THESIS

DESIGN METHODOLOGY FOR A LARGE SCALE SOIL ABSORPTION BED FOR SEPTIC TANK EFFLUENT

Submitted by

David L. Nettles

Department of Agricultural and Chemical Engineering

In partial fulfillment of the requirements

for the Degree of Master of Science

Colorado State University

Fort Collins, Colorado

Fall, 1984

ABSTRACT OF THESIS

DESIGN METHODOLOGY FOR A LARGE SCALE SOIL ABSORPTION BED FOR SEPTIC TANK EFFLUENT

In recent years there has been much interest expressed by small communities in alternatives to the conventional, centralized chemical-biological wastewater treatment system. One of the alternatives is a large scale soil absorption system. In these systems each home has its own septic tank with the tank effluent conveyed to a single, large absorption field.

One of the problems encountered with large scale absorption fields is the buildup of a ground water mound beneath them to the point that insufficient treatment of the effluent is provided. In an attempt to alleviate this problem a model was developed for use in designing a large scale leachfield. The model developed was based on the Rao and Sarma (1981b) model of ground water mound buildup.

The model was incorporated into a computer aided design (CAD) package consisting of three major divisions. These divisions were designed to: 1) determine the recharge area needed given the maximum acceptable ground water mound height buildup and the recharge rate, 2) determine the mound height given the recharge area and the recharge rate, and 3) determine the areal extent of the mound given the mound height, the recharge area, and the recharge rate.

Testing indicates that the CAD package estimates either the recharge area or the the mound height for a constant recharge rate with reasonable accuracy (within 3% of the actual value). Also, testing indicates that the dosing procedure (applying large effluent volumes for short time periods once or twice daily) used on most large scale leachfields can be approximated by a constant recharge rate applying the same effluent volume per day. The CAD package appears to be well suited to the design of large scale leachfields. Further field testing is needed to establish this fact conclusively.

The CAD package will be useful to persons concerned with on-site wastewater treatment for several reasons. 1) It estimates the leachfield area required based on the ground water mound buildup occuring beneath the leachfield. 2) The package can be used to estimate the height of the ground water mound beneath an existing leachfield. 3) The package can also be used to determine what effect the leachfield will have on other ground water influences, such as wells and streams.

The CAD package is currently a useful tool that can aid in the design of large scale leachfields. However, the package could be improved by the incorporation of a dynamic recharge component to account for infiltration from precipitation events. Also, additional work on bacterial die-off and nitrogen conversion rates would improve the design by providing a better understanding of the unsaturated zone thickness required for wastewater treatment beneath the leachfield.

David L. Nettles
Agricultural and Chemical Engineering Dept.
Colorado State University
Fort Collins, Colorado 80523
Fall, 1984

ACKNOWLEDGMENTS

The author gratefully acknowledges the support and guidance of Dr. Robert C. Ward and Dr. David B. McWhorter throughout the work on this project. Dr. Terry Podmore and Dr. Thomas Sanders are thanked for their thoughtful suggestions concerning the final document. Sincere appreciation is extended to the numerous professional people who kindly took the time to answer questions and send pertinent information. Finally, a warm thanks to Nancy for her editing skills and moral support.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF FIGURES

CHAPTER ONE

INTRODUCTION

In recent years there has been considerable interest in alternatives to the conventional chemical-biological wastewater treatment plant traditionally used in areas of high population density. Most of this interest has been expressed by small communities where the costs, both for construction and for operation and maintenance, of the conventional system would be prohibitively expensive (Goldstein, 1972).

One proposed solution to the problem of wastewater treatment in small communities is the use of an large scale septic tank-leachfield system (Otis, 1978; Diodato, 1980; Rubin and Carlile, 1982). In this system each home or business has an individual septic tank connected to a small diameter gravity or low pressure sewer system that conveys the tank effluent to an large scale soil absorption bed (commonly called a large scale leachfield and typically handling 13.23 m^3 (3,500 gal) to 71 m^3 (18,500 gal) per day). The treatment of the wastewater begun in the septic tank is then completed in the unsaturated soil beneath the leachfield.

The type of system described above has several advantages for an existing small community. The cost of construction is generally lower than the conventional system because small diameter plastic PVC pipe can be used for collection purposes since most of the solids have been settled out in the septic tank (Otis, 1978). The cost of operation and maintenance is lower since trained operators are not required to operate and maintain the plant 24 hours a day (Otis, 1978). Operation and maintenance on the septic tanks (pumping out solids) is also more uniform because pumping is no longer the responsibility of the individual home owner (Englehardt, 1983).

Another advantage of the above system is that in areas where only a limited area of soil is suitable for a leachfield, the suitable area can be used for a leachfield while homes can be built on areas without suitable soils (Rubin and Carlile, 1981). Also, this system replenishes ground water, but with added nitrates, whereas the conventional system usually discharges the treated effluent into a surface stream (Laak, 1980).

The major drawback of the large scale leachfield is the buildup of a ground water mound beneath the leachfield. Although this buildup is not a problem in itself, it becomes one if an unsaturated zone of sufficient thickness is not maintained below the leachfield (EPA, 1980). This zone of unsaturated soil must be maintained to allow for adequate treatment of the effluent before it reaches the ground water.

OBJECTIVES

There are two objectives for the proposed research. The first is to develop a model of ground water mound buildup which describes the impact of the effluent upon the maintenance of the unsaturated zone. The purpose of the model is to estimate the size of the recharge area which will limit the mound height to a value necessary for maintenance of an unsaturated zone.

The second objective is to place the model on a micro computer in a computer aided design mode to facilitate its use by county health departments, consulting engineers, and other interested parties.

SCOPE

The problem considered is the design of a large scale soil absorption bed for septic tank effluent. The primary design criterion will be the maintenance of an unsaturated zone beneath the leachfield. This unsaturated zone must be of sufficient thickness to insure that bacterial die-off and the conversion of organic nitrogen to nitrate is complete before the effluent reaches the water table. Criteria established by the U.S. EPA (1980) will be used to define the thickness of the unsaturated zone. No effort is made to verify that this thickness is in fact providing the treatment required.

The design will only consider soil absorption beds located in aquifers that can be approximated as infinite in areal extent. This condition is incorporated into the design because soil absorption beds should not be used in aquifers with obvious impermeable side-boundaries.

FURTHER PROBLEM DEFINITION

In 1978 Otis reported on the design of an large scale soil absorption system for a small town in northwest Wisconsin. Otis' method formed the conceptual basis for the design procedure used for this investigation.

Westboro, Wisconsin is a small town with very limited economic resources. Prior to the installation of the expanded system, all homes and businesses were on individual septic tank-leachfield systems. Because of a predominance of heavy clays in the area about 80% of these systems were discharging effluent above ground. Westboro was ordered by the Wisconsin Department of Natural Resources to upgrade the existing septic tank systems or construct a centralized collection and treatment system to alleviate the problem.

Westboro, in conjunction with the Small Scale Waste Management Project of the University of Wisconsin, examined eight alternatives for a centralized collection and treatment system. The most cost effective alternative was selected for use in Westboro.

The alternative selected consisted of individual septic tanks with small diameter gravity sewers conveying the effluent to a large soil absorption field serving the entire town.

Collection System

The collection system consisted of individual septic tanks followed by either small diameter gravity sewers or pressure sewers. Existing septic tanks were used where ever possible. Replacement tanks were all 3.78 m^3 (1000 gal.) reinforced concrete, regardless of home size.

The small diameter gravity sewers were designed using guidelines developed by the South Australia Department of Public Health (1968). Ten centimeter (4 in.) diameter mains set at a minimum gradient of 0.067% were used for a design peak flow of 0.011 m^3/d per capita (3 gal. per capita per day) with half full flow conditions as recommended by the guidelines.

The pressure sewers were designed by criteria developed in the United States (Kreissl et al., 1977). Small lift stations with high water alarms and one day of excess water storage were placed after each septic tank served by a pressure sewer. The small lift station pumps discharge into a 3.8 cm (1.5 in.) pressure sewer.

The collection system also required three community lift stations. The final lift station pumps the effluent from the entire town to the siphon chamber for dosing onto the soil absorption fields.

Soil Absorption Fields

The soil absorption field is divided into three beds with only two beds in service at any one time. Each spring the out of service bed is rotated into service so that each bed receives wastewater for two years and rests for one year. This three bed arrangement also allows a standby bed in case of an unexpected failure by one of the other beds.

The soil absorption field was designed for a 113.56 cubic meters per day (m^3pd) (30 000 gpd) loading. This design figure was calculated by assuming 0.95 m^3pd (250 gpd) per home for 120 homes. Each bed was designed to receive 56.78 m^3pd (15 000 gpd) or half the total flow. Each bed also had a pressure distribution system to distribute the wastewater uniformily over the entire bed.

The field soils were sand and loamy sand with long term infiltration rates estimated to be approximately 0.049 m^3pd/m^2 (1.2 gpd/ft^2). The bed area required was thus 1159 m^2 (12 500 ft^2) which is provided by a 30.5 meter (100 ft) by 45.7 meter (150 ft) bed.

The siphon chamber has three 25.4 centimeter (10 in.) siphons capable of discharging 3.79 m^3pm (1000 gpm) at the design head. The two siphons were designed to automatically alternate operation, discharging approximately 30.28 m^3 (8000 gal) per dose. Each bed was designed to receive two doses per day at design capacity.

Monitoring

The water monitoring program in Westboro consisted of two parts, quantity and quality. The quantity section examined the average daily wastewater flow for the system. The quality section examined the water quality of the area surrounding the soil absorption field.

The water quantity monitoring indicated that while 70% of the estimated maximum number of connections were used, only 25% of the design capacity was used, which reveals an overestimation of wastewater flow. This means that, although each bed was designed to receive a dose of 30.28 m^3 (8000 gal) every 0.5 days, each bed actually receives approximately 32.18 m^3 (8500 gal) every 2.5 days.

The water quality monitoring consisted of analyzing samples from wells in the expected wastewater plume area for ammonium, nitrite, nitrate, total phosporus, chloride, calcium, magnesium, total solids, total coliform, fecal coliforms, and fecal streptococous. The only change in ground water quality found in over a year of monitoring was in total nitrogen which increased from 0.5 mg/l - N to 15 mg/l - N. The form of the nitrogen has also changed from nitrate to ammonium. This indicates that the aerobic zone beneath the beds being loaded has ceased to exist.

There are two possible reasons for the disappearance of the aerobic zone beneath the absorption beds: (1) either a ground water mound has reached the bottom of the bed, or (2) the BOD (biochemical oxygen demand) of the effluent consumed all the available oxygen. The fact that anaerobic conditions developed beneath the beds despite a loading rate only 25% of design capacity indicates that future designs must consider both of the above design constraints very carefully.

CHAPTER TWO

LITERATURE REVIEW

The past ten years have seen an explosion in publications dealing with on-site or small flow wastewater treatment technology. This review of literature focuses on only a few of those more pertinent to the study — namely modeling the flow under large scale soil absorption systems. First, some of the more recent literature describing applications of large scale soil absorption systems is presented. Second, a review of the ground water models pertinent to the particular application dealt with in this study is presented. Finally, a review of papers dealing with computer-aided-design of on-site systems is presented.

APPLICATIONS OF LARGE-SCALE SOIL ABSORPTION SYSTEMS

One of the best documented applications of a large scale soil absorption system is the Westboro case as reported earlier. While the description will not be repeated here, it will be noted that the Westboro system is still subject to considerable discussion (Ward and Morrison, 1983) and study (Siegrist, 1984) among on-site professionals.

DeWalle (1981) reported on a failure analysis of large septic tank systems (66.25 m^3/day (17500 gpd) or more) in the state of Washington. It was found that the large system failure rate was 70% greater than the small system failure rate. However, the design regulations in the state of Washington were found to allow a significantly higher loading rate for large systems (from 45% to 230% greater, depending on percolation rates) than for small systems. The report concluded by proposing a further study of large scale septic tank systems to determine what changes need to be made in design procedures and siting to improve large system performance.

Carlile et al. (1981) reported on the movement and treatment of septic tank effluent from soil absorption beds in the North Carolina coastal plain. Water table location was found to be the most important factor affecting effluent movement and treatment.

Fifteen of the seventeen systems studied were at least seasonally saturated. The systems which experienced nearly continuous saturation had the highest concentration and greatest movement of contaminants in the ground water. It was shown that better maintenance and effluent pressure distribution systems within the leachfield improved treatment.

Rubin and Carlile (1981) presented a report on the design of four different large scale on-site systems in North Carolina. The four systems described were a flow reduction/mound system, a conventional soil absorption system, an effluent irrigation system, and a recirculating sand filter. Only the first two systems used soil absorption.

The flow reduction/mound system was designed for a small factory to be built in a rural area. The estimated waste water flow of 15.14 m^3/d (4000 gpd) was reduced by 50% using water conservation devices (low volume per flush toilets and spring actuated faucets) to require a leachfield area of 1860 m^2 (20 000 ft^2). The entire leachfield was created by installing the distribution system in an imported layer of loamy topsoil 0.46 m (1.5 ft) thick.

The conventional soil absorption system was installed at a small rural school experiencing severe septic tank failure because of poorly drained soil and a seasonally high water table. The problem was corrected by pumping the septic tank effluent 244 m (800 ft) to a new leachfield installed in a small area of well drained, sandy loam soil.

GROUND WATER MODELS

Before beginning a review of some of the ground water models considered for use in this design, a short discussion of the physics of unsaturated flow is in order.

As water percolates downward beneath the recharge area it forms a front that moves downward toward the water table in response to gravity and pressure gradients as shown in Figure 2-1.

When the wetting front reaches the capillary fringe just above the water table, its movement is refracted from vertically downward to a more horizontal flow outward from the center of the recharge area in response to the hydraulic gradient that has developed. The piezometric surface (water table) rises to form the classic mound shape associated with recharge areas because the thickness of the capillary fringe remains very nearly constant. The capillary fringe remains constant because it is essentially a function of the particular soil pore size.

It should be remembered that both the capillary fringe and the area below the water table contribute to the saturated thickness through which flow occurs. This is one reason why the capillary fringe height should be considered in the estimation of the maximum acceptable mound height increase.

The above discussion indicates that a large part of the fillable pore space above the water table is occupied by the percolating water.

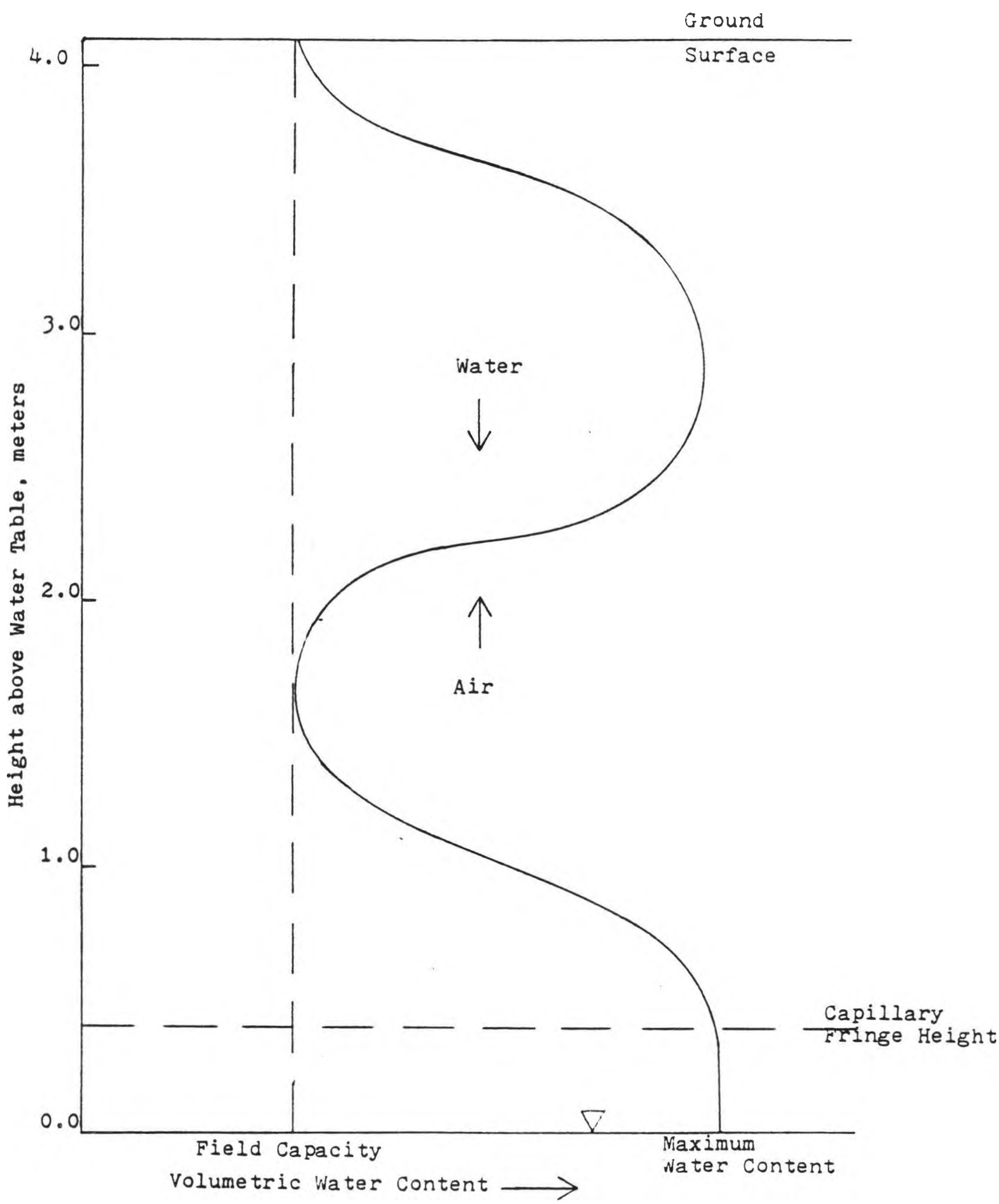

Figure 2-1. Description of Flow in the Unsaturated Zone.

This in turn, causes the effective fillable pore space (specific yield) to be less than the drainable porosity. The only ground water model examined which accounts for the reduction in the effective fillable pore space was that of Ortiz et al. (1979). This model was not considered for the design because it was felt that the greater accuracy of this model was not justified in view of the more extensive (and harder to obtain) physical parameters it required.

There are several other models currently available to predict the height of rise of a ground water mound in response to a constant recharge rate from a known rectangular area for an infinite unconfined aquifer (Glover, 1961; Hantush, 1967; Marino, 1975; Rao and Sarma, 1980). Rao and Sarma (1981a,b) also developed two models for a rectangular recharge area for a finite unconfined aquifer; one model for impermeable side-boundaries and another model for constant head side-boundaries. All of the above models were developed for a recharge rate applied over the entire area at the ground surface. Fielding (1981) also developed a model specifically for the increase in height of the ground water mound under a leachfield in an infinite aquifer.

Leachfields commonly have small diameter perforated pipes installed from 0.30 to 1.5 meters (1 to 5 ft) below the ground surface, separated by a horizontal distance of 0.91 to 1.83 meters (3 to 6 ft) (EPA, 1980). If a bed type system is used where there is a uniform layer of gravel laid on the bottom of the bed, there will be recharge over the entire area. Assuming that no positive pressures are developed in the bed, the fact that the bed will be 0.30 to 1.5 meters (1 to 5 ft) below the ground surface will have no effect on the model because the coordinate system can be oriented such that the bottom of the bed is the recharge interface. The assumption of no positive pressures developing in the bed is acceptable because of the large storage volume of the bed relative to the small application rate.

It has been determined by the EPA (U.S. Environmental Protection Agency) that an unsaturated zone from 0.61 to 1.22 meters (2 to 4 ft) thick is required for bacterial die-off and nitrogen conversion (EPA, 1980). An unsaturated zone thickness of 1.22 meters (4 ft) will be specified for use in this design. This thickness was chosen to assure that the minimum unsaturated zone thickness will be maintained, even if the field was underdesigned or overloaded to a small extent.

Most of the above noted models, with the setting as described, will now be reviewed for application to the problem of designing an expanded soil absorption bed.

Glover (1961) developed a model of flow beneath a rectangular recharge area for an unconfined aquifer of infinite areal extent. The equation formulated by Glover is as follows:

$$\alpha \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2}\right) = \frac{\partial h}{\partial t} \qquad \text{Eqn. (2.1)}$$

where:

h = height of the mound above the original water table level

t = time

α = KD/V

K = hydraulic conductivity

D = original saturated depth

V = drainable or fillable voids expressed as a ratio to the entire volume.

H = mound height at t=0 for instantaneous recharge applied uniformly over the entire area.

x, y, L, and D as defined in Figure 2-2.

The initial and boundary conditions are:

$$h = H \text{ for } \begin{matrix} -W2 < x < W/2 \\ -L/2 < y < L/2 \end{matrix} \quad \text{when } t = 0$$

$$h = 0 \text{ for } \quad x < -W/2,\ y < -L/2 \quad \text{when } t = 0$$

$$h = 0 \text{ for } \quad x > W/2,\ y > L/2 \quad \text{when } t = 0$$

If the recharge rate was constant at rate i and all the recharge was retained within the recharge area boundaries, the water table rise rate would be R = (i/V). The spreading of an increment of recharge Rd_ν occurring during the time interval d_ν may be integrated with respect to time to yield the rise, h, at time, t, in the form:

$$h = r \int_0^t \left(\frac{1}{\sqrt{\pi}} \int_{u_1}^{u_2} \exp(-u^2)du\right) \left(\frac{1}{\sqrt{\pi}} \int_{u_3}^{u_4} \exp(-u^2)du\right)d\nu \qquad \text{Eqn. (2.2)}$$

for

$$u_1 = \frac{(x-W/2)}{\sqrt{4\alpha(t-\nu)}}, \quad u_2 = \frac{(x+w/2)}{\sqrt{4\alpha(t-\nu)}}, \quad u_3 = \frac{(y-L/2)}{\sqrt{4\alpha(t-\nu)}}, \quad u_r = \frac{(y+L/2)}{\sqrt{4\alpha(t-\nu)}}.$$

Now let $\varepsilon = \nu/t \qquad d\varepsilon = \frac{1}{t}\, d\nu$, so that Equation (2.2) becomes

$$\frac{h}{Rt} = \frac{1}{4} \int_0^1 \left(\frac{2}{\sqrt{\pi}} \int_{u_1}^{u_2} \exp(-u^2)du\right) \left(\frac{2}{\sqrt{\pi}} \int_{u_3}^{u_4} \exp(-u^2)du\right)d\varepsilon \qquad \text{Eqn. (2.3)}$$

for

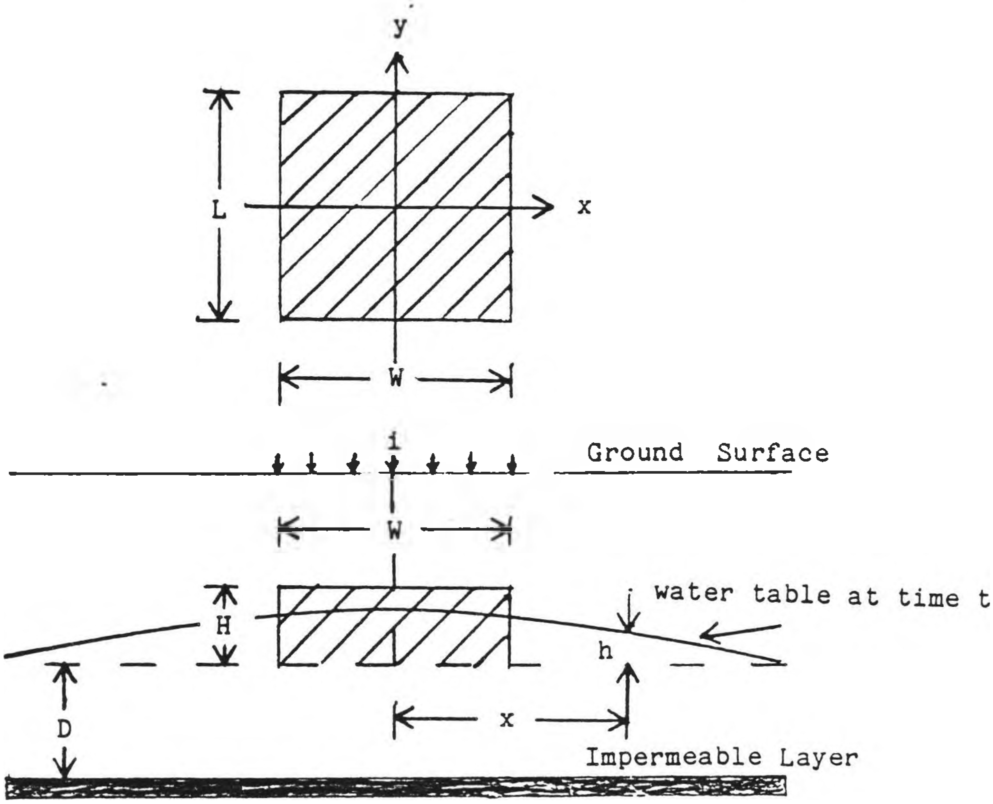

Figure 2-2. Boundary Conditions for Equation 2.1.
(Source: Glover, 1961)

$$u_1 = \frac{(x-W/2)}{\sqrt{4\alpha t(1-\varepsilon)}}, \quad u_2 = \frac{(x+W/2)}{\sqrt{4\alpha t(1-\varepsilon)}}, \quad u_3 = \frac{(y-L/2)}{\sqrt{4\alpha t(1-\varepsilon)}}, \quad u_4 = \frac{(y+L/2)}{\sqrt{4\alpha t(1-\varepsilon)}}.$$

Glover (1961) states that this integral may be evaluated using Simpson's rule and he provides an example to illustrate evaluation of this integral.

Hantush (1967) described the growth and decay of ground water mounds in response to constant recharge from rectangular or circular areas for an aquifer of infinite areal extent. Only the rectangular case will be examined here.

The analysis of the ground water mounding problem solved by Hantush (1967) makes the usual assumptions of an unconfined, homogeneous, isotropic aquifer resting on a horizontal impermeable base. The aquifer coefficients are also assumed constant in time and space with a constant rate of recharge.

Figure 2-3 shows the physical system described by the model. Note that there will be no flow across the x or y axes because of symmetry.

The model is represented by the following boundary-value problem:

$$\frac{\partial^2 Z}{\partial x^2} + \frac{\partial^2 Z}{\partial y^2} + \left(\frac{2W}{K}\right) f(x,y) = (1/\nu)\,\frac{\partial Z}{\partial t} \qquad \text{Eqn. (2.4)}$$

$Z(x,y,0) = 0$

$\partial Z(0,y,t)/\partial x = \partial Z(x,0,t)/\partial y = 0$

$\partial Z(\infty,y,t)/\partial x = \partial Z(x,\infty,t)/\partial y = 0$

for

$Z = h_2 - h_i^2$, $\nu = K\bar{b}/\varepsilon$

$\bar{b} = 0.5[h_i(0) + h(t_1))]$, $\quad t_1$ = period of recharge

K = hydraulic conductivity

ε = specific yield

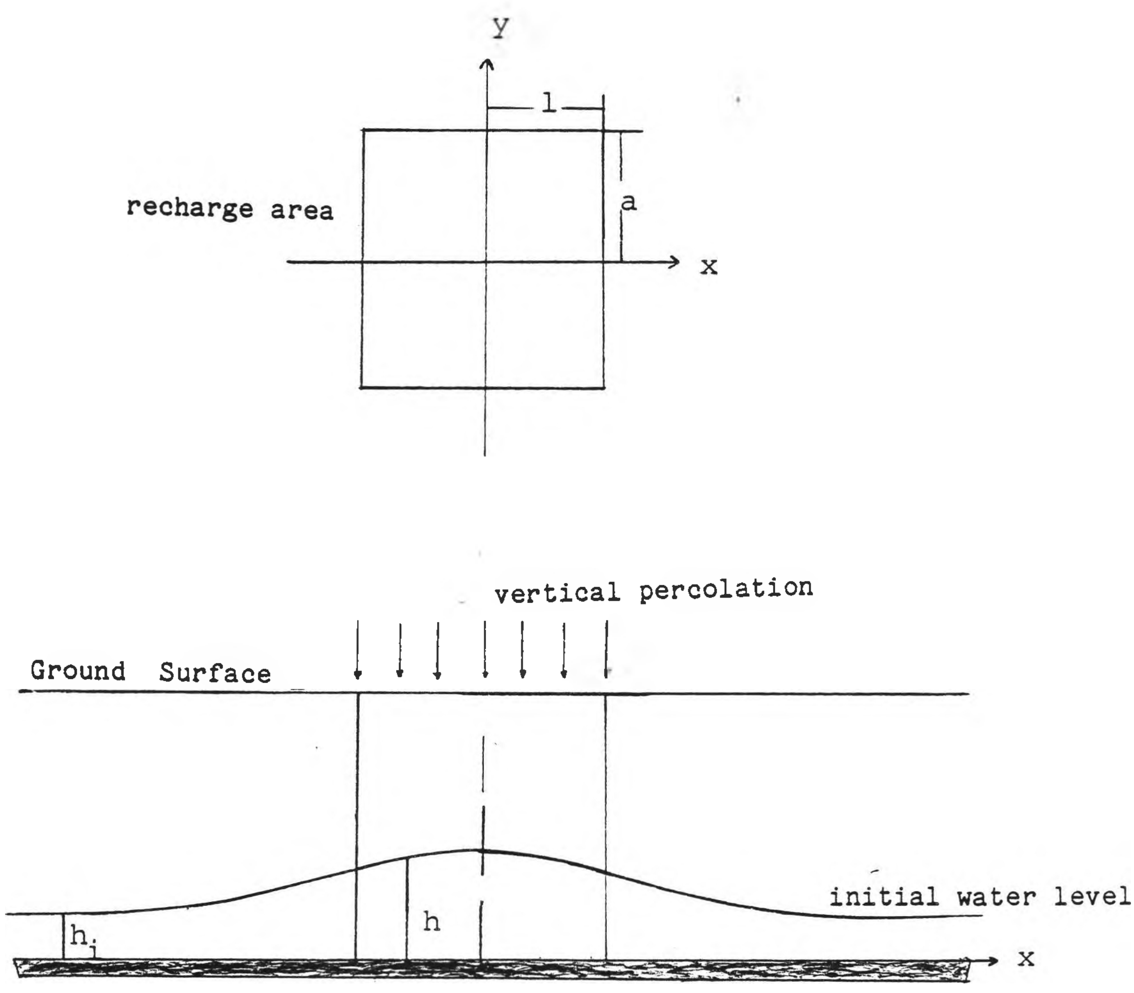

Figure 2-3. Physical System of Hantush Model.
(Source: Hantush, 1967)

W = constant recharge rate

$$f(x,y) = f_1(y) \qquad 0 < x < L$$

$$= 0 \qquad x < L$$

$$f_1(y) = 1 \qquad 0 < y < a$$

$$= 0 \qquad y < a$$

Equation 2.4, with the given boundary and initial conditions, is solved by applying the Laplace transform with respect to t and the Fourier Cosine Transform with respect to x and y. The respective inverse formulae are then used to return to the original variables. The general water table rise is given by:

$$h^2 - h_i^2 = \left(\frac{W}{2K}\right)(\nu t) \quad \left\{S^*\left(\frac{L+x}{\sqrt{4\nu t}}, \frac{a+y}{\sqrt{4\nu t}}\right) + S^*\left(\frac{L+x}{\sqrt{4\nu t}}, \frac{a-y}{\sqrt{4\nu t}}\right)\right.$$

$$\left. + S^*\left(\frac{L-x}{\sqrt{4\nu t}}, \frac{a+y}{\sqrt{4\nu t}}\right) + S^*\left(\frac{L-x}{\sqrt{4\nu t}}, \frac{a-y}{\sqrt{4\nu t}}\right)\right\} \qquad \text{Eqn. (2.5)}$$

where:

$$S^*(\alpha,\beta) = \mathrm{erf}(\alpha)\mathrm{erf}(\beta)+(4/\pi)\alpha\beta W(\alpha^2+\beta^2)$$

$$+(2/\sqrt{\pi})[\alpha\exp(-\alpha^2)\mathrm{erf}(\beta) + \beta\exp(\beta^2)\mathrm{erf}(\alpha)]$$

$$-2[\alpha^2 M^*(\beta/\alpha,\alpha^2)+\beta^2 M^*(\alpha/\beta,\beta^2)]$$

for

$W(x)$ = well function for nonleaky aquifers

$M^*(\alpha,\beta)$ = a function defined by Hantush (1967) and available in tabular form

α & β = transformation parameters given for a large range of values by Hantush (1967) in tabular form

Fielding (1981) described the ground water mounding under a leaching bed. Fielding (1981) made several simplifying assumptions to arrive at a relatively simple model. The maximum water table rise coordinate system is shown in Figure 2-4. For this coordinate system the inflow, Q_i, is assumed to be constant and the outflow, Q_o, at distance "x" is given by Darcy's Law in Equation 2.6.

$$Q_o = KA\left(\frac{h_m - h_x}{x}\right) \qquad \text{Eqn. (2.6)}$$

where:

K = hydraulic conductivity

A = cross-sectional flow area

h_m, h_x, x are as in Figure 2-4.

Assuming radial flow, the cross-sectional flow area equals

$$A = 2\pi x(D + h_x) \qquad \text{Eqn. (2.7)}$$

for D as in Figure 2-4.

Combining Equations 2.6 and 2.7, then rearranging yields

$$(h_m - h_x) = \frac{Q_o}{2\pi K(D+h_x)} \qquad \text{Eqn. (2.8)}$$

At $x=x_m$, $h_x=0$, and assuming $Q_i=Q_o$, the water table rise will be at a maximum which equals

$$h_m = \frac{Q}{2\pi KD} \qquad \text{Eqn. (2.9)}$$

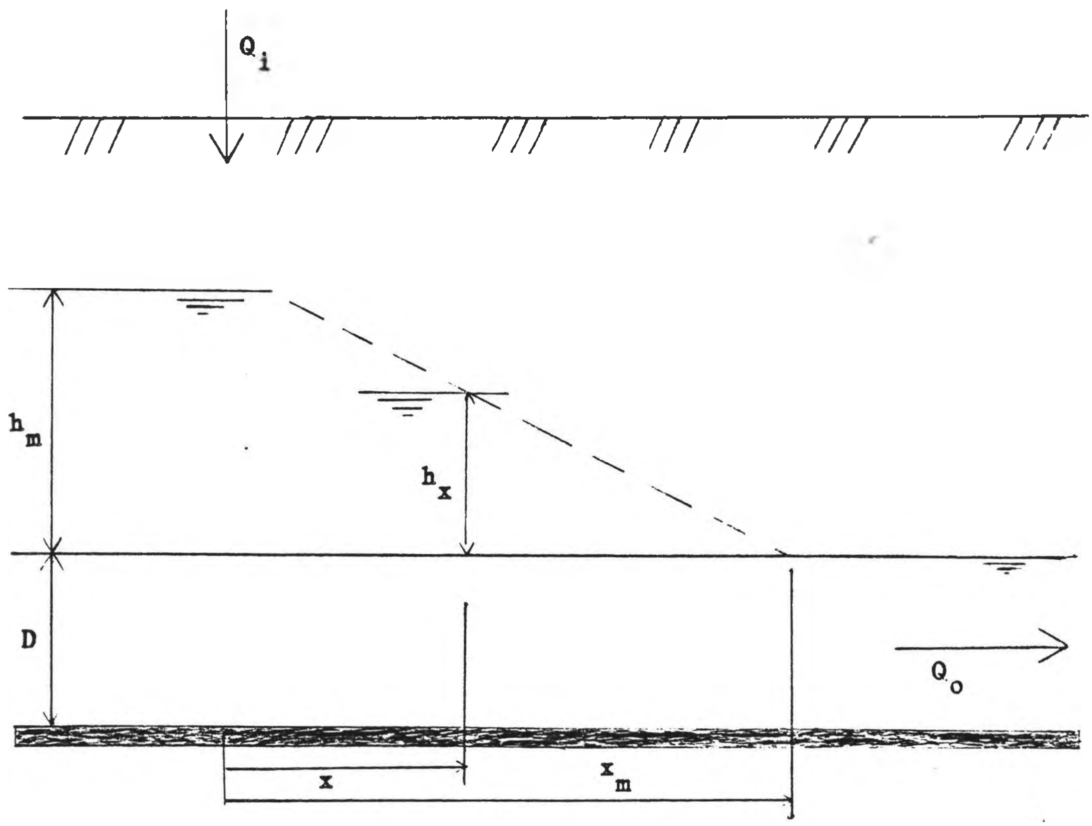

Figure 2-4. Maximum Ground Water Mound Height.
(Source: Fielding, 1981)

The water table rise rate coordinate system is shown in Figure 2-5. Assuming a constant inflow, Q_i, over an area A, with specific yield f, the outflow, Q_o, is variable with time as is the net input Q (Q_i - Q_o). For time Δt

$$\Delta V_1 = Af\Delta h$$

$$\Delta V_2 = Q\Delta t$$

$$= 2\pi KD(h_m - h)\Delta t$$

$$= \Delta V_i$$

therefore,

$$\frac{\Delta t}{\Delta h} = \left(\frac{Af}{2\pi KD}\right)\left(\frac{1}{h_m - h}\right) \qquad \text{Eqn. (2.10)}$$

Integrating Equation 2.10 and evaluating the constant of integration at t=0 and h=0 yields

$$t = \frac{Af}{2\pi KD}\ln\left(\frac{h_m}{h_m - h}\right) \qquad \text{Eqn. (2.11)}$$

Equation 2.11 is then rearranged to give

$$h = \frac{Q}{2\pi KD}\left[1 - \exp\left(\frac{-2\pi KD}{Af}\ t\right)\right] \qquad \text{Eqn. (2.12)}$$

Fielding (1981) tested his results on a large experimental leaching bed with the following physical parameters:

A= 85m X 65m= 5525 sq. m.

K= 0.142 to 2.0 m/d

D= 15 m (average, by seismic sounding)

f= 0.39 (average)

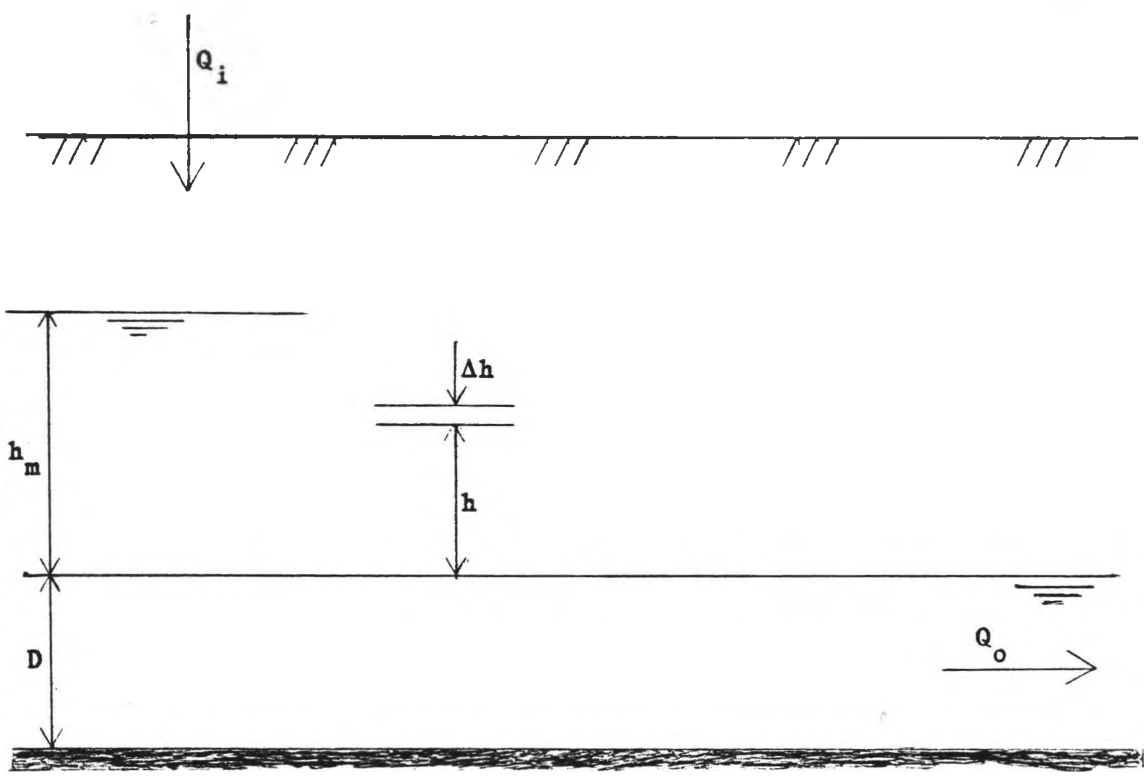

Figure 2-5. Change of h with Time.
(Source: Fielding, 1981)

The test was conducted by applying 123 cu. m/d (0.022 m/d-m^2) of water for 24 days. A regression analysis (r^2 = 0.996) of the observed water table rise gave a value of f= 0.17. Fielding (1981) states that the effects of rainfall and evaporation were negligible over the test period and that "...the high rate (of recharge) is probably responsible for the low value obtained on back calculation of the effective porosity (specific yield), in that the rate at which the soil voids could be filled was slower than the overall application rate."

Fielding's (1981) model appears to be an over simplification of the flow situation occurring beneath a recharge area. The model also appears to be awkward to use due to the "backing out" procedure used to check the accuracy of the model. Finally, the statement that Fielding (1981) makes to justify the poor results obtained from his check of the model seems unlikely since the recharge rate used was only 0.022 m/d-m^2, a relatively small recharge rate.

Rao and Sarma (1981b) studied the growth of a ground water mound in response to a constant recharge from a rectangular area for a finite aquifer. The aquifer in Rao and Sarma's (1981b) model was assumed to have a horizontal, impermeable base. The aquifer was further assumed to have constant parameters and receive a constant rate of recharge.

The ground water flow was described as in Figure 2-6 and the following equation:

$$\frac{\partial}{\partial x}\left(H\frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(H\frac{\partial H}{\partial y}\right) + \frac{p}{K} = \frac{e}{K}\frac{\partial H}{\partial t} \qquad \text{Eqn. (2.13)}$$

where:

K= hydraulic conductivity

e= specific yield

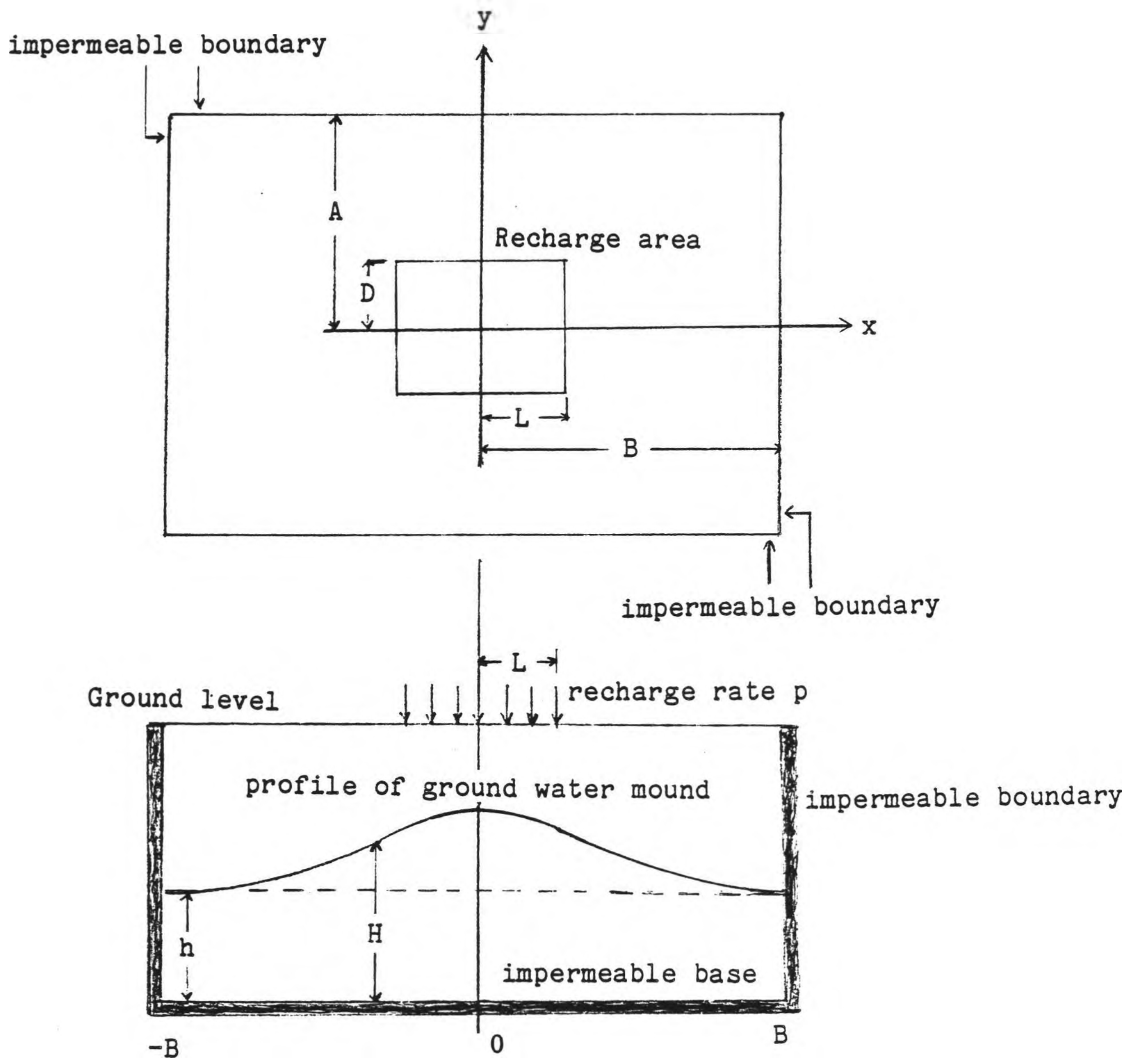

Figure 2-6. Physical System of Rao and Sarma Model.
(Source: Rao and Sarma, 1981b)

p= constant rate of recharge

H, h, x, and y are as in Figure 2-6.

Equation 2.13 was simplified by setting $s = H^2 - h^2$ to give

$$\frac{\partial^2 s}{\partial x^2} + \frac{\partial^2 s}{\partial y^2} + \frac{2p}{K} = \frac{1}{a}\frac{\partial s}{\partial t} \qquad \text{Eqn. (2.14)}$$

where:

$a = K\bar{h}/e$

$\bar{h} = \frac{1}{2}(H+h)$

Rao and Sarma (1981b) took the origin at the center of the recharge area because of symmetry. This means that there will be no flow across the axes, allowing only the positive quarter of the recharge area to be considered. Equation 2.14 was then solved subject to the following boundary conditions:

$$\frac{\partial s}{\partial x}(B, y, t) = \frac{\partial s}{\partial y}(x, A, t) = 0$$

$p = p \qquad \text{for } x \leq L; \quad y \leq D$

$\;\;= 0 \qquad \text{elsewhere}$

$s(x,y,0) = 0$

for A, B, D, and L as in Figure 2.6.

Equation 2.14 and the boundary conditions were solved by the Finite Fourier Cosine Transform and the inverse Fourier Cosine Transform on s(x,y,t) to give:

$$s(x,y,t) = \frac{8p}{K}\frac{A^2B^2}{4}\sum_{m=1}^{\infty}\sum_{n=1}^{\infty}\{\frac{1}{m\;n}\frac{1}{m^2A^2+n^2B^2}$$

$$[1-\exp\{-a\pi^2[(m^2A^2+n^2B^2)t/(A^2B^2)]\}]\ (\sin\frac{m\pi L}{B})$$

$$(\sin\frac{n\pi D}{A})\ (\cos\frac{m\pi x}{B})\ (\cos\frac{n\pi y}{A})\} + \frac{2paLD}{KAB}\ t \qquad \text{Eqn. (2.15)}$$

The accuracy of Equation 2.15 was evaluated by comparing the rise predicted by the equation to field data from Bianchi and Haskell (1975) for an infinite aquifer. The adjustment for an infinite aquifer was made by assigning large values to the ratio A/D (A/D = 50 was found to be an adequate simulation of an infinite aquifer.) The results of this comparison, along with the Glover (1961) model, are shown in Figure 2-7.

The ground water models just reviewed are by no means all the models examined for this design. The models are representative of the types of models available in the literature.

The Rao and Sarma (1981b) model was selected because it closely matches the actual ground water height increase under a recharge area, is relatively simple to program, and only requires a small amount of computer memory. The Glover (1961) and Hantush (1967) models also closely match the actual mound height increase, but are more complex to program and require more memory than the Rao and Sarma (1981b) model. Since the model used was to be incorporated into a computer aided-design package for a micro computer, simplicity in programming and small memory requirements were necessary. The Rao and Sarma (1981b) model, however, must be modified for purposes of this project. During detailed description of the model development in Chapter Three the required modifications will be presented.

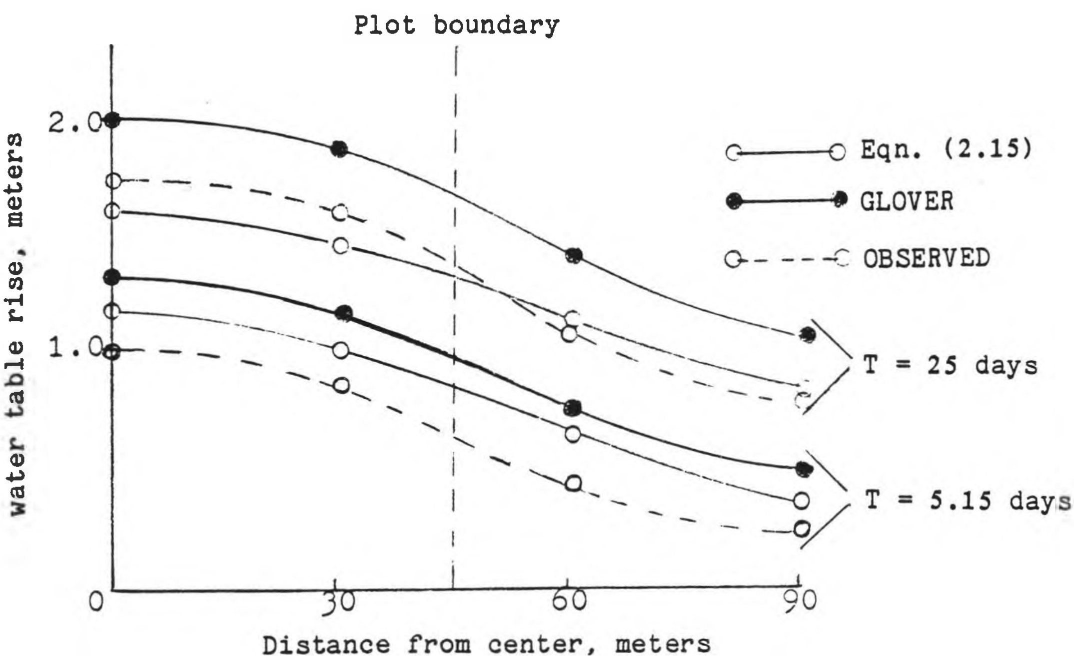

Figure 2-7. Comparison of Predicted and Observed Water Table Profiles. (Source: Rao and Sarma, 1981b)

The Fielding (1981) model, although designed specifically for a leachfield, was not selected because of the poor results obtained when comparing the actual and predicted values of specific yield.

COMPUTER-AIDED-DESIGN OF ON-SITE SYSTEMS

Computer aided design, as a more organized approach to the design of on-site wastewater treatment systems, has only recently been applied to soil absorption systems. This section will review most of the current computer aided design tools available for use in designing soil absorption systems.

Fritton et al. (1983a) presented a report describing a prototype computer information delivery system developed to improve site suitability decision making, on-site effluent disposal system selection, and soil absorption area sizing decisions. Fritton et al. (1983a) cited the poor reliability estimates of on-site disposal systems (less than 50% perform satisfactorily over their design life) as the justification for their efforts. The computer program was written in an interactive, user friendly form such that at the end of a session the user was given a choice of suitable system designs. The report also contains summaries of several individual studies undertaken to provide input concerning data or decision alternatives needed in the program.

Englehardt (1983), while not looking at initial design, reported on the quantification of on-site wastewater treatment operation and maintenance requirements. Englehardt collected on-site operation and maintenance requirement information and embodied this information in an interactive computer program. This program accepts site-specific input data and prints operation and maintenance recommendations and estimated annual costs for the site.

The National Small Flows Clearinghouse has a comprehensive computer listed bibliography of on-site wastewater treatment and disposal papers (Dix, 1984). While this bibliography is not a computer-aided-design in itself, it is a very valuable computer-aided tool to locate information on the design of on-site systems.

CHAPTER THREE

MODEL DEVELOPMENT

For the reasons discussed in Chapter Two, the Rao and Sarma (1981b) model will be modified for use in this design.

The Rao and Sarma (1981b) model is as follows:

$$s(x,y,t) = \frac{8p}{K} \frac{A^2B^2}{\pi^4} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \{\frac{1}{mn} \frac{1}{m^2A^2+n^2B^2}$$

$$[1-\exp\{-a\pi^2\{(m^2A^2+n^2B^2)t/(A^2B^2)\}\}] (\sin \frac{m\pi L}{B})$$

$$(\sin \frac{n\pi D}{A}) \quad (\cos \frac{m\pi x}{B}) \quad (\cos \frac{n\pi y}{A})\} + \frac{2paLD}{KAB} t \qquad \text{Eqn. (3.1)}$$

where:

H = height of water table above the base of the aquifer

K = hydraulic conductivity

e = specific yield

p = constant rate of recharge

h = initial height of the water table

x, y, L, and D as defined in Figure 3-1

$s = H^2 - h^2$

$a = K\bar{h}/e$

$\bar{h} = \frac{1}{2}(H+h)$

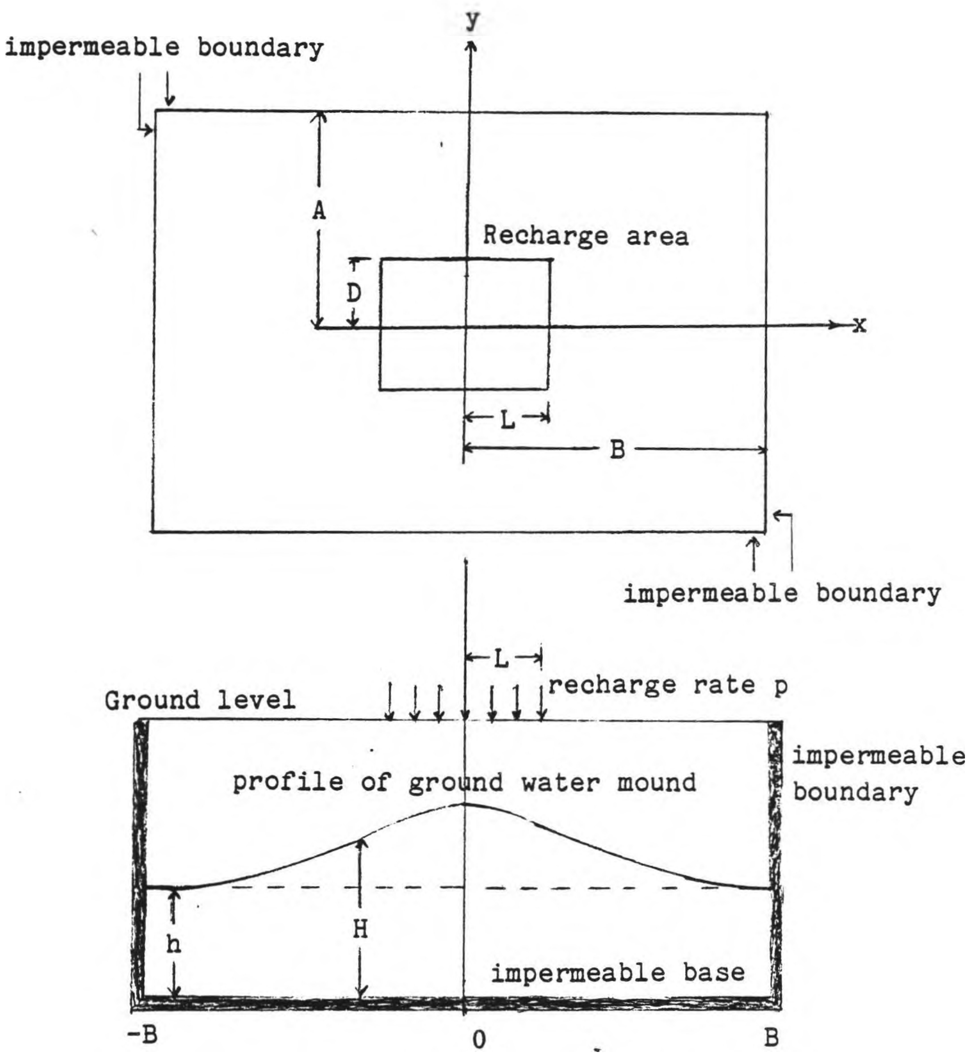

Figure 3-1. Physical System of Rao and Sarma Model.
(Source: Rao and Sarma, 1981b)

Equation 3.1 can be modified for purposes of this research by observing the following:

1. The mound height will be a maximum at $x = y = 0$. At this point the cosine terms in Equation 3.1 become unity; effectively removing these terms from the equation.
2. Equation 3.1 can be solved approximately without summing to infinity. A determination of the summation limits that will approximate summing to infinity must be made.
3. Expanded leachfields normally use a "dosing" procedure consisting of distinct times of wastewater application and rest periods. The model will be tested to determine if, for the same effluent volume, the "dosing" procedure must be accounted for or if it may be approximated by a smaller constant recharge rate.
4. Equation 3.1 is formulated to solve explicitly for $s(x,y,t)$, but it is desired to obtain L and D at $s(0,0,t)$. Since it is not possible to solve Equation 3.1 explicitly for either L or D, it will be necessary to solve for L and D implicitly by an iteration technique. This will be done by assuming L and D equal (i.e. a square leachfield).
5. After the area for a square leachfield has been calculated, the sizes of rectangular leachfields, which will give the same mound buildup, will be determined.
6. Equation 3.1 is formulated for use in a finite aquifer with side boundaries of distances A and B from the center of the recharge area. Rao and Sarma (1981b) found that Equation 3.1 approximated an infinite aquifer for ratios of $A/D > 50$. Since

leachfields should only be used in infinite aquifers, Equation 3.1 will be solved such that the ratios of A/D and B/L are always taken as greater than 50.

The modified equation is as follows:

$$s(0,0,t) = \frac{8p}{K}\frac{A^2B^2}{\pi^4}\sum_{m=1}^{i}\sum_{n=1}^{j}\{\frac{1}{mn}\frac{1}{m^2A^2+n^2B^2}$$

$$[1-\exp\{-a\pi^2[(m^2A^2+n^2B^2)t/(A^2B^2)]\}]$$

$$(\sin\frac{m\pi D}{B})\quad(\sin\frac{n\pi D}{a})\} + \frac{2paD^2}{KAB}\,t \qquad \text{Eqn. (3.2)}$$

i = j = integers after which the summation contribution will be negligible.

The method of solution of Equation 3.2 will be to set the maximum acceptable ground water mound height and calculate the square area needed to develop this height for a given recharge rate. The estimation of this maximum acceptable height will be one of the most important parts of the solution, as it must include both the actual water table height buildup and the corresponding height increase of the capillary fringe, which will also contribute to the saturated zone, although at pressures less than atmospheric (McWhorter and Sunada, 1977).

Of the above adaptations, the cosine adaptation (point 1), the summation limits (point 2), the time adaptation (point 3), and the iteration technique (point 4) will now be addressed. The ground water mound height difference between square and rectangular recharge areas (point 5) will be addressed in Chapter Four.

COSINE ADAPTATION

The origin of the coordinate system used in the Rao and Sarma (1981b) model is at the center of the recharge area. The center of the recharge area is also the point at which the ground water mound height will be a maximum, therefore this will be the critical point in a design concerned with the maximum mound height.

Using the origin as the critical point in the design implies that the values of x and y will be zero in the design. If the values of x and y are always zero in the design then the cosine terms in the model will always be one and have no effect on the maximum mound height.

The fact that the cosine terms are taken equal to one removes them from the numerical calculations, but does not change the fact that the model is two dimensional in space coordinates.

SUMMATION LIMITS

To determine the values of the summation limits that would yield an acceptable approximation of the summation, a small computer program was developed to evaluate the summation portion of the model for different values of m and n. The program was written in such a way that the value of the summation could be examined after each incrementation of m or n to determine if the value of the summation had become approximately constant.

It was originally thought that the values of m and n would be quite small (10 or less) because of the terms in the model involving one divided by m and n squared. It was found, however, that because of the terms involving the exponential raised to a negative power and the sine function, the terms involving one divided by m and n squared did not affect the value of the total summation to the extent expected.

The next step in the determination of the summation limits was to examine the sign of the sine terms. If the sign of successive sine terms had alternated, the convergence value would have been relatively easy to determine. However, the sign of successive sine terms did not alternate, thus eliminating this method for the determination of convergence also.

Figure 3-2 shows a general representation of the results obtained for the summation program. The results shown in Figure 3-2 were similar for a wide range of values of the physical parameters (hydraulic conductivity and specific yield) contained within the summation. It was found that the "approximation value" (the constant value the summation approached at large values of m = n) changed, but that the maximum and minimum points remained at the same values of m and n (i.e. the maximum and minimum points remained at m=n=50.0,100.0,...).

An examination of the summation terms reveals the reason for the results shown in Figure 3-2. At small values of m and n the sine terms dominate the summation, but as the values of m and n increase the sine terms become progressively more dampened by the other terms until the summation approaches a constant.

This knowledge of the approximation value was used, in conjunction with the range of values of aquifer properties considered acceptable for use with a leachfield, to determine the smallest values of m and n that would approximate the summation value as m and n approach infinity. For an infinite aquifer (A/D>50) the values of m and n were found to be m=n=32 for the smallest acceptable aquifer properties and m=n=40 for the largest acceptable aquifer properties.

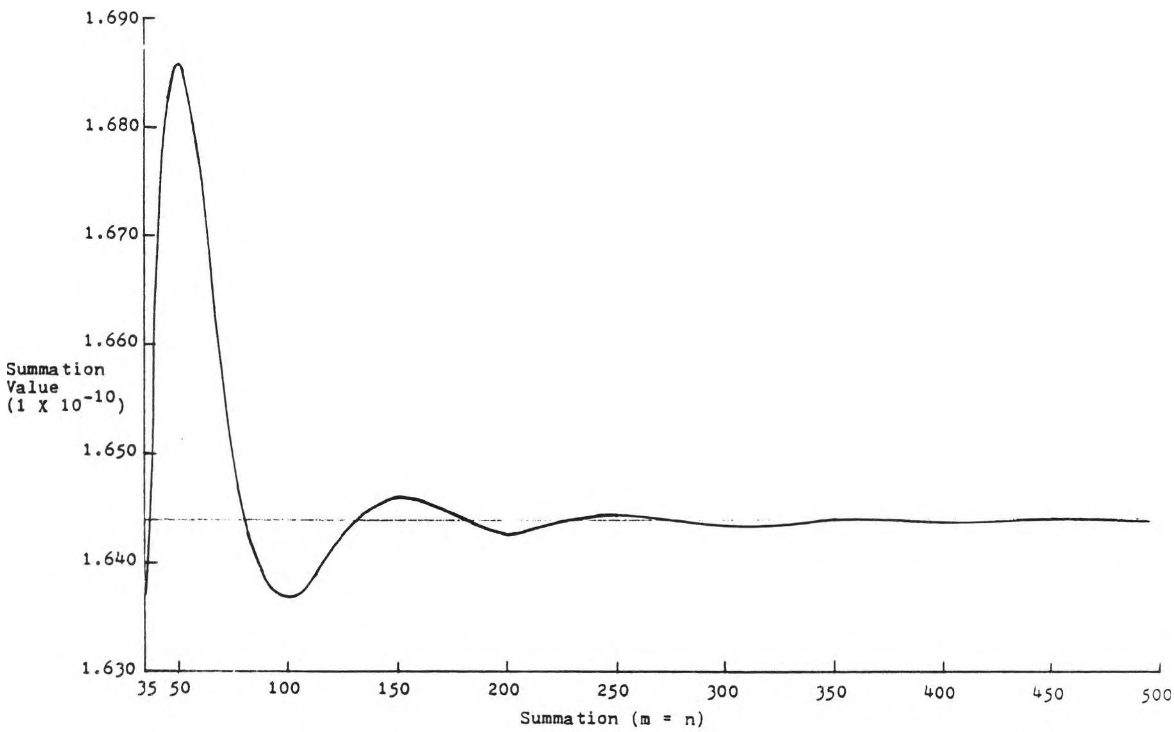

Figure 3-2. Plot of Change in Summation Value with a Change in Summation Limits.

Personal correspondence with Mr. Rao, one of the model developers, indicated that Rao and Sarma (1981b) used a value of m=n=40 in their testing of the model. How they arrived at this value for m and n was not indicated.

The results cited lead one toward the adoption of a value of m=n=36 for use in the design procedure because it represents a median value for the infinite aquifer which should minimize both over and under estimation of the summation value. However, an examination of Figure 3-2 shows that the curve is at its greatest slope here. This means that the summation should be truncated at a larger value of m=n to minimize the error involved in the truncation of the summation before infinity.

The value of the summation, for the data used to generate Figure 3-2, was calculated for values of m=n from 1 to 950 to investigate where the summation should be truncated. It was found that between m=n=900 and m=n=950 the summation value was approximately constant at a value of 1.64366×10^{-10}. The values of m=n which best approximate the constant summation value are shown in Table 3-1.

The data shown in Table 3-1 and factors to be discussed in the section on Time Adaptation caused the summation truncation point to be set at m=n=130 for the design. This value of m=n was chosen for two reasons: 1) it is two orders of magnitude closer to the symmetric value than m=n=36 and one order of magnitude closer than m=n=82, but the same order of magnitude as m=n=179 and m=n=229; 2) it requires 11.82 times as much run time as m=n=36, but only 2.48 times as much run time as m=n=82 and less run time than either m=n=179 or m=n=229. Because of these two factors, it was felt m=n=130 represented the best compromise between accuracy and run time.

Table 3-1. Approximate Values of Summation

Value of m=n	Summation Value (10^{-10})	Absolute Value Difference	Increase in Run Time (%)
36	1.64225	1.4×10^{-13}	0
82	1.64378	1.2×10^{-14}	476
130	1.64362	4.0×10^{-15}	1182
179	1.64365	1.0×10^{-15}	2203
229	1.64367	1.0×10^{-15}	3286

TIME ADAPTATION

The Rao and Sarma (1981b) model was developed using a continuous and constant rate of recharge. Most expanded leachfield systems are designed for use with a "dosing" procedure consisting of distinct times of effluent application and rest periods. Therefore, testing must be done to determine if the "dosing" procedure needs to be accounted for or if it may be approximated by a uniform average recharge rate.

The dosing procedure can be represented graphically as shown in Figure 3-3. Figure 3-3 suggests that a superposition procedure may provide an acceptable method of solution to the dosing problem.

A superposition procedure may only be applied to a linear, homogeneous partial differential equation with linear boundary conditions (Kreyszig, 1979). Since the Rao and Sarma (1981b) model satisfies these conditions in s, a superposition procedure may be applied. The superposition procedure used in the leachfield design was somewhat unorthodox in concept, but was relatively simple to program and more importantly, was quite compact in terms of computer memory.

The usual superposition procedure has a summation of positive and negative terms in increasing order (from 1.0 to 365.0 for example). The superposition procedure used in the design (Equation 3.3) has a summation of positive and negative terms in decreasing order (from 365.0 to 1.0). This decreasing summation makes Equation 3.3 very easy to use with a DO loop and uses a minimum of computer memory, which is not the case with the usual superposition procedure.

$$s(0,0,t) = G(t_E-t_{SL})-G(t_E-t_{eL})+G(t_E-t_{SL-1})-G(t_E-t_{eL-1})+\ldots$$

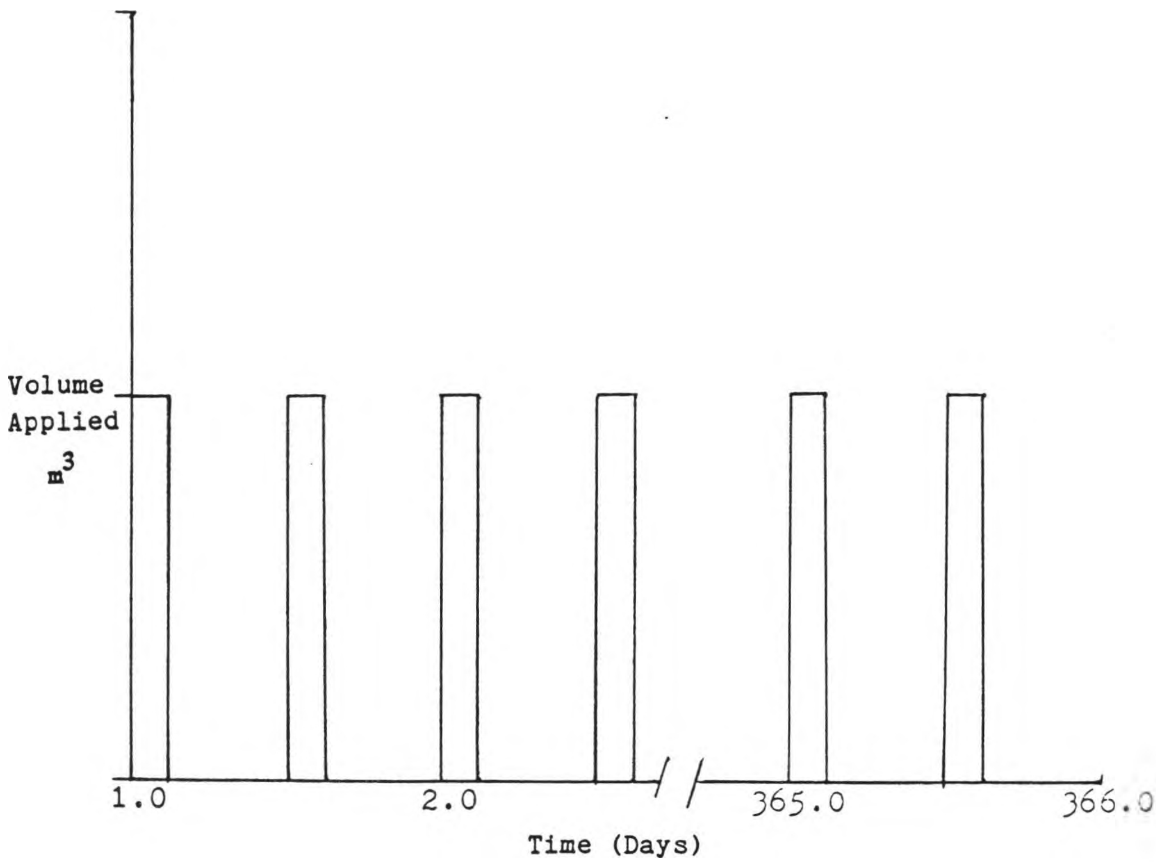

Figure 3-3. Plot of Recharge vs. Time for a Typical Expanded Leachfield.

$$+G(t_E-t_{SL-L})-G(t_E-t_{eL-L}) \qquad \text{Eqn. (3.3)}$$

where:

$G(t)$ = represents Equation 3.2

t_E = time at end of dosing period

t_{SL} = time at start of last dosing period

t_{eL} = time at end of last dosing period

t_{SL-1} = time at start of next-to-last dosing period

t_{eL-1} = time at end of next-to-last dosing period

t_{SL-L} = time at start of first dosing period

t_{eL-L} = time at end of first dosing period

A comparison of results obtained from running the model with the superposition procedure and with an equivalent average constant recharge rate are shown in Table 3-2. The variable values used in both cases were the same, with the exception of recharge rate, and were: specific yield, e=0.089; original saturated thickness, h=4.88 meters (16.0 ft); maximum acceptable mound height, H=5.79 meters (19.0 ft). The recharge rate for the superposition case was set at 0.10 m/d (0.32 ft/d) for two, three hour recharge periods per day for 365 days. The recharge rate for the constant recharge rate was set at 0.025 m/d (0.08 ft/d) for 365 days to give the same recharge volume as the superposition case.

The data in Table 3-2 clearly show that there is no significant difference in the mound height calculated by the superposition procedure and the constant recharge rate case. There is no significant mound height difference between the two cases because the twice daily applications used in the superposition procedure resulted in the soil

Table 3-2. Comparison of Superposition Procedure Constant and Recharge Rate

Hydraulic Conductivity (cm/sec)	Side Length (m)	Area (m^2)	Superposition Mound Height (m)	Constant Rate Mound Height (m)
3.7×10^{-2}	128.0	16,384	5.81	5.87
3.5×10^{-3}	41.5	1,722	5.82	5.84
3.5×10^{-4}	16.9	286	5.82	5.85

merging the individual wetting fronts into one continuous wetting front. If the time between applications was lengthened (to one dosing per week, for example) the wetting fronts would probably not merge and the two procedures would give decidedly different mound heights. Since leach-fields usually dose once or twice a day the constant recharge rate case will be used because it (1) takes less time to run, (2) allows the model to be used once for the entire design life of the system, and (3) allows the summation truncation point to be taken at larger values of m and n because of the shorter run times.

ADAPTATION FOR COMPUTER-AIDED-DESIGN PACKAGE

One of the objectives stated in Chapter One was to incorporate the model in a computer aided design package for use by designers. This design package was developed by using the model to solve, separately, for three different variables which would be of interest to the designer. This section discusses the adaptations made to the model, in addition to those described previously, to solve for a particular variable of interest.

Area

The area portion of the design package determines the square recharge area required to give a specified mound height buildup for a specified set of physical parameters.

Any attempt to solve the model, assuming a square recharge area, by an iterative technique gives rise to two major questions. 1) What iteration technique is best suited to this situation? 2) What is an acceptable accuracy?

Three possible iteration techniques were examined: a) using an array, b) using an incrementation with interpolation (sometimes called a line search), c) using an incrementation.

The use of an array was first considered because it would arrive at a solution with a minimum amount of run time. However, it was not used because it requires more memory than either of the other two methods.

The use of an incrementation procedure until the correct value was bracketed, and then interpolating, was considered next because it would arrive at a solution fairly quickly, but not use a large amount of memory. However, the use of incrementation with interpolation was rejected because it was more complicated and required almost as much memory as the array.

The use of an incrementation procedure alone was selected because it was much less complicated, and used much less memory, than the other two procedures.

The procedure used in the the model is as shown in Figure 3-4. It is seen from Figure 3-4 that the upper and lower limits of acceptability are very important to the iteration procedure. This leads to the consideration of the second major question stated previously regarding an acceptable accuracy level for the solution.

It was stated in Chapter Two that an unsaturated zone thickness of 1.22 meters (4 ft) would be specified for this design. The "window" of acceptable solutions was set as $\pm$ 0.15 meters ($\pm$ 0.5 ft) so that the minimum saturated thickness possible would be 1.07 meters (3.5 ft). This will insure that an unsaturated zone of adequate thickness (i.e. sufficient filtration of wastewater) is provided.

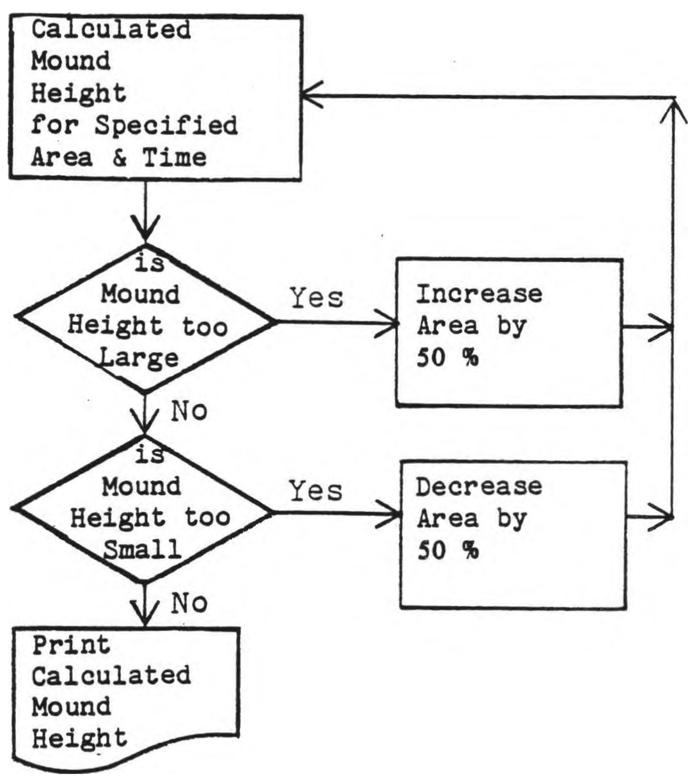

Figure 3-4. Flow chart of the Incrementation Procedure Used.

The iteration window limits were chosen such that they would correspond with a reasonably accurate measurement of such physical parameters as saturated hydraulic conductivity, specific yield, and aquifer thickness. It is realized that in most cases the measurement of the previously stated physical parameters is not very precise. This gives rise to the possibility of a false sense of security in the accuracy of the solution (i.e., the answer appears to be more precise than is possible because of imprecision in the measurement of the physical parameters which go into the answer). A warning to the design package users must be provided to place this apparent accuracy in proper perspective.

Height

The height portion of the design package determines the mound height build-up under a rectangular recharge area for a specified set of physical parameters (including the length and width of the recharge area). This portion of the design package was intended primarily for the determination of the mound buildup beneath existing leachfields.

The procedure used here was not to treat the mound height, H (defined in Figures 2-6 and 3-1), as a known, constant quantity, as was done in the other two parts of the design package, but to treat the mound height as variable. The value of H may now be changed until an acceptable solution is reached. This is essentially the same procedure Rao and Sarma (1981b) used in testing their model.

The procedure adopted for use in the design package consists of increasing the maximum ground water mound height, H, by a user specified increment from the original saturated thickness until the assumed maximum mound height approximately coincides with the mound height calculated by the model. When the difference between the calculated and assumed mound heights is less than 1.0 millimeter the program exits the iteration procedure and prints the calculated ground water mound height.

Distance

The distance portion of the design package is intended to be used for determination of the separation distance required between adjacent leachfields. This portion of the design package determines the distance at which the ground water mound height increase is negligible (less than 1.0 centimeter). If the mound height does not become less than 1.0 centimeter within the specified side-boundaries, the program prints the mound height at the side-boundary and stops.

Determination of negligible buildup is accomplished by re-inserting one of the cosine terms from the original model, Equation 3.1, into the adapted model, Equation 3.2. The distance value is now set equal zero (coordinate at the center of the mound) for the first run and is increased by a user specified increment until the height increase is equal to or less than 1.0 centimeter. The program prints the ground water mound height at each distance increment.

CHAPTER FOUR

SENSITIVITY ANALYSIS

With the model constructed as described in the previous chapter, it must now be tested to establish its accuracy and its sensitivity to variations in model parameters. First the model is verified by comparing actual field measurements of mound height buildup and recharge area with model results for these quantities. The model is then tested to evaluate its sensitivity to variations in the input parameters.

MODEL VERIFICATION

To insure that there were no errors in the program the model was tested with a time constant recharge rate using the data from Bianchi and Haskell (1975). The input data was as follows: hydraulic conductivity = 3.7×10^{-2} cm/s (52.0 in/hr), specific yield = 0.089, recharge rate = 9.7 cm/d (0.32 ft/d), time of recharge = 5.15 days, and an original water table height of 4.88 meters (16 ft). It was found that both the height and the area portions of the model calculated values within 3.0% of the measured values, as shown in Table 4-1.

The data in Table 4-1 shows that, for a constant recharge rate, the model does an accurate job of estimating either the mound height or the recharge area.

EVALUATING PARAMETER SENSITIVITY

The sensitivity testing shown in the following sections deals with the change in recharge area with a variation of a particular physical

Table 4-1. Comparison of Measured and Predicted Values

	Measured Value m (ft)	Calculated Value m (ft)	Percent Error
Height	5.79 (19.0)	5.96 (19.5)	2.8
Side Length	90.0 (295.0)	91.4 (300.0)	1.7

parameter. The reader will recall that there are three options contained within the design package; area, height, and distance. Because all three options of the design package use the same equation, the sensitivity of any one option to parameter variation will parallel the sensitivity of the other options; therefore, only the testing with the area option is shown here.

Change of Area with Hydraulic Conductivity

In testing the sensitivity to hydraulic conductivity, all the variables within the area model were held constant except hydraulic conductivity. The specific yield was set at 0.05 because this was felt to be about as small a value of specific yield as would be practical for use with an expanded leachfield. The original saturated thickness was taken from the Bianchi and Haskell (1975) data as 4.88 meters (16 ft). The daily effluent volume was established by assuming an effluent generation rate of 0.284 cubic meters per capita per day (75 gpcd) for a population of 250 people to give a total daily volume of 71 cubic meters (18,750 gal). The time was set as 3650 days (10 years). The maximum acceptable mound buildup was also taken from the Bianchi and Haskell (1975) data as 5.79 meters (19 ft) $\pm$00.15 meters ($\pm$0.5 ft). The results are shown in Table 4-2.

One may observe from Table 4-2 that as hydraulic conductivity decreases the recharge area required increases. This is expected because, for the same effluent volume applied, the effluent with a high (4.2×10^{-2}) hydraulic conductivity should flow away from the recharge more easily and quickly than with a low (5.0×10^{-3}) hydraulic conductivity. Thus, for the same mound height increase, less area is

Table 4-2. Change in Recharge Area with a Change in Hydraulic Conductivity.

Hydraulic Conductivity cm/sec (in/hr)	Side Length m (ft)	Area m^2 (ft^2)	Mound Height m (ft)
4.2×10^{-2} (59.5)	51 (167)	2,583 (27,889)	5.83 (19.13)
2.97×10^{-2} (42.1)	54 (177)	2,930 (31,329)	5.80 (19.03)
1.73×10^{-2} (24.5)	72 (236)	5,208 (55,696)	5.66 (18.57)
5.0×10^{-3} (7.1)	458 (1,503)	209,556 (2,259,009)	5.93 (19.46)

needed with a high value of hydraulic conductivity than with a low value of hydraulic conductivity.

Table 4-2 also reveals that the model is more sensitive to variations in hydraulic conductivity on the high end of the scale considered than on the low end. This indicates that the most accurate measurements of hydraulic conductivity possible need to be made to avoid undersizing the leachfield.

The last recharge area required in Table 4-2 appears to be very large. Laak (1980) states that soil absorption areas should not be used in areas with a hydraulic conductivity of less than 5×10^{-3} cm/s. The results shown in Table 4-2 suggest that for large soil absorption systems, this is the extreme lower limit of hydraulic conductivity that should be considered acceptable. A more reasonable lower limit on hydraulic conductivity for use with large soil absorption systems would be 1.0×10^{-2} cm/s.

Change of Area with Specific Yield

In testing the sensitivity to specific yield, all the variables were held constant except specific yield. The values used for the variables were the same as in the previous section with hydraulic conductivity set at $K=2.97 \times 10^{-2}$ cm/s (42.1 in/hr). The results are shown in Table 4-3.

One may observe from Table 4-3 that there is an inverse relationship between recharge area and specific yield. This inverse relationship between specific yield and recharge area means that if measured values of specific yield vary greatly over the proposed recharge area, taking the smallest value of specific yield will insure

Table 4-3. Change in Recharge Area with a Change in Specific Yield.

Specific Yield	Side Length m (ft)	Area m^2 (ft^2)	Mound Height m (ft)
0.01	124 (407)	15,378 (165,649)	5.77 (18.93)
0.05	54 (177)	2,930 (31,329)	5.80 (19.03)
0.10	36 (118)	1,302 (13,924)	5.89 (19.32)
0.20	25 (82)	646 (6,724)	5.89 (19.32)
0.30	21 (69)	454 (4,761)	5.85 (19.19)

that the design will provide an adequate area to treat and dispose of the expected effluent volume.

The previous discussion assumes that space for the recharge area is unlimited, which is not always a realistic assumption. When space is limited it is possible to use an average value of specific yield which should give acceptable treatment and disposal of the effluent, but with a reduced safety factor.

Change of Area with Varying Acceptable Mound Buildup

In this sensitivity test the maximum mound height increase was changed to determine what effect this had on the recharge area. The variable values were the same as those used previously with specific yield taken as 0.05. The results are shown in Table 4-4.

Table 4-4 shows that as the maximum mound height increases, the recharge area decreases. This is logical because, for the same effluent volume applied, less recharge area is required to give a higher mound height increase. This is due to the greater water storage possible in the vertical direction with a higher mound height.

Change of Area with Effluent Volume Applied

Effluent volume applied was varied to determine the sensitivity of the recharge area to such changes. The variable values used were the same as those stated before with the maximum mound buildup set as 5.79 meters (19.0 ft). The results are shown in Table 4-5.

From Table 4-5 it may be shown that, for effluent flow volumes of 142 m^3/d or below the relationship between effluent volume and recharge area required is linear (r=0.989). This implies that one of the easiest ways to reduce the recharge area required is to reduce the flow volume

Table 4-4. Change in Recharge Area with a Change in Maximum Mound Height.

Maximum Mound Height m (ft)	Side Length m (ft)	Area m^2 (ft^2)	Calculated Mound Height m (ft)
5.18 (17.0)	200 (656)	40,000 (430,336)	5.16 (16.93)
5.49 (18.0)	72 (236)	5,124 (55,696)	5.51 (18.08)
5.79 (19.0)	54 (177)	2,930 (31,329)	5.80 (19.03)
6.10 (20.0)	44 (144)	1,970 (20,736)	6.14 (20.01)

Table 4-5. Change in Recharge Area with a Change in Effluent Volume Applied.

Effluent Volume Applied m^3/d (gpd)	Side Length m (ft)	Area m^2 (ft^2)	Mound Height m (ft)
35.5 (9,375)	36 (118)	1,302 (13,924)	5.79 (19.00)
71 (18,750)	54 (177)	2,930 (31,329)	5.80 (19.03)
142 (37,500)	98 (322)	9,683 (103,684)	5.75 (18.86)
284 (75,000)	246 (807)	60,516 (651,249)	5.88 (19.29)

to the area. Reduction of the flow volume applied is accomplished, in general, by two major approaches.

One method of reducing the volume applied is to install water conservation devices in the homes served. Rubin and Carlile (1981) reported on the use of water conservation devices for recharge area size reduction for a small factory in North Carolina. Fritton et al. (1983b) reported on rehabilitating failing leachfields by the use water conservation devices also. Both of the above cited studies indicated favorable results were obtained with the use of flow reduction devices along with good user acceptance.

The second major method of reducing the volume applied is to divide the flow between several recharge areas. Flow division may be accomplished by having several recharge areas with individual collection systems (Abney, 1978) or by having one collection system with several recharge areas (Otis, 1978). Primary factors influencing the selection of a flow reduction method appear to be cost, site topography, and land availability.

Since reducing the volume applied is effective in reducing the recharge area required it should be strongly considered as a means for areas with low hydraulic conductivities (not lower than 5×10^{-3} cm/s), low specific yields (below 0.05), or high water tables (0.305 meters maximum acceptable rise) to be used with soil absorption systems without needing unreasonably large recharge areas.

Limits on Use of Large Scale Systems

The previous sections have revealed two areas where the model would probably indicate a site not being appropriate for a recharge area.

1. Where a small increase in mound buildup is acceptable.

2. Where the value of hydraulic conductivity is small ($K < 5 \times 10^{-3}$ cm/s).

If only a small ground water mound buildup is acceptable the corresponding recharge area will be large. This is because where a large increase in mound height is available there is more water storage space in the vertical plane. More storage is needed in the horizontal plane when only small ground water mounds can be tolerated. In this case there are two options: 1) reduce the effluent volume applied to the recharge area by methods previously discussed or 2) use another type of wastewater treatment and disposal system, such as a lagoon system.

If a small value of hydraulic conductivity is found at a recharge site, it should be considered unacceptable for use as a large scale soil absorption system. If this is the case, another type of waste water treatment and disposal system should be used.

Square vs. Rectangular Recharge Areas

Chapter Three stated that after the area needed for a square leachfield was determined, the area of a rectangular leachfield necessary to yield the same mound buildup would be calculated. To this end the mound buildup for a square leachfield was calculated. Next the mound buildup for rectangular leachfields, with the same area as the square leachfield, was calculated for various length-to-width ratios. The results of some of these calculations are shown in Figure 4-1.

It is seen from the above figure that a square leachfield has the greatest mound buildup. This means that for a rectangular leachfield to achieve an equal mound buildup with a square leachfield, the rectangular leachfield must have a larger effluent volume applied than the square leachfield. Since there is no special requirement to achieve the same

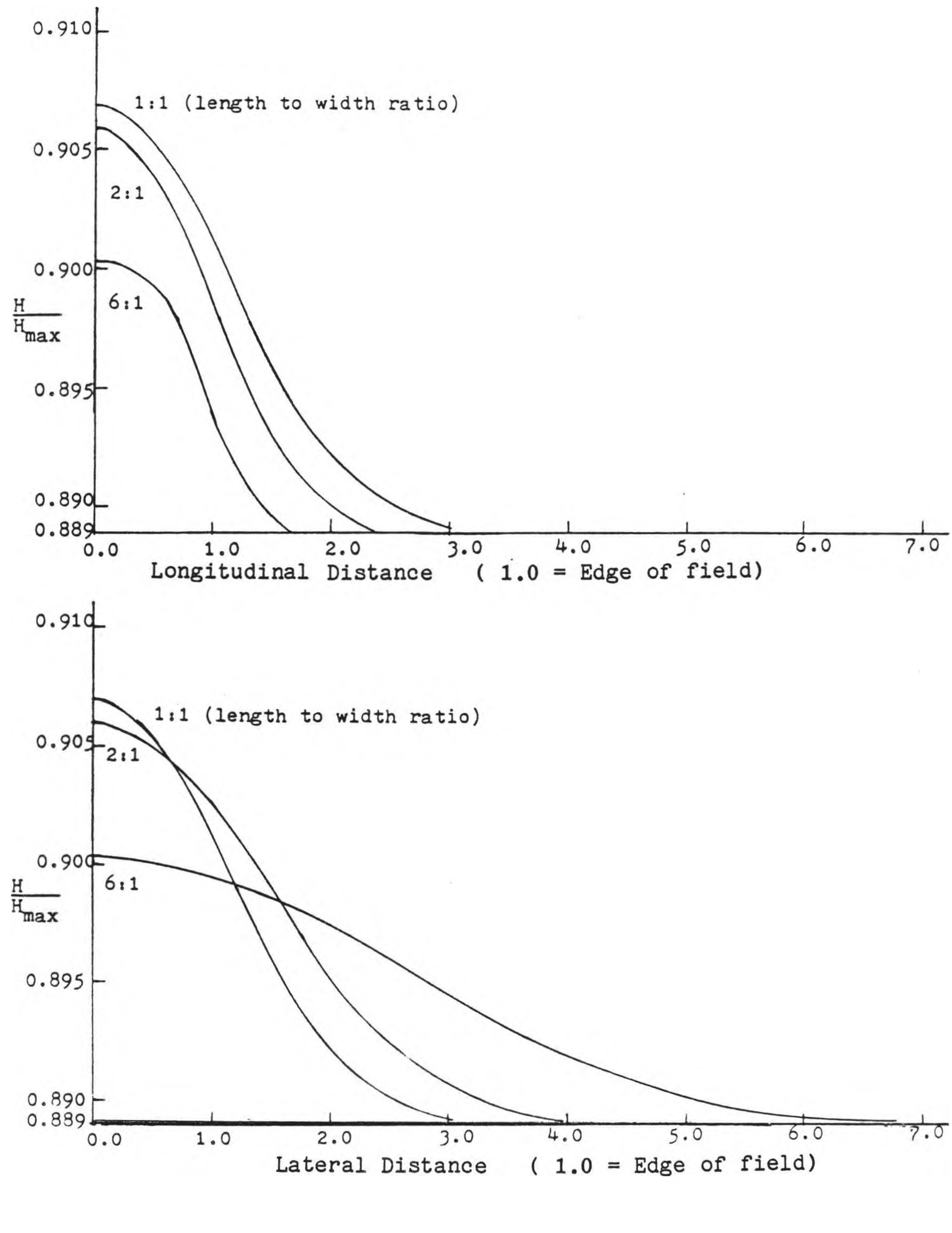

Figure 4-1. Dimensionless Mound Profiles for Square and Rectangular Recharge Areas.

mound buildup, using the area calculated for a square leachfield to size a rectangular leachfield is acceptable. There are two major advantages in using the calculated square leachfield area to size rectangular leachfields.

<u>Capillary Fringe</u>: As stated in Chapter Two, the capillary fringe needs to be accounted for in estimating the acceptable mound buildup. If the capillary fringe is not accounted for or is underestimated, the lower mound height under a rectangular leachfield will serve as a safety factor to insure that an acceptable mound buildup is maintained.

<u>Precipitation Events</u>: It is assumed that some allowance for the infiltration and deep percolation of precipitation (as opposed to runoff) is made when estimating flow volume for the system in humid areas. However, in arid or semi-arid areas little or no allowance may be made for deep percolation. If no allowance is made for deep percolation from precipitation events, the lower mound height will again serve as a safety factor for the increased mound buildup after precipitation events.

CHAPTER FIVE

INTERACTIVE USE OF THE DESIGN PACKAGE

This chapter is an overview of the use of the design package. The first section is a discussion of the applications and the data required for each of the three options contained in the design package. A discussion of the hardware required to run the package is presented in the second section. The third section presents a discussion of a few of the methods available to estimate the physical parameters required to run the design package. The final section is an illustration of how the "Area" option of the design package could be used to estimate the leachfield area required.

DESIGN PACKAGE OPTIONS

This section deals with some of the expected uses of the three options contained in the design package as well as showing the data required to run each option.

Area

The Area option of the design package was developed to estimate the leachfield area required for a large scale soil absorption bed. It is anticipated that the Area option will be used by engineers designing a single soil absorption area to serve a small community. The Area option could also be used by a regulatory agency to determine if a soil absorption area design submitted for approval was appropriate for proper removal of the wastewater in a timely manner.

Figure 5-1 is a copy of the introduction and data required statements that greet the user upon entering the Area design option. The data required have been defined previously. Estimation of the required input parameters is discussed later in this chapter.

Height

The Height option of the design package was developed to estimate the height of the ground water mound built up beneath the leachfield at a particular time. This option could be used by a regulatory agency to check the ground water mound buildup under a proposed or existing large scale soil absorption bed. It could also be used by a designer to check the mound buildup beneath a leachfield designed by another method (using the EPA's percolation rate design table (EPA, 1980) for example). A copy of the introduction and data required statements greeting the user entering the Height design option is shown in Figure 5-2.

Distance

The Distance option of the design package was developed to estimate the distance at which the ground water mound buildup could be considered negligible (less than 1.0 centimeter). This option calculates the ground water mound buildup at user specified distances from the center of the leachfield until the mound buildup is less than 1.0 centimeter.

The Distance option would be useful in establishing the separation distance between two or more adjacent leachfields. It would also be useful in determining the effect a leachfield would have on other ground water influences, such as streams and wells.

Figure 5-3 is a copy of the introductory and data requirement statements greeting the user of the Distance option.

```
THIS PRORGAM IS DESIGNED TO ESTIMATE THE AREA
REQUIRED FOR A LARGE SCALE LEACHFIELD.  THE PRIMARY
DESIGN CONSIDERATION IS THE MAINTENANCE OF AN
UNSATURATED ZONE OF ADEQUATE THICKNESS BENEATH
THE LEACHFIELD.

ENTER THE FOLLOWING DATA

FIRST ESTIMATE OF ONE-HALF FIELD SIDE LENGTH(M)

NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD

VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)

SPECIFIC YIELD

HYDRAULIC CONDUCTIVITY (CM/SEC)

ORIGINAL WATER TABLE HEIGHT (M)

MAXIMUM ACCEPTABLE WATER TABLE HEIGHT (M)
```

Figure 5-1. A Copy of the Area Welcome and Input Screens.

```
THIS PROGRAM IS DESIGNED TO ESTIMATE THE HEIGHT OF
THE GROUND WATER MOUND THAT DEVELOPES BENEATH A
LARGE LEACHFIELD.

ENTER THE FOLLOWING DATA

ONE-HALF LENGTH (M) OF EACH SIDE OF FIELD SEPERATED
BY A COMMA.

NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD

VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)

SPECIFIC YIELD

HYDRAULIC CONDUCTIVITY (CM/S)

ORIGINAL WATER TABLE HEIGHT (M)

DESIRED INCREMENTATION ON RISE OF WATER TABLE
HEIGHT (E.G. 0.10 M, 0.20 M, ETC: NOTE; LARGER
RUN TIMES ARE REQUIRED FOR SMALLER VALUES OF THIS
INCREMENTATION.)
```

Figure 5-2. A Copy of the Height Welcome and Input Screens.

```
THIS PROGRAM IS DESIGNED TO ESTIMATE THE HEIGHT OF
THE GROUND WATER MOUND BENEATH A LARGE LEACHFIELD AT
USER SPECIFIED POINTS ALONG THE LEACHFIELD AXIS.

ENTER THE FOLLOWING DATA

ONE-HALF LENGTH (M) OF EACH SIDE OF FIELD SEPERATED
BY A COMMA (ENTER SEPERATION CALCULATION DISTANCE AS
SECOND HALF LENGTH)


NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD

VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)

SPECIFIC YIELD

HYDRAULIC CONDUCTIVITY (CM/S)

ORIGINAL WATER TABLE HEIGHT (M)

MAXIMUM WATER TABLE HEIGHT (M)

INCREMENT ON DISTANCE FOR SEPERATION CALCULATIONS
(10M, 25M, ETC: NOTE: LARGER RUN TIMES ARE REQUIRED
FOR SMALLER VALUES OF THIS INCREMENTATION.)
```

Figure 5-3. A Copy of the Distance Welcome and Input Screens.

HARDWARE REQUIRED TO RUN THE DESIGN PACKAGE

The design package was developed on a Cyber 205 mainframe computer in FORTRAN 77. The package was then converted to Microsoft FORTRAN77 and run on an IBM Personal Computer XT.

The design package executable statements require approximately 115 kilobytes (k) of memory with each design option requiring approximately 38.333 k. Thus, the entire design package will fit on a single, single sided diskette (175 k available) with enough space left to include a users manual on the diskette.

From the above discussion it can be seen that any IBM compatible micro computer with a Microsoft FORTRAN77 capability and 64 k of memory should be able to run the design package. The specification of 64 k of memory results from the package requirements for the storage of program generated values and the storage requirements of internal machine operations.

At this point it should be said that, although a printer is not required to run the design package, it is convenient to have a printed copy of the user's manual to refer to. It is also much more convenient to have a printout of the results of the Distance option because of the table of values generated.

ESTIMATING REQUIRED PARAMETERS

The accuracy of any design is only as good as the accuracy of the physical parameters which go into it. This section is presented to give the reader an idea of some of the methods available to estimate the required parameters.

Time

The time after recharge begins, at which it is desired to know the ground water mound height, is one of the most important parameters needed for the design. It is recommended that the time used be the design life of the leachfield.

The only exception to using the design life of the field for the value of time in the design package would be when it is anticipated that the field will be removed from service for extended periods (one year or more). In this case the sum of the years that the field will not be in service should be subtracted from the design life and this number be used as the time value. However, if there are several expanded soil absorption beds adjacent to one another (as in the Westboro case) which have the flow alternated between them on a yearly basis, one should consider this a single leachfield and use the field design life as the time value.

Flow Volume

Wastewater flow is essentially equal to water use when there is no lawn sprinkling or other consumptive use and when infiltration to and exfiltration from the collection system is negligible (Clark et al., 1977). It is also generally reported that 60-70 % of the total water supplied becomes wastewater (Clark et al., 1977).

In light of the above facts it is recommended that the average water use per day of the community be determined by monitoring the water supply system. If no daily water use data is available, a defensible estimate of flow, such as that suggested by the state health department, should be used.

Monitoring water supply will probably overestimate the wastewater flow slightly because of consumptive uses, but this overestimation should help to account for infiltration into the collection system. Using the average water use per day to estimate flow volume will not make any allowance for precipitation or system expansion. However, these factors are influenced so greatly by climate and location that a decision of how to account for them is best left to the individual designer.

Specific Yield

Specific yield is a term that is defined as the difference between the porosity and the specific retention of a soil (McWhorter and Sunada, 1977). A similar parameter is the apparent specific yield, which is defined as the ratio of the volume of water added or removed from the saturated aquifer to the resulting change in aquifer volume below the water table (McWhorter and Sunada, 1977). Specific yield may be thought of as the theoretical value while apparent specific yield is the field value.

Specific yield may be estimated from a laboratory analysis by determining the porosity and specific retention (field capacity) of a soil. Apparent specific yield may be estimated by the Theis method, the Jacob method, or the distance-drawdown method discussed by McWhorter and Sunada (1977) in their treatment of aquifer tests.

Hydraulic Conductivity

There are a variety of methods (both field and laboratory) available to estimate the hydraulic conductivity of a soil. Field methods of determining hydraulic conductivity are generally preferred for use with unconsolidated material because laboratory methods usually destroy any anisotropy present in the sample.

A discussion of several field methods for estimating hydraulic conductivity is presented in most general ground water hydrology texts. McWhorter and Sunada (1977) discuss, for example, five methods by which one can estimate the hydraulic conductivity of an aquifer; the Theis method, the Jacob method, the distance-drawdown method, the recovery test, and the slug test.

Aquifer Thickness

The depth to the impervious aquifer bottom may be obtained from a test hole boring or from the well logs of adjacent water wells. If a test boring is used, care should be taken to insure that an actual impervious layer is encountered, rather than a clay lens or other small impervious abnormality.

Capillary Fringe Height

The height of the capillary fringe is not essential for the design, but as discussed earlier, it is important to include it in the acceptable mound height increase.

One method for estimating the capillary fringe height was presented by McWhorter and Nelson (1980).

$$h_d = 9.66 \left(\frac{K}{S_{ya}}\right)^{-0.401} \qquad \text{Eqn. (5.1)}$$

h_d = capillary fringe height, cm

K = saturated hydraulic conductivity, cm/s

S_{ya} = specific yield

McWhorter and Nelson (1980) found the correlation coefficient of Equation 5.1 to be $r^2 = 0.878$ over a range of measured capillary fringe heights of 13 to 240 centimeters of water.

Maximum Acceptable Ground Water Height Buildup

The maximum acceptable ground water height buildup should be calculated as the distance from the base of the aquifer to a point that is the capillary fringe height plus 1.22 meters (4 ft) below the bottom of the leachfield. This should insure that an unsaturated zone of sufficient thickness for proper treatment of the effluent is provided beneath the leachfield.

Comment

The preceding methods of parameter estimation were provided only for discussion and should not be used to the exclusion of other valid methods with which the designer is more comfortable.

One source of information in parameter estimation that should not be overlooked is local experience. The knowledge of local persons experienced in working with the required parameters (such as Agronomists, Geologists, and Engineers) should be used, if it is available.

EXAMPLE APPLICATION

An example of how the design package is used is presented in this section. This is accomplished by running the Area option of the design package with hypothetical data.

A 30 unit housing development is proposed for installation on a site with no obvious ground water flow boundaries. An aquifer test on the proposed site revealed that the aquifer saturated thickness was approximately 6.71 meters (22 ft) and that the depth to the base of the aquifer was approximately 9.75 meters (32 ft). Analysis of the drawdown data, by the Theis method, yielded the following results: the storage coefficient (apparent specific yield) is estimated to be 0.09 and the transmissivity (hydraulic conductivity multiplied by the aquifer thickness) is estimated as 8.72 cm^2/sec (0.56 in^2/min) to give a hydraulic conductivity of

$$8.72\ cm^2/s \div 671\ cm = 1.3 \times 10^{-2}\ cm/s\ (18.4\ in/hr).$$

If it is assumed that the bottom of the leachfield is 1.52 meters (5 ft) below the ground surface and the capillary fringe is calculated from Equation 5.1 as 0.21 meters (0.69 ft), the maximum acceptable ground water height increase is calculated as

$$9.75\ m - 1.52\ m - 0.21\ m = 8.02\ m.$$

Once the aquifer related properties have been established, a determination of the flow volume and design life of the leachfield must be made. Since there are no records of water use for this proposed community, the daily volume of effluent applied to the leachfield must be estimated. Information from the developer indicates that all 30 units in the development are to have 3 bedrooms. Assume that the State Health Department estimates wastewater generation to be 0.57 cubic meters (150 gal) per bedroom per day, from which the daily effluent volume is calculated as

$$0.57\ m^3/d\ (30\ units * 3\ bedrooms/unit) = 51.3\ m^3/day.$$

The design life of this system is established as 20 years (7300 days).

The data required to run the Area option is listed, in the order it is needed, in Table 5-1.

Figure 5-4 is a copy of the welcome, input, and output screens for this example.

Figure 5-4 shows that for the example data used, a square leachfield of 43.0 meters (141 ft) per side with an area of 1815 square meters (19 902 ft^2) will have a maximum ground water mound buildup of 7.90 meters (25.9 ft) after 20 years. A rectangular leachfield with the same surface area would have an even lower ground water mound buildup.

The design values shown above should not be accepted by the designer as the final design area and ground water mound buildup for the leachfield. Any design should be critically examined to determine if the design seems reasonable. This is especially true of this design because of the large spatial variability of hydraulic conductivity over small distances, which can induce a false sense of security in the accuracy of this design.

Table 5-1. Data for Example Application.

first estimate of one-half field side length = 30 m
design life = 7300 days
daily effluent volume = 51.3 m^3
specific yield = 0.09
hydraulic conductivity = 0.013 cm/s
original water table height = 6.71 m
maximum acceptable water table height = 8.02 m

```
THIS PRORGAM IS DESIGNED TO ESTIMATE THE AREA
REQUIRED FOR A LARGE SCALE LEACHFIELD.  THE PRIMARY
DESIGN CONSIDERATION IS THE MAINTENANCE OF AN
UNSATURATED ZONE OF ADEQUATE THICKNESS BENEATH
THE LEACHFIELD.

ENTER THE FOLLOWING DATA

FIRST ESTIMATE OF ONE-HALF FIELD SIDE LENGTH(M)
30
NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD
7300
VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)
51.3
SPECIFIC YIELD
0.09
HYDRAULIC CONDUCTIVITY (CM/SEC)
0.013
ORIGINAL WATER TABLE HEIGHT (M)
6.71
MAXIMUM ACCEPTABLE WATER TABLE HEIGHT (M)
8.02

              HEIGHT (M.)          =           7.90

              SIDE LENGTH (M.)     =          43.

              FIELD AREA (SQ M.) =          1815.
```

Figure 5-4. Screen Appearance of Example.

CHAPTER SIX

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

The objectives of this research, as stated in Chapter One, were to: 1) develop a model of ground water buildup for use in designing a large scale soil absorption bed and 2) utilize the model as the basis for a micro computer aided design package. Both objectives were accomplished.

The Rao and Sarma (1981b) approach to mathematically describing the buildup of a ground water mound was selected as the basis for a model to incorporate the configuration of a large scale soil absorption bed as described in Chapter Three. Adaptations to the original model had very little effect on the accuracy with which it predicted the ground water mound buildup.

The model was used to develop a micro computer based design package as described in Chapter Five. This package can now be made available in diskette form to persons interested in large scale soil absorption bed design.

CONCLUSIONS

1. The design package is a useful tool to aid in the design of large scale soil absorption systems. The design package cannot, however, be used without a clear understanding of its limitations.

2. The model proved to be quite accurate with the limited data available for testing.
3. Prior to any widespread use of the computer aided design package, the model needs further testing.
4. Approximating the dosing procedure used on most large scale soil absorption beds with a constant recharge rate applying the same effluent volume per day is acceptable in this design.
5. The model illustrates several points about the design of large scale systems that warrant careful consideration.
   a. The acceptable hydraulic conductivity range appears to be 5 X 10^{-3} to 4.2 x 10^{-2} cm/sec.
   b. With the above hydraulic conductivities, flow through the soil is often very rapid, which raises questions as to the ability of the 1.22 meter (4 ft) unsaturated zone to adequately treat the wastewater.
6. The design package is currently a useful tool for the design of large scale soil absorption beds. However it could be greatly improved by the incorporation of a dynamic recharge component to account for precipitation infiltration and a better understanding of the unsaturated zone required for bacterial die-off and nitrogen conversion.

RECOMMENDATIONS

The following recommendations would serve to greatly increase the usefulness of the design package.

1. There needs to be more testing of the model with field data. Despite the encouraging results obtained in testing the model with limited field data, the model should be tested with more

field data (especially actual leachfield data) before it is released to the public.

2. A dynamic recharge component should be incorporated into the model. Such a component would allow the designer to account for the effects of rainfall or snowmelt infiltration into the leachfield directly.

3. More research needs to be done on the required residence time of the effluent in the unsaturated zone for bacterial die-off and nitrogen conversion. At present, little work has been done on either bacterial die-off or nitrogen conversion rates in the unsaturated zone beneath a leachfield. A better understanding of the rate at which bacteria die and organic nitrogen is converted to nitrate in the unsaturated zone would allow the designer to make a more informed judgment of the thickness required of this zone beneath the leachfield. A more informed judgment of the unsaturated zone would lead to better leachfield design on all sites and especially on those sites which are now marginally acceptable for large scale soil absorption systems.

REFERENCES

Abney, J.L. 1978. Fountain Run, Kentucky (A Case Study). National Conference on Less Costly Wastewater Treatment Systems for Small Communities. U.S. EPA, Washington D.C., EPA 600/9-79-010: pp. 54-67.

Bianchi, W.C. and E.E. Haskell 1975. Field Observations of Transient Ground-Water Mounds Produced by Artifical Recharge into an Unconfined Aquifer. Rep. W-27, Agricultural Research Service, U.S. Department of Agriculture, Washington, D.C.

Carlile, B.L., C.G. Cogger, M.D. Sobsey, J. Scandura, and S.J. Steinbeck 1981. Movement and Fate of Septic Tank Effluent in Soils of the North Carolina Coastal Plain. Division of Health Services, North Carolina Department of Public Health, Raleigh, North Carolina.

Clark, J.W., W. Viessman, Jr., and M.J. Hammer. 1977. Water Supply and Pollution Control, Third Edition. Harper and Row, Publishers, Inc. New York, New York.

DeWalle, F.B. 1981. Failure analysis of large septic tank systems. Journal of the Environmental Engineering Division, ASCE, 107(EE1): 229-240.

Diodato, F.D. 1980. Community onsite disposal systems--the Iselin demonstration project. Proceedings of the Seventh National Conference on Individual On-Site Wastewater Systems, National Sanitation Foundation, Ann Arbor, Michigan.

Dix, S.P. 1984. Personal Communication. Director, National Small Flows Clearinghouse, Morgantown, West Virginia.

Englehardt, J.D. 1983. Quantification of onsite wastewater treatment operation and maintenance. Unpublished Masters Thesis, Colorado State University, Fort Collins, Colorado.

U.S. Environmental Protection Agency. 1980. Design Manual for Onsite Wastewater Treatment-Disposal Systems. Technology Transfer Program, U.S. EPA, Washington D.C., EPA 625/1-80-012.

Fielding, M.B. 1981. Ground water mounding under leaching beds. Proceedings of the Third National Symposium on Individual and Small Community Sewage Systems, American Society of Agricultural Engineers, Saint Joseph, Missouri.

Fritton, D.D., D.A. Long, G. Aron, and G.W. Peterson 1983a. On-Site Sewage Disposal: Site Suitability, System Selection, and Soil Absorption Area Sizing. Institute for Research on Land and Water Resources, Pennsylvania State University, University Park, Pennsylvania.

Fritton, D.D., W.E. Sharpe, A.R. Jarrett, C.A. Cole, and G.W. Peterson 1983b. Restoration of Failing On-Lot-Sewage Disposal Areas. Institute for Research on Land and Water Resources, Pensylvania State University, University Park, Pennsylvania.

Glover, R.E. 1961. Mathematical Derivations Pertaining to Ground Water Recharge. Agricultural Research Service, U.S. Department of Agriculture, Fort Collins, Colorado.

Goldstein, S.N. 1972. Community sewage systems vs. on-site sewage treatment systems. On-Site Waste Management Series. Hancor, Inc., Findlay, Ohio, 1:2-27.

Hantush, M.S. 1967. Growth and decay of groundwater mound in response to uniform percolation. Water Resources Research, 3:227-234.

Kreissl, J.F., I.A. Cooper and J.W. Rezek. 1977. Alternatives for Small Wastewater Treatment Systems-Pressure Sewers/Vacuum Sewers, Volume II. Technology Transfer Program, U.S. EPA, Cincinnati, Ohio.

Kreyszig, E. 1979. Advanced Engineering Mathematics, Fourth Edition. John Wiley and Sons, New York, New York.

Laak, R. 1980. Wastewater Engineering Design for Unsewered Areas. Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan.

Marino, M.A. 1975. Artifical groundwater recharge. II. Rectangular recharging area. Journal of Hydrology. 26:29-37.

McWhorter, D.B. and D.K. Sunada. 1977. Ground-Water Hydrology and Hydraulics. Water Resources Publications, Fort Collins, Colorado.

McWhorter, D.B. and J.D. Nelson. 1980. Seepage in the partially saturated zone beneath tailings impoundments. Mining Engineering. 27(4): 432-439.

Ortiz, N.V., D.W. Zachmann, D.B. McWhorter, and D.K. Sunada 1979. Effects of in-transit water on ground water mounds beneath circular and rectangular recharge areas. Water Resources Research. 15(3):577-582.

Otis, R.J. 1978. An Alternative Public Wastewater Facility for a Small Community. Small Scale Waste Management Project, University of Wisconsin, Madison, Wisconsin.

Rao, N.H. and P.B.S. Sarma. 1980. Growth of groundwater mound in response to recharge. Ground Water 18(6):587-595.

Rao, N.H. and P.B.S. Sarma. 1981a. Recharge from rectangular areas to finite aquifers. Journal of Hydrology. 53:269-275.

Rao, N.H. and P.B.S. Sarma. 1981b. Groundwater recharge from rectangular areas. Ground Water 19(3):271-274.

Rao, N.H. 1983. Personal Communication. Scientist S-1, Water Technology Centre, Indian Agricultural Research Institute, New Delhi, India.

Rubin, A.R. and B.L. Carlile. 1981. Large scale on-site systems. Proceedings of the Eighth National Conference on Individual On-Site Wastewater Systems, National Sanitation Foundation, Ann Arbor, Michigan.

Siegrist, R.L. 1984. Personal Communication. Principal Partner, RSE, Incorporated, Madison, Wisconsin.

South Australia Department of Public Health 1968. Septic Tank Effluent Drainage Schemes, Public Health Inspection Guide No. 6, Norwood, South Australia.

Ward, R.C. and S.M. Morrison, Editors 1983. Research Needs Relating to Soil Absorption of Wastewater, National Science Foundation, Washington, D.C.

APPENDIX A

PROGRAM LISTING

AREA OPTION LISTING

```
       PROGRAM SHORT1
      WRITE (*,1)
  1   FORMAT(1X,'THIS PRORGAM IS DESIGNED TO ESTIMATE THE AREA')
      WRITE (*,2)
  2   FORMAT(1X,'REQUIRED FOR A LARGE SCALE LEACHFIELD.  THE PRIMARY')
      WRITE (*,3)
  3   FORMAT(1X,'DESIGN CONSIDERATION IS THE MAINTENANCE OF AN')
      WRITE (*,4)
  4   FORMAT(1X,'UNSATURATED ZONE OF ADEQUATE THICKNESS BENEATH')
      WRITE (*,5)
  5   FORMAT(1X,'THE LEACHFIELD.')
C     ENTRY OF INITIAL DATA
      WRITE (*,10)
 10   FORMAT(/,1X,'ENTER THE FOLLOWING DATA',/)
      WRITE (*,12)
 12   FORMAT(1X,'FIRST ESTIMATE OF ONE-HALF FIELD SIDE LENGTH(M)')
      READ (*,*) D
      WRITE (*,16)
 16   FORMAT(1X,'NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD')
      READ (*,*) T
      WRITE (*,20)
 20   FORMAT(1X,'VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)')
      READ (*,*) V
      WRITE (*,24)
 24   FORMAT(1X,'SPECIFIC YIELD')
      READ (*,*) E
      WRITE (*,28)
 28   FORMAT(1X,'HYDRAULIC CONDUCTIVITY (CM/SEC)')
      READ (*,*) HP
      WRITE (*,32)
 32   FORMAT(1X,'ORIGINAL WATER TABLE HEIGHT (M)')
      READ (*,*) HO
      WRITE (*,36)
 36   FORMAT(1X,'MAXIMUM ACCEPTABLE WATER TABLE HEIGHT (M)')
      READ (*,*) HM
C     ITERATION WINDOW
        HU=HM+0.15
        HL=HM-0.15
C     CALCULATION OF MUCH USED CONSTANTS
        HK=(HP*86400.0)/100.0
        HA=(HM+HO)/2.0
        AA=(HK*HA)/E
        AE=-AA*3.14159**2
        W2=8/(HK*3.14159**4)
        A2=(2.0*AA*T)/HK
C     LOOP TO CALCULATE HALF FIELD LENGTH
 40     P=V/(4*D**2)
        A=D*50
        AS=A**2
        CE=AS*AS
        W1=W2*CE
        A1=A2/AS
        WS=W1*P
        AP=A1*P
        WP=AP*D**2
```

```
          PA=(3.14159*D)/A
          SG=0.0
C        CALCULATION OF OUTTER SUMMATION
           DO 93 II=1,130
           AQ=II**2*AS
           S1=SIN(II*PA)
C        INNER SUMMATION CALCULATIONS
            DO 92 IJ=1,130
            PG=((AQ)+(IJ**2*AS))
            SF=(1.0/(II*IJ))*(1.0/PG)
            TE=AE*((PG*T)/CE)
C        CHECK ON VALUE OF EXPONENTIAL
            IF (TE .LE. -675.00) GO TO 60
            SS=1.0-(EXP(TE))
            GO TO 70
 60         SS=1.0
 70         ST=S1*(SIN(IJ*PA))
            SM=SF*SS*ST
            SG=SG+SM
 92         CONTINUE
 93        CONTINUE
C        CALCULATION OF S AND ACTUAL MOUND HEIGHT
          HT=(WS*SG)+WP
          HC=SQRT(HT+(HO**2))
C        CHECK ON ACTUAL MOUND HEIGHT & INCREMENTATION OF D
          IF (HC .GT. HU) GO TO 120
          IF (HC .LT. HL) GO TO 110
          GO TO 150
 110      D=D*0.71
          GO TO 40
 120      D=D*1.23
          GO TO 40
C        CALCULATION OF FIELD SIDE LENGTH & FIELD AREA
 150      D2=D*2
          DA=D2**2
        WRITE (*,160)HC
 160    FORMAT(//,15X,'HEIGHT (M.)          = ',8X,F9.2,//)
        WRITE (*,170)D2
 170    FORMAT(15X,'SIDE LENGTH (M.)    = ',6X,F9.0,//)
        WRITE (*,180)DA
 180    FORMAT(15X,'FIELD AREA (SQ M.) = ',F15.0,//)
        STOP
        END
```

HEIGHT OPTION LISTING

```
       PROGRAM RECTRY
      WRITE (*,1)
    1 FORMAT(1X,'THIS PROGRAM IS DESIGNED TO ESTIMATE THE HEIGHT OF')
      WRITE (*,2)
    2 FORMAT(1X,'THE GROUND WATER MOUND THAT DEVELOPES BENEATH A')
      WRITE (*,3)
    3 FORMAT(1X,'LARGE LEACHFIELD.')
C     ENTRY OF INITIAL DATA
      WRITE (*,10)
   10 FORMAT(/,1X,'ENTER THE FOLLOWING DATA',/)
      WRITE (*,12)
   12 FORMAT(1X,'ONE-HALF LENGTH (M) OF EACH SIDE OF FIELD SEPERATED')
      WRITE (*,13)
   13 FORMAT(1X,'BY A COMMA.')
      READ (*,*)D,DL
      WRITE (*,14)
   14 FORMAT(1X,'NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD')
      READ (*,*)T
      WRITE.(*,18)
   18 FORMAT(1X,'VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)')
      READ (*,*)V
      WRITE (*,22)
   22 FORMAT(1X,'SPECIFIC YIELD')
      READ (*,*)E
      WRITE (*,26)
   26 FORMAT(1X,'HYDRAULIC CONDUCTIVITY (CM/S)')
      READ (*,*)HP
      WRITE (*,30)
   30 FORMAT(1X,'ORIGINAL WATER TABLE HEIGHT (M)')
      READ (*,*)HO
      WRITE (*,32)
   32 FORMAT(1X,'DESIRED INCREMENTATION ON RISE OF WATER TABLE')
      WRITE (*,33)
   33 FORMAT(1X,'HEIGHT (E.G. 0.10 M, 0.20 M, ETC: NOTE; LARGER')
      WRITE (*,34)
   34 FORMAT(1X,'RUN TIMES ARE REQUIRED FOR SMALLER VALUES OF THIS')
      WRITE (*,35)
   35 FORMAT(1X,'INCREMENTATION.)')
      READ (*,*)HS
C     CALCULATION OF CONSTANTS
       A=D*50.0
       B=DL*50.0
       P=V/(4*(D*DL))
       HK=(HP*86400.0/100.0)
       AS=A**2
       BS=B**2
       WS=((8*P)/HK)*((AS*BS)/3.14159**4)
       CE=BS*AS
       PB=(3.14159*DL)/B
       PA=(3.14159*D)/A
C     LOOP TO INCREMENT WATER TABLE HEIGHT INCREASE
       HU=HO
        DO 130 IM=1,1000
        HM=HO+(HS*IM)
        HA=(HM+HO)/2.0
        AA=(HK*HA)/E
        AE=-AA*3.14159**2
        SG=0.0
C     PART OF EQUATION PAST PLUS SIGN
```

```
          AF=(2.0*F*AA*D*DL*T)/(HK*A*B)
C      START OF OUTTER SUMMATION
           DO 97 II=1,130
           AQ=II**2*AS
           S1=SIN(II*PB)
C      START OF INNER SUMMATION
            DO 95 IJ=1,130
            PG=((AQ)+(IJ**2*BS))
            SF=(1.0/(II*IJ))*(1.0/PG)
            TE=AE*((PG*T)/CE)
C      CHECK ON VALUE OF EXPONENTIAL TERM
            IF (TE .LE. -675.00) GO TO 60
            SS=1.0-(EXP(TE))
            GO TO 70
 60         SS=1.0
 70         ST=S1*(SIN(IJ*PA))
            SM=SF*SS*ST
            SG=SG+SM
 95         CONTINUE
 97        CONTINUE
C      CALCULATION OF MOUND HEIGHT
          HT=(WS*SG)+AF
          HC=SQRT(HT+(HO**2))
          HV=HC-HU
          IF(HV .LE. 0.001) GO TO 150
          HU=HM
 130    CONTINUE
 150    D=D*2
        DL=DL*2
 155    WRITE (*,160)HC
 160    FORMAT(//,15X,'CENTER HEIGHT (M) = ',F6.2,//)
        WRITE (*,165)D,DL
 165    FORMAT(15X,'SIDE LENGTHS ARE ',F5.0,' (M) X ',F5.0,' (M)')
        STOP
        END
```

DISTANCE OPTION LISTING

```
      PROGRAM XLDIST
      WRITE (*,1)
 1    FORMAT(1X,'THIS PROGRAM IS DESIGNED TO ESTIMATE THE HEIGHT OF')
      WRITE (*,2)
 2    FORMAT(1X,'THE GROUND WATER MOUND BENEATH A LARGE LEACHFIELD AT')
      WRITE (*,3)
 3    FORMAT(1X,'USER SPECIFIED POINTS ALONG THE LEACHFIELD AXIS. ')
C     ENTRY OF INITIAL DATA
      WRITE (*,10)
10    FORMAT(/,1X,'ENTER THE FOLLOWING DATA',/)
      WRITE (*,12)
12    FORMAT(1X,'ONE-HALF LENGTH (M) OF EACH SIDE OF FIELD SEPERATED')
      WRITE (*,13)
13    FORMAT(1X,'BY A COMMA (ENTER SEPERATION CALCULATION DISTANCE AS')
      WRITE (*,14)
14    FORMAT(1X,'SECOND HALF LENGTH)')
      READ (*,*)D,DL
      WRITE (*,18)
18    FORMAT(1X,'NUMBER OF DAYS EFFLUENT IS APPLIED TO FIELD')
      READ (*,*)T
      WRITE (*,22)
22    FORMAT(1X,'VOLUME OF EFFLUENT APPLIED TO FIELD PER DAY (CU. M)')
      READ (*,*)V
      WRITE (*,26)
26    FORMAT(1X,'SPECIFIC YIELD')
      READ (*,*)E
      WRITE (*,30)
30    FORMAT(1X,'HYDRAULIC CONDUCTIVITY (CM/S)')
      READ (*,*)HP
      WRITE (*,34)
34    FORMAT(1X,'ORIGINAL WATER TABLE HEIGHT (M)')
      READ (*,*)HO
      WRITE (*,38)
38    FORMAT(1X,'MAXIMUM WATER TABLE HEIGHT (M)')
      READ (*,*)HM
      WRITE (*,40)
40    FORMAT(1X,'INCREMENT ON DISTANCE FOR SEPERATION CALCULATIONS')
      WRITE (*,41)
41    FORMAT(1X,'(10M, 25M, ETC: NOTE; LARGER RUN TIMES ARE REQUIRED')
      WRITE (*,42)
42    FORMAT(1X,'FOR SMALLER VALUES OF THIS INCREMENTATION.)')
      READ (*,*)LS
C     CALCULATION OF MUCH USED CONSTANTS
       HK=(HP*86400.0)/100.0
       P=V/(4*(D*DL))
       A=D*50.0
       B=DL*50.0
       L=0
       LX=B
       HA=(HM+HO)/2.0
       AA=(HK*HA)/E
       AS=A**2
       BS=B**2
       WS=((B*P)/HK)*((AS*BS)/3.14159**4)
       AE=-AA*3.14159**2
       CE=BS*AS
       PB=(3.14159*DL)/B
       PA=(3.14159*D)/A
       PX=(3.14159/B)
```

```
       AP=(2.0*P*AA*D*DL*T)/(HK*A*B)
       WRITE (*,50)
 50    FORMAT(//,5X,'DISTANCE FROM RECHARGE AREA CENTER',10X,'HEIGHT')
C      LOOP TO CALCULATE DISTANCE TO NEGLIGABLE MOUND HEIGHT INCREASE
        DO 150 IX=L,LX,LS
        X=IX
        SG=0.0
C      START OF OUTTER SUMMATION
         DO 97 II=1,130
C      CALCULATION OF CONSTANTS FOR OUTTER SUMMATION
         AQ=II**2*AS
         SX=II*X*PX
         S1=II*PB
         CP=(SIN(S1))*(COS(SX))
C      INNER SUMMATION CALCULATIONS
          DO 95 IJ=1,130
          PG=((AQ)+(IJ**2*BS))
          SF=(1.0/(II*IJ))*(1.0/PG)
          TE=AE*((PG*T)/CE)
C      CHECK ON VALUE OF EXPONENTIAL
          IF (TE .LE. -675.00) GO TO 60
          SS=1.0-(EXP(TE))
          GO TO 70
 60       SS=1.0
 70       ST=(SIN(IJ*PA))*CP
          SM=SF*SS*ST
          SG=SG+SM
 95       CONTINUE
 97      CONTINUE
C      CALCULATION OF S AND ACTUAL MOUND HEIGHT
        HT=(SG*WS)+AP
        HC=SQRT(HT+(HO**2))
        HS=HC-HO
C      CHECK ON MOUND HEIGHT INCREASE
        WRITE (*,130)X,HC
 130    FORMAT(//,19X,F5.0,25X,F5.2)
        IF (HS .LE. 0.01) GO TO 160
        IF (X .GE. LX) GO TO 153
 150    CONTINUE
 153   WRITE (*,154)
 154   FORMAT(1X,'THE PROGRAM IS NOT DESIGNED TO CALCULATE MOUND')
       WRITE (*,155)
 155   FORMAT(1X,'HEIGHT AT DISTANCES GREATER THAN 50 TIMES THE ')
       WRITE (*,156)
 156   FORMAT(1X,'FIELD HALF LENGTH.')
 160   STOP
       END
```