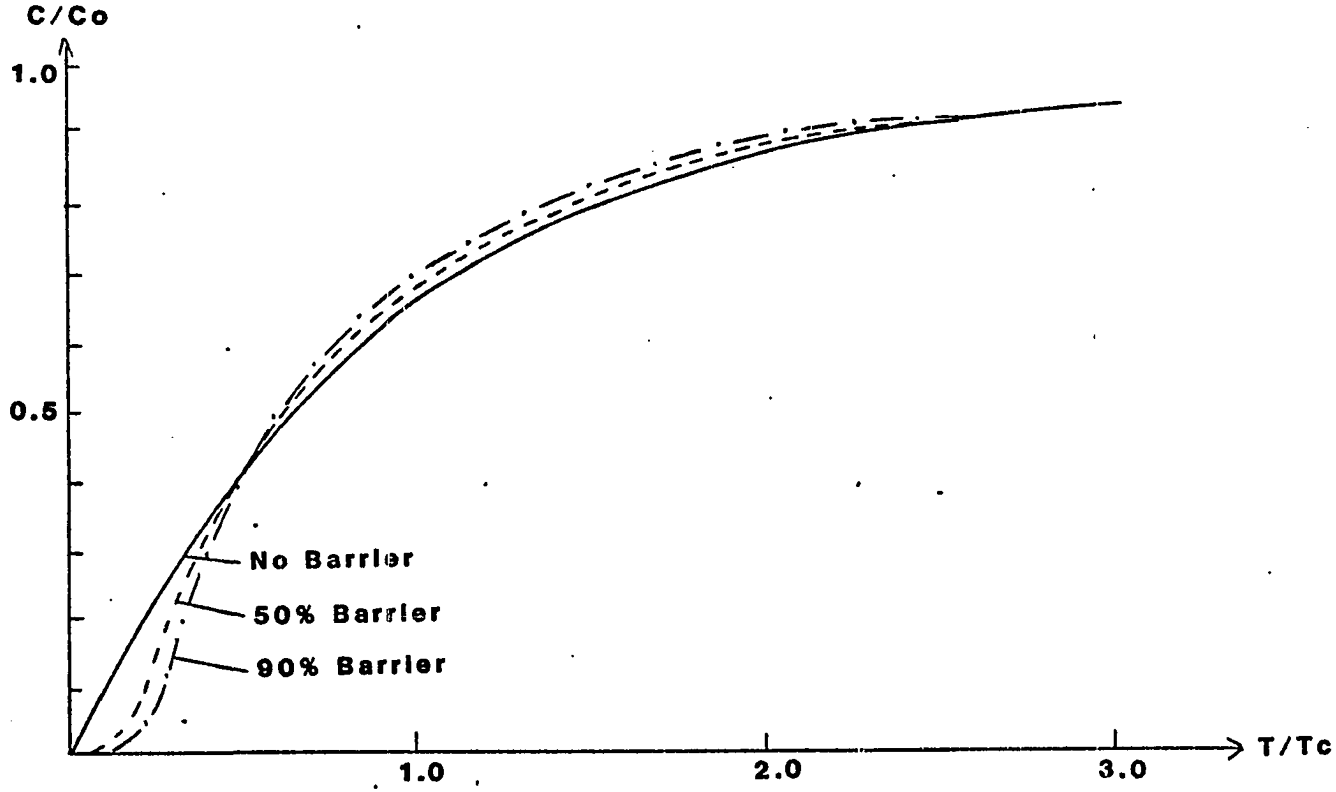


Figure V-3: Pollutant Breakthrough Curves for Aquifer Subjected to Full Recharge.

the depth of the barrier increases, the initial response of the system decreases. In other words, the two "with barrier" cases show a time lag before a dramatic response over that of the "no barrier" case. The curves also show that the "with barrier" cases actually catch and then surpass the "no barrier" case in terms of relative concentration.

The time lag for the "with barrier" cases can be explained by examining the $\overline{\Psi}$ distributions under the barrier (Figure V-4). As might be expected, placement of a barrier significantly increases the flow distance for the near streamlines. The far streamlines are affected less by placement of the barrier. The increase in flow path distances explains the time lag seen in Figure V-3.

The reason for the deeper barrier actually overtaking the more shallow barrier and the no barrier cases in terms of relative concentration over time can be explained by the spacing of the streamlines around the opening in the barrier in Figure V-4. The Cauchy-Riemann conditions state that velocity (specific discharge) vectors are proportional to the rate of change of $\overline{\Psi}$ in the orthogonal direction.

$$V_x = K_x \frac{\delta\Psi}{\delta y} \quad \text{and } V_y = K_y \frac{\delta\Psi}{\delta x}$$

Since the streamlines are closely spaced near the opening, the velocities in both directions are increasing. Notice for the deeper barrier the velocity gradient around the opening is steeper and its effects are felt further into the aquifer. So, even though the travel distances for the deep barrier are increased, the contaminant is actually subjected to a more intense velocity gradient and as such, will overtake the cases of shallow or no barrier with their mild

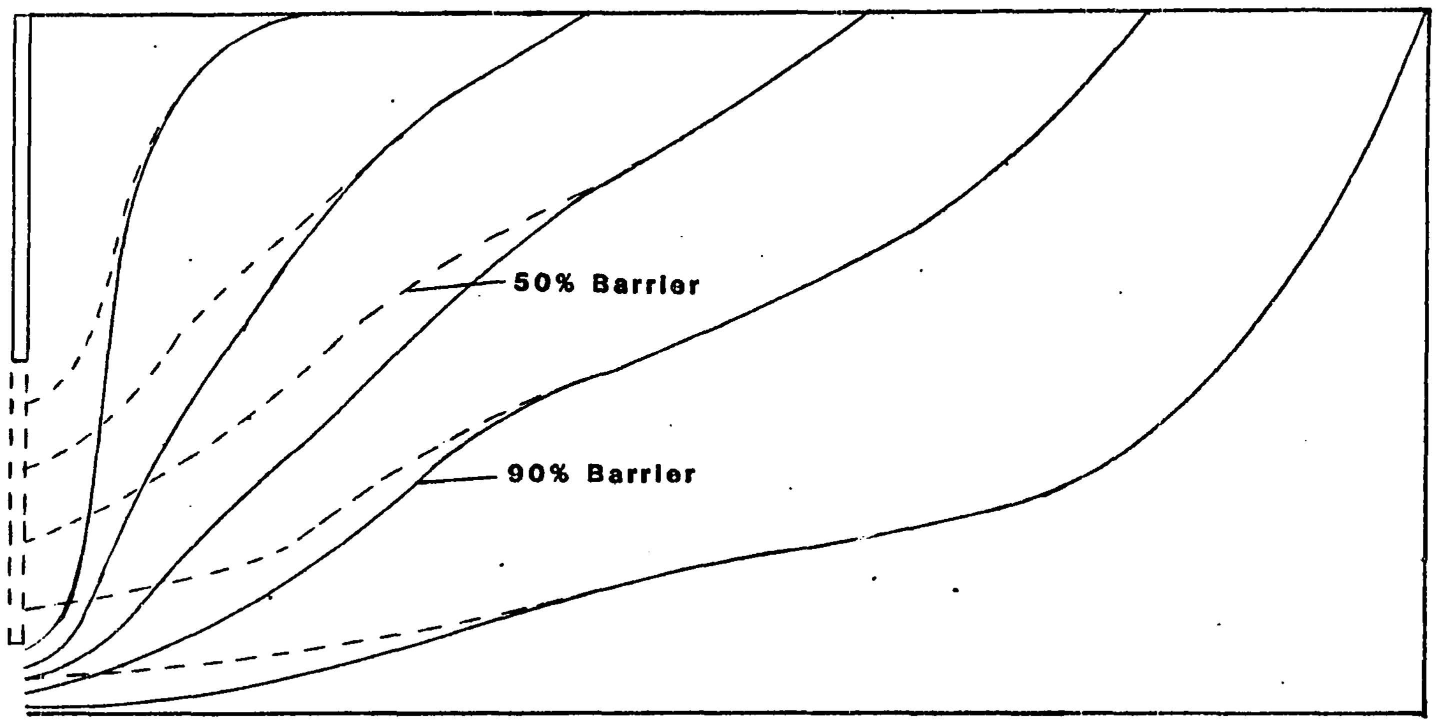


Figure V-4: Streamline Distributions for 50% and 90% Penetration Barrier.

velocity gradients. From Figure V-3 it is seen that the deep barrier overtakes the other cases at a relative concentration of about 0.4. This phenomena may or may not be significant depending on what relative concentration is considered critical. This critical concentration will be pollutant specific.

Problem 2: Variable Length of Recharge

One of the variations imposed on the aquifer was to change the length of the recharge area. A comparison was made for full recharge and recharge over the outer 3/4, 1/2 and 1/4 of the aquifer. Conceptually, this is equivalent to placing the barrier downgradient from a source of contaminated recharge.

Plotted in Figures V-5, V-6, and V-7 are the pollutant breakthrough curves for three different depths of penetration for recharge over the outer 3/4, 1/2 and 1/4 of the aquifer respectively. These figures show a pattern similar to that of Figure V-3; that is, the deeper barriers provide an initial time lag and then seem to overtake the shallow barriers in terms of relative concentration increases.

Note that although moving the barrier further away from the source of recharge provides a greater initial time lag, this increased lag can only be attributed to the increased travel distance and not to the performance of the barrier. In fact, the effectiveness of the barriers is diminished by placing them downgradient. Plotted in Figure V-8 are the pollutant breakthrough curves for the deep barrier for the four different lengths of recharge. Notice that although the barrier

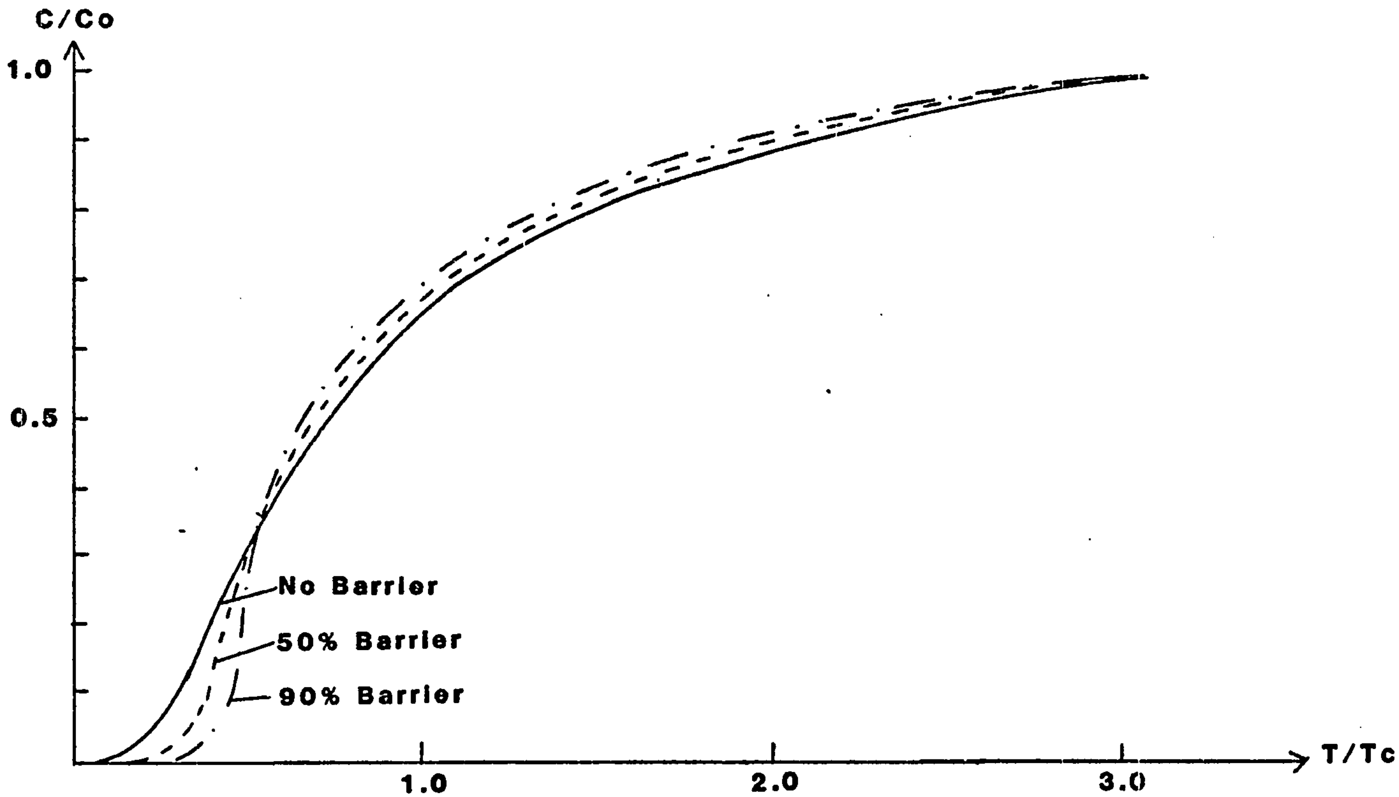


Figure V-5: Pollutant Breakthrough Curves for Aquifer Subjected to 3/4 Recharge.

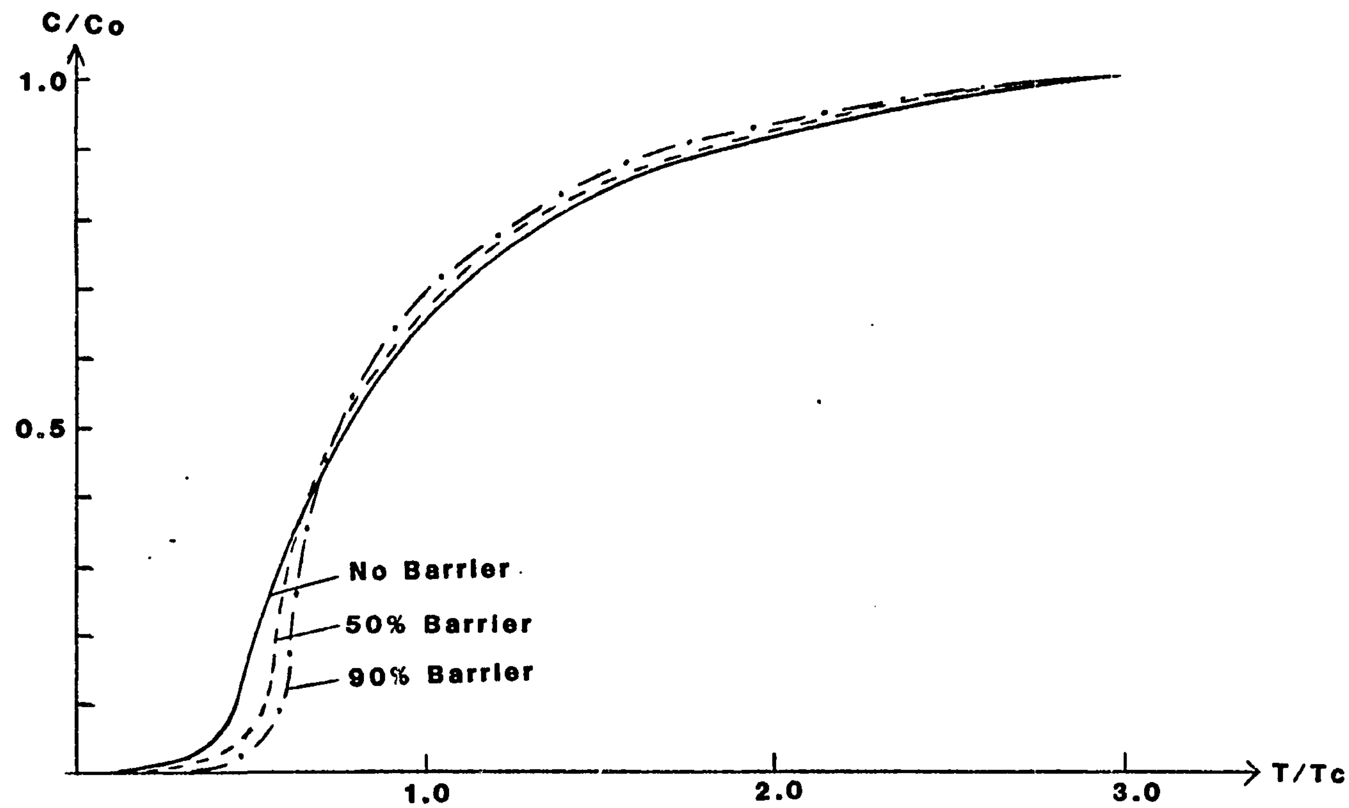


Figure V-6: Pollutant Breakthrough Curves for Aquifer Subjected to 1/2 Recharge.

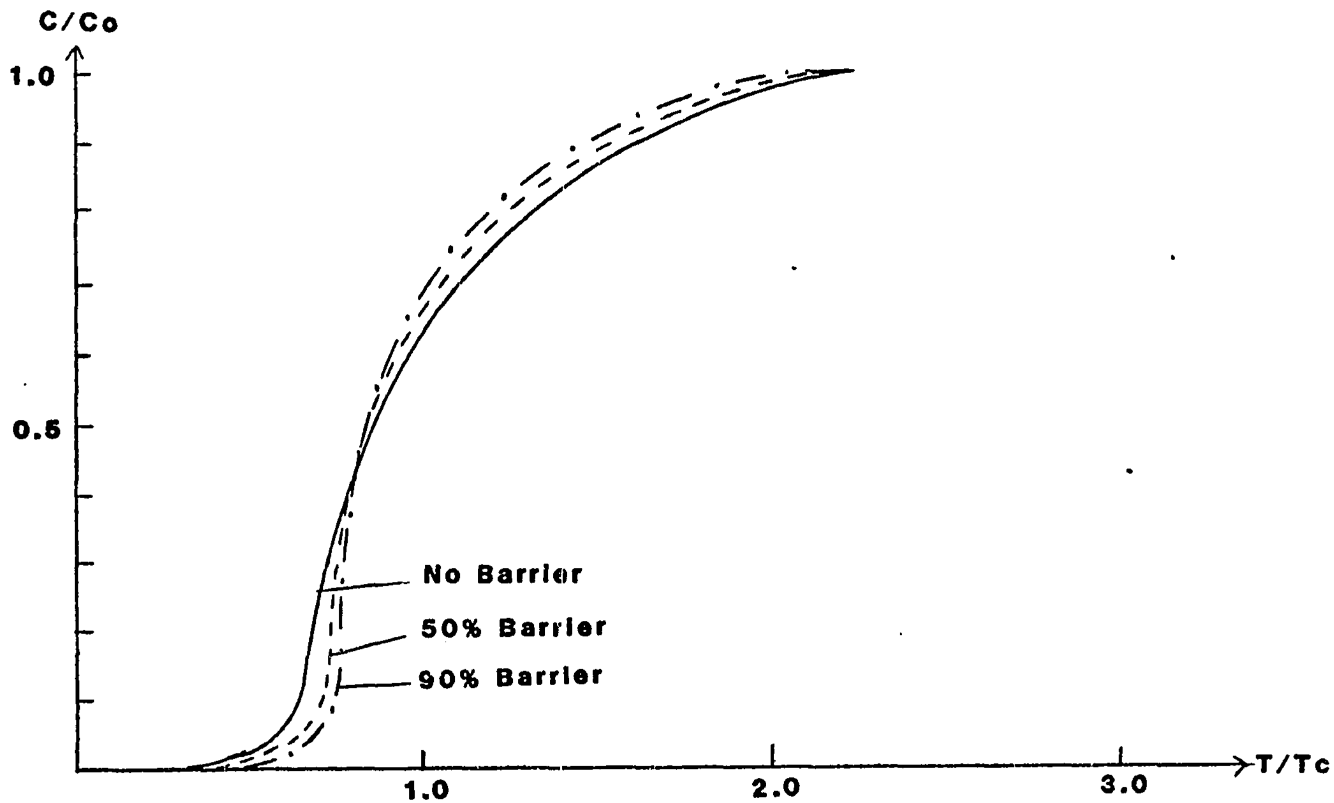


Figure V-7: Pollutant Breakthrough Curves for Aquifer Subjected to 1/4 Recharge.

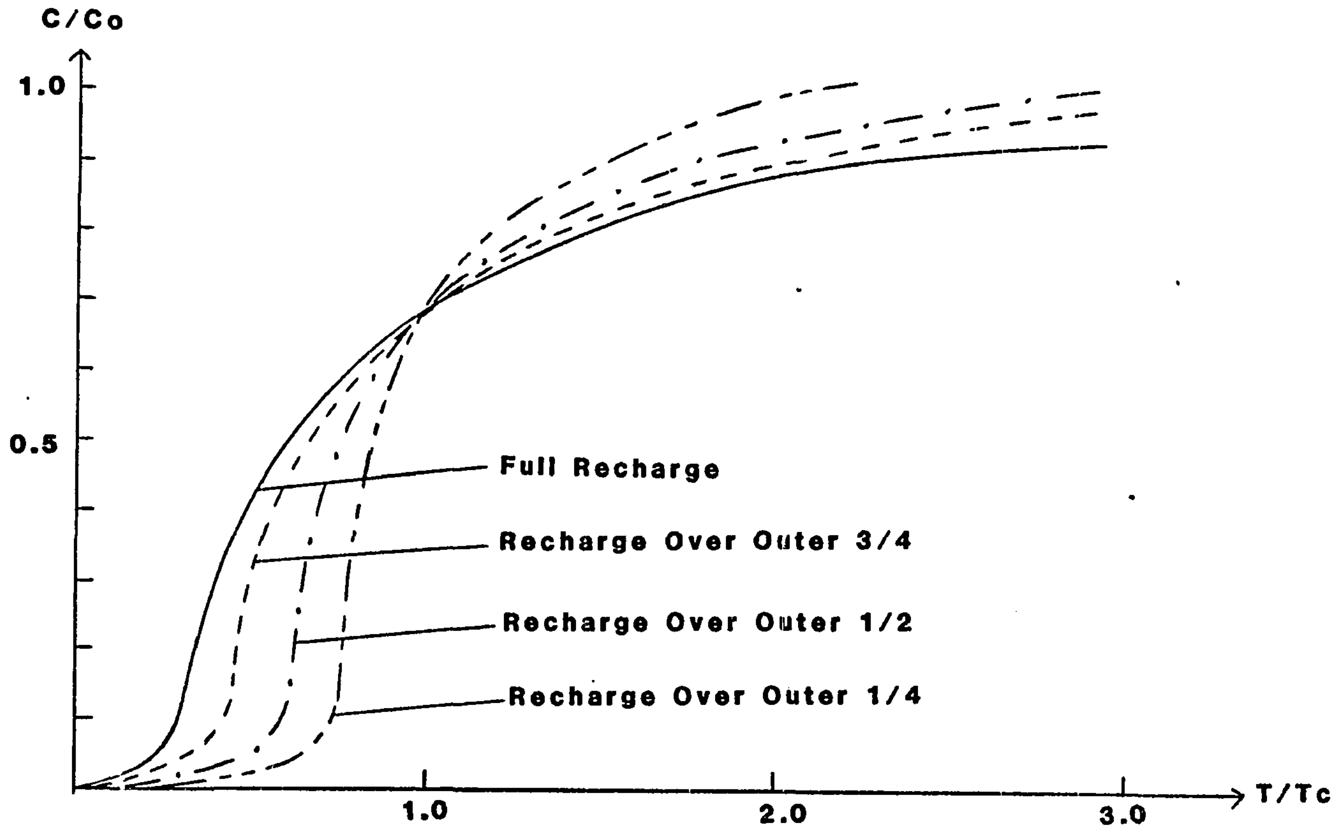


Figure V-8: Pollutant Breakthrough Curves for 90% Penetrating Barrier at Variable Recharge Lengths.

placed furthest downgradient provides the greatest time lag, it actually has the greatest rate of concentration increase and in fact overtakes the barrier placed right next to the source at a relative concentration of about 0.7. The explanation for this phenomena is that placement of a barrier downgradient from the recharge allows the flow pattern to become established before entering the area of influence of the barrier.

Plotted in Figure V-9 are $\overline{\Psi} = 0.5$ streamlines for the full recharge and outer 1/4 recharge cases. As seen from Figure V-9, the flow distance for the outer recharge streamline is greater than that of the full recharge streamline. However, the outer recharge streamline drops vertically initially and proceeds to take a more direct route to the opening in the barrier. Since acceleration is greater in the X direction than the Y direction, it can be seen that the streamline that is lower will move faster once in the area of influence of the velocity gradient. This behavior is exemplified by the fact that in the full recharge case, the inner streamtubes are <u>not</u> the first streamtubes to arrive under the barrier. The inner streamtubes are delayed by the barrier and the outer streamtubes are accelerated by the barrier. This causes all the streamtubes to arrive under the barrier at or about the same time. Hence, the relative concentration for the furthest recharge case seems to increase almost vertically.

Figure V-8 is a plot of pollutant breakthrough for a constant rate of recharge. If the rate of recharge is increased when the length of recharge area is decreased, breakthrough curves can be developed for different recharge scenarios at a constant amount of recharge. These

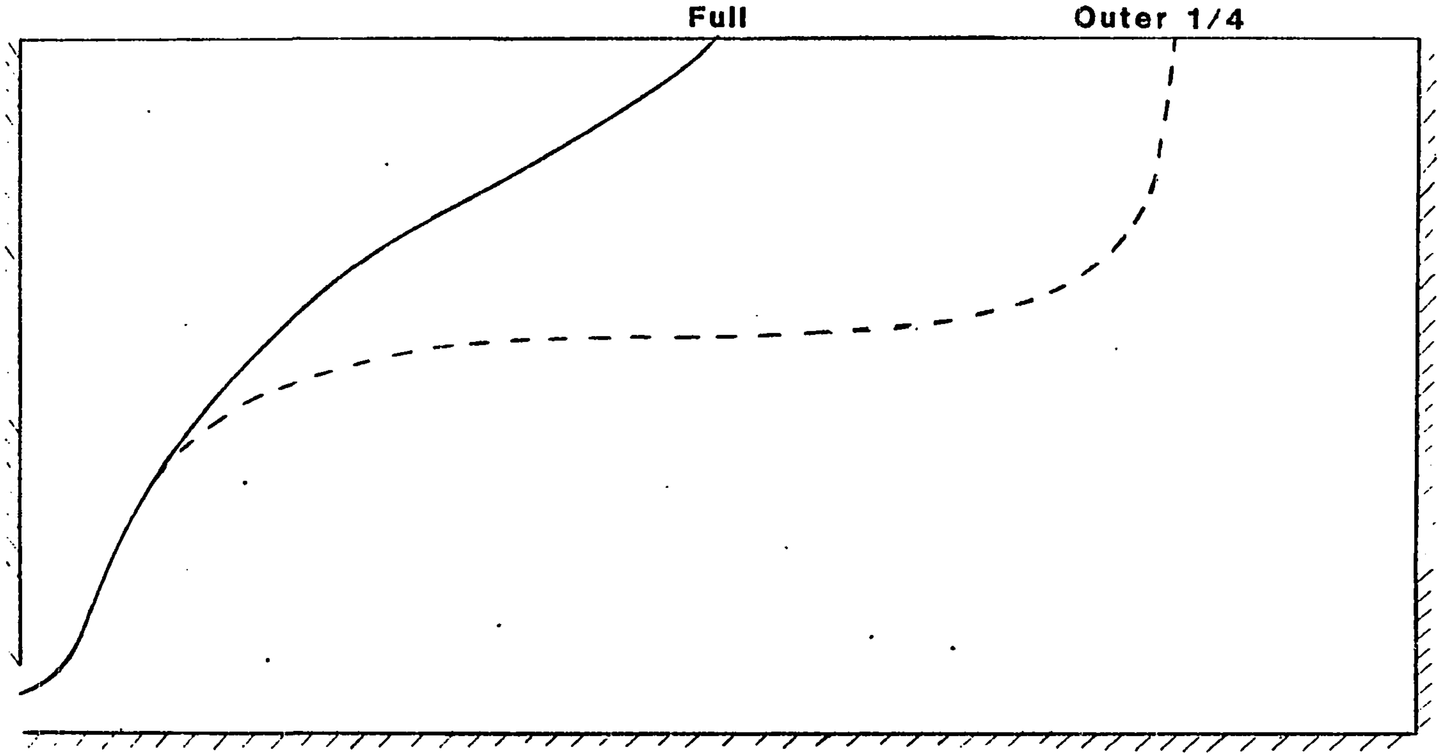


Figure V-9: $\overline{\Psi}=0.5$ Streamlines for Full Recharge and 1/4 Recharge.

curves are plotted in Figure V-10. These curves also show that placing the barrier downgradient of the source actually increases the rate of concentration increase under the barrier. More specifically, these curves show that the further the barrier is removed, the poorer it performs in relation to its own aquifer response time (t_c). For example, the furthest barrier shows a dramatic concentration increase when time has only reached about two-tenths the aquifer response time for that recharge rate. Conversely, the no barrier curve shows a more gradual concentration increase and does not even start to increase significantly until time has reached about three-tenths the aquifer response time for that recharge amount. Therefore, placing a barrier downgradient of a source of contaminated recharge is a poor application of barrier technology.

Problem 3: Anisotropic Soil

Another variation imposed on the aquifer was to place anisotropic soils within the aquifer. Two cases will be examined. First, the soil will be assumed to be highly conductive in the horizontal direction ($K_x >> K_y$). Conceptually, this is equivalent to a highly stratified soil; perhaps layers of sand and gravel between layers of clay. The second case will be that of a soil highly conductive in the vertical direction ($K_y >> K_x$). Conceptually, this is equivalent to a highly fractured soil.

The approach used was to apply an overall directional conductivity reflecting the ability of the soil to transmit water in that direction. Also, because the hydraulic conductivities for this

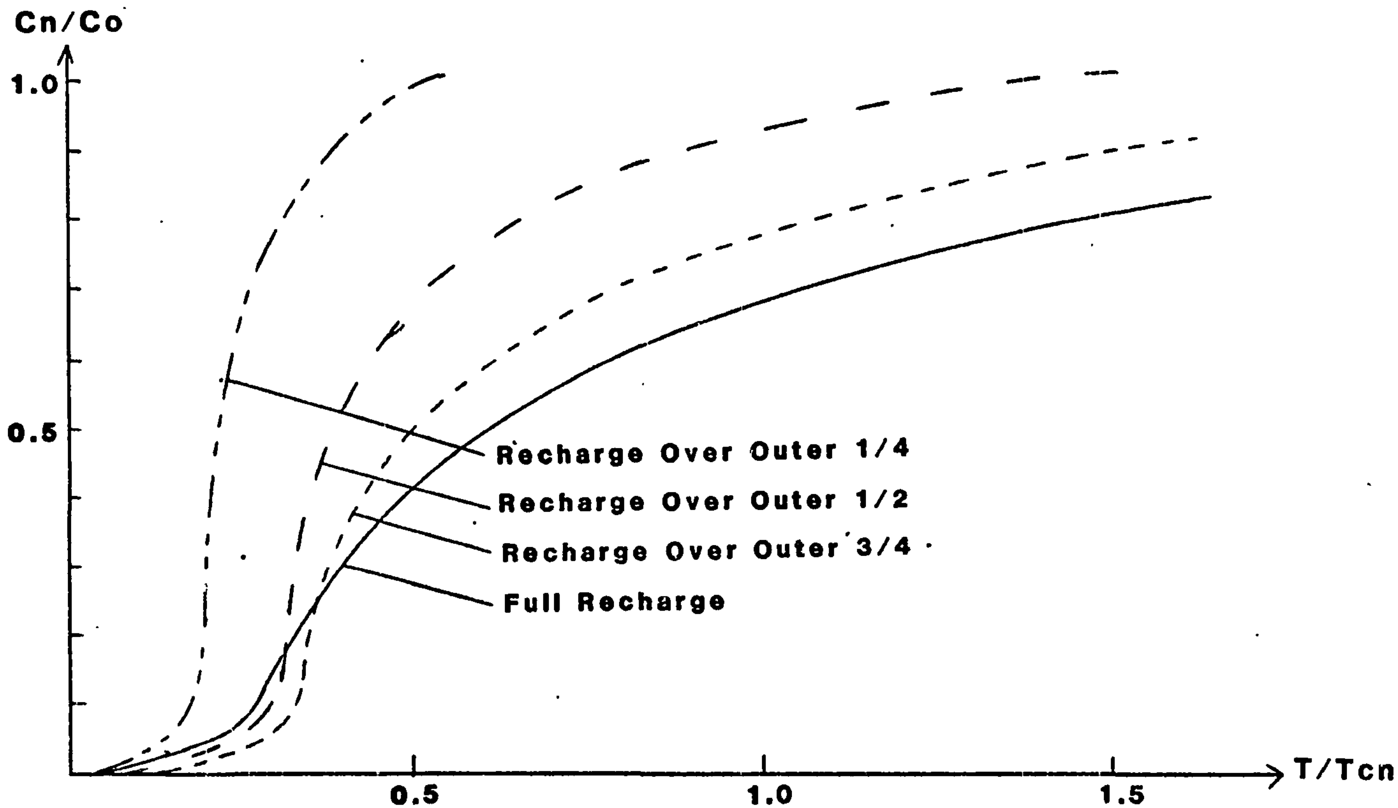


Figure V-10: Pollutant Curves for 90% Penetrating Barrier at Constant Amounts of Recharge.

model are incorporated in an aspect ratio ($K = (K_x/K_y)(D/S)^2$), only the ratio of horizontal conductivity to vertical conductivity is needed. Hence, if a conductivity ratio of 10 ($K_x/K_y = 10$) is used, this implies that the aquifer has a greater propensity to transmit the water laterally than vertically.

Plotted in Figure V-11 are the pollutant breakthrough curves for no barrier and a deep (90%) barrier in a horizontally-conductive ($K_x/K_y = 10$) aquifer subjected to full length recharge. These curves follow the pattern established previously, i.e., the deep barrier provides an initial time lag but eventually overtakes the no barrier case. The interesting aspect of these curves, however, is the very gradual concentration increase at the higher values of C/C_o. What is happening is that the furthest streamtubes (the streamtubes that must travel furthest downward) are having trouble moving vertically downward initially. Hence their time of arrival under the barrier is being delayed.

Plotted in Figure V-12 are the same curves except the soil is now dominated by vertical conductivity ($K_x/K_y = 0.1$). These curves show a more gradual rate of concentration increase. This phenomena reflects the fact that the streamtubes are dropping vertically with relative ease but are having trouble moving laterally. Since the difference in horizontal distance (x-direction) each streamtube must travel is greater than the distance difference in the vertical direction, the streamtubes will tend to arrive in proportion to the horizontal distance which they must travel. This accounts for the gradual rate of concentration increase.

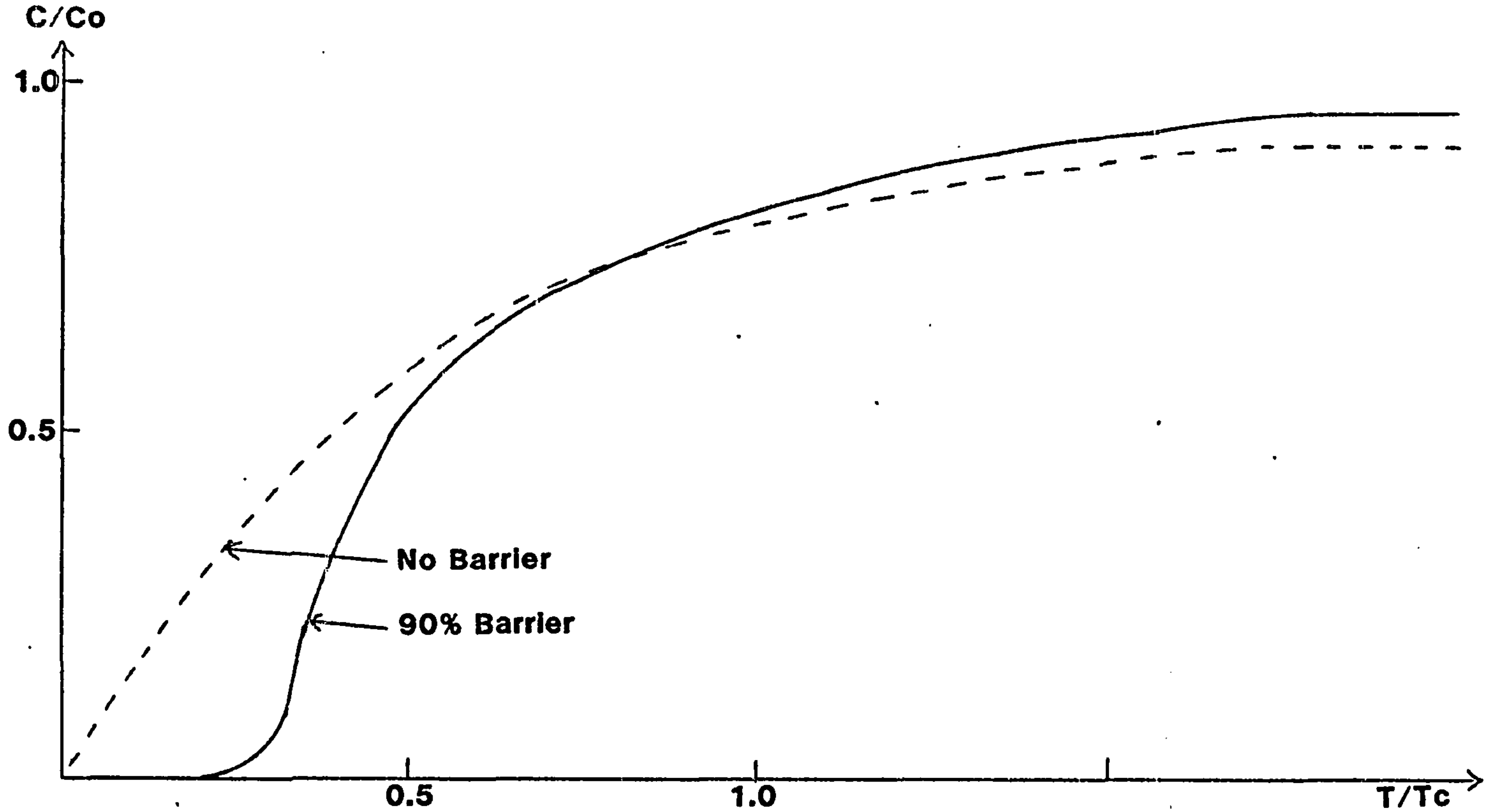


Figure V-11: Pollutant Breakthrough Curves for No-Barrier and 90% Barrier In Horizontally Conductive Soil.

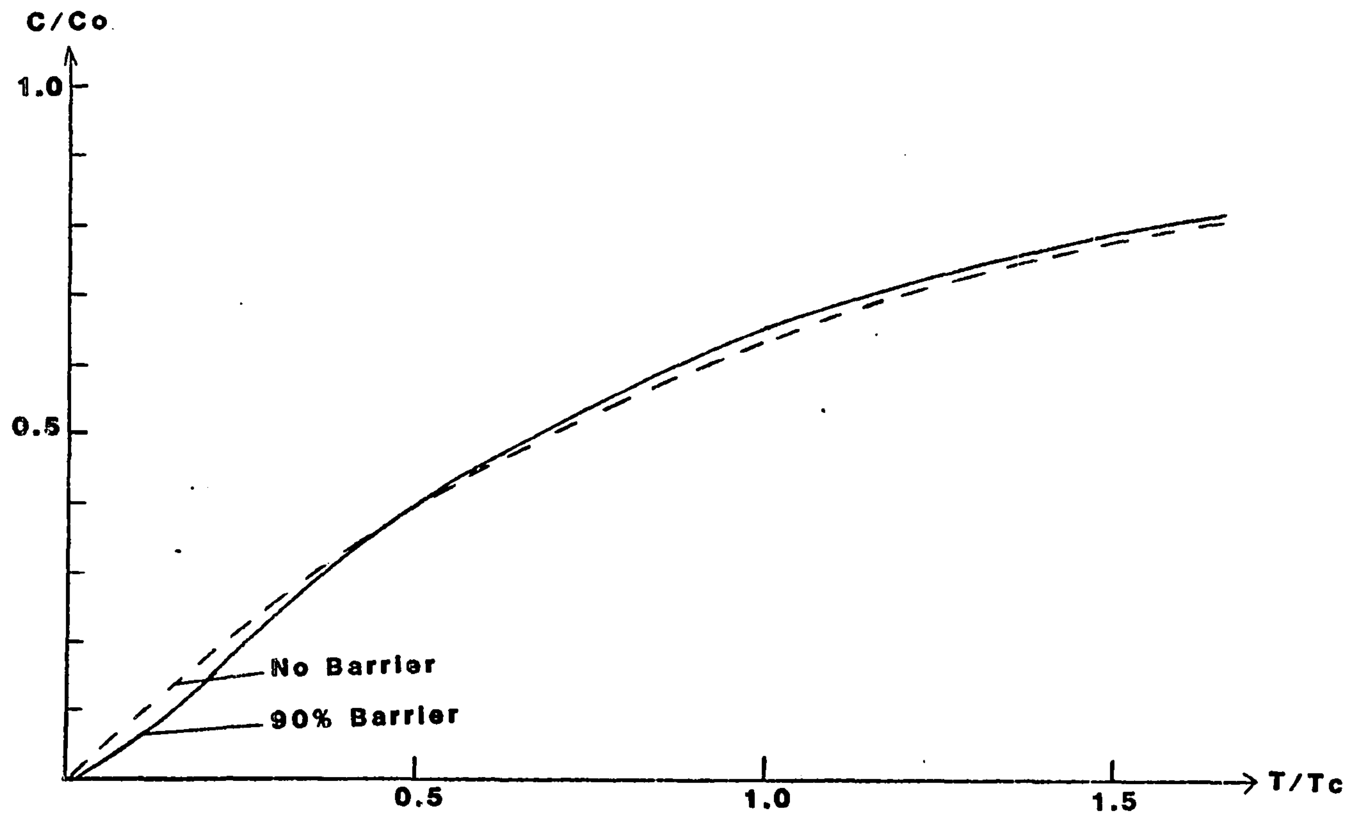


Figure V-12: Pollutant Breakthrough Curves for No-Barrier and 90% Penetrating Barrier in Vertically Conductive Soil.

The interesting thing to note from Figure V-12 is that practically nothing is gained from placement of the barrier. The nature of the aquifer itself retards lateral pollutant migration significantly. Hence, the placement of the barrier has little effect in retarding pollutant migration. This is directly opposed to the performance of the barrier in a horizontally conductive aquifer (Figure V-11) which provides a significant time lag.

Plotted in Figure V-13 are the breakthrough curves for deep barriers in a horizontally-dominated soil for two different recharge lengths. Once again, the distant recharge case provides a greater initial time lag but soon overtakes the adjacent recharge case. It has already been shown that placement of a barrier in a vertically-dominated soil has little effect on pollutant migration, and as such, it would be totally inappropriate to place such a barrier far downgradient in a vertically dominated soil. These two facts further exemplify that moving a barrier downgradient from a recharge source is inappropriate.

Mass Balance

In these simplified analyses, it has been inherently assumed that the aquifer was initially saturated with uncontaminated water. The pollutant breakthrough curves, therefore, represent the time required to completely flush the aquifer with contaminated recharge. A good check of the validity of the model would be a mass balance. In other words, the mass (amount) of recharge required to flush a given aquifer should be the same regardless of whether or not a partially penetrating

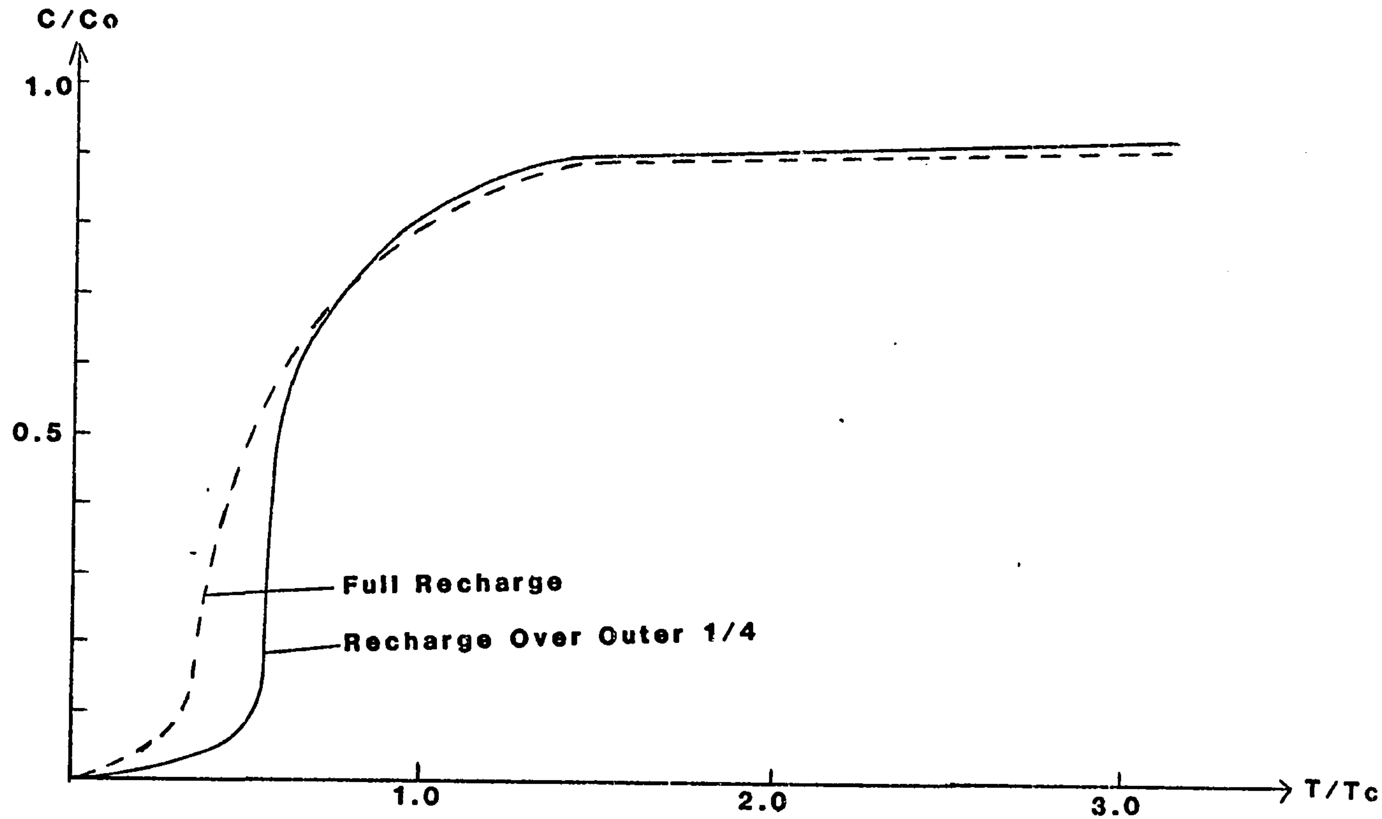


Figure V-13: Pollutant Breakthrough Curves for 90% Penetration Barrier in Horizontally Conductive Soil Under Variable Recharge Lengths.

barrier is present. Integrating the area under the concentration versus time (pollutant breakthrough) curves gives the mass added to the aquifer for a constant rate and length of recharge. The mass added should be equal for the with barrier and without barrier cases.

By inspection, Figures V-3, V-5, V-6, V-7 and V-12 appear to show good mass balances. In other words, the areas under the curves all seem to be about equal and in fact, the difference in areas for the curves in all these figures is less than about 1%. Figure V-11 presents the only situation that would not appear to satisfy the mass balance requirements by visual inspection. Integrating the area under these curves reveals that the area under the no-barrier case exceeds that of the with barrier case by about 6% up to a t/t_c value of 2.0. However, this is still only a small difference percentage and it will continue to decrease as the graphs are continued out to larger t/t_c values. The close mass correlations between the with-barrier and without-barrier cases for each of the variations imposed on the aquifer confirm the accuracy of this simplified approach.

Conclusions

This phase of the analysis utilized a simplified (plug flow) model to examine the travel of pollutants from a source of contaminated recharge through an aquifer and under a partially penetrating impermeable barrier. The model was applied to a variety of simple input conditions. From this analysis the following conclusions can be drawn:

1. The most efficient use of barrier technology is to place the barrier directly adjacent to the source of

contaminated recharge. Placement of the barrier downgradient has been examined under a variety of conditions and in each case it was found to be less effective than an adjacent barrier. Placing a barrier downgradient allows the pollution front to become established and approach the gap in the barrier as a wall. Hence, concentration increase under the barrier is quite dramatic following a small initial time lag. The initial time lag is due solely to aquifer geometry. When these facts are coupled with the added consideration that placing a barrier downgradient will increase its total length (and cost), it becomes obvious that this is less than optimum application of the technology.

2. Unless a perfect connection can be made between the barrier and the underlying impermeable formation, the barrier does not provide complete containment. The barrier will provide an initial time lag before the appearance and increase in concentration of a contaminant under the barrier. However, this positive attribute is counterbalanced by the fact that the concentration under a partially penetrating barrier will actually increase faster than the outflow concentration in an aquifer with no barrier. This phenomena will be significant depending on the pollutant and the relative concentration considered to be critical.

3. When an aquifer with a waste source is highly conductive in the vertical direction, the effectiveness of a barrier is practically negligible. Conversely, aquifers conductive in the horizontal direction are more amenable to barrier technology. The reason for this is that the pollutants tend to run laterally initially until they reach the barrier. They then must travel vertically downward to get by the barrier. In essence, the barrier dramatically increases the flow path distance the pollutant would normally follow.

4. The simple numerical model and analysis employed in this phase of the research was relatively simple, quick, and economical. However, the results generated only provide information on the general behavior of barriers. The results would be hard to apply to a field problem unless specific parameters such as aquifer response time (t_c) could be related to input data such as hydraulic conductivity. Additionally, the simple numerical model does not handle some of the more complex variations such as dispersion, layered soils, or semi-permeable barriers.

Phase III -- Analysis by Numerical Solute Transport Model

This phase of the analysis involved applying the Konikow-Bredehoeft (K-B) numerical solute transport model to the proposed problem to examine three phenomena. First, the effects of dispersion on the pollutant breakthrough curves were examined. Second, the effects of keying the barrier into a relatively impermeable yet not totally impermeable formation were analyzed. Finally, the behavior of a totally penetrating but semi-permeable barrier was assessed.

The K-B model will be applied to a hypothetical aquifer. The grid representation of the aquifer is in Figure V-14. The K-B model requires that the outermost boundaries of the aquifer be designated no-flow boundaries. The second row from the top of the grid represents a recharge source of constant concentration on the right side of the barrier and a no flow layer on the left side of the barrier. The first column left of the barrier represents a constant head condition. The constant head difference across the barrier simulates a constant recharge condition.

Listed in Table V-2 are the input parameters that remain constant throughout the analysis. The porosity and hydraulic conductivity are arbitrary but can be considered typical. The hydraulic gradient or head difference across the barrier will be kept constant so that aquifers of varying geometry can be compared on a common basis. The storage coefficient is set equal to zero to generate steady-state conditions in the K-B model. The ratio of horizontal to vertical hydraulic conductivity is constant and equal to one because the effects of anisotropy were analyzed previously. The ratio of

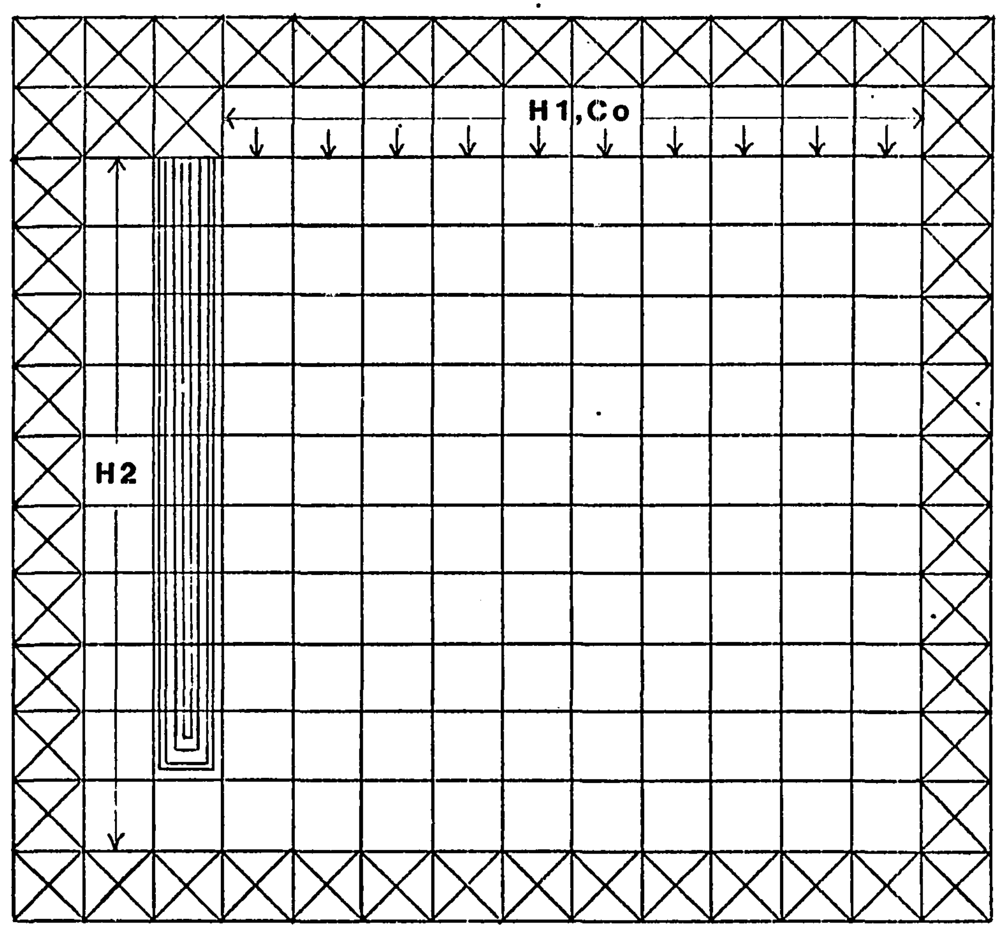


Figure V-14: Grid Representation of Aquifer Subjected to Contaminated Recharge.

Table V-2: Constant Parameters fcr Numerical Model

1. Hydraulic Conductivity = 0.0001 ft/sec (2.63 m/day)

2. Kx/Ky = 1.00

3. Initial Hydraulic Gradient Across Barrier = 0.25 ft/ft

   or

4. Initial Head Difference Across Barrier = 10.0 ft

5. Porosity = 0.35

6. Storage Coefficient = 0.0

7. Aquifer Dispersitivies = 0 or 1 or 5 or 10

8. Longitudinal Dispersivity/Transverse Dispersivity = 1.0

9. Pumping Period = 5.0 years

longitudinal to transverse dispersivity is constant and set equal to one because the input data is strictly hypothetical and one would only want to examine variations in dispersivity with actual field data.

Problem 4: Effects of Dispersion

Plotted in Figure V-15 are the pollutant breakthrough curves at 90% depth for the no barrier case and three dispersive cases for a 90% penetrating barrier. The aquifer is 100 feet deep and 400 feet wide. This approaches the depth limit to which barriers can be placed and maintains the 4:1 width to depth ratio used in the previous analyses.

The first thing to note about Figure V-15 is that the placement of a partially penetrating barrier in the aquifer is ineffective in terms of retarding pollutant migration. The no-barrier case shows an almost immediate response to the contaminated recharge. However, the with-barrier cases only provide time lags on the order of 0.5 years over that of the no-barrier case. Note also that the rate of concentration increase under the barrier is at least as great if not greater than that of the no-barrier case. In essence, all the barrier does is provide a small initial time lag. The actual numerical values of these time lags is not important in that the input data is hypothetical, but the relative behavior of the barrier to the no-barrier case is important and it shows that partially penetrating barriers are not completely effective in retarding pollutant migration.

One positive note about partially penetrating barriers is that they confine the escaping pollutant to a localized area where it could

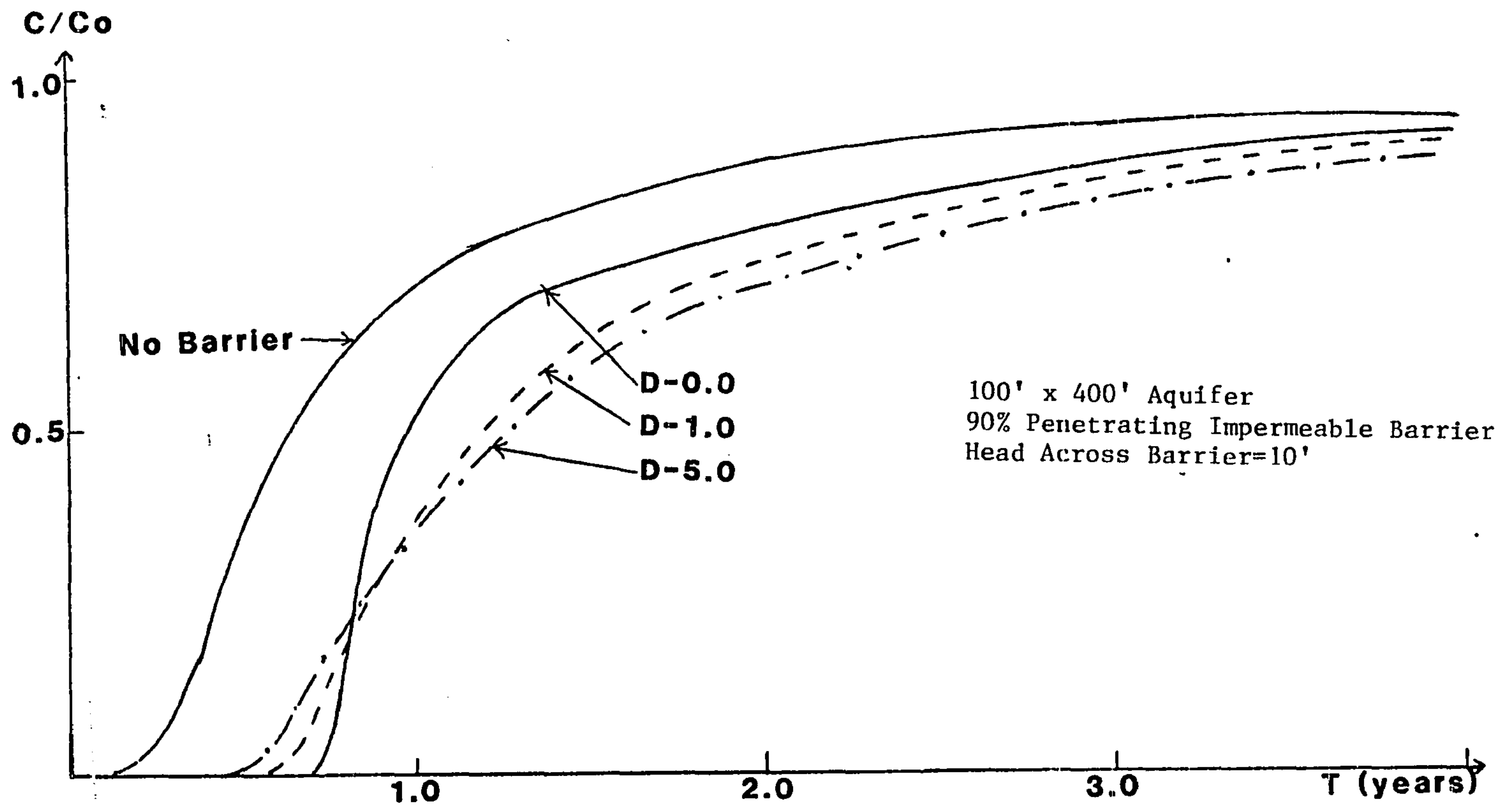


Figure V-15: Pollutant Breakthrough Curves for 100 Foot Deep Aquifer.

be monitored or captured. The no-barrier case allows the pollutant to migrate laterally for the full depth of the aquifer.

The second thing noted about Figure V-15 is that the effects of dispersion are somewhat anticipated and for the most part negligible. The effects of dispersion are to show an earlier initial arrival of the contaminant, a more gradual increase in concentration and an assymptotic approach to the input concentration (C_o). The dispersive effects cannot be totally disregarded in that they could become important depending on the pollutant in question. Since dispersion shows an earlier initial arrival of pollutants under the barrier, this could be critical for a highly toxic or perhaps carcinogenic pollutant. On the other hand, the more gradual increase in concentration shown by the dispersive cases shows these partially permeable barriers to be somewhat more effective in delaying the concentration from reaching input levels (C_o). This could be significant, again depending on the type of pollutant and the concentration considered to be critical. However, the cases of dispersion do not appreciably affect the overall performance of these barriers. The barrier shows only a small initial time lag whether the effects of dispersion are included or not.

This behavior is not specific to this aquifer. Plotted in Figures V-16 and V-17 are the same relationships for 50-foot and 25-foot deep aquifers, respectively, under the same initial hydraulic gradient across the barrier. The only conclusion to be drawn is that effects of dispersion are even less pronounced in the shallower aquifers.

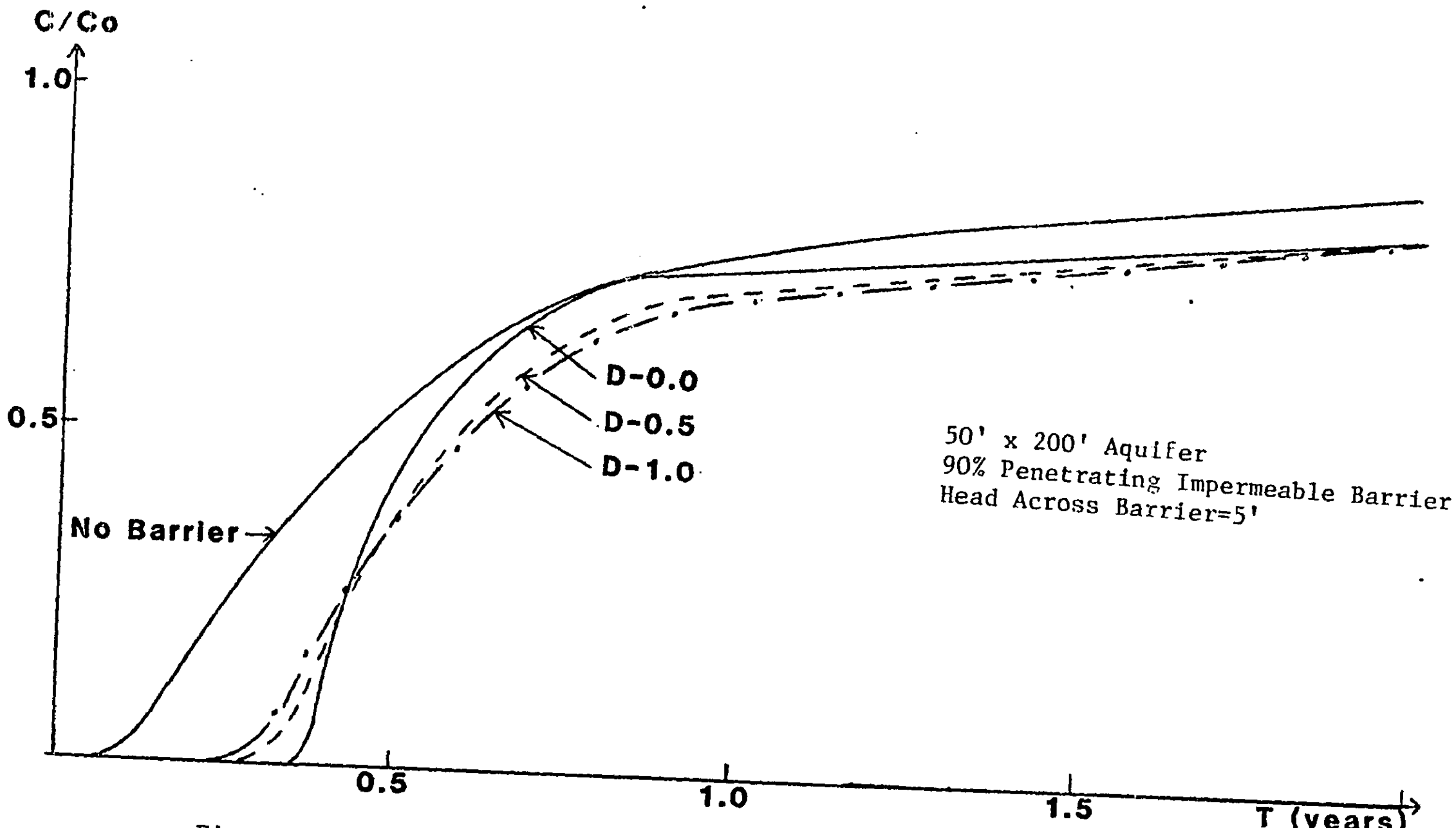


Figure V-16: Pollutant Breakthrough Curves for 50 Feet Deep Aquifer.

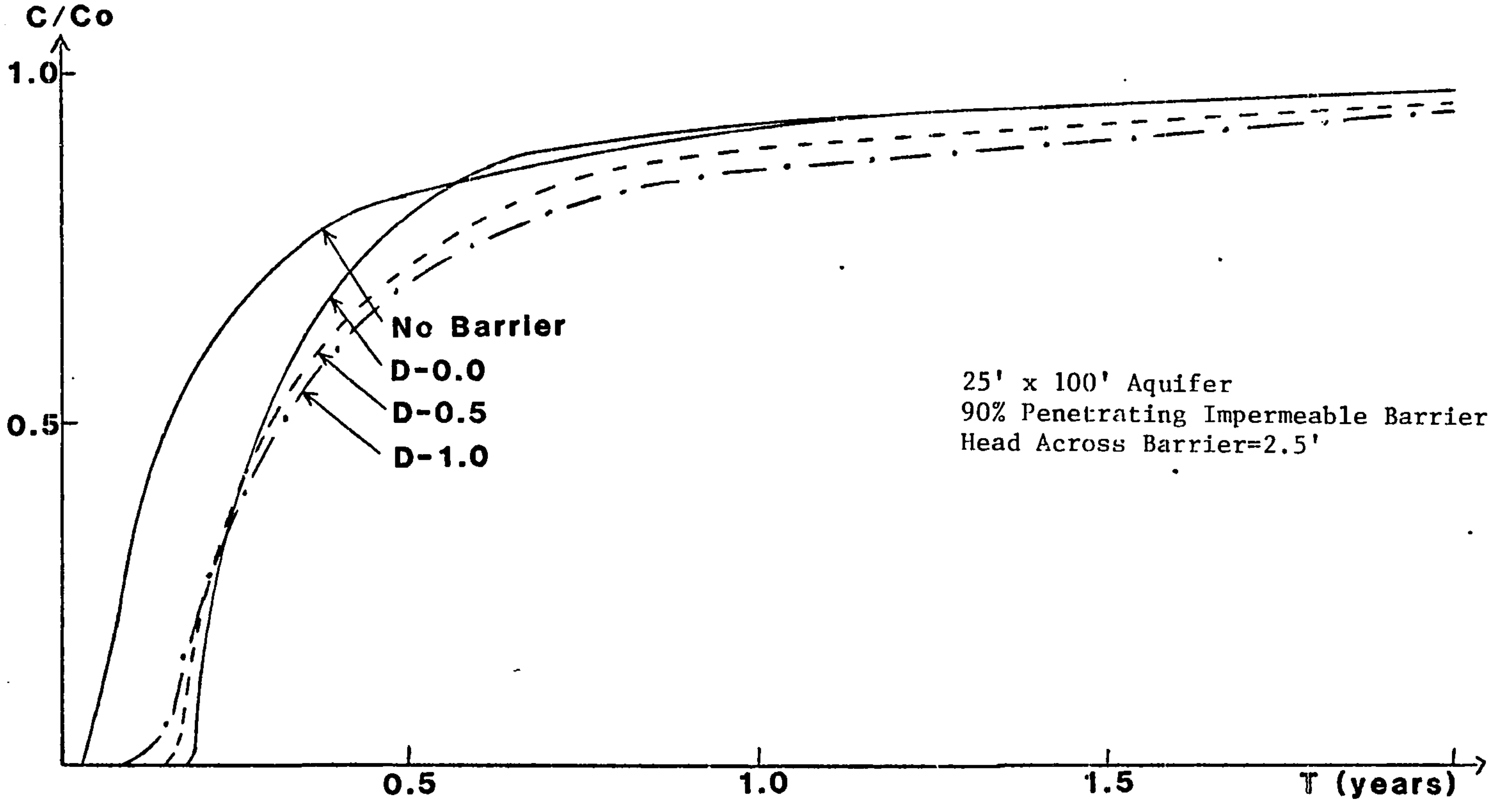


Figure V-17: Pollutant Breakthrough Curves for 25 Foot Deep Aquifer.

If the 50-foot and 25-foot deep aquifers are subjected to a total head difference (rather than hydraulic gradient) equal to that of the 100-foot deep aquifer a predictable pattern develops. Plotted in Figures V-18 and V-19 are the pollutant breakthrough curves for these aquifers subjected to a comparable head difference as the 100-foot deep aquifer. As expected, this increased head difference causes the arrival and increase in concentration under the barrier to be pushed forward in time. The behavior of the pollutants is similar to that generated earlier. The barriers provide small initial time lags but tend to behave like the no-barrier case as time proceeds.

Figure V-20 is a plot of the behavior of the pollutant under the barrier in a 50-foot deep aquifer for two different recharge rates. In essence, this figure demonstrates the sensitivity of the systems to the driving force, in this case the total head difference across the barrier. In general terms, the initial head difference influences the first arrival of pollutants under the barrier but does not affect the rate of concentration increase. Figure V-20 shows that the smaller the driving force working on a partially penetrating barrier, the better the barrier behaves in terms of retarding pollutant migration. However, this must be considered a case of the lesser of two evils as the partially penetrating barrier is relatively ineffective in either case.

If one subjects an elongated aquifer (width/depth = 5.0) to the same initial hydraulic gradient across the barrier as the 100 foot deep aquifer, an interesting but not unexpected result is produced. Plotted in Figure V-21 are the pollutant breakthrough curves for an elongated

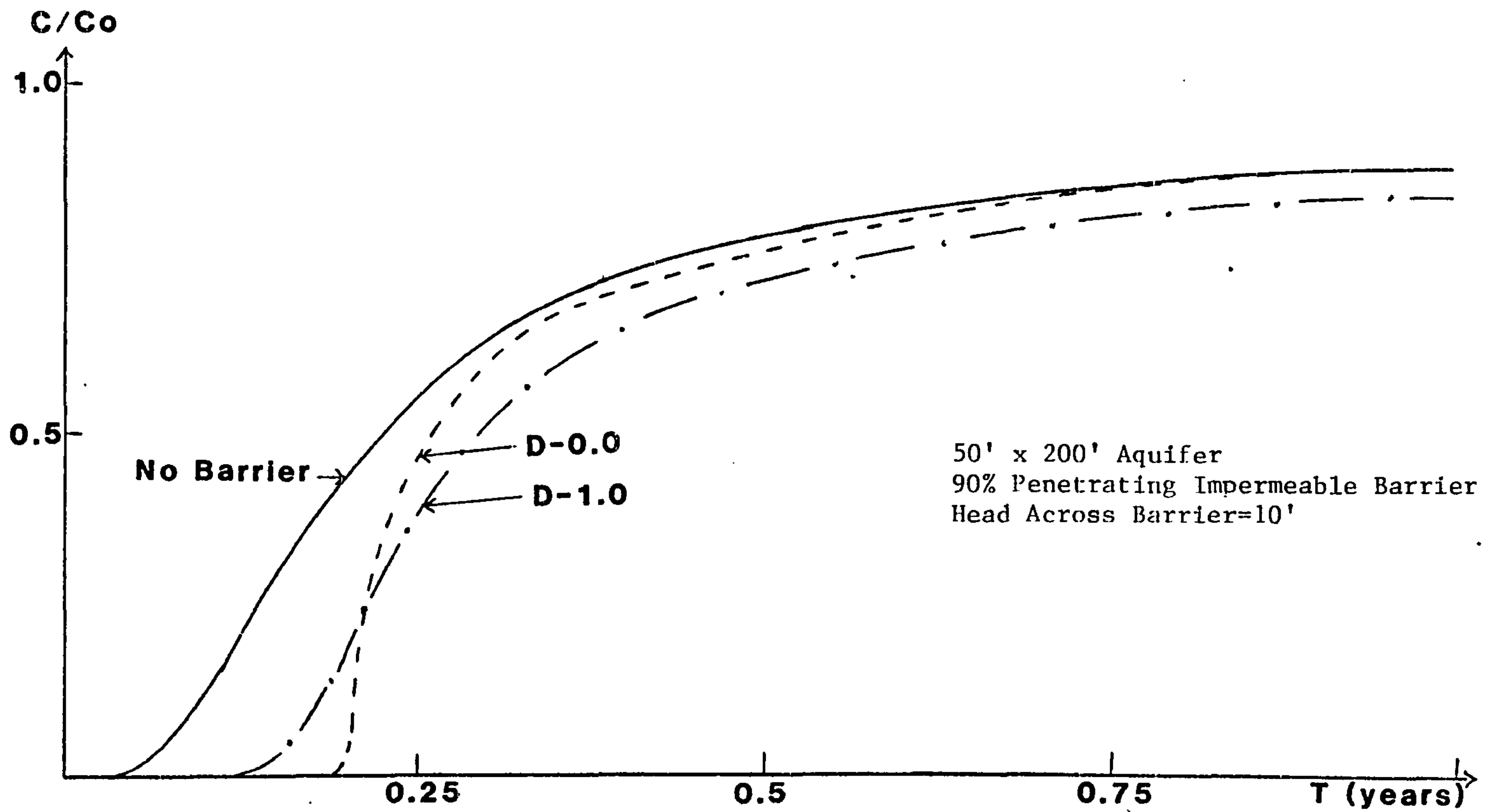


Figure V-18: Pollutant Breakthrough Curves in 50 Foot Deep Aquifer for Head Difference Comparable to 100 Foot Deep Aquifer.

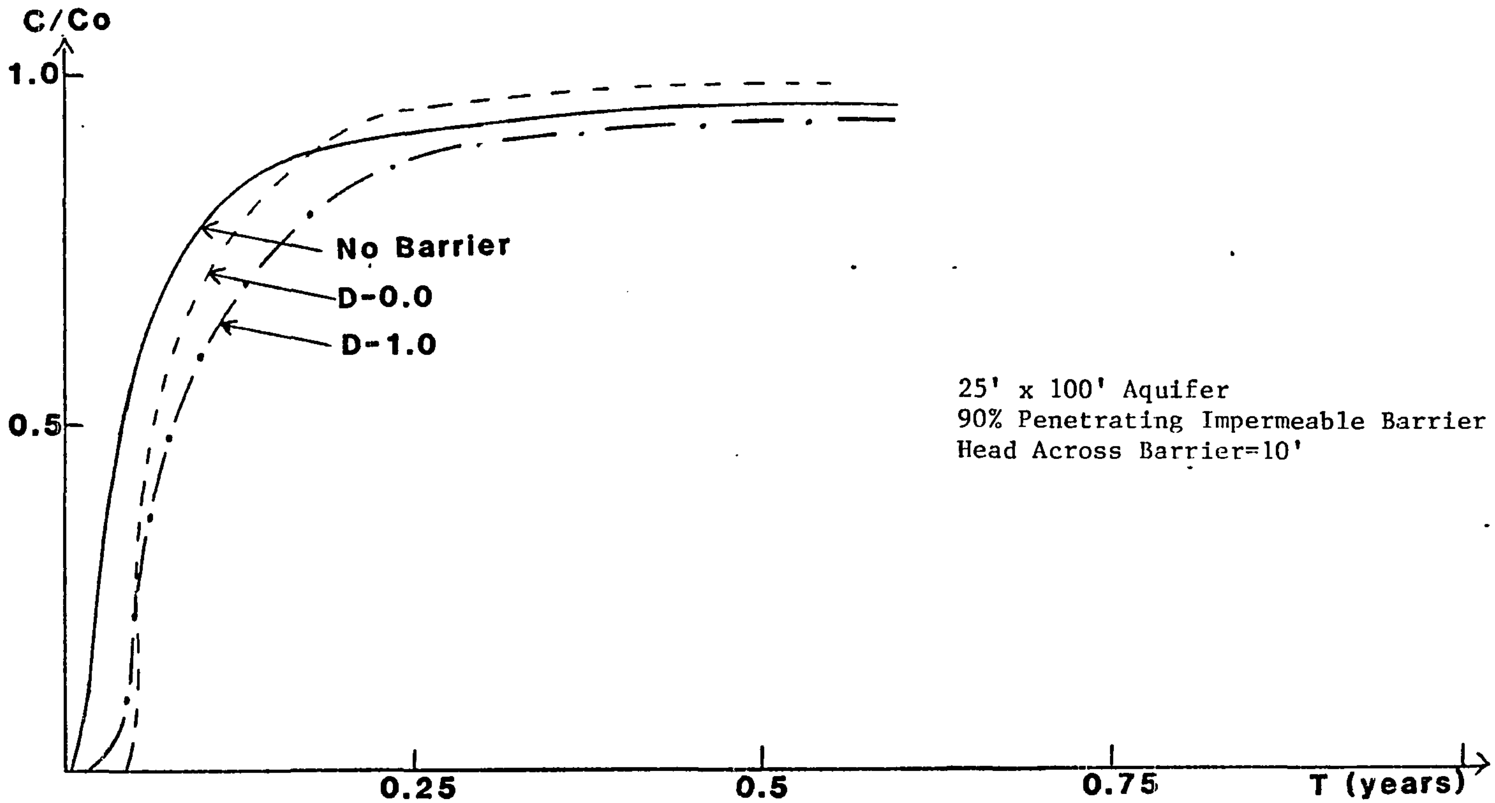


Figure V-19: Pollutant Breakthrough Curves in 25 Foot Deep Aquifer for Head Difference Comparable to 100 Foot Deep Aquifer.

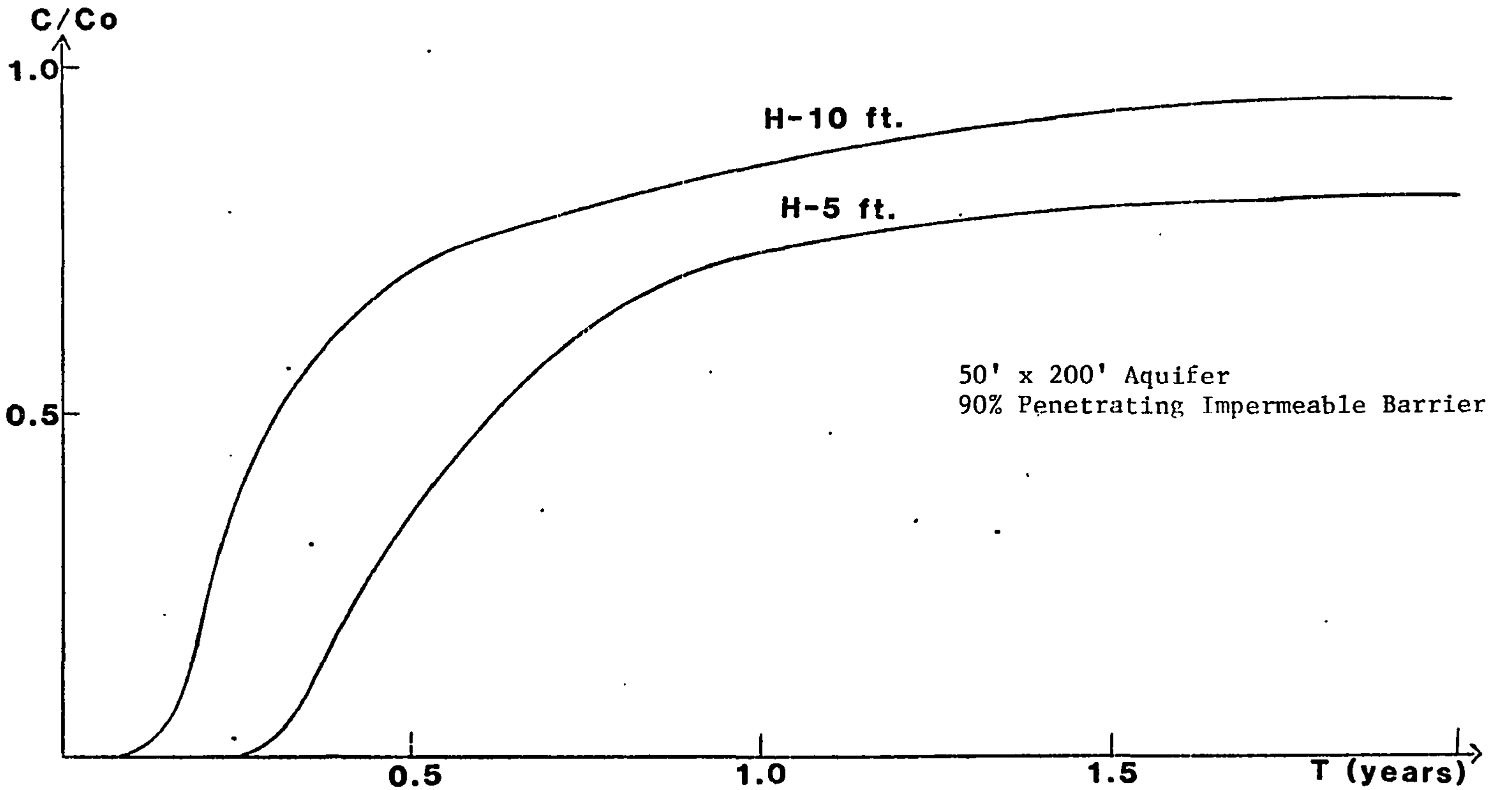


Figure V-20: Pollutant Breakthrough Curves for 50 Foot Deep Aquifer for Two Recharge Rates.

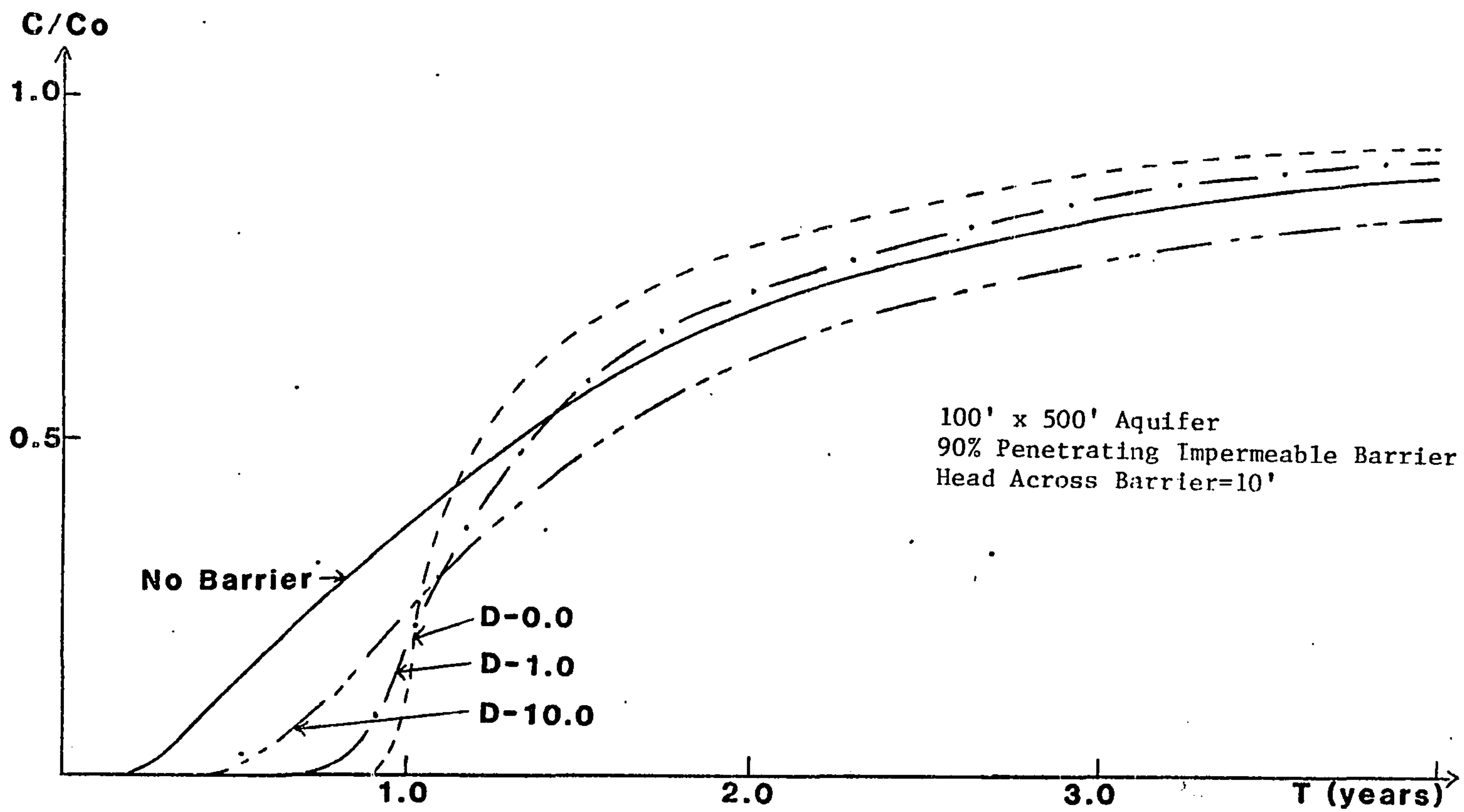


Figure V-21: Pollutant Breakthrough Curves for an Elongated Aquifer.

aquifer subject to a comparable hydraulic gradient across the barrier. The pattern is similar to that for the previous aquifers except that the behavior now parallels that predicted by the previous simplified model. The barrier provides a small initial time lag over the no-barrier case but actually catches and surpasses the no-barrier case in terms of concentration increase for all but the most dispersive case. Note that even though the highly dispersive case does not surpass the no-barrier case in terms of concentration increase, it does show a significantly earlier time of first arrival of pollutant under the barrier. This tradeoff will be significant depending on the type of pollutant.

Problem 5: Layered Soils

One means of alleviating the problem of a partially penetrating barrier would be to construct an adequate key into an impermeable formation. If an impermeable formation does not exist at a reasonable depth one would probably not use a barrier. However, if a relatively impermeable yet not totally impermeable layer exists at a reasonable depth one might key into it. Plotted in Figure V-22 are the breakthrough curves for a partially penetrating barrier (90%) and a barrier that has been keyed into a formation only slightly less permeable than the overlying aquifer ($K_{layer} = 0.1\ K_{aquifer}$). The results are quite dramatic. Not only does the impermeable layer delay the initial appearance of contaminant under the barrier but also decreases the rate at which the concentration of the escaping contaminant increases. This effect is even more pronounced for deeper

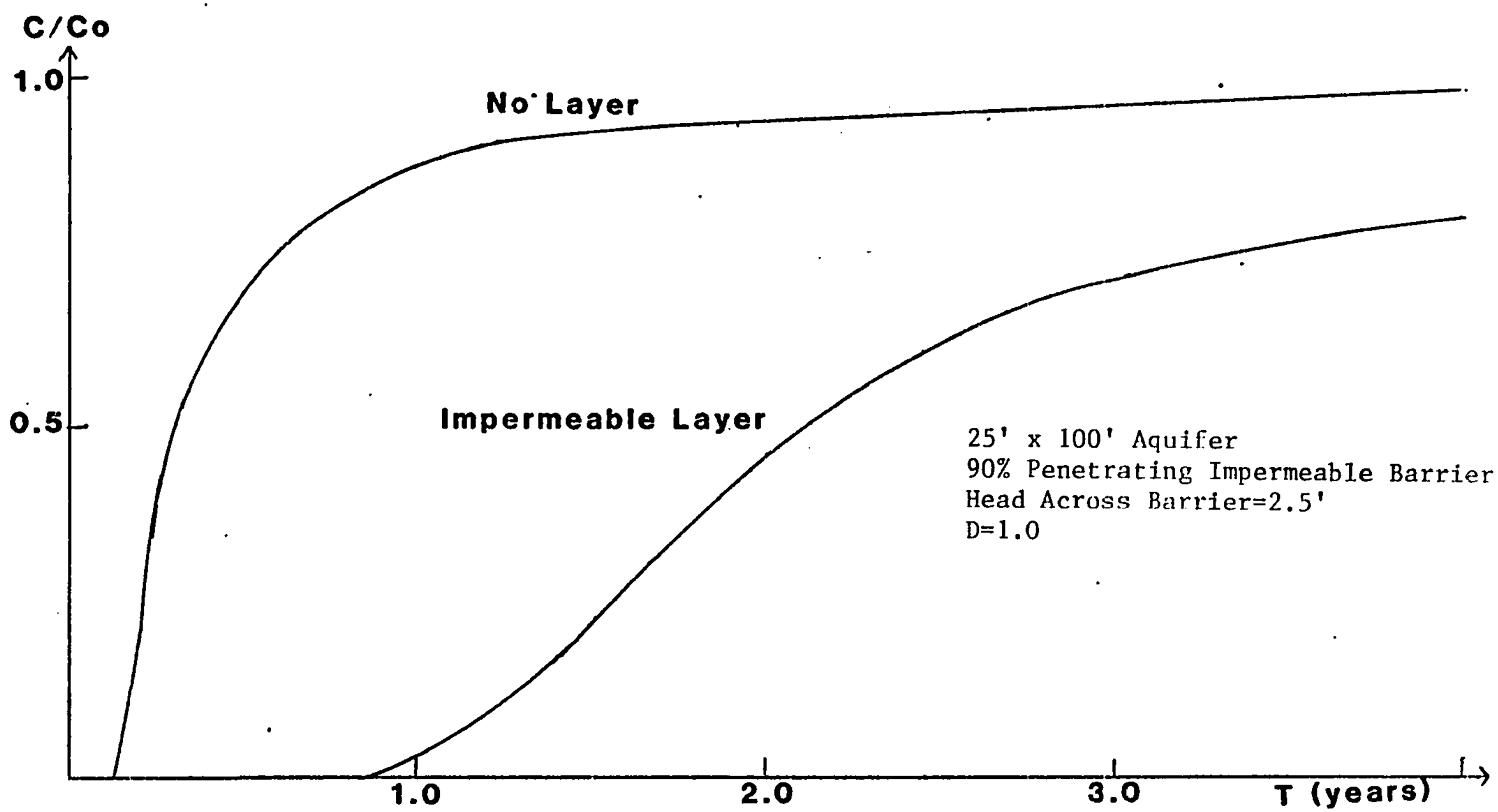


Figure V-22: Pollutant Breakthrough Curves for Barrier Keyed to Impermeable Layer (25 foot).

aquifers. Plotted in Figure V-23 are the same curves for an aquifer twice as deep as that of Figure V-22. The time lag is approximately twice that of Figure V-22 and this is explained by the fact that the contaminant must travel twice the distance vertically before attempting to penetrate the layer.

The above phenomenon has practical significance in that it implies that barriers might be useful at future facilities where an artificial impermeable layer (such as a clay liner) has been placed. In these situations an adequate key can be assured by simply sinking the barrier down past the liner. Such is not the case with deep impermeable bedrock where one must be concerned with both the quality and depth of penetration of the key into the bedrock.

Problem 6: Semi-Permeable Barriers

Another problem that has plagued barriers is that the impermeability of the wall itself cannot be assured. As stated in Chapter III, steel sheet-piles are not initially water-tight and grout curtains and slurry walls are both subject to permeability increases upon exposure to certain contaminants. Plotted in Figure V-24 are the concentration breakthrough curves at 10%, 30%, 50%, 70%, and 90% depths on the left side of the barrier. The barrier is fully penetrating but only possesses a permeability equal to one-tenth that of the aquifer. Note that although the contaminant appears first at the uppermost monitoring point, the concentration actually increases fastest at the second monitoring point. This is due to the fact that the uppermost monitoring point is topped by a no-flow boundary and only feels the

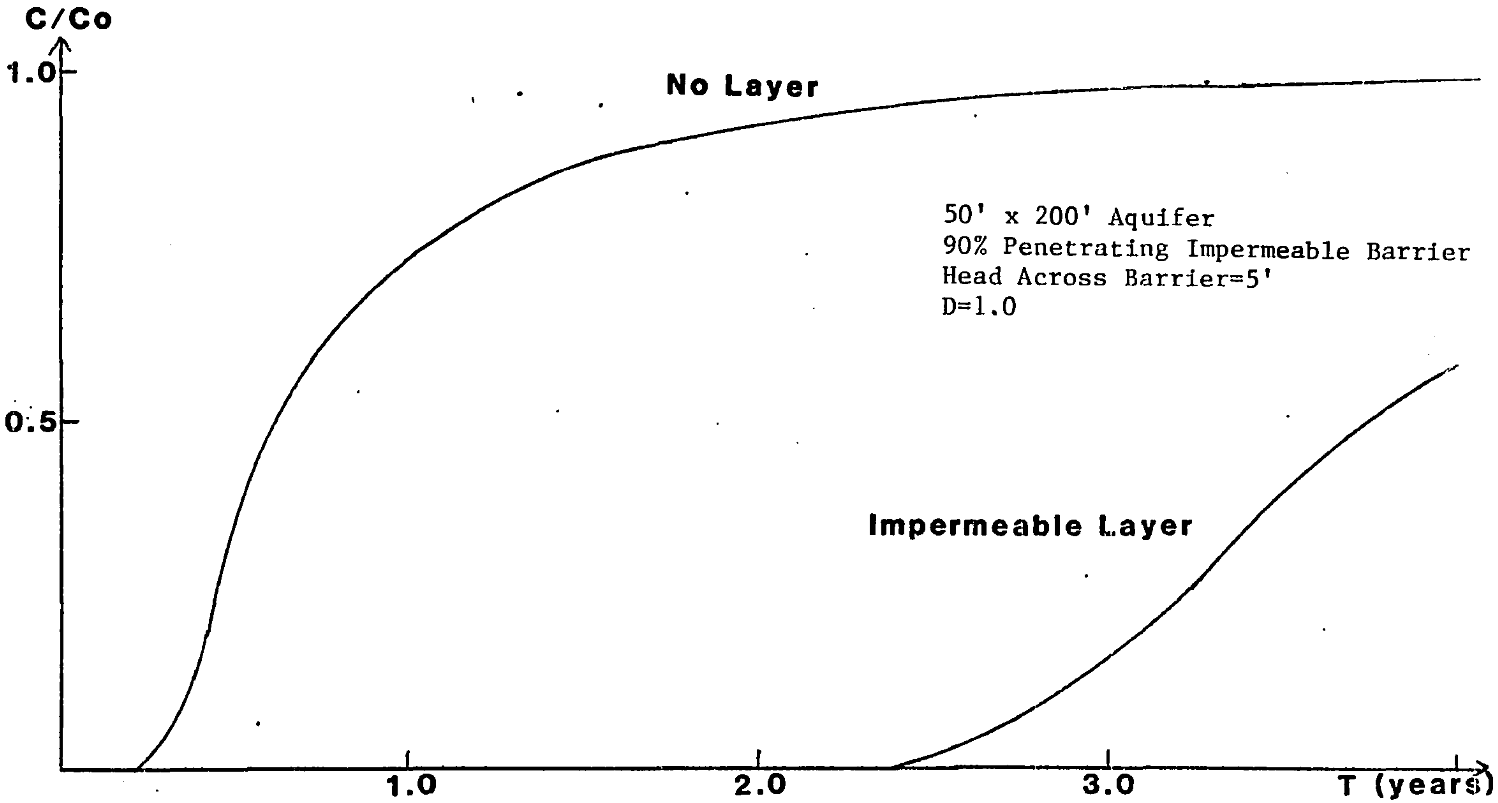


Figure V-23: Pollutant Breakthrough Curve for Barrier Keyed to Impermeable Layer (50 foot).

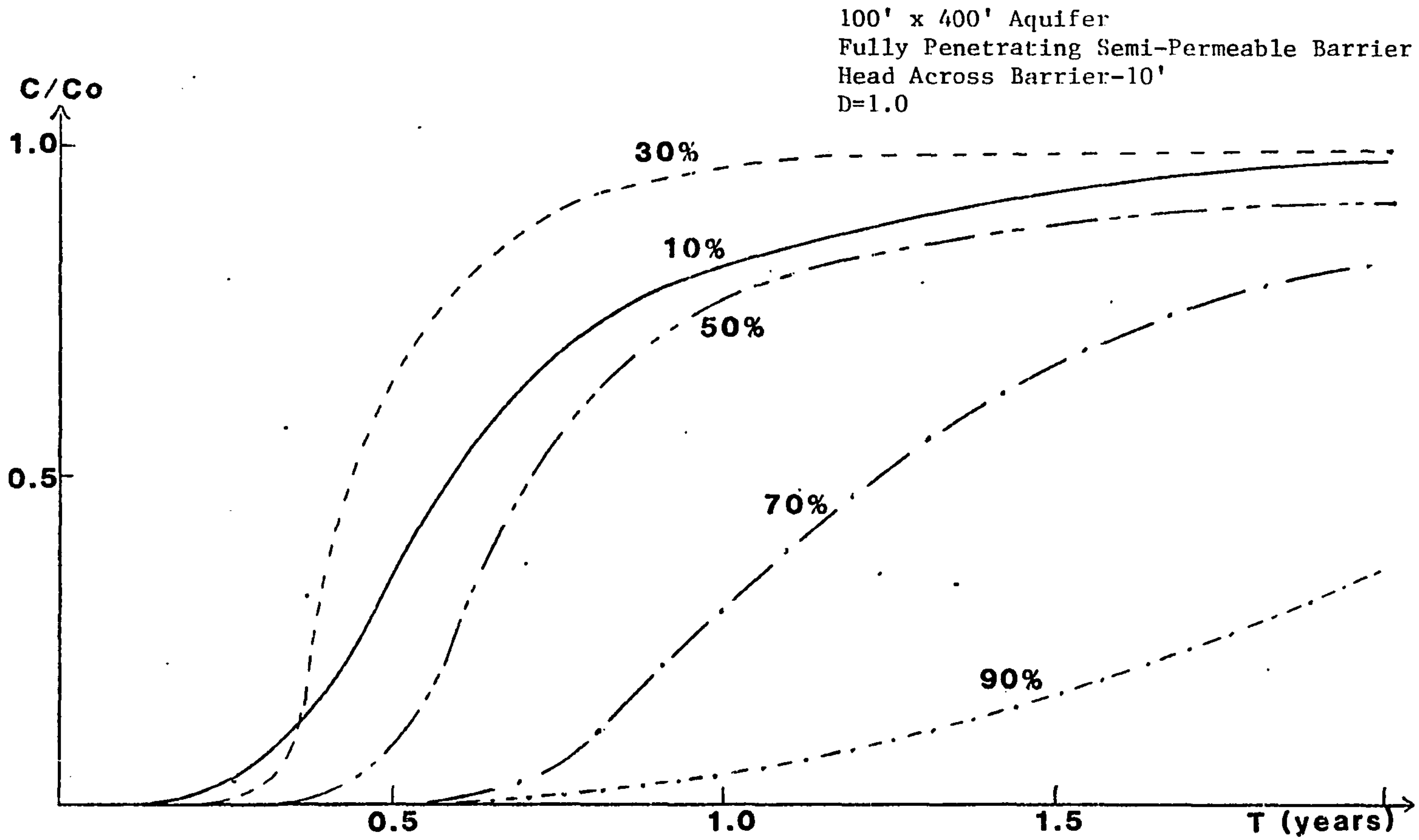


Figure V-24: Pollutant Breakthrough Curves at Selected Depths Adjacent to a Semi-Permeable Barrier.

influence of flow in the uppermost layer while the second monitoring point feels the effects of contaminant flow above, below and at its same level. The deepest monitoring point is the last to receive contamination and increases most gradually. This is because the flow paths of contaminants are no longer directed straight down the barrier. Instead, they move down slightly before penetrating the semi-permeable barrier.

Insuring the impermeability of the barrier appears to be a more critical factor than insuring an adequate key. Note that keying the barrier into a semi-permeable layer was extremely effective in retarding pollutant migration. From Figure V-25 it can be seen that a fully penetrating barrier does not perform effectively unless the permeability of the barrier is at least two orders of magnitude lower than the adjacent aquifer ($K_{barrier} = 0.01 \ K_{aquifer}$). This is reasonable in that there exists a significant stress in the horizontal direction while stresses in the vertical direction are minor, i.e., in this situation the tendency is for the pollutant to migrate laterally more than downward.

Plotted in Figure V-26 is a summary of the behavior of the barrier systems outlined above plus an additional one. The fifth curve is a plot of the breakthrough curve for a partially penetrating (90%) semi-permeable barrier. As seen from the figure, this is a highly undesirable situation in that this system does not behave appreciably different from the system with no barrier. The general trend of these figures is the same irrespective of aquifer depth.

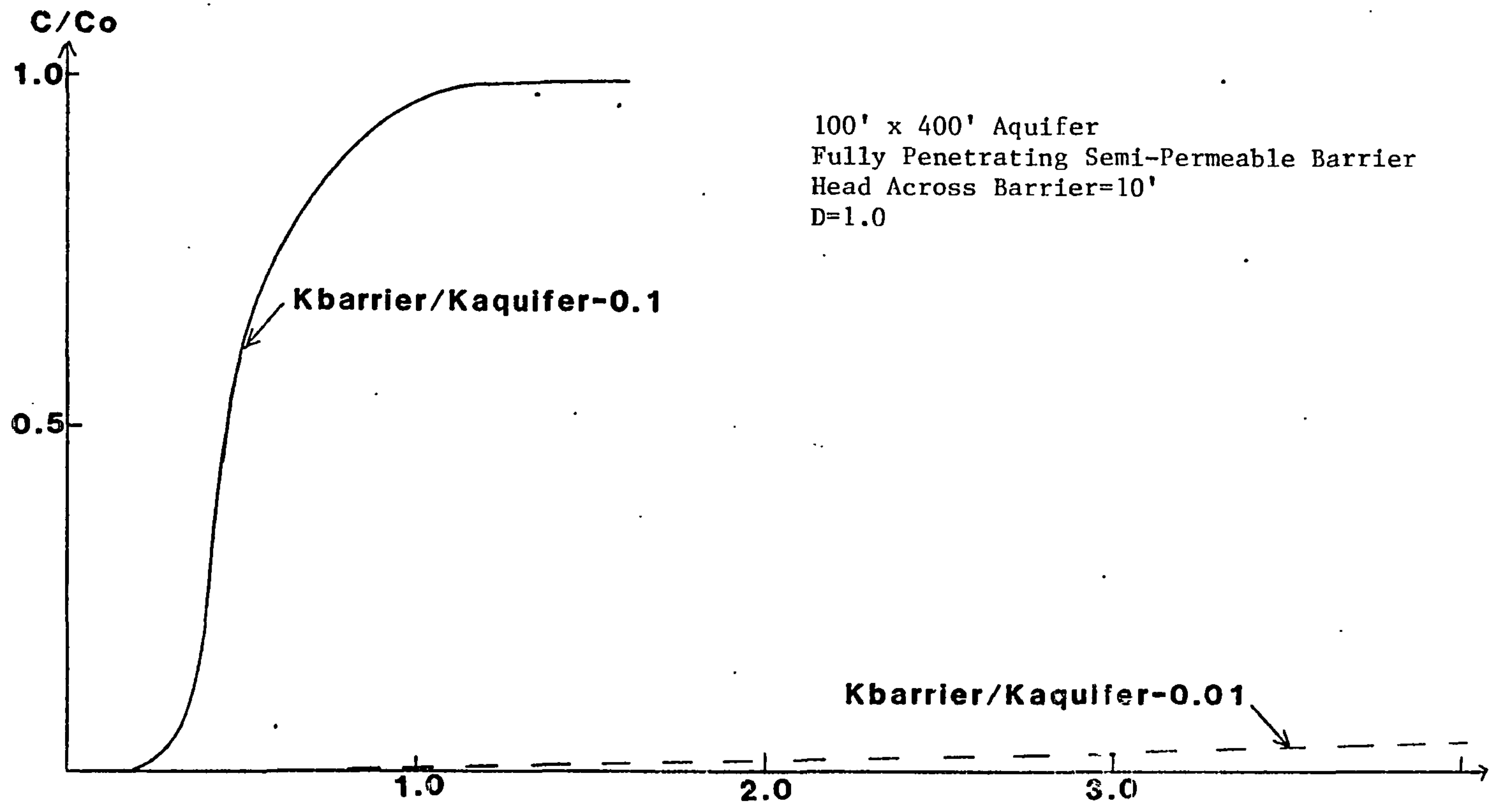


Figure V-25: Pollutant Breakthrough Curves Adjacent to a Semi-Permeable Barrier at 30% Depth.

NB - No Barrier
PPPP - Partially Penetrating, Partially Permeable Barrier
PPFP - Partially Penetrating, Fully Impermeable Barrier
FPPP - Fully Penetrating, Partially Permeable Barrier
IL - Barrier Keyed to Impermeable Layer

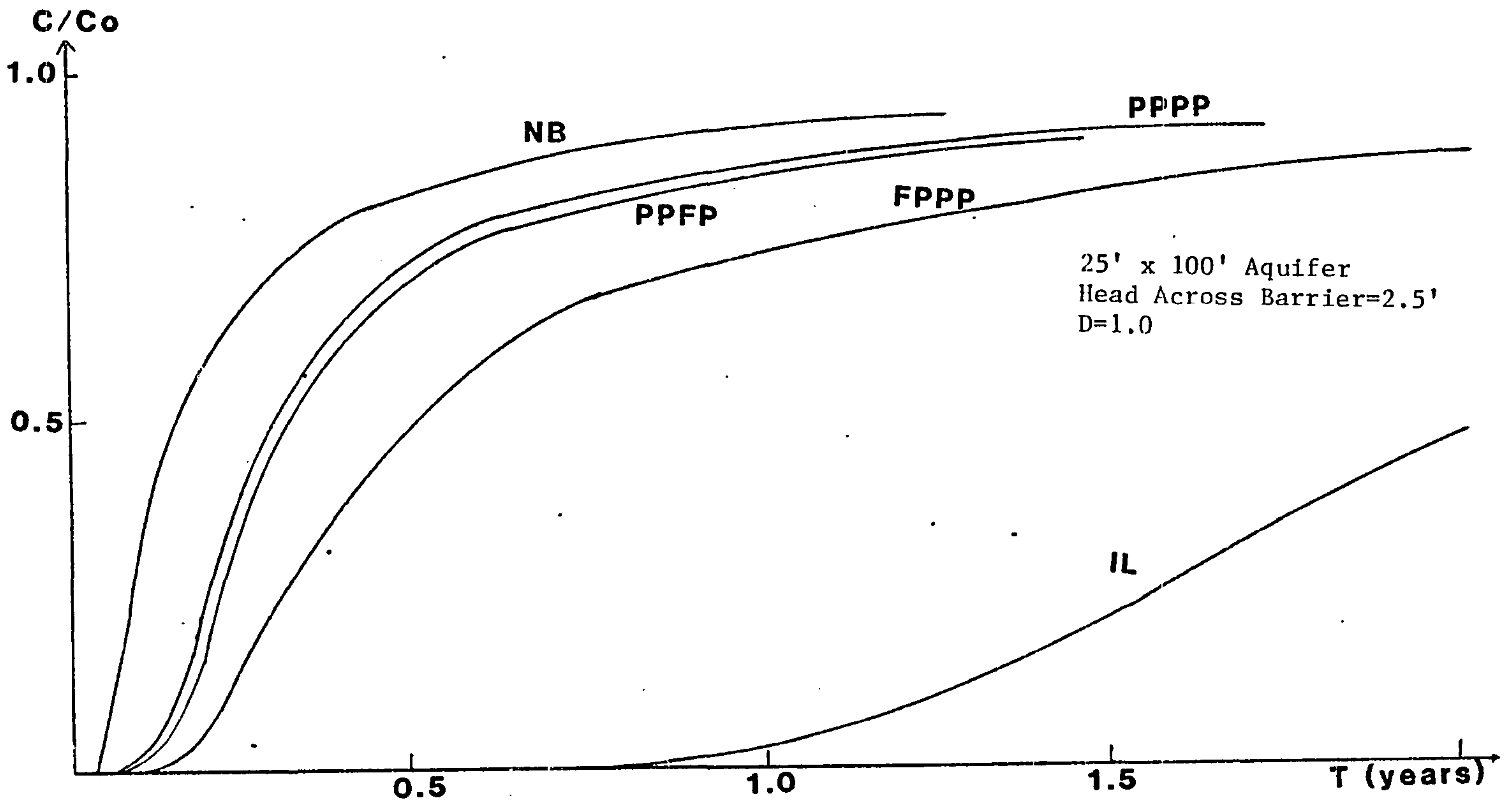


Figure V-26: Summary Plot of the Behavior of Barriers.

Conclusions

As a result of the previous analysis a number of conclusions can be drawn as to the general behavior of barriers. Because the input data was hypothetical, no specifics can be discussed but the trends can be outlined. The conclusions are:

1. The effects of dispersion on the overall behavior of barrier systems is negligible. The general pattern of an initial time lag prior to first appearance of contaminant, followed by a rapid increase in concentration, and concluded with an assymptotic approach to input concentration is followed whether the pollutant is dispersive or not. Dispersive pollutants tend to arrive earlier than non-dispersive, but this factor is not significant when one considers that the time lags are in the order of fractions of years.

2. The ability to key a truly impermeable barrier to an impermeable formation, be it totally or only moderately impermeable, dramatically improves the performance of the barrier. If an adequate key cannot be assured, barriers are a poor (and expensive) choice of ground water pollution control.

3. The placement of a barrier that is not dramatically less permeable than the native aquifer material is ineffective. Moreover, the contaminant tends to first penetrate the upper parts of a semi-permeable barrier and this fact should be considered in the design of monitoring networks for barriers. Monitoring these systems only at full depth may not detect the movement of contaminant out of the upper layers.

Comparison of Models

By imposing a constant head difference across the impermeable barrier (Figure V-14), a constant flow through the opening in the barrier is developed. Since the aquifer is assumed to be at steady state conditions, the flow through the opening in the barrier must be equal to the amount of recharge coming into the aquifer. The recharge rate the aquifer is subjected to is the recharge flow divided by the length of the aquifer. With this known recharge rate it is possible to use the simple analysis and calculate the response time ($t_c = \frac{nD}{\varepsilon}$) for a given aquifer. With a known t_c, the pollutant breakthrough curves can be converted to include an absolute time (t) rather than a dimensionless time parameter (t/t_c). This will allow for a comparison of the results of the simple plug flow model and the sophisticated K-B model.

Plotted in Figure V-27 are the pollutant breakthrough curves for the 100 foot deep, 400 foot wide aquifer. Two of the curves were generated by the K-B model and they reflect the case of no dispersion and mild dispersion. The third curve represents the results generated by the simple plug flow model, but now converted to an absolute time scale by the process outlined above. The curves illustrate that the simple model tends to predict an earlier first arrival and a more gradual concentration increase of pollutant under the barrier. An interesting aspect of Figure V-27 is that the results of both the K-B model and the simple plug flow model are not significantly different. This same trend was found when converting the simple plug flow model results to a comparable basis with the K-B model results for other depths of aquifers. Hence, it is concluded that the simple model not only gives an accurate prediction of the general behavior of these barrier systems, but can also provide fairly comparable actual values relative to the results of the K-B model if the parameter t_c can be calculated.

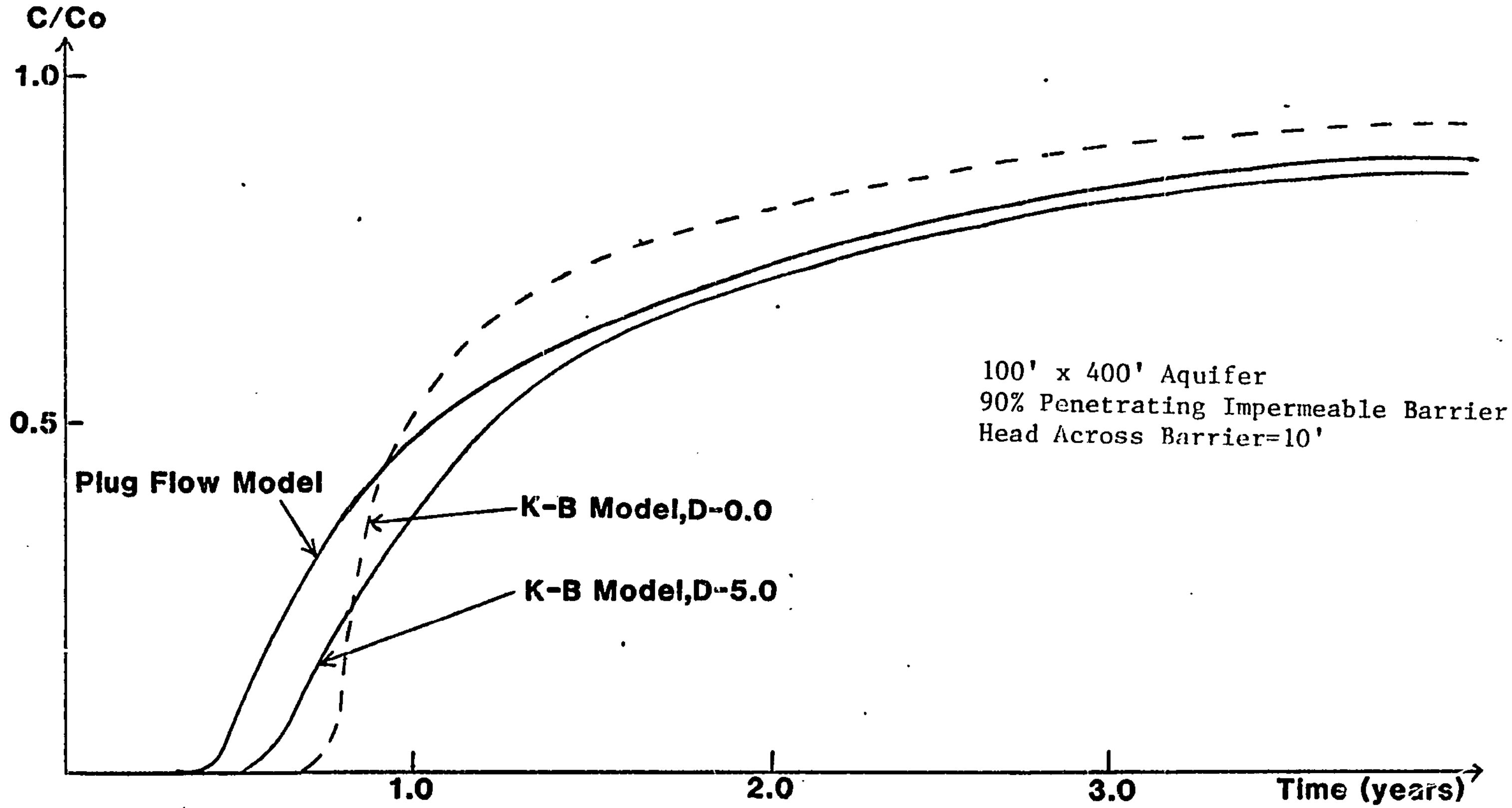


Figure V-27: Comparison of Simple and K-B Models.

Discussion

The previous two analyses seem to point to the general conclusion that partially penetrating barriers are not effective in retarding pollutant migration as compared to the case of no-barrier. However, before a total condemnation of barriers can be made, consideration of a few practical implications should be made.

It should be noted that the overall effect of the partially penetrating barrier is to channel the movement and distribution of contaminant in the aquifer to a specific, localized area. This makes the contaminant amenable to monitoring and/or removal. The no barrier case allows the contaminant to move through the aquifer for its full depth.

It is important to consider the implications of the contaminant moving out under the barrier. Depicted in Figure V-28 are three possible well locations outside the barrier. The first well is located adjacent to the barrier and is screened at the outflow depth of the barrier. Obviously, if this is a fresh water well, it is doomed to rapid failure. However, if this represents a removal or monitoring well, it is situated perfectly. The second well represents a shallow well with a screened interval above the outflow plume. A fresh water well in this position probably will not be affected and could continue to operate as such. However, if this represents a well for monitoring the effectiveness of the barrier, it is totally ineffective. The third well is an intermediate well placed downgradient of the barrier with a screened interval located within the outflow plume. A fresh water well at this location is probably faced with ultimate closure after time,

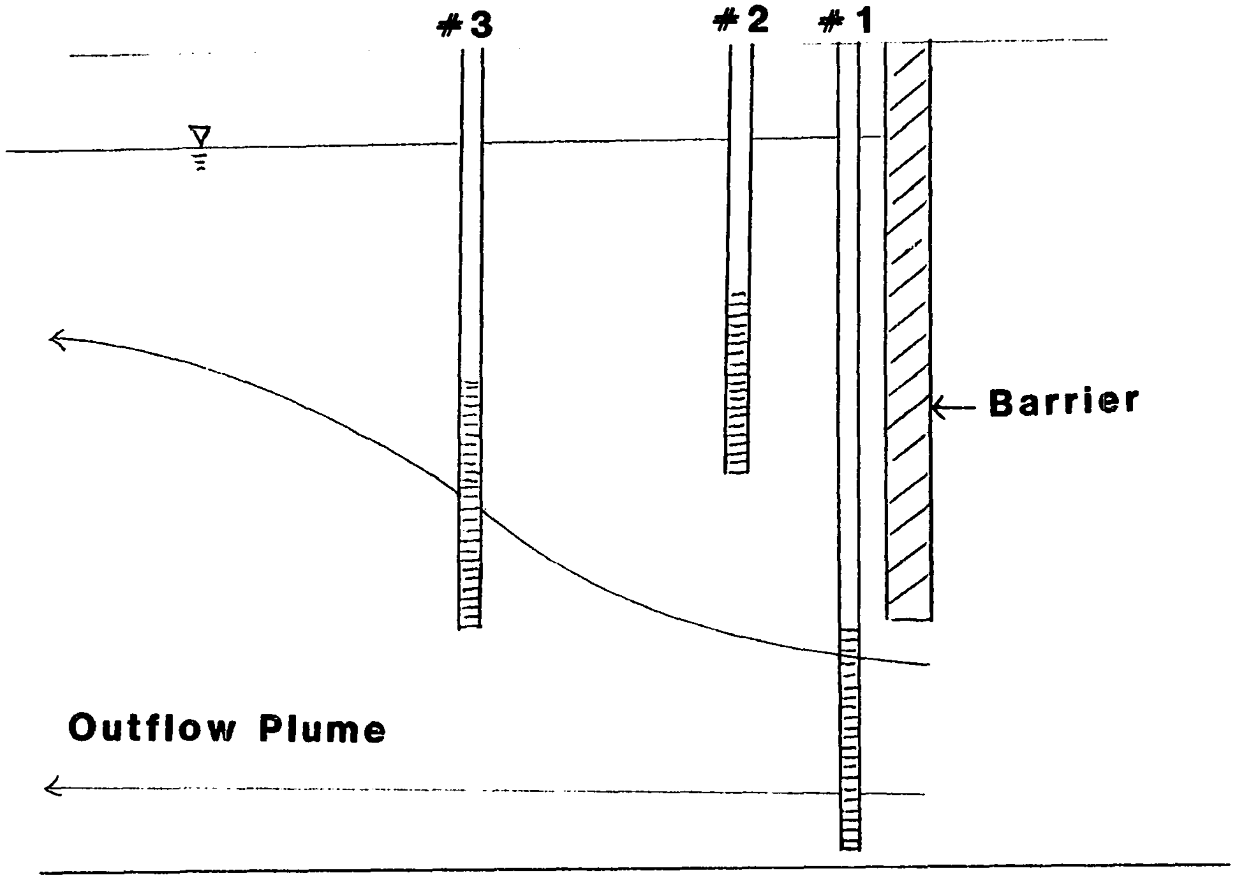


Figure V-28: Well Locations Outside An Imperfect Barrier.

but a monitoring well in this location seems to be well-placed. This is a misleading case in that the concentration of the contaminant reaching this well could be significantly reduced due to dispersion and/or dilution effects. Hence, the monitoring well at this location becomes less desirable and the fresh water well less threatened. The point to be made by this discussion is that the use of barriers for ground water pollution control cannot be condemned based solely on the results of this analysis.

One final point to be made is that the previous analyses were intended to isolate the performance of barriers by themselves. The analyses showed partially-penetrating barriers to be ineffective. However, it must be emphasized that barriers are rarely used by themselves for ground water pollution control. Barriers require some complimentary technologies to be effective, whether they are fully penetrating or not. The implications of these complimentary technologies are offered as a research recommendation in Chapter VI.

CHAPTER VI

SUMMARY AND RECOMMENDATIONS

This chapter contains a summary of the findings of the analysis of the movement of contaminated recharge under and/or through an imperfect impermeable barrier. Also included is a set of recommendations related to the operation of these systems and future research needs.

Summary

One of the most popular methods of retarding ground water pollutant migration is to place an impermeable subsurface barrier in the flow path of the contaminant. These barriers usually take one of three forms: slurry walls, grout curtains or steel sheet-piles. Slurry walls are the most commonly employed barrier technology because of their relative ease of construction and material cost savings. For these barriers to be effective and/or feasible, three criteria must be met. First, the in-place permeability of the barrier must be very small and should remain so over time. Second, an impermeable formation to which the barrier can be keyed must exist at a reasonable depth. Third, an adequate key between the underlying impermeable formation and the barrier must be constructed. To date the literature on barrier technology has been dominated by construction and design information.

Little information exists as to the effectiveness of barriers applied for the purpose of ground water pollution control.

The area of ground water modeling has received considerable attention in recent years, with the emphasis being toward the development of sophisticated numerical models. The development of simplified analytical models has found its greatest application in the field of irrigated agriculture. Only a few studies related to modeling of barriers have been identified from the literature.

This research has focused on the effectiveness of impermeable barriers given that either they are not adequately keyed or they are not predominantly impermeable. The analysis was divided into three phases. The first phase was to develop an analytical solution for the streamline distribution under a partially penetrating barrier. Second, a simplified numerical model was developed and verified by the analytical solution. The simple numerical model was utilized to analyze variations in barrier depth, recharge rates and lengths, and anisotropic soils. Third, a numerical solute transport model was utilized to examine the effects of hydrodynamic dispersion, layered soils and semi-permeable barriers.

The simple numerical model showed that barriers are more effective when placed adjacent to a contaminant source. Additionally, barriers are more effective in soils that are more conductive horizontally than vertically, e.g. stratified soils. The most important result of this analysis was that in the long run a partially penetrating barrier is actually less effective in retarding pollutant migration than no barrier. The effect of partially penetrating barriers is to provide an initial delay in contaminant movement, but then actually accelerate the movement of the contaminant out under the barrier.

As for the simple numerical model itself, it was moderately difficult to develop but extremely easy to program and utilize. The computer program simulating the analytical solution was short and inexpensive to use. The simple model produced results as to the general behavior of partially penetrating barriers in elongated (width/depth ≥ 4) aquifers. This behavior was subsequently verified by the numerical solute transport model. Because the analytical solution was developed in dimensionless terms, its applicability to a variety of problems is increased. This could include models on a laboratory scale.

The simple numerical model was not without its limitations. The model was not directly applicable to the cases of interest which were analyzed by the numerical solute transport model. Additionally, the results produced are really just applicable to the general behavior of these systems. Actual numbers cannot be generated unless one can relate certain input parameters, such as hydraulic conductivity, to the solute response time (t_c).

The numerical solute transport model produced some interesting yet not totally unexpected results. First, the effects of dispersion were found to have a very minor influence on the relative performance of partially penetrating barriers. The first appearance of contaminant under the barrier was accelerated only slightly and the rate of concentration increase was not affected by these barriers whether the pollutant was considered to be dispersive or not. The effects of keying the barrier into a relatively impermeable layer were very favorable in terms of retarding pollutant migration. A fully penetrating barrier was found to be effective only if its permeability was predominantly smaller than that of the adjacent aquifer ($K_{barrier} = 0.01\ K_{aquifer}$).

The numerical solute transport model was quite versatile and capable of handling most any variation imposed on the system. But, these increased capabilities were realized at the expense of increased complexity, computer time, and costs. The model showed sensitivity to input data and had problems handling the high velocity gradients around the opening in the barrier. This was exemplified by numerical dispersion in the results generated. This problem can be overcome by decreasing the grid spacing and the time steps. However, the practicality of these measures must be questioned in light of the fact that the simple numerical model generated acceptable results with much less effort.

One conclusion from the above summary is that unless a barrier can be constructed with an adequate key to an underlying impermeable formation and its impermeability assured over time, the long-term effectiveness of this technology is questionable. A number of variations were imposed on both the aquifer properties and geometry. In each case, the partially penetrating or semi-permeable barriers showed an outflow concentration increase greater than or equal to that of no barrier, after a small initial delay. Barriers are expensive to construct and represent an irretrievable commitment of the flow field in the aquifer. An imperfect barrier could represent a poor choice both economically and environmentally.

Another conclusion is that not every ground water flow problem requires a sophisticated numerical model for analysis. In certain cases, simplifying assumptions can produce a problem amenable to analysis by less complex models. The assumption of plug flow and the development of pollutant breakthrough curves through numerical integration of aquifer streamlines in this analysis, predicted behavior patterns for the aquifer that were reproduced by the sophisticated solute transport model. The results from the simple numerical model were generated with much less effort and lower costs.

Recommendations

Based on this study two general recommendations can be made concerning the operation of a barrier system. The first recommendation is that the barrier be placed as close to a pollution source or the leading edge of the contaminant plume as possible. This research has shown that any time delay advantages gained by placing a barrier downgradient of a contaminant source are due solely to the aquifer geometry and not to the performance of the barrier. Furthermore, if the barrier is constructed in any shape other than a straight line, moving downgradient will increase the size (and subsequently the cost) of the barrier without an increase in efficiency. The second recommendation is that monitoring systems be located throughout the entire depth of the aquifer. The analysis has shown that pollutants that are able to penetrate a partially permeable barrier will do so near the top of the barrier initially and more dramatically. Monitoring at full depth to determine if the connection between the barrier and the underlying impermeable formation is adequate could possibly miss the effects of a partially permeable barrier.

Perhaps the most important outcome of this research is the multitude of future research possibilities generated. The first research recommendation would be to develop laboratory models of the systems described herein. Use of a two-dimensional Hele-Shaw apparatus would be possible. Since the simple numerical model used in this analysis was developed in dimensionless terms, its applicability to laboratory scale models would seem possible and should be verified.

The problem of the permeability of a bentonite barrier increasing upon exposure to certain contaminants is now being promoted as solvable through use of contaminant-resistant bentonites. However, the performance of these new bentonites has only been assessed and published by the bentonite manufacturers themselves. Soil column type studies on the performance of these new bentonites needs to be undertaken by an independent research body.

Another area of needed research is on in-situ measurement of aquifer parameters. In addition, there is also need for methods of measuring the in-place permeability of barriers, and some means of assessing the adequacy of connections made between barriers and underlying impermeable formations.

The use of the numerical solute transport model was based on hypothetical input data. Hence, the validity of any of the calculated values could be questioned. However, the counter argument is that none of the values were out of the range of normally encountered real-life values. Trying to make a case for or against any of the specific values is difficult without having actual data as a basis. It is also difficult to determine compatible input data for hypothetical situations. For example, is it justifiable to use a homogeneous isotropic aquifer with a hydraulic conductivity of X having a dispersivity of Y? It simply cannot be done without field data. This leads to one of the more important research recommendations, and that is that an evaluation of an existing barrier system be undertaken. The results of extensive monitoring could then be used in a truly representative modeling study.

The final recommendation is a broad one that involves specific variations. As this research proceeded the possible variations for study increased dramatically. A partial listing of possible variations that could be analyzed includes:

1. Aquifer Characteristics -- A number of variations in aquifer characteristics omitted from this analysis deserve attention. Variations in the ratio of dispersivities is one example. Analyzing the effects of a more realistic recharge scenario would involve representing the recharge boundary as a mounding situation. Transient flow in the aquifer may be more typical of actual field situations. The inclusion of retardation or decay factors for non-conservative pollutants represents an interesting case. All of these variations would most probably require field data and some would require a more sophisticated version of the solute transport model.

2. Barrier Characteristics -- The behavior of the barrier over time represents an interesting case. This analysis included the effects of a semi-permeable barrier that remained constant over time. The more representative case would be that of a barrier that increases in permeability only after saturation by the contaminant.

3. Complementary Technologies -- This analysis was aimed at assessing the effectiveness of barriers alone. In real life applications, barriers usually have some complementary technology such as removal wells or surface capping. The inclusion of wells increases the number of interesting variations almost exponentially. Wells could be used in a pumping mode inside the barrier for removal and treatment of the contaminant, or they could be used as freshwater injection wells outside the barrier to further inhibit pollutant migration. The number, spacing, pumping rates, depth, etc., of the wells represents an interesting study with almost endless variations. An optimization study could be initiated to determine the appropriate combinations of influencing factors.

Some final remarks on the intent of this study are appropriate. This analysis and the results are not meant as an attack on subsurface impermeable barriers. It should be noted that this study was designated as a preliminary assessment and as such, the findings should be taken as a first step to a better understanding of this technology. The study has shown that under certain circumstances, subsurface barriers are ineffective in retarding pollutant migration. This does not mean that these problems will exist with every application of barrier technology. The most important recommendation is that further studies be undertaken to better understand barrier behavior. Barrier technology is being used extensively in an attempt to solve one of the most pressing issues of the day -- ground water pollution. The value of natural ground water resources is immeasurable, and efforts should be made to minimize mistakes in efforts to preserve and protect these measures.

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APPENDIX I

SOURCE PROGRAM FOR ANALYTICAL SOLUTION

```
C A PROGRAM TO CALCULATE THE DIMENSIONLESS
C STREAMLINE DISTRIBUTION AS PREDICTED BY ANALYTICAL SOLUTION
      DIMENSION X(41),Y(11),PSI(11,41)
      B=0.5
      SD=4.
      X(1)=0.
      DO 13 J=1,41
      X(J+1)=X(J)+.025
      Y(1)=0.
   13 CONTINUE
      DO 14 I=1,11
      Y(I+1)=Y(I)+.1
   14 CONTINUE
      I=1
    8 J=1
    9 N=0
      SUM=0.
   19 N=N+1
      VARB=(N*3.14159*B)
      SNE=SIN(VARB)
      VARY=(N*3.14159*Y(I))
      SN1=SIN(VARY)
      VARX=(N*3.14159*SD*(1-X(J)))
      SNCH1=(EXP(VARX)-(1/EXP(VARX)))/2
      VARN=(N*3.14159*SD)
      SNCH2=(EXP(VARN)-(1/EXP(VARN)))/2
      IF(X(J).EQ.0)GO TO 40
      S=(SNE*SN1*SNCH1)/((SNCH2)*(N**2))
      GO TO 12
   40 S=(SNE*SN1)/(N**2)
   12 SUM=SUM+S
      IF(N.GE.10)GO TO 18
      GO TO 19
   18 P=1.-Y(I)+(X(J)*Y(I))
      PSI(I,J)=P-(2/((3.14159**2)*B))*SUM
  108 FORMAT(2X,F10.4,2X,F10.4,2X,F10.4)
      J=J+1
      IF(J.LE.41)GO TO 9
      I=I+1
      IF(I.LE.11)GO TO 8
  111 YDY=.1
      PS=0.05
   93 WRITE(6,910)PS
      J=1
   60 I=11
   77 IF(PSI(I-1,J).GE.PS)GO TO 81
      I=I-1
      GO TO 77
```

```
 81 IF(PSI(I,J).GT.PS)GO TO 50
    TY=((YDY)*(PS-PSI(I,J)))/(PSI(I-1,J)-PSI(I,J))
    AI=1
    YC=(AI-1.)/10-TY
    AJ=J
    XC=(AJ-1.)/40
910 FORMAT(1X,'COORDINATES FOR PSI =',F10.4)
    WRITE(6,911)XC,YC
911 FORMAT(1X,'X =',F10.4,'Y =',F10.4)
 50 J=J+1
    IF(J.LT.41)GO TO 60
    PS=PS+.05
    IF(PS.LT.1.00)GO TO 93
    CONTINUE
    STOP
    END
```

APPENDIX II

SOURCE PROGRAM FOR SIMPLE NUMERICAL MODEL

```
C A PROGRAM TO CALCULATE THE DIMENSIONLESS STREAMLINE
C DISTRIBUTION UNDER AN IMPERMEABLE BARRIER
        DIMENSION PSI(21,81),TEM(81),TEMY(81),VX(21,81),VY(21,81),
       2VT(21,81),C(21,81),DL(21,81),CC(21,81),CN(21,81),CP(21,81),
       2DT(21,81),DXX(21,81),DYY(21,81),DXY(21,81)
        REAL KX,KY,K
        DY=.05**2
        DX=.0125**2
C NPC = NUMBER OF COLUMNS IN GRID
C NPR = NUMBER OF ROWS IN GRID
        NPC=81
        NPR=21
C KX,KY = CONDUCTIVITIES IN X AND Y DIRECTIONS
        KX=1
        KY=1
C TIME STEP
        DTS=.001
C DS = RATIO OF DEPTH TO LENGTH
        DS=.25
C K = ASPECT RATIO
        K=(KX/KY)*(DS**2)
        XK=0.
        TK=1./(2*((K*DY)+DX))
        XTK=1./(2*((KX*DY)+DX))
        NC=NPC-1
        NR=NPR-1
        XNR=NR*1.
C ND = DEPTH OF BARRIER
        ND=1
        XND=(ND-1)*1.0
        XNX=(XND/XNR)*100.
        NB=NPR-ND
        NDD=NB+1
C NP = LENGTH NOT SUBJECTED TO RECHARGE
        NP=41
        NCC=NP+1
        XNP=NP
        XNCC=NPC-NP
        XNB=NB
        ITER=0
C ASSIGN INITIAL STREAM VALUES TO NODES AND
C ASSIGN BOUNDARY CONDITIONS
        WRITE(6,157)
    157 FORMAT(1X,'THIS ANALYSIS UTILIZES THE FOLLOWING')
        WRITE(6,158)XNX,NP,K,AT
    158 FORMAT(1X,'PENETRATION OF BARRIER =',1X,F7,3,1X,'%',
       1/1X,'RECHARGE OVER ALL BUT',1X,I2,1X,'NODES',/1X,
       1'ASPECT RATIO =',F7.2)
```

```
      DO 10 I=1,NPR
      PSI(I,NPC)=1.0
   10 CONTINUE
      DO 11 I-1,NB
      AI=I
      PSI(I,1)=1.-((AI-1.)/XNB)
   11 CONTINUE
      IF(ND.EQ.0)GO TO 12
      DO 12 I=NDD,NPR
      PSI(I,1)=0.
   12 CONTINUE
      IF(NP.EQ.0)GO TO 18
      DO 18 J=1,NP
      PSI(NPR,J)=0
      PSI(1,J)=1.
   18 CONTINUE
      DO 27 J=1,NPC
      PSI(1,J)=1.
   27 CONTINUE
      DO 13 J=NCC,NPC
      AJ=J
      PSI(NPR,J)=((AJ-XNP)/XNCC)
      PSI(1,J)=1.
   13 CONTINUE
      DO 14 I=2,NR
      DO 15 J=2,NC
      PSI(I,J)=1.0
   15 CONTINUE
   14 CONTINUE
    1 RESID=0.
      ITER=ITER+1
C RELAX INTERIOR NODES BY COLUMN THEN BY ROW
      DO 16 I=2,NR
      DO 17 J=2,NC
      IF (XK.LE.0.0)GO TO 55
      IF (I.GT.NDD)GO TO 55
      K=XK
      TK=XTK
   55 TEMP=(TK*((K*DY)*(PSI(I,J+1)+PSI(I,J-1))))+(TK*((DX)*(PSI(I+1,J)
     2+PSI(I-1,J))))
    9 TEMPX=PSI(I,J)+(1.5*(TEMP-PSI(I,J)))
      RESID=RESID+ABS(TEMP-PSI(I,J))
      PSI(I,J)=TEMPX
   17 CONTINUE
   16 CONTINUE
      DO 33 J=2,NC
      DO 34 I-2,NR
      TEMP=(TK*((K*DY)*(PSI(I,J+1)+PSI(I,J-1))))+(TK*((DX)*(PSI(I+1,J)
     2+PSI(I-1,J))))
      RESID=RESID+ABS(TEMP-PSI(I,J))
      TEMPX=PSI(I,J)+1.5*(TEMP-PSI(I,J))
```

```
      PSI(I,J)=TEMPX
   34 CONTINUE
   33 CONTINUE
      IF(RESID.LE.TOL)GO TO 4
      IF(ITER.LT.30)GO TO 1
  780 FORMAT(11F7.3)
    4 CONTINUE
C CALCULATE LINES OF CONSTANT PSI AND
C THE AREA ABOVE EACH CURVE
      YDY=.05
      PS=0.05000
   93 AT=0.0
      M=0
      J=1
   60 I=NPR
   77 IF(PSI(I-1,J).GE.PS)GO TO 81
      I=I-1
      GO TO 77
   81 XP=(PS-PSI(I,J))
      IF(XP.LT.-.0001)GO TO 50
   51 M=M+1
      Y=((YDY)*(PS-PSI(I,J)))/(PSI(I-1,J)-PSI(I,J))
      AI=I
      YC=(AI-1.)/NR-Y
      TEMY(M)=1-YC
      AJ=J
      XC=(AJ-1.)/NC
      TEMX(M)=XC
      IF(M.EQ.1)GO TO 47
      DA=((TEMX(M)-TEMX(M-1))/2)*(TEMY(M-1)+TEMY(M))
      GO TO 48
   47 DA=XC*(1-YC)
   48 AT=AT+DA
      WRITE(6,913)XC,YD,PS
  913 FORMAT(1X,'X= ',F7.3,1X,'Y= ',F7.3,1X,'FOR PSI= ',F7.3)
C WRITE OUT PSI VS AREA RELATIONSHIPS
  912 FORMAT(1X,'FOSPSI =',F10.2,1X,'THE AREA =',F10.5)
   50 J=J+1
      IF(J.LT.NPC)GO TO 60
      WRITE(6,912)PS,AT
      PS=PS+0.05000
      IF(PS.LT.1.000)GO TO 93
      CONTINUE
   58 XDX=.0125
      YDY=.05
      C(NPR,NPC)=.5
C CALCULATE VELOCITY DISTRIBUTIONS
      DO 82 I=1,NB
      VX(I,1)=(PSI(I,1)-PSI(I+1,1))/YDY
      VY(I,1)=0.
```

```
   82 CONTINUE
      DO 83 I-NDD,NPR
      VX(I,1)=0.
      VY(I,1)=(PSI(I,2)-PSI(I,1))/XDX
   83 CONTINUE
      DO 84 J=2,NC
      VY(NPR,J)=(PSI(NPR,J+1)-PSI(NPR,J-1))/(2*XDX)
      VX(NPR,J)=(PSI(NR,J)-PSI(NPR,J))/YDY
      VY(NPR,NPC)=(PSI(NPR,NPC)-PSI(NPR,NC))/XDX
      VX(NPR,NPC)=(PSI(NR,NPC)-PSI(NPR,NPC))/YDY
      VY(1,J)=0.
      VX(1,NPC)=(PSI(1,NPC)-PSI(2,NPC))/YDY
      VX(1,J)=(PSI(1,J)-PSI(2,J))/YDY
      VY(1,NPC)=0.
   84 CONTINUE
      DO 85 I=2,NR
      VX(I,NPC)=0.
      VY(I,NPC)=(PSI(I,NPC)-PSI(I,NC))/XDX
   85 CONTINUE
      DO 86 I=2,NR
      DO 87 J=2,NC
      VY(I,J)=(PSI(I,J+1)-PSI(I,J-1))/(2*XDX)
      VX(I,J)=(PSI(I-1,J)-PSI(I+1,J))/(2*YDY)
   87 CONTINUE
   86 CONTINUE
      DO 701 I=1,NPR
      DO 702 J=1,NPC
      TVX=4*(VX(I,J))
      TVY=VY(I,J)
      VX(I,J)=KX*TVX
      VY(I,J)=KY*TVY
  702 CONTINUE
  701 CONTINUE
C WRITE OUT VELOCITY DISTRIBUTIONS
      DO 708 J=1,NPC,2
      WRITE(6,709)(VX(I,J) ,I=1,NPR,2)
  708 CONTINUE
      DO 710 J=1,NPC,2
      WRITE(6,709)(VY(I,J) ,I=1,NPR,2)
  710 CONTINUE
  709 FORMAT(1X,11F7.3)
      STOP
      END
```