USE OF A FINITE DIFFERENCE COMPUTER MODEL TO ESTIMATE 
DEWATERING NEEDS AND EFFECTS OF A PROPOSED UNDERGROUND, 
URANIUM MINE, GAS HILLS, WYOMING 

by 
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A Thesis Submitted to the Faculty of the 
DEPARTMENT OF HYDROLOGY AND WATER RESOURCES 
In Partial Fulfillment of the Requirements 
For the Degree of 
MASTER OF SCIENCE 
WITH A MAJOR IN HYDROLOGY 

In the Graduate College 
THE UNIVERSITY OF ARIZONA 

1978
STATEMENT BY AUTHOR

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ACKNOWLEDGMENTS

I would like to thank my thesis chairman, Dr. John W. Harshbarger, for his time and suggestions in the preparation of this thesis. I would also like to thank the members of my committee, Drs. Eugene S. Simpson and L. Graham Wilson for their assistance. Much of the test data contained within were obtained from the Tennessee Valley Authority. Mr. William M. McMaster provided moral support and helped to obtain permission to use data from the pumping tests. Mr. Benjamin K. Bryan provided suggestions on making the model more efficient and helped greatly in debugging the program. Mr. James D. Sutton did most of the illustrations, and his care is greatly acknowledged. The typing was done by Mrs. Roxine Kalvels and her diligence and hard work are greatly appreciated. Lastly, to all those people, especially my wife Jo Ellen, who kept the pressure on me to finish this work, I owe an eternal favor.
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ABSTRACT

The increased demand for energy related minerals has caused renewed interest in mining. Many of the ore deposits are below or within aquifers. To retrieve this ore, the water must be removed from the mine. Ascertaining the need for dewatering/depressuring requires the use of elementary and complex procedures. Digital computer models have been used to simulate aquifer systems and can be used to test dewatering/depressuring schemes if certain precautions are taken. A digital model using a program developed by T. A. Prickett and C. G. Lonnquist has been developed in the Gas Hills, Wyoming Uranium Mining District to test a proposed dewatering/depressuring scheme for an underground uranium mine. Results of this test indicate that digital models can be used to predict dewatering/depressuring scheme effects, but continued monitoring of the piezometric surface in the area of the mine once dewatering/depressuring commences is needed in order to validate the predictions of the model.
CHAPTER 1

INTRODUCTION

The energy and mineral shortage of the last few years and associated increased prices for minerals and energy related materials have made the mining industry more aware of deposits that were once abandoned or passed as being uneconomical and/or technologically infeasible. Uranium reserve cutoffs in the 1960's were in the tenths of percent of $U_3O_8$ where some companies presently use a cutoff grade (the lowest point at which ore reserves are calculated) of as low as .05 percent $U_3O_8$. Advances in technology in the past 10 years and the economic incentive of higher prices are making mines once abandoned viable for production at present and in the future. Similarly other deposits are becoming more economical to mine.

Many of these deposits lie below or are contained in aquifers. These conditions in most cases necessitate the removal of the water so that mining might begin or continue. The mining industry has recognized the need for trained experts to deal with this problem (Fox, 1971). However, often in the past a geohydrologist has not been called upon or his advice has not been heeded or followed. At the Lisbon Shaft in the Gas Hills, Wyoming the shaft and mine were lost to flooding because a drift was driven to one of the peripheral dewatering wells, resulting in the collapse of the drift and shaft from fluidization.¹ Such an

1. See Appendix A for definitions of mining terms.
action would probably not have been recommended by a knowledgeable geohydrologist. Much of the technology developed in geohydrology has been slanted toward water production, and much of today's literature deals with water procurement and not water control. Although the technology does exist to aid mining engineers in assessing dewatering needs, it is often overlooked or considered an insignificant item in mine planning.

Recent years have seen the creation and incorporation into modern geohydrology of the digital computer. This device has seen some use as applied to mine dewatering problems although digital models are capable of aiding in the prediction and assessment of such needs as optimal pumping patterns and changes in piezometric surfaces with time (Mount, 1968). If mining engineers and geohydrologists had the capability of using a computer model for rough predictions, a great deal of time and money could possibly be saved. The use of a model in such a manner is illustrated in this paper.
CHAPTER 2

THE NEED FOR DEWATERING AND METHODS FOR ACHIEVEMENT

The Concept of Depressuring/Dewatering

Henry Darcy (1856) discovered that the flow rate \( Q \) through horizontally stratified beds of sand was proportional to the energy loss or head \( h \) and inversely proportional to the length of the flow path \( l \). The concept of depressuring or dewatering employs this principle.

The term dewatering has been used for years in the mining industry to describe the process of water removal from the mine. In an artesian system, there are two parts to this process. The first part is the reduction in hydraulic head or piezometric surface over the mine itself. This is referred to as depressuring. Once the head has been reduced to a level at which the piezometric surface is at or below the top of the aquifer, the system is under non-artesian or water table conditions at those places in and around the mine area where such a condition exists. In this second part of the dewatering process, actual removal of water from aquifer storage is accomplished so that the aquifer in the area of the mine will approach being dewatered if pumped for a long enough time.

It has been noted that mines in areas of water table or near water table aquifers yield less water than those mines in artesian aquifers. For example, Gulf Minerals' shaft at Mariano Lake, New Mexico
during its construction produced a reported 100 GPM from the Poison Canyon Member of the Morrison Formation. This shaft is located near the southern boundary of the formation and near its recharge area. Another mine in the same formation at Church Rock, New Mexico approximately 14 miles west reportedly produces 4000 to 5000 GPM (Patterson, Stoneman, and Trauger, 1976). The mine at Church Rock is under greater head than the Mariano Lake operation.

The concept behind the depressuring/dewatering technique is to reduce the artesian head over the mine workings to a point at which all flow into the mine is the result of gravity flow (water table conditions). Flow into the mine workings under water table conditions is less than under artesian conditions because of the reduction of saturated thickness and thus the transmissivity (Johnson Drillers Journal, 1977). In addition there is less hydraulic head, and the gradient of the hydraulic head is more gentle.

If a mine exists in an aquifer under water table conditions, there is no need to depressure as flow can be usually handled in the shaft or mine workings and pumped to the surface unless the mine exists in an area of extremely high transmissivity.

Mine geometry can affect the performance of a dewatering system. If drifts are driven in opposite directions simultaneously, more water inflow can be expected than if only one were driven. Distortion of the cone of depression along the axis of the drift will also occur. If drifts are driven individually, each drift will help to dewater the mine area so that the head over each succeeding drift is reduced.
Experience at a central Wyoming uranium mine has shown that a good depressuring/dewatering program coupled with effective grouting can reduce expected flows into a shaft by a significant amount. Based on yield of test wells (about 70 GPM) it was estimated that a non-depressured/dewatered shaft would yield about 100 GPM when it had penetrated a sandstone aquifer in the Powder River Basin. A series of dewatering wells located around the shaft and along the ore trend was constructed. All wells were surface wells, i.e., the casing is exposed at the surface instead of being contained in the mine workings. A drawdown of approximately 175 feet at the shaft site was required to depressure the aquifer at that point. Total pumpage for the area was an average of about 200 GPM for depressuring and began in the winter of 1975. By spring of 1977, a drawdown of about 175 feet had been attained at the shaft site. Upon entry into the aquifer, the shaft yielded about 25 GPM or 25 percent of the estimated non-depressured/dewatered flow. In January 1978 the surface wells were shut down, and water inflow was controlled in the shaft. Discharge from the shaft averaged 130 GPM after the wells were shut down.

Reasons for Dewatering

A question that might arise is why dewater in the first place. Perhaps it would be easier to just mine and let the water take care of itself. Unfortunately this approach has too often been taken, most times with disastrous results.

One reason for dewatering is that unsaturated rock is many times stronger than saturated rock because of the lack of hydrostatic
pressure and the lubricating action of water. Estimates of enhanced rock strength of as much as 2 to 3 times have been made with a resulting savings in shaft sinking time and/or mining costs. In addition, a stronger rock means a safer mine because there is less chance of cave-in.

Another advantage of dewatering is there would be a lesser possibility of fluidization of sandstone. Large artesian heads can cause the bottom of a shaft to boil up or heave. This condition is also associated with artesian burst. Under such conditions the floor may have a quicksand condition. In this case, not only is water pumped but also sand, resulting in deterioration of pumps and loss of efficiency.

A "dry" mine is safer than one that is wet because there is less likelihood of flooding, better traction, and greater rock strength. Mining operations are easier because machinery has better traction. Haulageways will be clearer and more easily traveled in a "dry" mine (Loofbourow, 1973). In addition to a more efficient operation, equipment maintenance, down time to clear equipment of plugs, and fuel consumption can be decreased.

Water derived from dewatering can be used for many purposes. When properly treated, it can be used for the domestic supply for the mine. It can be used in mine operations and for process water if it does not contain constituents that would interfere with milling operations. The water can be used for recreation or for irrigation if of suitable quality. It can provide a mitigating measure for water supplies that are disturbed or destroyed by mining functions by
providing replacement water for those supplies lost as a result of dewatering, destruction of wells by mine workings, and the lowering of pumping levels. Mine water can also be recycled to extract minerals in the water such as uranium or copper. Recovery of these metals and minerals would increase overall production.

Economics on a whole play an important role in any mining operation. The object of mining is to extract minerals at the lowest possible cost. Many shaft sinkers will receive a fixed amount of money per day plus an additional charge for any problems that were not previously agreed upon. If large quantities of water are encountered, an operation may be shut down for some time as Gulf Minerals' operation at San Mateo, New Mexico. If a contractor's operating cost is $20,000 per day for example, he will charge that amount plus pumping costs even though there may be no additional shaft sinking. One shaft sinking contractor, Harrison Western, Inc. of Denver, Colorado estimated that it costs an average of 3c for every gallon of water pumped from the shaft in delays, equipment, and energy to remove the water (Harrison Western, Inc., personal communication, 1976). Appendix B illustrates how a dewatering program that reduces the inflow into a shaft by 75% can reduce construction costs.

Methods of Dewatering or Water Control

Four basic questions should be answered before a dewatering program is begun: (1) Is dewatering feasible?; (2) Is it the best preparatory step?; (3) What is the best method to use and then how should it be implemented?; and (4) How long will it take, and how much
Another important question that might be asked is what is (are) the environmental impact(s) of dewatering, and can the impact(s) be economically and technologically mitigated?

One course of action in a dewatering program is to take no action and handle the water as it flows into the mine or shaft. This method can lead to disastrous results ranging from shaft sinking or mine development delays to loss of life and the shaft or mine itself.

One method of controlling water inflow is to reduce the permeability of the aquifer by injecting cement grout under pressure ahead of the shaft. A long hole is drilled in advance of the bottom of the shaft some 30–40 feet until water is encountered. Then other holes are drilled in the bottom of the shaft, and the cement grout is pumped into them until the flow of water from the long hole is reduced to an acceptable flow. The shaft is advanced along with the long hole with grouting continuing as needed until the planned depth of the shaft is reached. This operation can also be performed from the surface. One danger of cement grouting is if the grout is injected under too great a pressure, it can actually increase the hydraulic conductivity through "fracturing", and the grout can fail to seal off these areas because it has been diluted by the ground water. These additional fractures can actually increase the flow of water into the shaft if they are intercepted.

One method used to stop flow into the shaft and provide additional ground support is freezing. This method is particularly good in areas prone to caving or that are known to have loose ground or
rock. It has been used in mines in Saskatchewan and the Gas Hills, Wyoming. Briefly the process consists of injecting chilled, saline water, whose freezing point is below that of the ground water, down holes that are bored around the circumference of a circle outside the periphery of the proposed shaft. Pairs of concentric pipes transport the solution down and then back to the surface. The holes are spaced at 2-4 foot intervals. The result is that the water in the ground in the vicinity of these holes becomes frozen. This method is very useful in areas of highly unstable ground (Peele, 1941). The reduction in the price of liquid nitrogen in recent months has added new interest to using it in the freezing process. The problem with this method is that the ground must be continually frozen in the area of the shaft because water under predevelopment head would enter the mine until the shaft could be cement lined if the ground were not frozen. Environmentally, this method is perhaps one of the best because water levels are minimally disturbed.

One of the most common methods in use to dewater prior to shaft sinking is the use of surface wells to relieve the artesian head. Wells have also been used to reduce the head above the mine workings, such as at the United Nuclear Corporation mine at Morton Ranch, Wyoming, although such a method is considered to be somewhat inefficient in many cases. Dewatering wells differ from normal production wells in being arranged and constructed or designed to maximize mutual interference. With an appropriate array, the wells can reduce the artesian head near the shaft or mine site so that the area around the shaft or mine is under non-artesian (water table) conditions. Once the shaft or mine is
under non-artesian conditions, the flow of water into the mine or shaft is much less than under artesian conditions, and it usually can be handled by sump pumps in the shaft. Upon completion of the shaft and development of stations and installation of the main pumps, the shaft can be used as one "big well", and the dewatering wells can be cut off or their pumping rate reduced. Because the area around the shaft is usually dewatered or depressured by the time of station development, the aquifer near the shaft is usually under water table conditions; and wells near the shaft contribute little to dewatering in future mine development. Vent holes along ore trends can also be used as dewatering wells to depressure haulageways or drifts before they are driven.

One common practice in shaft sinking is the use of a water ring (as shown in Figure 1). A water ring consists of a small indentation or groove in the side of the shaft. This indentation or groove intercepts the falling water which is then led to a sump and then pumped to the surface. A splash board is used to insure that most of the water flows into the ring. The groove is about 2 feet deep with a dam of timber embedded with concrete or clay to form the channel behind which the water collects. Several of these rings may be placed in a shaft. These devices are used when complete waterproofing is too costly or to save on vertical lift in deep shafts (Peele, 1941).

In conjunction with a water ring and sometimes alone, several sump pumps may be located in the bottom of the shaft to control flow into the shaft itself. These pumps are located in a low spot of the shaft and pump water to the surface or to another set of pumps higher
Figure 1. Water Ring. -- (Stevens, 1973)
in the shaft. This is the most common method of controlling water in the shaft itself.

During drift or haulageway development, surface wells can be used to depressure the rock into which the haulageway or drift is being constructed. However, this is somewhat inefficient unless there is a time restraint. Perhaps the best method is the use of long holes. Long holes are drilled in drift headings for two purposes: (1) exploration and (2) dewatering. By using these long holes, the drift itself acts as a well, and the effective radius of the "well" is increased. Long holes are drilled some tens of feet into the top, sides, and bottom of a haulageway or drift. Many times these long holes will intercept saturated rock and aid in dewatering. The flow is commonly directed down the haulageway or drift to the shaft sump and pumped to the surface.

Determining Dewatering Needs and Effects

Before a mining operation is initiated, the need for and requirements for depressuring or dewatering should be ascertained.

One method is a guess based upon experience. This experience can come from working in rocks of similar type or environment or from the history of mines in the same geographical location or rock type. While this method is better than a pure guess, it leaves much to be desired because the hydrologic conditions may be different in just a few hundred feet in some areas such as fracture systems or intrusives. A commonly used rule-of-thumb in the mining industry is to provide one
gallon per minute pumping for each foot of shaft. Obviously onsite
information of the hydrologic characteristics is very important.

In some cases laboratory tests on cores can be used to calculate
the hydraulic conductivity and porosity of such rocks as sandstones.
The problem here is that a core sample is usually disturbed and likely
will not indicate the true characteristics of the rock. The core also
represents only a small sample of the entire aquifer system, and the
results obtained from its analysis may be anomalous and not true of the
aquifer as a whole. Geophysical logs such as resistivity and self or
spontaneous potential have also been used to estimate characteristics
such as hydraulic conductivity or permeability, porosity, and water
quality. Qualitative information is usually obtained by these logs and
cannot be used to accurately determine such hydrologic parameters as
transmissivity and storage coefficient.

A commonly used method for determining dewatering needs is the
use of a drill stem test (see Figure 2). While this test is not
considered the best, its low cost and fairly reliable results make it a
possible choice when the exploration budget is small. Briefly a drill
stem test consists of the lowering of a special tool on the end of a
drill pipe down the well. The tool is usually placed between 2 packers,
which isolate the aquifer of interest from the rest of the hole. A
pressure gauge records the pressure changes during the test. Pressures
are recorded as the tool is lowered, as the test section is being shut-
in, and as the tool is removed. By allowing the fluid to flow from the
ground through the tool and recovering a sample; potential production
Figure 2. Schematic Illustration of Drill Stem Test
of the aquifer can be calculated using the rate of flow and the bottom hole pressures (Loofbourow, 1973; Hackbarth, 1978).

The most common test and perhaps the best is the aquifer or pump test. These tests are performed using water wells to obtain data from which hydrologic properties such as transmissivity and storage coefficient can be calculated. One type of pump test consists of pumping a well in a series of increasing rates and measuring the resulting drawdown in the pumped well. This test gives an idea of the yield of the well, and a rough estimate of how much water the shaft might produce if not depressured or dewatered can be made employing a rule-of-thumb that states that doubling the diameter of a well will increase its yield by 7% for artesian wells (UOP Johnson, 1972).

Another test, constant discharge, can be used to calculate hydrologic characteristics of the aquifer. It is common in this test to measure the change in head or piezometric surface (drawdown) in a number of observation wells (piezometers) that have been placed in the area of interest. If a record of the water levels in these piezometers and the pumped well at particular times is known, it is possible to depict the shape of the resulting cone of depression and thus an idea of how much head has been reduced or should be reduced to depressure the area of interest.

Once a pump test has been concluded and the changes in head with respect to time after pumping began are known, the data can be used to calculate values of transmissivity and storage coefficient. When these values are known and if a few assumptions are made concerning the aquifer system, estimates of drawdown at any point in time after
pumping starts at any radial distance from the pumped well can be calculated for a given discharge using analytical means. In addition, estimates of leakage from overlying aquifers possibly can be made. Thus knowing the distance from the well(s) to the shaft site, it is possible to predict the drawdown at various times at that site for a given pumping rate. It is also possible to study if more than one well will be needed to effect depressuring or dewatering.

Because analytical solutions require assumptions and simplifications, e.g., aquifers which are homogeneous, isotropic, and infinite in areal extent, the use of other types of models to simulate the actual conditions has become popular in recent years. One of the earliest models was the analog model. By using properties of electricity (capacitance for storage, resistance for transmissivity) and measuring the voltage drops (changes in head), predictions could be made on the response of an aquifer system. However, the construction of such models is tedious and very time consuming. In addition, these models are bulky and many times quite large and cannot be easily reproduced or transported.

With the advent of the digital computer, hydrologists and geologists had a tool that would give them numerical solutions to governing equations with complex boundary conditions. Such models had their beginnings in the 1960's (Pinder and Bredehoef, 1968; Prickett and Lonnquist, 1968) and are continuing to be formulated today with one of the most recent models being created for the United States Geological Survey (Trescott, Pinder, and Larson, 1976).
CHAPTER 3

DIGITAL COMPUTER MODELS

The present day digital computer lends itself quite well to simulating geohydrological conditions. Great advances in reducing the size and cost of these machines have been made in recent years, and their ability to make millions of complex computations in a matter of seconds gives them great flexibility. Computer service is usually readily available in the United States and Canada via batch processing or a time sharing system. More and more countries are also developing sophisticated data processing facilities, and the existence of global communications via satellites makes almost any part of the world accessible to some facility.

The Art of Modeling

Huntoon (1974) lists 7 steps in simulation modeling:

1. Formulate the problem and define the project objectives.

2. Develop an adequate mathematical description of the physical system to be modeled. This normally involves the choice of an appropriate partial differential equation or a series of equations and boundary conditions.

3. Develop the necessary numerical analogues to the partial differential equation.

4. Write a computer program or develop a solution algorithm to solve the numerical scheme.
5. Test the model against known analytical solutions to the partial differential equation.

6. Calibrate the model using the data from the system to be modeled.

7. Apply the model as a predictive or problem solving device.

Sensitivity tests are usually performed on the model after it has been tested against known analytical solutions. These tests consist of varying input parameters over a wide range of values to study the effect of these changes on the model. These tests can also be used to test the adequacy of boundary conditions.

It is in the last 3 steps that simulation modeling becomes as much art as science. The investigator must use his or her judgement and knowledge to decide whether the model is performing satisfactorily and if not what changes must be made. The investigator must also decide what degree of accuracy is needed or desired. In modeling because generalities and the use of a two-dimensional model to describe a three-dimensional system create error, it is perhaps more desireable to approximate trends and magnitudes of changes rather than exactly matching data from the system being modeled.

Desired Qualities of a Mining Hydrology Model

A digital model suitable for a user to simulate geohydrological conditions should be based on a number of factors. First of all its operation should be easy and simple. As the general company may not have a hydrologist on its staff, the model should be one that can be basically understood by an engineer or a systems programmer
or analyst. Many mining companies today have such trained people on their staffs. The model should be straightforward in its form and not contain so large a number of options as to confuse the user.

Because no two areas are completely identical, changes to the basic program and the input data should be uncomplicated and quick. Changes in the source program should be short and efficient. Changes should require a minimum increase in core storage requirements.

The model should require as small a core storage as possible so it can be run on small computers. Many times a computer will be available but have only a few tens of thousands of bytes of core storage. If the core storage requirement of the model is too large, such a computer could not be used. Also the entire operation of the computer would have to be committed to the execution of the model, thus preventing the processing of other programs. Many time sharing systems do not allow the use of more than a few hundred thousand bytes of core storage. Thus quick changes and answers via this system would be difficult.

Data needs of a hydrological model consist of such variables as transmissivity, storage coefficient (artesian and non-artesian), boundary conditions, geological fabric, and the rate of recharge or leakage. These data can be obtained from pump tests, publications, or personal observations. Once the data have been assembled, input into the model should be simple and straightforward.

The output of the model should be reliable, clear, and easily understood. It should characterize the item of interest that gives an image with respect to another dimension such as time or space. Such output can take on several different forms, but probably the best is a
finite difference grid in the form of a matrix showing the distribution of head with respect to areal extent and time. A graphical output would also be desirable. The model should be able to handle several different situations whose solutions are already known and closely agree with those answers.

In the same vein, the model should be able to handle many different geohydrologic situations or a combination of several situations such as leaky aquifers, changeover from artesian to non-artesian conditions, boundaries, anisotropic aquifers, and variable pumping.

Prickett and Lonnquist Programs

A series of digital computer programs was devised in 1971 by T. A. Prickett and C. G. Lonnquist at the Illinois State Water Survey (Prickett and Lonnquist, 1971). In addition to a basic program, routines to simulate such geohydrologic conditions as nonsteady flow of ground water in heterogeneous aquifers under non-artesian (water table) and artesian, leaky and non-leaky conditions were written. The routines are capable of simulating anisotropic conditions in one, two, or three dimensional flow.

The listed programs are capable of handling time varying pumping from wells, natural or artificial recharge, induced infiltration from surface waters, evapotranspiration, conversion from artesian to non-artesian conditions, and variable grid spacing. The simulation of both recharge and negative boundaries is possible with the program. A number of printout options are listed, including numerical printouts of heads
and drawdowns, category printout, and time-water level graphs. Results may be contoured.

Another version of these programs was released by the same two authors in 1973 (Prickett and Lonnquist, 1973). This listing allows the use of the program on computers that do not have large storage capability. By computing small sections of the program and reducing the output of each section to one or a few variables and storing these on tape or disk for recall as needed, smaller core storage is required. This listing is ideal for use in small installations where such devices as tape drives and disk packs can be used to store information until needed.

Mathematical Description of Programs

Jacob (1950) developed what is termed by many as the "General Equation for Ground-Water Flow" based upon the principle of conservation of mass. He combined the continuity equation with Darcy's law to give the following equation:

\[
\frac{\partial}{\partial x} (K_x \frac{\partial h}{\partial x}) + \frac{\partial}{\partial y} (K_y \frac{\partial h}{\partial y}) + \frac{\partial}{\partial z} (K_z \frac{\partial h}{\partial z}) = S' \frac{\partial h}{\partial t}
\]  

(1)

where

\begin{align*}
K & = \text{hydraulic conductivity} \\
h & = \text{head} \\
x & = \text{x direction} \\
y & = \text{y direction} \\
z & = \text{z direction (vertical)} \\
S' & = \text{specific storage} \\
t & = \text{time}
\end{align*}
If it is assumed that $K_x = K_y = K_z = K$ (the aquifer is isotropic and homogeneous), equation 1 becomes

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S'H}{Kb}$$

(2)

In the case of an artesian aquifer of essentially uniform thickness $b$, the right side of equation 2 is multiplied by $\frac{b}{b}$ to give

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S'H}{T'b}$$

(3)

where

$S = S'b$ (storage coefficient) \hspace{1cm} T = Kb$ (transmissivity)

Equation 3 is that for three dimensional flow in a homogeneous, isotropic aquifer under non-steady state conditions.

In large areal aquifer systems, the lateral flow components (those in the $x$ and $y$ directions) are the dominant parts of the system. The flow component in the vertical direction ($z$ direction) is so much appreciably smaller than those in the two other directions that it can be considered for all practical purposes to be equal to zero. Thus if any flow resulting from the vertical direction is neglected, equation 3 becomes for two-dimensional flow

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S'H}{T'b}$$

(4)

Taylor series can be used to write the finite difference form of equation 4.
Equation 4 uses the following assumptions:

1. The aquifer is compressible in the vertical direction.
2. The aquifer is isotropic.
3. Changes in fluid density are negligible.
4. The storage coefficient is constant.
5. The aquifer is of essentially uniform thickness, i.e., the transmissivity is constant.
6. Flow is two dimensional (Huntoon, 1974).

In the Prickett and Lonnquist programs the numerical method also follows the principle of conservation of mass. All inputs and outputs are accounted for and following the law of continuity the net change in storage in the system can be calculated.

If \( Q_1, Q_2, Q_3, \) and \( Q_4 \) are used to represent node-to-node transfer rates, \( Q_5 \) is used to represent the flow rate of the amount of water taken into or released from storage per unit time increment, and \( Q_6 \) is a net constant withdrawal rate, the continuity equation for water flow in and out of a finite difference cell is

\[
Q_n + Q_1 + Q_3 = Q_2 + Q_4 + Q_5 + Q_6 \quad (5)
\]

where \( Q_n \) is a flow rate associated with such specialized conditions as leakage, induced infiltration, and evapotranspiration (Figure 3).

Prickett and Lonnquist term the portions of the aquifer included in the flow terms as "vector volumes" so as to emphasize volume and also direction of flow. Since all vector volumes extend the full depth of the aquifer and have dimensions of the grid space, they represent the
Figure 3. Finite Difference Grid and Flow Rates. — (Prickett and Lonnquist, 1971)

Figure 4. Vector Volumes for Node-to-Node Flow Rates. — (Prickett and Lonnquist, 1971)
movement of a particular mass or volume of water through the vector volume for a discretized time interval (flow rates \( Q_1 \) through \( Q_4 \)).

There are then four separate vector volumes of water to be considered in each grid block: (1) \( i,j \) to \( i+1,j; \) (2) \( i-1,j \) to \( i,j; \) (3) \( i,j+1 \) to \( i,j; \) (4) \( i,j \) to \( i,j-1 \) (see Figure 4). \( Q_5, Q_6, \) and \( Q_n \) are treated in a similar manner except that \( Q_6 \) is usually a constant, and \( Q_n \) is set to zero unless one of the special conditions it represents is modeled.

Applying Darcy's law to the flow rates illustrated in Figure 4 the following equations are derived:

\[
Q_1 = T_{i-1,j,2}(h_{i-1,j} - h_{i,j})\Delta y/\Delta x \quad (6a)
\]

\[
Q_2 = T_{i,j,2}(h_{i,j} - h_{i+1,j})\Delta y/\Delta x \quad (6b)
\]

\[
Q_3 = T_{i,j,1}(h_{i,j+1} - h_{i,j})\Delta x/\Delta y \quad (6c)
\]

\[
Q_4 = T_{i,j-1,1}(h_{i,j} - h_{i,j-1})\Delta x/\Delta y \quad (6d)
\]

where

\( h_{i,j} \) = calculated heads at the end of a time increment measured from an arbitrary reference level at node \( i,j \)

\( T_{i,j,1} \) = aquifer transmissivity within the vector volume between nodes \( i,j \) and \( i,j+1 \)

\( T_{i,j,2} \) = aquifer transmissivity within the vector volume between nodes \( i,j \) and \( i+1,j \)

Furthermore, \( Q_5 \) is defined as

\[
Q_5 = S\Delta x\Delta y(h_{i,j} - h_{i+1,j}) / \Delta t \quad (7)
\]
where

\[ h_{i,j}^{\text{coeff}} = \text{calculated head at node } i,j \text{ at the end of the previous time increment } \Delta t \]

\[ \Delta t = \text{time increment elapsed since last calculation of heads} \]

(Prickett and Lonnquist, 1971)

\[ Q_6 \] is the constant net withdrawal rate and equals \( Q_{i,j} \). \( Q_n \) is the special source term and equals itself.

Substitution of equations 6 and 7 and \( Q_{i,j} \) into equation 5 results in

\[
T_{i-1,j,2} (h_{i-1,j} - h_{i,j}) / \Delta x^2 + T_{i,j,2} (h_{i+1,j} - h_{i,j}) / \Delta x^2 + T_{i,j,1} (h_{i,j+1} - h_{i,j}) / \Delta y^2 + T_{i,j,1} (h_{i,j-1} - h_{i,j}) / \Delta y^2 = S (h_{i,j} - h_{i,j}^{\text{coeff}}) / \Delta t + Q_{i,j} / \Delta x \Delta y - Q_n / \Delta x \Delta y
\]

(Prickett and Lonnquist, 1971)

(8)

where

\[ i = \text{column number} \quad j = \text{row number} \quad i,j = \text{node number} \]

\[ h_{i,j}^{\text{coeff}} = \text{calculated head at node } i,j \text{ at the end of the previous time increment } \Delta t \]

\[ Q_n = \text{flow rate of specialized conditions such as leakage, induced infiltration, and evapotranspiration} \]

If \( Q_n \) equals zero, equation 8 is the finite difference form, an equation for non-steady state, non-homogeneous, two dimensional flow in an artesian aquifer.

Each node now has an equation in the form of equation 8. Using the iterative alternating direction implicit method of Peaceman and
Rachford (1955) which involves the use of Gauss elimination to solve many equations simultaneously and what Peaceman and Rachford (1955) call G and B arrays applied to tri-diagonal matrices, the resulting heads can be calculated at each node.

Abilities of the Program in Dewatering Problems

The program offers some attractive features for use in determining dewatering needs. Its job set-up is rather simple and easily understood. It has variable pumpage and can use a variable grid pattern. It can account for the change from artesian to non-artesian conditions. It requires relatively little core storage, and the output can be digital or in contour form. The programs with certain modifications can simulate one to three dimensional flow. Most importantly of all, it can simulate many of the conditions and practices found in mining.

It must be cautioned that any digital model is not the magic "black box" that has long been searched for. The model should be used to help predict dewatering needs (once it has been properly tested and calibrated) and to check out dewatering schemes that have been designed from analytical means as is done in this paper. As the calibration of the model reflects only one solution of many possible solutions, it must be continually updated. The model can require months to be validated. However the model can be used to predict the effect of different dewatering schemes. It can also be used to check the consistency of aquifer parameter estimates. At all times it should be remembered that the output of the model is only as good as the input and that errors in aquifer parameter estimation and boundary conditions can cause the
model to reflect incorrect or misleading output. The output of the program can be very inaccurate. Also answers that agree exactly with observed data are rarely obtained, and the results should be treated accordingly.

**Modifications to Program for Dewatering Predictions**

In order to use the program for prediction of mine dewatering needs, it was necessary to modify slightly and incorporate certain methods used for mine dewatering.

A problem encountered in the program itself was that of head predictions. This section is supposed to predict the heads for the next time step and thus increase precision and reduce the number of iterations. The problem arose that if the heads at the first time step at each node were not the same as the head on the default card, the value of the default card was read in, resulting in erroneous results. This was easily corrected by making the value of the head at each node equal to that read on a node card (if a node card existed for that node); or if no node card were available for a particular node, the default value was used. This was accomplished by inserting the statements at the end of the "Start of Simulation" section of the composite aquifer listing (Appendix C).

Dewatering/depressuring wells are simulated by placing discharge wells on the grid at appropriate locations. A correction formula is necessary in most cases as the effective radius is generally not the default value of the program (see Appendix D).
Long holes are simulated by placing small discharge wells along the mine workings. Generally more than one long hole is drilled at each location so the cumulative discharge is treated as one well at each location. Long holes can be created by using the variable pumping section of the model, making discharge equal to zero until the time step in which the long holes are drilled is computed.

**Use of Model for Assessment**

Hydrologic data for the model can be acquired by many means. These include drill stem tests, slug tests, laboratory tests, and aquifer or pump tests. The pump test is the best source because it is the best method for accurate estimates of aquifer parameters. Drilling information and geophysical logs can be used to determine lithologic data. Other mines in the area can be used to estimate the amount of inflow into the long holes. Aerial photography can be used, along with drilling data, to locate structural features such as faults and intrusives. Geophysical techniques can also be used to delineate these features.

Once all available data have been compiled and placed into the model, the process of calibration can begin. This consists of matching the actual observed changes to those predicted by the model, given certain conditions of discharge and well location(s). This process can consist of varying the aquifer parameters to see what the effect is, locating additional structural features or re-estimating the effects of known features, or increasing or reducing discharge and/or recharge boundary effects. The attempt here is to match as closely as possible
the actual observed conditions in the area to those computed by the model. This can be done by calibrating the model against the data of a pump test. However, the model can be used when very little data are known to give a very rough estimate of the effects of dewatering.

Once the calibration of the model is complete, it can be used to project future water levels in response to different pumping configurations, long hole spacings, haulageway or drift development and to examine such environmental effects as radius of influence and water supplies affected by pumping. Since the output can be both digital and contoured, a visual picture can be gained to give an idea of the physical location of the head in an area.

It must be remembered that the output of the model is only a prediction and that it does not necessarily mean that this is the way the system operates or will operate. As previously mentioned, the calibrated solution is only one of many possible solutions and should be continually updated. Problems also arise from trying to impose a two-dimensional system on a three-dimensional one. The effect of partially penetrating wells is also a problem not resolved. While the output can be very precise, it can also be very inaccurate. Misinterpretation of boundaries or incorrect estimates of aquifer parameters especially in anisotropic areas can make the model invalid for long term predictions.

The best way to use the digital model appears to be to confirm or support hypotheses and designs that are solved analytically. Thus dewatering/depressuring schemes are designed based upon pump test and hydrologic information, and then the proposed scheme is tested with the model to check its validity. By doing this, the digital model can
play an important role in predicting dewatering/depressuring requirements and effects.
CHAPTER 4

URANIUM POINT ZONES, GAS HILLS, WYOMING TEST CASE

The Uranium Point Zones (UPZ) is located in the Gas Hills Uranium District of Central Wyoming (Figure 5). The Gas Hills is an area of intense uranium mining that commenced in the early 1950's and is continuing today. Both underground and open pit mines have been developed in the area, the largest pit being the Lucky Mc of the Lucky Mc Uranium Corporation.

The Gas Hills is located in the southern part of the Wind River Basin, a large regional structural depression of about 8,500 square miles in Central Wyoming. Within the area rocks of Paleozoic, Mesozoic, and Tertiary age are as much as 27,000 feet thick. The Wind River Basin is bounded on the south by the Sweetwater Arch and by the Wind River and Owl Creek Mountains to the west and north. The Casper Arch, an anticlinal feature, and the Rattlesnake Hills form the eastern boundary. Beaver Rim, a large escarpment, forms the southern boundary between the Basin and the Sweetwater Arch on the project site. It dips southward from the Gas Hills and provides a very vivid boundary at an elevation of 7400 feet. The elevations near the project site are about 7000 feet. The area is semi-arid in climate with less than 12 inches of precipitation annually.
Site Geology

The Uranium Point Zones (UPZ) is located in the southern part of the Gas Hills, at the base of Beaver Rim and adjacent to Uranium Point (Figure 6).

The project site is underlain by 2000 feet of Precambrian granite, Paleozoic and Mesozoic sediments, and about 1100 feet of Tertiary sediments (Table 1). The formations consist of Precambrian granites; Paleozoic limestones, sandstones, shales, and siltstones; Mesozoic sandstones; and Tertiary sandstones.

The stratigraphically lowest formation of interest in this study is the Chugwater Formation of Triassic Age. The Chugwater is a very distinctive formation in Wyoming, characterized by its reddish-brown color. It can be described as a reddish-brown siltstone and sandstone which is shaley and/or calcareous in part. An upper unit of the Chugwater found in some areas of central Wyoming is the Alcova Limestone Member. Thickness of the Chugwater is given as 1100 feet in the area of Beaver Rim (Van Houten, 1964). The Chugwater is the remnant of an old erosional surface, and a map of its surface shows a series of buttes and mesas with associated deep valleys. The Chugwater occurs from depths of 900 feet on the project site to surface outcrops east of the UPZ.

Overlying the Chugwater and creating a Tertiary unconformity is the lower Eocene Wind River Formation. The Wind River is a lenticular, arkosic sandstone with lenses of mudstone. Some conglomerate is also
Table 1

Stratigraphic Section, Uranium Point Area, Fremont County, Wyoming. —

After Van Houten and Weitz, (1956); Van Houten (1964)

<table>
<thead>
<tr>
<th>Geologic Age</th>
<th>Formation</th>
<th>Thickness (feet)</th>
<th>Lithology</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tertiary</td>
<td>Split Rock</td>
<td>150‡</td>
<td>Well sorted and massive sandstone and conglomerate with coarse, angular pebbles and cobbles.</td>
</tr>
<tr>
<td></td>
<td>White River</td>
<td>225‡</td>
<td>Mudstone, bentonitic and tuffaceous with lenses of arkose and conglomerate.</td>
</tr>
<tr>
<td></td>
<td>Wagon Bed</td>
<td>350‡</td>
<td>Bentonitic mudstone and sandstone with some volcanic sandstone and conglomerate.</td>
</tr>
<tr>
<td></td>
<td>Wind River</td>
<td>300-400</td>
<td>Lenticular, arkosic sandstone with lenses of mudstone; some conglomerate.</td>
</tr>
<tr>
<td>Triassic</td>
<td>Chugwater</td>
<td>1100‡</td>
<td>Sandstone and siltstone, generally red.</td>
</tr>
<tr>
<td>Lower Triassic</td>
<td>Dinwoody</td>
<td>60‡</td>
<td>Sandstone and shale, gray to tan.</td>
</tr>
<tr>
<td>Permian</td>
<td>Phosphoria</td>
<td>350‡</td>
<td>Dolomitic siltstone and sandstone, bedded chert, this phosphatic zones, gray to brown.</td>
</tr>
<tr>
<td>Pennsylvanian</td>
<td>Tensleep</td>
<td>270‡</td>
<td>Sandstone, gray to rusty.</td>
</tr>
<tr>
<td>Pennsylvanian</td>
<td>Amsden</td>
<td>180‡</td>
<td>Sandstone and shale, red to gray</td>
</tr>
</tbody>
</table>
Table 1 (cont.)

<table>
<thead>
<tr>
<th>Geologic Age</th>
<th>Formation</th>
<th>Thickness (feet)</th>
<th>Lithology</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Cambrian</td>
<td>Gallatin</td>
<td>500</td>
<td>Limestone, gray to red.</td>
</tr>
<tr>
<td>Middle Cambrian</td>
<td>Gros Ventre</td>
<td>350+</td>
<td>Siltstone and sandstone, glauconitic, gray to reddish brown.</td>
</tr>
<tr>
<td>Precambrian</td>
<td>Sweetwater</td>
<td></td>
<td>Granites and metasediments.</td>
</tr>
</tbody>
</table>
present. The Wind River contains two distinct sandstone units which are separated by a mudstone layer in many areas. One sandstone is the host rock for the uranium, which was deposited in a roll type front, in the area and is a yellowish-orange to yellowish-gray arkose. The other sandstone is pale yellowish-gray to very pale olive. Thickness of the Wind River varies from 300 to 600 feet near Beaver Rim (Van Houten, 1964).

Conformably overlying the Wind River is the middle and upper Eocene Wagon Bed Formation. The Wagon Bed consists of 130 to 700 feet of bentonitic mudstone and sandstone with some volcanic sandstone and conglomerate.

Unconformably overlying the Wagon Bed is the Oligocene White River Formation which consists of 100 to 650 feet of bentonitic and tuffaceous mudstone with lenses of arkose and conglomerate (Van Houten, 1964). Its color is a light yellowish-gray to grayish-orange. Slump blocks of this formation can be seen on the slope of Beaver Rim at the UPZ.

Unconformably overlying the White River and forming the cap rock for Beaver Rim is the Miocene Split Rock Formation. The Split Rock consists of over 150 feet of conglomerate and well sorted sandstone.

The post Chugwater formations dip to the southeast at about 2 degrees and pinch out against the Rattlesnake Hills. A synclinal structure exists at this point as the formations are wedged up against these hills.

Structurally the project site is believed to occur in a horst structure (see Figure 6) with normal faults to the north and south of
the site. The entire Gas Hills region is intensely faulted, and most faults are normal and are generally en echelon, parallel to Beaver Rim. The two faults forming the horst run east to west and are intercepted and bounded on the west by another fault that trends northeast-southwest. Several other faults are suggested by drilling data in the area but are unconfirmed.

**Site Hydrology**

No surface, perennial stream exists at the UPZ, but the area is marked by several usually dry channels and streambeds that transport runoff from site. No spring is known to exist at the UPZ although a small one does exist near the south side of Uranium Point. Springs in the area are more or less aligned with the fault system paralleling Beaver Rim, which suggests strong structural control by faults.

Some recharge to the regional aquifer originates on the Granite Mountains and the Rattlesnake Hills from precipitation as the regional ground-water gradient is to the northwest. Marks (1958) has theorized that recharge to the aquifer at the site may be the result of direct precipitation on outcrops or/and at depth by confined or unconfined water.

The Wind River Formation contains the principal aquifers of the project site. It consists of 300 to 400 feet of sandstone with an intervening siltstone layer. Movement of ground water is primarily along fracture systems because the formation is fairly well cemented. Two aquifers are contained in the Wind River at the project site. The upper aquifer is 50 to 125 feet thick and consists of medium to coarse
grained arkosic sandstone. It is separated from the lower aquifer by about 100 feet of siltstone. The lower sand has a fine grained layer at its base but is mainly medium to coarse grained and is 200 to 300 feet thick in the area. Piezometric heads in observation wells show there is a difference of about 7 feet between the aquifers in head, the higher head being in the upper sand unit. On the UPZ, the intervening siltstone layer is present, but it thins to the east and west and is absent approximately one and a half miles to the east. Thus the upper and lower aquifers become one unit in some areas. The siltstone layer is discontinuous throughout the area (Bill King, Geologist, Federal-American Partners, personal communication, 1978). However the two sandstone units are considered two separate aquifers on the UPZ for reasons that will follow.

There is disagreement over the piezometric surface gradient at the UPZ. Marks (1958) shows the area as having a gradient to the northwest, which is the regional gradient. Measurements made by Woodward-Clyde Consultants (1975) indicate that the piezometric gradient in the UPZ is to the southeast. Two new piezometers were installed in the summer of 1976, and new measurements were made in September, 1976. These measurements (Figure 7) show a gradient in the northwest direction as shown by Marks. After the Woodward-Clyde measurements, a survey error of 30 feet was discovered at the UPZ, but the corrected water levels still showed a southeastern gradient. However, the Woodward-Clyde measurements were made during a time of several pump tests, and it is possible that water levels had not recovered to static conditions when those readings were made. Measurements made in the spring of 1977 support the belief that the site gradient is the same as the regional
Figure 7. Uranium Point Zones Piezometer Scheme and Major Structural Features — Lower Sand Piezometric Surface (950-76)
gradient. Personal communication with Neil Jacquet, who conducted the Woodward-Clyde study, in 1977 revealed the possibility that water levels did not attain their static levels is certainly real. It appears this is the case since the water levels recorded over a year's time period as well as historical data indicate a northwestern flow. It is concluded that flow is to the northwest as shown by Marks (1958) and illustrated in Figure 7.

Wells in the Wind River Formation are presently yielding a total of about 1000 GPM in the area of the Lucky Mc Mine. John Russell of Utah International (now Lucky Mc Mining Company) estimates that 95% of this flow is derived from a north-south trending fault that is parallel to the west wall of the pit (Woodward-Clyde, 1975). This along with the orientation of springs along known faults suggests strong structural control of ground-water movement.

Pump Tests and Observation Well Network

In order to ascertain the hydrologic conditions and monitor changes in water levels, an observation well network was installed on the UPZ (Figure 7). The network consists of seven observation wells on the UPZ site proper.

The UPZ piezometers consist of 5 observation wells screening the lower sand (UPZ-415P, UPZ-416P, UPZ-438P, UPZ-3P, and UPZ-4P) and two observation wells screening the upper sand (UPZ-417P and UPZ-418P). Piezometers were drilled as 6½ inch holes with 2-3/8 inch O.D. casing installed and torch slotted opposite the appropriate sand unit. Three piezometers were completed into the fault system on the south
(UPZ-416P, UPZ-418P, and UPZ-438P). UPZ-3P and UPZ-4P were positioned so they could be used for pump testing both test wells and also to provide additional monitoring within the ore body during mining.

Two test wells (UPZ-414W and UPZ-433W) were located on the project site to ascertain hydrologic conditions and also to help to depressurize/dewater the aquifers once construction of the mine and shaft had begun. Both wells were drilled as 11 inch boreholes with mill slot casing installed opposite the sand units. Both wells fully penetrated the aquifers and were open to both and were cemented from the top of the slotted casing near the surface.

A series of pump tests was conducted on the area. The first test was conducted October 1975 and consisted of two separate 24 hour pump tests conducted by Woodward-Clyde Consultants of Denver, Colorado. Wells UPZ-433W and UPZ-414W were pumped, and the associated piezometers measured.

No attempt to determine the contribution of flow from each sand was made during the test, and the sands were treated as one aquifer. Well UPZ-414W was pumped for 1095 minutes at 100 GPM and 120 GPM for the remaining 345 minutes. Total drawdown was about 135 feet for a specific capacity of .89 GPM/ft and a transmissivity of 1150 GPD/ft. Piezometer UPZ-415P (lower sand) showed a transmissivity of about 1025 1025 GPD/ft, and UPZ-417P (upper sand) had a calculated transmissivity of about 1900 GPD/ft based upon the Cooper-Jacob semi-log plot. UPZ-433W was pumped at varying rates from 120 GPM to 100 GPM for 24 hours with an estimated specific capacity of .39 GPM/ft. A transmissivity of about 450 GPD/ft was calculated for this well. UPZ-438P
showed a transmissivity of about 625 GPD/ft, and UPZ-416P had a calculated transmissivity of about 800 GPD/ft. Both these observation wells were in the lower sand and in or near the southern fault. A storage coefficient of about .0002 was calculated for the area (Woodward-Clyde, 1975). Table 2 gives a resume of these tests. Because no attempt to segregate flows from each individual sand was made, the reported transmissivity values are not considered reliable.

Because of the brevity of the 1975 tests and the need to identify significant boundaries, two extended pump tests were run in late 1976 and early 1977. The purpose was to obtain long term hydrologic parameters as well as to identify boundary conditions.

Well UPZ-433W was test pumped commencing November 30, 1976, with a submersible pump rated at 85 GPM at 700 feet of head installed at 700 feet. Although discharge of 90 GPM was obtained in the early part of the test, the average discharge was about 85 GPM as the water level declined. Equipment malfunctions made it impossible to obtain water level information in the test well, and the test was plagued by generator problems. As most of the slotted casing in this well is opposite the lower sand unit, it was presumed that 100% of the discharge came from that unit. When well UPZ-433W was pumped solely, no measurable drawdown was noted in an upper sand piezometer (UPZ-417P). This suggests that the well is totally within that lower unit, and that there is little or no hydrologic interconnection between the upper and lower sands in the area of UPZ-417P. The test was run until December 22, 1976. Analysis of the data revealed long term transmissivities as shown in Table 2. Appendix E illustrates the semi-log plots of this test.
Table 2

Transmissivity Values for Various Pump Tests, UPZ Wells

Transmissivities in gallons per day per foot (GPD/ft)

<table>
<thead>
<tr>
<th>Piezometer/Well</th>
<th>Woodward-Clyde Test</th>
<th>Test I (UPZ-433W)</th>
<th>Test II (UPZ-414W)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UPZ-415P</td>
<td>1116(J)</td>
<td>1206(J)</td>
<td>977(J)</td>
</tr>
<tr>
<td>UPZ-414W</td>
<td>1147(J), 1200(T)</td>
<td>NM</td>
<td>1500(J)*</td>
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<tr>
<td>UPZ-438P</td>
<td>2631(J), 467(T)</td>
<td>954(J)</td>
<td>1200(J)</td>
</tr>
<tr>
<td>UPZ-433W</td>
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<tr>
<td>UPZ-416W</td>
<td>806(J), 667(T)</td>
<td>758(J)</td>
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<td>NM</td>
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<td>1359(J)</td>
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<tr>
<td>UPZ-417P</td>
<td>1910(T), 1886(J)</td>
<td>No Drawdown</td>
<td>422(J)</td>
</tr>
</tbody>
</table>

J = Jacob method
T = Theis method
* Well 414W screens both sands, and T is for both units combined
Woodward Clyde did not differentiate between flows from each individual sand unit.
Well UPZ-414W was test pumped beginning January 26, 1977, at about 128 GPM. The test lasted only 8 days (the result of generator failure) with a final discharge of 118 GPM. The pump in this well was a 40 HP submersible. Observation wells UPZ-415P, UPZ-417P, UPZ-4P, and UPZ-438P were monitored during this test. At the end of six days total drawdown in the pumped well was 145 feet at an average discharge of 122 GPM for a specific capacity of .84 GPM/ft. The construction of the well indicates that 80% of the slotted casing was opposite the lower sand. Thus it was estimated that 80% of the discharge originated from this aquifer (presuming the two aquifers had similar hydraulic conductivities). In addition, well UPZ-433W had yielded about 100 GPM in the 1975 test, and it was exposed almost fully to this unit. It was estimated that about 100 GPM (80% of 128 GPM) came from the lower sand. The lower sand was modeled because it is the unit in which mining will occur.

Table 2 shows the different computed values for each test at each piezometer. Log-log plots were used for the Woodward-Clyde test in addition to semi-log Cooper-Jacob plots. Semi-log plots were used for the extended tests to better understand the effects of boundaries on the area. In the case of UPZ-4P in the UPZ-414W test, one and possibly two negative boundaries were noted and an "average" transmissivity was calculated (see Appendix F). The time interval between 300 and 7400 minutes was used for those computations.
Conceptual Depressuring/Dewatering Scheme

Based upon the results of the series of pump tests at the UPZ, a system to depressure the lower and upper sands was devised. It was recognized that a potential water problem existed (based upon hydrologic investigations), and a scheme such as this was recommended because it was believed to be the most economical and safest of all methods considered as well as the most efficient.

It is presumed for this study that the shaft is located within 200 feet of UPZ-414W. It is also presumed that a single haulageway would be driven westward from this shaft for about 1200 feet or near UPZ-4P. Long holes would be used to dewater haulageways and drifts.

The electrical geophysical log of well UPZ-414W indicates the top of the upper sand to be about 540 feet below land surface. The static water level is about 465 feet below land surface. Thus about 75 feet of drawdown are needed at the shaft site to depressure the upper sand. Similarly the top of the lower sand is about 695 feet below land surface with a static water level of about 470 feet below land surface. This requires 225 feet of drawdown for depressurization.

The long term specific capacity of well UPZ-414W is believed to be about .7 GPM/ft. Its total depth is 910 feet, and its available drawdown is about 400 feet if the pump is set about 40 feet from the bottom. It is estimated that the well will yield at least 260 GPM of which about 80% would be withdrawn from the lower sand (200 GPM) and the remaining from the upper (60 GPM) based upon a specific capacity of .7 GPM/ft. Well UPZ-433W is believed to yield about 90 GPM. It is desired that depressuring will occur about 180 days after the pumps are turned on.
Using the Theis non-equilibrium formula and assuming a transmissivity of 1200 GPD/ft and a storage coefficient of $2 \times 10^{-4}$, pumping from well UPZ-433W would reduce the piezometric head in the lower sand at the shaft site by about 52 feet after 180 days (see Appendix D). An additional 173 feet of drawdown would be necessary to depressure the shaft site.

Using the same aquifer characteristics and presuming well UPZ-414W is 200 feet from the shaft and pumping the lower sand at 200 GPM, a drawdown of 172 feet is projected after 180 days. This drawdown when added to the drawdown from UPZ-433W would be sufficient to depressure the shaft site.

The upper sand will be depressured by well UPZ-414W. Assuming a transmissivity of 422 GPD/ft and a pumping rate of 60 GYM and the same coefficient of storage, the predicted drawdown at the shaft site would be 129 feet, which is more than adequate to depressure the upper sand in the area of the shaft site.

It is necessary to estimate the amount of water the shaft would yield if no depressuring were used in order to design sump pumps. These are needed to remove water not withdrawn by surface wells and as a standby system in case all peripheral pumping is lost. Using the Jacob equation and presuming a transmissivity of 1200 GPD/ft, a storage coefficient of $2 \times 10^{-4}$, a 14 foot diameter shaft, and a drawdown of 425 feet; it is estimated that the shaft would yield 422 GPM after one day (see Appendix D). Using a formula for a non-penetrating well (Jaiswal, Chauhan, and Childyal, 1977) and the same constants, except for a drawdown of 215 feet, it is predicted that the shaft would yield 39 GPM
upon entry into the lower aquifer (Appendix D). This latter method is considered to be a better estimate because it more closely models the actual conditions, i.e., shafts are not fully penetrating wells upon entry into the aquifer as required by the Jacob equation.

The Thiem equation can be used to estimate the amount of pumping required to hold steady state conditions although this system is not a steady state condition in reality. Presuming the same aquifer parameters in the lower sand, a head difference of 150 feet between the shaft and the pumped well, and a radius of 200 feet for the well from the shaft, it is estimated that about 150 GPM would be required to maintain the water level at equilibrium (Appendix D).

Because it is not certain that UPZ-414W can yield 260 GPM on a long term basis and the desire for back-up systems, an additional well on the other side of the shaft at a radius of about 200 feet is needed. This well would be designed to screen only the lower sand as well UPZ-414W can produce the desired degree of depressuring in the upper sand alone.

If two wells (each 200 feet from the shaft) were pumped at 150 GPM each in the lower sand, their mutual interference would result in almost 260 feet of drawdown at the shaft site after 180 days. In addition these two wells would lower the piezometric head over 150 feet at the far end of the mine. If the effect of UPZ-433W is added to the drawdown at the shaft, over 300 feet of drawdown is projected at the shaft and over 200 feet at UPZ-4P (see Appendix D).

Mutual interference will also cause pumping levels to be lower in each well. It is estimated that the wells at the shaft will add
about 110 feet of additional drawdown to each other. Well UPZ-433W will add another 50 feet to each for a total of about 450 feet after 180 days. This level would place the pumping level in each well near the bottom of each well (see Appendix D).

At this time there is no known analytical solution or method to predict accurately the inflow into a haulageway or drift. Good estimates have been obtained by using the Darcy equation. In this instance it is presumed that the hydraulic conductivity of the lower sand is 4.5 GPD/ft². It is also presumed that the haulageway is 1200 feet long and has a maximum head at 1200 feet from the shaft and a minimum head at the shaft. An average head difference across the entire haulageway of 250 feet is presumed (remembering that the shaft dewatering has reduced the piezometric head over 200 feet at 1200 feet and over 300 feet at the shaft). The saturated thickness of the lower sand is 210 feet and the haulageway has an area of 18,800 ft². Using this method of approach, it is estimated that 70 GPM will enter the haulageway. If long holes are spaced at 125 foot intervals and are presumed to handle all inflow, each set of long holes will flow at 7 GPM (see Appendix D). This condition is that of maximum inflow, and the rates will decay as the piezometric head is reduced.

In summary the depressuring/dewatering scheme for the mine would be as follows:

a. Two wells, each 200 feet from the shaft would be pumped. Each well would withdraw 150 GPM from the lower sand, and one of the wells would withdraw 60 GPM from the upper. In addition, well UPZ-433W would be pumped at 90 GPM.
b. All wells are to be pumped for at least 180 days prior to entry into the upper sand.

c. Sump pump capacity of at least 250 GPM should be maintained in the shaft.

d. Dewatering of haulageways should be accomplished by long holes in conjunction with the wells if possible. Station pumps should be capable of handling at least 140 GPM per haulageway (100% overdesign).

e. The use of water doors to isolate the shaft should extremely high flows be encountered and vent holes as auxiliary wells should be given serious consideration.

Assumptions Used in Model

Using the following assumptions, a model of the mine site was constructed:

1. The area is homogeneous and anisotropic. The east to west transmissivity is 1300 GPD/ft and the north to south is 900 GPD/ft. Storativity is .0002 for artesian conditions and .1 for non-artesian conditions.

2. Datum is the static water level and is flat.

3. Faults in the area are negative boundaries and have transmissivities of 50 to 100 GPD/ft.

4. A 25 x 25 finite difference grid was used with a grid spacing of 125 feet.

5. The sides of the model were treated as "recharge" boundaries and were assigned high storage factors. This was done because the aquifer extends much further than the limits set by the model, necessitating a
larger $\Delta x \Delta y$ spacing. The sides of the model were not treated as constant head boundaries because it is probable that the effect of pumping can be seen beyond the limits of the model.

6. The mine area is not influenced by any other pumping in the area.

7. The aquifer is of uniform thickness, is bounded on top and bottom by impermeable boundaries, and is horizontal.

Figure 8 illustrates the finite difference grid and boundary conditions.

**Digital Model Calibration**

Because the mine will be wholly (except for the shaft) within the lower sand unit, the digital model was prepared only for this aquifer. Once the shaft is completed in the upper sand, it is expected that the upper sand will not be disturbed during mining.

The model was programmed with the values given previously and listed in Appendix C. Datum was considered to be the static water level; and because the gradient is so slight for the area, the datum was considered to be a flat surface.

The model was run for 20 days in the first trial. Wells UPZ-414W and UPZ-433W were simulated at nodes (17,12) and (12,13) respectively and were pumped 100 GPM and 90 GPM respectively. Each well was simulated in separate trials, and then its predicted results were compared to those obtained during the actual pump test. Tables 3 and 4 show the comparison between the computed and actual drawdowns. A short test was run with both pumps in operation, and a comparison of the
Table 3
Pump Test I. Computed vs. Actual Drawdowns

Pumped Well: UPZ-433W (node 12,13)
Drawdowns in feet

<table>
<thead>
<tr>
<th>Piezometer node</th>
<th>UPZ-3P (12,9)</th>
<th>UPZ-4P (8,11)</th>
<th>UPZ-438P (*)</th>
<th>UPZ-416P (**)</th>
<th>UPZ-415P (17,11)</th>
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</table>

<table>
<thead>
<tr>
<th>Times after pumping began (min.)</th>
<th>Com Act</th>
<th>Com Act</th>
<th>Com Act</th>
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</table>

*Exact location not near node. Gradient between nodes (12,13) and (13,14) used. Piezometer located 121 ft. SE on node (12,13).

**Exact location not near node. Gradient between nodes (11.15) and (11,16) used. Piezometer located about 30 ft. south of (11.15).

Com = computed drawdown from model
Act = actual drawdown in piezometer
NM = not measured
Table 4

Pump Test II. Computed vs. Actual Drawdowns

Pumped Well: UPZ-414W (node 17,13)
Drawdowns in feet

<table>
<thead>
<tr>
<th>Piezometer/Well</th>
<th>UPZ-415P</th>
<th>UPZ-438P</th>
<th>UPZ-4P</th>
<th>UPZ-414W</th>
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<thead>
<tr>
<th>Time after pumping began (min.)</th>
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<th>Act</th>
<th>Com</th>
<th>Act</th>
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</table>

Com = computed drawdown from model
Act = actual drawdown in piezometer
drawdowns obtained during that test and those computed by the model is illustrated in Table 5.

Table 5 shows that the model comes close (for a short time period) to duplicating the actual drawdowns at each individual piezometer. Values are not for the same time periods because data were obtained during the test at long time intervals. However, the values computed by the model are close approximations of the actual measurements. A problem with the last test was that static water levels were not taken accurately before the test began; however the measurements are thought to be accurate enough for model calibration since they closely approximate static water levels obtained at other times.

Model Simulation

After the model had been calibrated, the proposed pumping scheme was placed on the finite difference grid. Two wells (nodes 18,12 and 18,9) each within about 190 feet of the shaft (between nodes 18,10 and 18,11) were pumped at 150 GPM and well UPZ-433W was pumped at 90 GPM. All three wells were pumped for 370 days at these rates, and then the two wells at the shaft site were reduced to 110 GPM each. This was done because it was presumed that it would take 180 days to sink the shaft and about the same time to cut stations and begin haulageway construction. Once this was done, pumping rates could be reduced to hold the piezometric surface at a steady state condition although pumping from the mine workings would continue to lower the piezometric surface.

After 370 days, long holes were placed along the haulageway to the west for 1250 feet at 125 foot intervals. These long holes were
Table 5

Computed vs. Actual Drawdowns UPZ-433W and UPZ-414W Pumped Simultaneously

Drawdowns in feet

<table>
<thead>
<tr>
<th>Piezometer/Well</th>
<th>UPZ-414W</th>
<th>UPZ-415P</th>
<th>UPZ-438P</th>
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* node is approximate location of piezometer
Com = computed drawdown from model
Act = actual drawdown in well/piezometer
presumed to flow at an average of 7 GPM. Because of the increased
interval between time steps, it was necessary to simulate all long holes
at once. In reality the haulageway is expected to be driven at 24 to 30
feet per day. At this rate the haulageway would be completed in 40 to
50 days. A correction factor of about 4 feet was computed for each long
hole.

The model was allowed to run through 16 time steps (approximately
700 days). It was expected that near equilibrium would be obtained by
this time. Only a single haulageway was used although the model could
incorporate other haulageways and drifts.

A beginning time of 1.0 days was used. An error factor of 1.0
was used even though it was much lower than the value of $2.4 \times 10^3$
recommended by Prickett and Lonnquist (1971) (see Appendix D). It was
desired to obtain as great amount of convergence as possible within
certain processing time limitations. Because of the small scale of the
model, it was doubtful that the error value would be reached in the early
times, and a limit of 25 iterations was imposed. None of the time steps
exceeded $2.4 \times 10^3$, and a value of .88 was obtained after 387 days.
Most early time error values were from 10 to 40.

The model computations indicated that it would take about 75
days to depressure the shaft site. This would be adequate advance of
the time until the shaft actually penetrated the lower unit (160 days).
Thus a fairly long delay in pumping could be encountered before
adversely affecting the depressuring operations. At 171 days, the model
computed there would be about 272 feet of drawdown at the shaft or 47
feet more than necessary. The far end of the mine would also be
virtually depressured. At 260 days it was presumed that station pumps would be installed and would handle most of the pumping. At this time the entire mine is predicted to be depressured and in most cases actually in a state of being dewatered. Appendix G contains contour maps of selected time intervals.

At 389 days the long holes were placed on the model. Because of the expected low flows (the area is partially dewatered), the long holes had only a minor effect on the piezometric surface, contributing only a few feet of drawdown. Concurrent with the installation of long holes, the shaft area pumping was reduced to 220 GPM. The Thiem equation estimate had indicated that about 150 GPM were necessary to hold the piezometric surface at equilibrium at the shaft site. The model indicated 18 feet of additional drawdown from 389 to 644 days in the area of the shaft. The shaft area pumping was further reduced to about 200 GPM with the result of rising water levels. Thus the model estimates that the pumpage required for steady-state is between 200 and 220 GPM.

Along the sides of the model showed large drawdowns, suggesting that the cone is extensive and will interfere with other pumping in the area. It also suggests that pumping at the UPZ will be affected by other pumping if it is great enough and there are no significant boundaries between that pumping and the UPZ. Because the UPZ exists in a mining district with the need for dewatering, such interference would benefit all parties affected. There is only a limited need for domestic supply, and at mine sites water from the dewatering operation could be used as a domestic supply when properly treated.
Although mining would not be completely "in the dry", the model does show that the proposed depressuring/dewatering scheme is viable. Analytical solutions predicted slightly more than 310 feet of drawdown at 180 days near the shaft site. The model predicted about 272 feet. The analytical solutions also predicted about 230 feet at the far end of the mine (UPZ-4P) at this same time. The model predicted 224 feet of drawdown at UPZ-4P. The difference between the analytical solutions and the computed drawdowns is probably caused by boundary conditions and structural features which could be accounted for by the model but not by the analytical solutions, which presume homogeneous, isotropic aquifers. Also, the analytical methods are unable to account for the change from artesian to non-artesian conditions.

It is believed that the solutions approximate each other close enough to warrant the use of the model as a tool in predicting future piezometric surfaces.

A model's results are only as good as the input data and the continued updating of results. Once the actual depressuring/dewatering operation begins, it would be desirable to set up a monitoring program to compare actual results to those predicted by the model. Continued updating of the model would allow even better predictions.
CHAPTER 5

CONCLUSION

Digital computer models can assist in the environmental and engineering analysis of a mine depressuring/dewatering system. It has been shown that such a model when properly calibrated against known data can approximate results obtained by analytical solutions. The model can take into account boundary conditions and changeover from artesian to non-artesian conditions. Analytical solutions are often very complicated or only rough approximations in these cases.

Once the model has been properly calibrated and tested, it can be continually updated and used to make predictions as to future changes in piezometric head or future pumping needs. It can be used to simulate loss of all pumping in order to estimate the amount of time allowed before flooding occurs or the amount of in-shaft pumping needed to maintain water levels in the shaft until repairs can be made. Environmentally it can give qualitative information and in some cases quantitative results as to the effect of the program on the surrounding area.

There remains the need for a model to ascertain the effects of long holing on the mining system. The procedure used in this paper is only an approximation of the situation and does not take fully into account such conditions as anisotropy. A model that would accurately estimate the inflow into the mine from long holes would be most helpful.
Such a model would also make correction for the change in head resulting from shaft pumping and long holing and make the appropriate changes to flow predictions.

It would be desirable to apply data from the actual depressuring/dewatering operation and compare it to this model's predictions. Such a comparison might bring forth a solution to the above problem and also would aid in future use of this model in conjunction with other underground mines.
APPENDIX A

GLOSSARY OF SELECTED MINING TERMS
Cut: To excavate (Thrush, 1968)

Dewater: To remove water from a mine (Thrush, 1968)

Dewatering: Removing water by pumping, drainage (Thrush, 1968)

Drift: A horizontal opening, lying in or near the orebody, parallel or nearby parallel to its strike (Peele, 1941)

Haulageway: The gangway, entry, or tunnel through which loaded or empty mine cars are hauled by animal or mechanical power (Thrush, 1968)

Grout: A pumpable slurry of neat cement or a mixture of neat cement and fine sand, commonly forced into a borehole to seal crevices in a rock to prevent ground water from seeping or flowing into an excavation. Also, the act or process of injecting a grout into a rock formation through a borehole (Thrush, 1968)

Heave: A lifting of the floor of an underground working (Thrush, 1968)

Long hole: Underground boreholes and blastholes exceeding 10 feet in depth or requiring the use of 2 or more lengths of drill stem or rods coupled together to attain the desired length (Thrush, 1968)

Roll: A form of orebody that has a curved outline that cuts sharply across the bedding of the host sandstone (Harshman, 1970)

Shaft: An excavation of limited area compared to its depth, made for finding or mining ore or coal; raising rock, water, ore, rock, or coal; hoisting and lowering men and material; or ventilating underground workings (Thrush, 1968)

Sink: To excavate strata downward in a vertical line for the purpose of winning and working minerals (Thrush, 1968)
Station: The excavation adjoining the shaft at each of the different levels where men and materials are removed or delivered (Thrush, 1968)

Sump: That portion of the shaft below the normal winding level which is used for the collection of water for pumping (Thrush, 1968)
APPENDIX B

ECONOMIC COMPARISON BETWEEN DEPRESSURED AND NON-DEPRESSURED SHAFT
Assumptions

1. Three wells used for depressuring.
2. Cost of handling water in shaft is $.03 per gallon.
3. Cost of electricity is $.05 per kilowatt hour.
4. Surface well pumping precedes shaft sinking and average pumping is 2000 GPM against 1400 feet of head for the first 19 months and 1200 GPM against 1600 feet of head for the last 19 months.
5. Shaft inflow averages 650 GPM for 13 months of shaft sinking and station cutting in a non-depressured system.
6. Depressuring program reduces inflow into shaft by 75%.
7. Surface wells with pumps cost $350,000 each and require $330,000 in total additional maintenance over 38 months.
8. Wire to water efficiency is 50%.
9. Station cutting and completion of shaft in aquifer requires 13 months.

Formula used: Illinois State Water Survey Formula for Computing the Cost of Pumping Water

Fuel cost for surface wells

For 2000 GPM

\[ \text{KW-cost} = (0.628)(0.05)(1400 \text{ ft})(2000 \text{ GPM}) \]

\[ = \frac{0.88}{100 \text{ ft} 1000 \text{ GPM}} \]

or for 570 days

\[ (0.88)(570 \text{ days})(1440 \text{ min/day}) = 7.2 \times 10^5 \]

For 1200 GPM

\[ \text{KW-cost} = (0.628)(0.05)(1600 \text{ ft})(1200 \text{ GPM}) \]

\[ = \frac{0.60}{100 \text{ ft} 1000 \text{ GPM}} \]

or for 570 days

\[ (0.60)(570 \text{ days})(1440 \text{ min/day}) = 4.9 \times 10^5 \]
Inflow into depressured shaft

25% of 650 GPM = 163 GPM

Cost of handling this flow = (163 GPM)($0.03)(1440 min/day)(390 days = $2.7 \times 10^6

Summary of Costs for Dewatering/Depressuring by Wells

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
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<tbody>
<tr>
<td>Cost of 3 wells</td>
<td>$1.05 \times 10^6</td>
</tr>
<tr>
<td>2000 GPM, 1400 feet head @ $0.05/kwh, 19 months</td>
<td>7.2 \times 10^5</td>
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<tr>
<td>1200 GPM, 1600 feet head @ $0.05/kwh, 19 months</td>
<td>4.9 \times 10^5</td>
</tr>
<tr>
<td>Sump pumping in shaft, 163 GPM at $0.03/gal for 13 months</td>
<td>2.7 \times 10^6</td>
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<tr>
<td>Well and pump maintenance</td>
<td>3.3 \times 10^5</td>
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</tbody>
</table>

Cost of Handling All Water in the Shaft

(650 GPM)($0.03/gal)(1440 min/day)(390 days) = $1.1 \times 10^7
APPENDIX C

SOURCE PROGRAM AND INPUT DATA SET
A DIGITAL COMPUTER MODELING PROGRAM FOR SIMULATING AQUIFERS WITH LEAKY CONDITIONS, CHANGEOVER FROM ARTESIAN TO WATER TABLE.
CONDITIONS, WATER EVAPOTRANSPIRATION, AND NEGATIVE AND POSITIVE BOUNDARY EFFECTS. IT IS A MODIFIED VERSION OF THE FINITE DIFFERENCE AQUIFER SIMULATION PROGRAM BY PRICKETT AND LOVEQUIST (1971), ILLINOIS STATE WATER SURVEY BULLETIN 59.

DEFINITION OF VARIABLES

J=MODEL COLUMN NUMBER
J+=MODEL ROW NUMBER
H(J,J)=HEADS AT START OF TIME INCREMENT (FT)
H(J,J)=HEADS AT END OF TIME INCREMENT (FT)
CH(J)=STORAGE FACTOR (GAL/FT³)
CH(J)=ELEVATION OF TIP OF AQUIFER (FT)
CH(J)=ELEVATION OF TOP OF AQUIFER (FT)
CH(J)=ELEVATION OF BOTTOM OF AQUIFER (FT)
CH(J)=ELEVATION OF STREAM SURFACE (FT)
PERMI(J,1)=HYDRAULIC CONDUCTIVITY OF AQUIFER BETWEEN J AND J+1 (GAL/DAY/FT²)
PERMI(J,2)=HYDRAULIC CONDUCTIVITY OF AQUIFER BETWEEN J AND J+1 (GAL/DAY/FT²)

JOB SETUP FOR THE MULTI-PURPOSE AQUIFER MODEL
JOB CARD AND JCL CARDS FOR PARTICULAR COMPUTER USEFD
SEPARATION CARD BETWEEN DATA AND SOURCE PROGRAM
PUMPING SCHEDULE FOR ALL PUMPS
ANY COL 1-60 (ENCLOSED BY PARENTHESES)
PARAMETER CARD
MSTEPS COL 1-50 RIGHT JUSTIFIED (RJ)
DELTA COL 9-12
ERROR COL 12-18
NC, COL 1-5 RJ
NL COL 7-11 RJ
TITLE COL 13-18
TJ COL 19-24
ML COL 25-30
MM COL 31-36
XQ COL 37-42
KX COL 43-48
YR COL 49-54
RD COL 55-60
DJ COL 61-66
CL COL 67-72
PP COL 73-78
SECOND CARD
BDTJ COL 1-6
DELT COL 7-12
PUMP PARAMETER CARD
NP COL 1-6 RJ
NSP COL 7-12 RJ
CONTINUE

READ NODE CARDS

READ (5,110,END=120) I,J,(T(I,J),11)1,J))11,J),CH(I,J),PERM(J,J),P(E(M1,J,J),211,J),

FORMAT

DO

TIME = TIME + DELTA

ISTEP = 1,NSTEPS

KC = 1

DO

J = 1,NR

IF

F = D/DL(I,J)

IF

F.GT.5) F = 5.0

IF

F.LT.0.0) F = 0.0

190

IF

H(I,J) = H(I,J) + D*F

IF

IF

H(I,J).LE.BOT(I,J)) H(I,J) = BOT(I,J) + 0.01

DO

CALCULATE B AND G ARRAYS

STREAMED INFILTRATION CONTROL

IF

RE = 0,0

END
IN CASPER.AQSIMKIM.FORT

- MEMBER FIRST

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IN CASPER.AQSIMKIM.FORT

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MEMBER FIRST

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WED 27 JUL 77

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J1)*(AMIN(J1,J+1),CH(J1,J+1)-BOT(J1,J+1))

460

DD = DD + H(I,J)*#(I,J,1)

470

IF (I-1) 460,470,460

480

BB = BB + T(I,J,1)

490

IF (I-NC) 500,500,500

500

IF (PERM(I,J,2)>E+0.0) GO TO 510

510

BB = BB + TII,J,2

520

W = BB - A*K(J-1)

530

G(J) = (DD-AA)*(J-1)/W

C RE-ESTIMATE HEADS

540

E = E + ABS(H(NC,J)-GNC)

550

MA = G(NC,J) - B(NC)*MIN(1,J)

560

E = E + ABS(H(NC,J)-MA)

570

CONTINUE

580

IN A (NC-LE-25) NC = 25

590

WRITE (6,580) I,TIME,E,ITER

600

DO 590 J=1,NC

610

WRITE (6,610) IH(I,J),XJ

C PRINT RESULTS

620

CONTINUE

630

WRITE (6,630)

640

CONTINUE

650

STOP

C END
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APPENDIX D

SHAFT AND MINE DEWATERING/DEPRESSURING REQUIREMENTS CALCULATIONS
Assumptions used in calculations:

\[ T = 1200 \text{ GPD/ft} \quad r = \text{as given} \quad t = 180 \text{ days} \]
\[ S = 2 \times 10^{-4} \quad Q = \text{as given} \]

I. Effect of UPZ-433W on shaft site (lower sand unit)

Formula used: Theis

\[ Q = 90 \text{ GPM} \quad r = 850 \text{ feet} \]

\[ u = \frac{1.87(850 \text{ ft})^2(.0002)}{(1200 \text{ GPD/ft})(180 \text{ days})} = 1.25 \times 10^{-3} \]

\[ W(u) = 6.1094 \]

\[ s = \frac{(114.6)(90 \text{ GPM})(6.1094)}{1200 \text{ GPD/ft}} = 52.5 \text{ feet} \]

II. Effect of UPZ-414W on shaft site (lower sand)

Formula used: Theis

\[ Q = 200 \text{ GPM} \quad r = 200 \text{ feet} \]

\[ u = \frac{1.87(200 \text{ ft})^2(.0002)}{(1200 \text{ GPD/ft})(180 \text{ days})} = 6.9 \times 10^{-5} \]

\[ W(u) = 9.0043 \]

\[ s = \frac{(114.6)(200 \text{ GPM})(9.0043)}{(1200 \text{ GPD/ft})} = 172 \text{ feet} \]

III. Effect of UPZ-414W on shaft site (upper sand)

Formula used: Theis

\[ Q = 60 \text{ GPM} \quad r = 200 \text{ feet} \]

\[ u = \frac{1.87(200 \text{ ft})^2(.0002)}{(422 \text{ GPD/ft})(180 \text{ days})} = 2.0 \times 10^{-4} \]

\[ W(u) = 7.9402 \]

\[ s = \frac{(114.6)(60 \text{ GPM})(7.9402)}{422 \text{ GPD/ft}} = 129 \text{ feet} \]
IV. Non-depressured shaft inflow estimate

Formula used: Jacob

\[ T = \frac{160 \text{ ft}^2/\text{day}}{r} \]
\[ S = \cdot0002 \]
\[ t = 1 \text{ day} \]
\[ s = 425 \text{ feet} \]

\[ 425 = \frac{(2.3)(Q)}{\log (2.24)(160 \text{ ft}^2/\text{day})(1 \text{ day})} \]
\[ 4 (160 \text{ ft}^2/\text{day}) \]
\[ (49 \text{ ft}^2)(.0002) \]

\[ Q = 8.1 \times 10^4 \text{ ft}^3/\text{day} = 422 \text{ GPM} \]

Formula used: Jaiswal, Chauhan, and Childyal Transient Spherical Flow to a Cavity Well

\[ u = \frac{r^2S}{4Tt} = \frac{(7)^2(.0002)}{(4)(160 \text{ ft}^2/\text{day})(1)} = 1.5 \times 10^{-5} \]
\[ u = 3.9 \times 10^{-3} \]

\[ \frac{r}{b} = .1 \text{ (limit of tables)} \]

\[ C(Ju, r/b) = 1 \]

\[ s = \frac{Q}{2 \text{ Kr} C(Ju, r/b)} = \]
\[ Q = (215 \text{ ft})(2\pi)(.8 \text{ ft/day})(7 \text{ ft})(1) = 7,500 \text{ ft}^3/\text{day} = 39 \text{ GPM} \]

V. Mutual interference of wells at shaft site

Effect of UPZ-433W = 52.5 feet

2 wells at 200 feet from shaft at 150 GPM each

\[ W(u) = 9.0043 \]
\[ s = \frac{(114.6)(300 \text{ GPM})(9.0043)}{1200 \text{ GPD/ft}} = 258 \text{ feet} \]

Total effect = 258 ft + 52.5 ft = 310.5 ft
VI. Estimate of pumpage required to hold steady state

Formula used: Thiem

\[ \begin{align*}
  h_1 &= 60 \text{ feet} \\
  h_2 &= 210 \text{ feet} \\
  r_1 &= 1 \text{ foot} \\
  r_2 &= 200 \text{ feet} \\
  T &= 160 \text{ ft}^2/\text{day} \\

  Q &= \frac{(2)(160 \text{ ft}^2/\text{day})(60 \text{ ft} - 210 \text{ ft})}{2.3 \log \frac{1 \text{ ft}}{200 \text{ ft}}} = 2.8 \times 10^4 \text{ ft}^3/\text{day} = 148 \text{ GPM}
\end{align*} \]

VII. Interference between wells themselves

Shaft site wells \( r = 400 \) feet

\[ u = \frac{(1.87)(400 \text{ ft})^2(0.0002)}{(1200 \text{ GPD/ft})(180 \text{ days})} = 2.8 \times 10^{-4} \]

\[ W(u) = 7.6038 \]

\[ s = \frac{(114.6)(150 \text{ GPM})(7.6038)}{1200 \text{ GPD/ft}} = 109 \text{ feet} \]

VIII. Effect of pumping on far end of mine (UPZ-4)

UPZ-433W: \( r = 530 \) feet \( Q = 90 \text{ GPM} \)

\[ T = 950 \text{ GPD/ft} \]

Formula used: Theis

\[ u = \frac{(1.87)(530 \text{ ft})^2(0.0002)}{(950 \text{ GPD/ft})(180 \text{ days})} = 6.1 \times 10^{-4} \]

\[ W(u) = 6.8254 \]

\[ s = \frac{(114.6)(90 \text{ GPM})(6.8254)}{950 \text{ GPD/ft}} = 74 \text{ feet} \]

Shaft wells: \( Q = 300 \text{ GPM} \) \( r = 1200 \) feet

\[ u = \frac{(1.87)(1200 \text{ ft})^2(0.0002)}{(1200 \text{ GPD/ft})(180 \text{ days})} = 2.5 \times 10^{-3} \]

\[ W(u) = 5.4167 \]

\[ s = \frac{(114.6)(300 \text{ GPM})(5.4167)}{1200 \text{ GPD/ft}} = 155 \text{ feet} \]

Total = 74 feet + 155 feet = 229 feet
IX. Haulageway inflow estimate

\[ K = \frac{T}{M} = \frac{1000 \text{ GPD/ft}}{210 \text{ ft}} = 4.5 \text{ GPD/ft}^2 = 0.6 \text{ ft}^2/\text{day} \]

Formula used: Darcy  \( K = 0.6 \text{ ft}^2/\text{day} \quad A = 18,800 \text{ ft}^2 \)

\( h = 250 \text{ feet} \quad l = 210 \text{ feet} \)

\[ Q = (0.6 \text{ ft}^2/\text{day})(18,800 \text{ ft}^2)(250 \text{ ft}) \]

\[ (210 \text{ ft}) \]

\[ Q = 1.35 \times 10^4 \text{ ft}^3/\text{day} = 70 \text{ GPM} \]

Presume 10 sets of long holes

Each set produces 7 GPM average

X. Well UPZ-414W pumping node correction

Formula used: Prickett and Lonnquist pumping node correction

\[ Q = 144,000 \text{ GPD} \quad T = 1550 \text{ GPD/ft} \quad a = 125 \text{ feet} \quad r_w = 1 \text{ foot} \]

\[ s_{\text{cor.}} = \frac{(0.3664)(144,000 \text{ GPD}) \log 125 \text{ ft}}{1550 \text{ GPD/ft}} \frac{125 \text{ ft}}{(4.81)(1 \text{ ft})} \]

\[ s_{\text{cor.}} = 48 \text{ ft} \]

Long hole nodes correction factor

\[ Q = 10,000 \text{ GPD} \quad T = 1200 \text{ GPD/ft} \quad a = 125 \text{ feet} \quad r_w = 1 \text{ foot} \]

\[ s_{\text{cor.}} = \frac{(0.3665)(10,000 \text{ GPD}) \log 125 \text{ ft}}{1200 \text{ GPD/ft}} \frac{125 \text{ ft}}{(4.81)(1 \text{ ft})} \]

\[ s_{\text{cor.}} = 4.3 \text{ ft} \]

XI. Initial value of ERROR calculation

Formula used: Prickett and Lonnquist ERROR estimate

\[ Q = 562,000 \text{ GPD} \quad \text{DELTA} = 1.0 \text{ day} \quad \text{SF1} = 23 \]

\[ \text{ERROR} = \frac{(Q)(\text{DELTA})}{(10)(\text{SF1})} \]

\[ \text{ERROR} = \frac{(562,000 \text{ GPD})(1.0)}{(10)(23)} = 2.4 \times 10^3 \]
APPENDIX E

SEMI-LOG PLOTS OF TEST I DATA (TEST WELL UPZ-433W)

The following are the semi-log plots of the data from Test I which was conducted from November 30, 1976, to December 22, 1976, at the Uranium Point Zones.
APPENDIX F

SEMI-LOG PLOTS OF TEST II DATA (TEST WELL UPZ-414W)

The following are the semi-log plots of the data from Test II which was conducted from January 26, 1977, to February 2, 1977, at the Uranium Point Zones.
\[ T = \frac{(264)(128)}{22.5} = 1500 \text{ gpd/ft} \]

\( \Delta s = 22.5 \text{ ft} \)
\[ T = \frac{(264)(28)}{17.5} = 420 \text{ gpd/ft} \]

\[ \Delta s = 17.5 \text{ ft} \]

**Test 2**
- Obs. Well 417
- \( R = 101 \text{ ft} \)
- \( Q = 128 \text{ gpm} \)

**Jacob Plot**
APPENDIX G

PROJECTED DRAWDOWNS FOR SELECTED TIMES FROM MODEL
Scale:

200  0  200  400  600  800 1000 FEET

Conour interval = 10 FEET

Legend:

- Contour of drawdown, feet below original water level
- U.P. Zones - Gas Hills
- Projected drawdown in lower sand after 50 days pumpage

- Location of pumped well
SCALE:
200 0 400 600 800 FEET

CONTOUR INTERVAL 10 FEET

LEGEND:
433W - LOCATION OF PUMPED WELL

U.P. ZONES - GAS HILLS
PROJECTED DRAWDOWN IN LOWER SAND AFTER 270 DAYS PUMPAGE
SCALE:

200 0 200 400 600 800 FEET

CONTOUR INTERVAL = 10 FEET

LEGEND:

CONTOUR OF DRAWDOWN, FEET BELOW ORIGINAL WATER LEVEL

LOCATION OF PUMPED WELL

U.P. ZONES - GAS HILLS

PROJECTED DRAWDOWN IN LOWER SAND AFTER 644 DAYS PUMPAGE
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