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Simulations of dry well recharge in the Tucson Basin, Arizona

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SIMULATIONS OF DRY WELL RECHARGE IN THE TUCSON BASIN, ARIZONA

by

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ABSTRACT

The variably saturated flow model Unsat 2 was used for three case study simulations of dry well recharge in the Tucson Basin, Arizona. Dry well design, and rainfall/runoff and vadose zone conditions representative of the locality were assumed in the simulations to address travel time to the regional aquifer, rates and extent of radial flow, and relative degree of solute attenuation by sorption and dilution with regional groundwater. Soil specific surface was used to estimate relative degree of sorption among the three cases. One case of uniform soil composition and two cases of layered soil composition were simulated. Clay content had the greatest influence on specific surface. Hydraulic conductivity had the greatest influence on soil water velocities and degree of radial flow. The presence of layered subsurface conditions that included strata of low hydraulic conductivity enhanced the degree of subsurface solute attenuation by sorption and dilution.

INTRODUCTION

High rates of growth in metropolitan centers in the southwestern United States have lead to increased volumes of urban runoff. The increased paved area in growing cities is resulting in volumes of storm water runoff that exceed the capacity of existing routing and detention/retention facilities. As a result, use of the "dry well," a storm water drainage well designed to provide localized runoff disposal by routing water from the ground surface to permeable vadose zone sediments, has increased in cities of the Southwest. Accompanying the increased implementation of dry well drainage systems has been public concern over their effect on underlying groundwater supplies. Dry wells are often placed near commercial and industrial developments where there exists a high likelihood for toxic substances deposited on paved areas by normal industrial activities, or accidental spillage, to be carried into dry wells along with runoff water. Long-term degradation of groundwater via sustained percolation of tainted urban runoff to the water table through dry wells is a major concern in areas where dry wells are used on a large scale.

DRY WELL REGULATION

This concern has led to regulation of dry wells at the federal, state, and local level. Dry wells are included under the classification of Class V injection wells by the U.S. Environmental Protection Agency (EPA). All types of Class V injection wells are regulated under Part C of the Safe Drinking Water Act (PL-93-523), which requires state development of regulations through Underground Injection Control (UIC) programs. EPA regulations stated in Title 40 of the Code of Federal Regulations, Parts 124 and 144 through 147, take effect in states where adequate UIC programs have not been developed. The default EPA regulations allow the construction and use of all Class V injection wells until more stringent regulations are developed.

Arizona state regulation of dry wells was included under its Environmental Quality Act and House Bill 2229, effective in 1986. This legislation called for rules governing location, construction, maintenance, and closure of dry wells, as well as the establishment of a dry well inventory and permitting process. This legislation and its follow-up regulation of dry wells preempts federal regulation under the Safe Drinking Water Act.

Pima County enacted the Interim Dry Well Policy in 1986, which has also been adopted by the City of Tucson. The Dry Well Policy prohibited new construction of dry wells in industrial and landscaped areas, and imposed requirements for siting, operation, and maintenance of new dry wells in other areas. The policy requires 75 feet of separation between the dry well and the regional water table in residential areas, and 125 feet of separation in commercial areas. The policy requires 300 feet of horizontal separation from cased water supply wells and 500 feet from uncased supply wells. Additional requirements include a suspended sediment trap device in or near each well, time limitations on drainage of ponded water for mosquito control, and routine annual maintenance, including removal of accumulated sediment.

PURPOSE

In 1985 Pima County authorized funding for computer simulations of dry well recharge under hydrologic conditions representative of the Tucson Basin. The results of the study were intended to aid in assessing the potential for degradation of groundwater quality via dry well recharge and in refining guidelines for the regulation of dry well construction in Pima County.

The purpose of this research was to assess how varying layered subsurface conditions and soil composition affected: A) time of arrival of recharged water at the regional water table, B) the rate and extent of lateral movement of recharged water in the vadose zone, and C) the potential for subsurface attenuation of water-borne pollutants. As the variably saturated flow model used in the simulations did not treat solute transport phenomenon, part C above had to be addressed within the limitations of a hydraulic model. As such, the research addressed the variability of subsurface attenuation insofar as it may be associated with physical and chemical interactions of recharged water with vadose zone soil particle surfaces.

Arrival time of recharged water at the regional water table represents the time that the "first flush" of recharged runoff water contacts the regional water supply. This is of concern to regulators in that the first flush of runoff from a commercial or industrial area is usually the most contaminated. The extent of lateral movement of recharged water is of concern in that the water may directly impact nearby water supply wells by seepage into a wellbore or percolation into the area of its cone of depression. Subsurface attenuation is of primary concern as dry-well recharged water typically undergoes no above-surface treatment for contaminant removal.

OVERVIEW

The research objectives were addressed by performing three case study simulations of dry well recharge from rainfall/runoff events and subsurface conditions representative of the Tucson Basin. The variably saturated flow model incorporated data collected during construction and testing of an experimental dry well at the University of Arizona Water Resources Research Center (WRRC) in Tucson, Arizona. Simulated subsurface conditions, though generally representative of the Tucson Basin, were derivative of those observed at the WRRC site. The three case study scenarios are briefly summarized below.

CASE 1

The first case study was intended to simulate dry well recharge under subsurface conditions where exposure of drainage water to soil particle surfaces would be minimized. As such, the Case 1 simulation assumed the composition of the vadose zone to be that of a high permeability gravelly sandy loam soil horizon in which the WRRC experimental dry well was completed **(Figure 1a)**. This soil material was designated as Material 1 for the purposes of this study. Dry well recharge into a uniform material of this type would be characterized by relatively rapid movement through the vadose zone and minimal attenuation of contaminants, and minimal lateral flow of perched water.

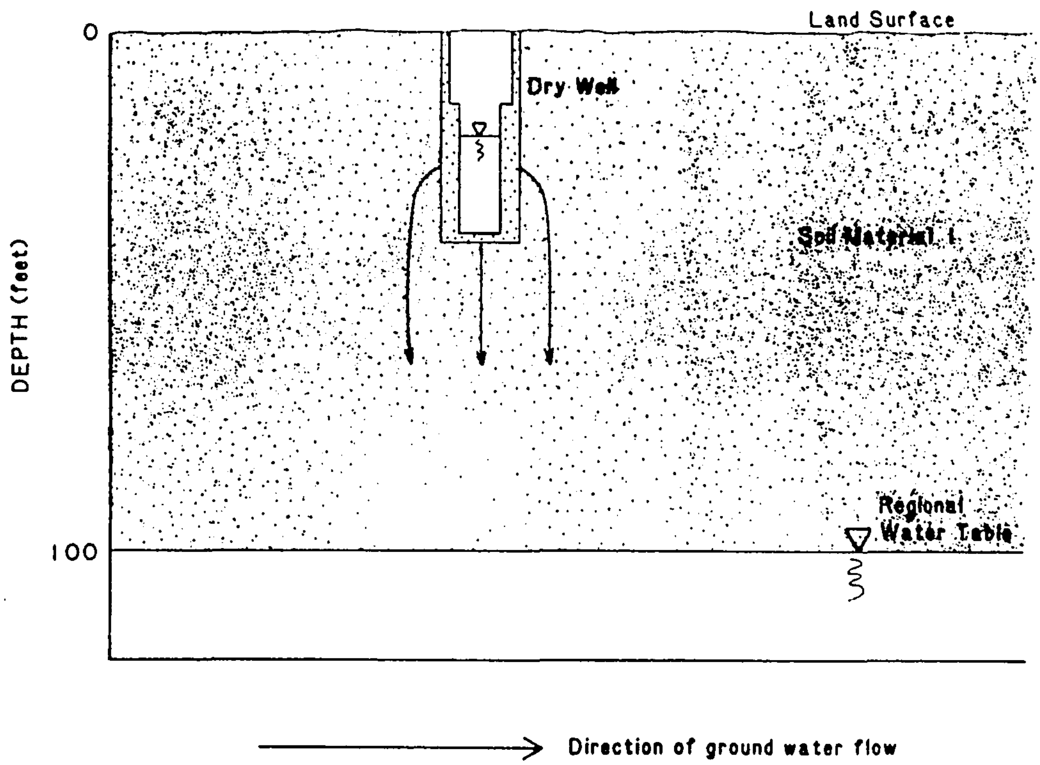


FIGURE 1a. CASE 1 DRY WELL RECHARGE SCHEMATIC

CASE 2

The second case study assumed a two-layer vadose zone patterned after layered subsurface conditions observed at the WRRC Field Laboratory. The upper layer consisted of the Material 1 soil used in Case 1, underlain by a soil material of similar composition, but much lesser permeability **(Figure 1b)**. This soil, designated as Material 2 for this study, is patterned after soil types observed below a depth of about 30 feet at the WRRC Field Laboratory. Drainage water introduced via dry well into the coarse-grained Material 1 would build up and move laterally as well as vertically along the layer transition to the fine-grained Material 2. As such, drainage water moves more slowly through the vadose zone, as well as being exposed to a larger area of soil. The Case 2 simulation of this study is characterized by greater travel time to the water table, exposure to soil particle surfaces for dry well drainage water and lateral flow of perched water, relative to Case 1.

CASE 3

The third case study simulation was intended as an intermediate case relative to Cases 1 and 2. This case study assumes the same layered conditions as Case 2, with Material 2 replaced by a soil material of moderate permeability **(Figure 1c)**. This soil, designated as Material 3 for this study, illustrates the effect of some perching and lateral movement at the layer transition, but less so than in Case 2. The Case 3 study illustrates moderate rates of vertical movement,

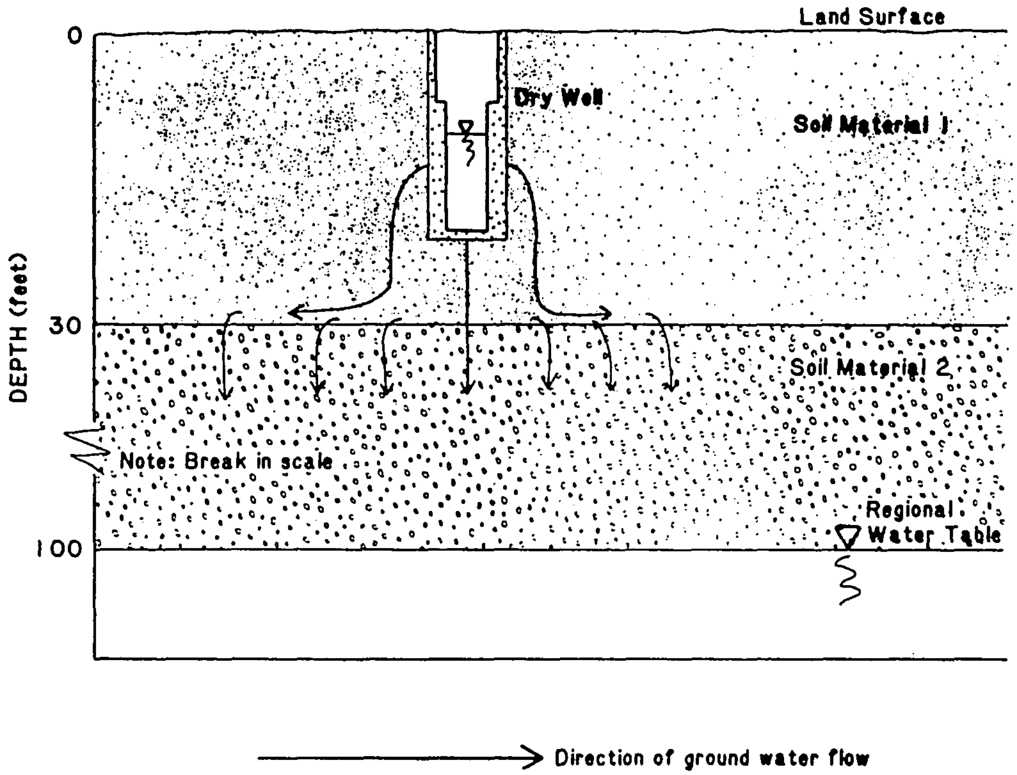


FIGURE 1b. CASE 2 DRY WELL RECHARGE SCHEMATIC

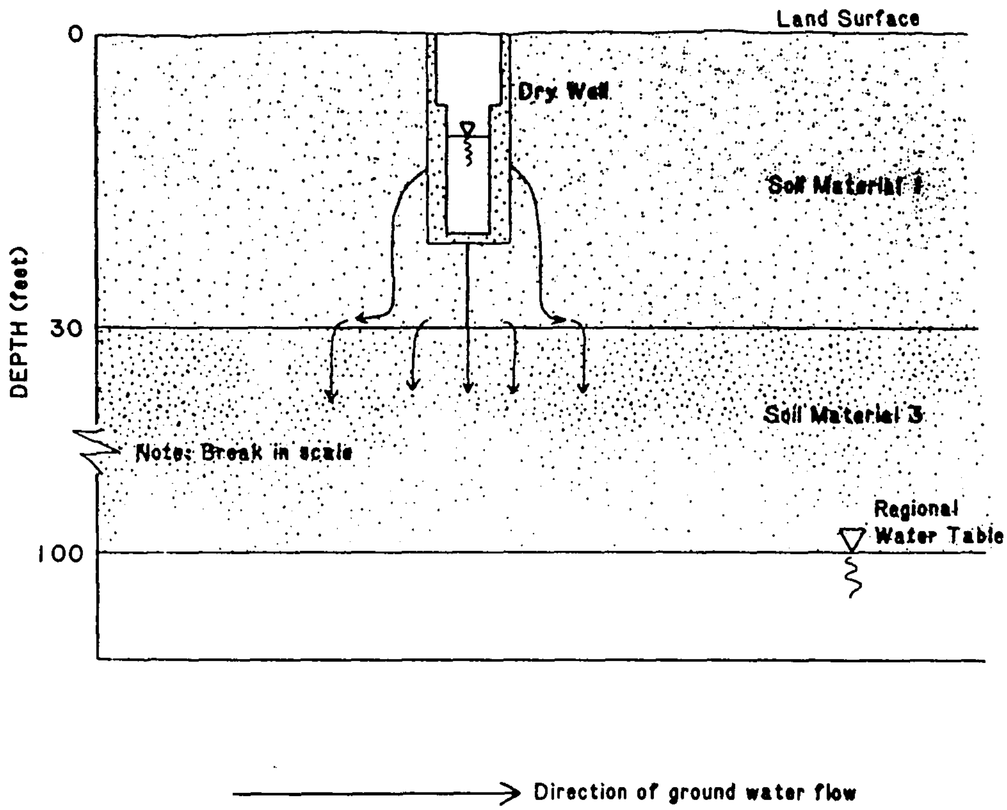


FIGURE 1c. CASE 3 DRY WELL RECHARGE SCHEMATIC

moderate exposure to soil particles, and moderate lateral movement of perched water, relative to Cases 1 and 2.

BACKGROUND

STRUCTURE, FUNCTION AND USE OF DRY WELLS

Generally, a dry well is any structure that facilitates the rapid movement of runoff water into permeable vadose zone sediments. Dry wells are commonly used in and near commercial parking lots such as those found at shopping centers, restaurants and hotels, near industrial plants, golf courses and residential areas, or in a number of other common urban locations where substantial amounts of surface runoff are generated. The most common dry well design employed in Arizona cities consists of an upper compartment which acts as a settling chamber and a lower compartment where water drains into vadose zone sediments **(Figure 2)**. The upper compartment consists of a pre-cast concrete liner, which is perforated with weep holes to allow some drainage from this compartment. An asbestos-cement overflow pipe extends from the base of the debris screen to the base of the upper compartment, where it connects to the perforated tubing and injection screen of the lower compartment. The base of the upper compartment is lined with a fibrous screen to restrict movement of fine sediment into the lower compartment. The annular space is filled with washed gravel. The concrete liner is completed at the surface with a manhole cone and covered with a traffic grate (Wilson, 1983).

This design minimizes the entry of particulates into the lower compartment, which may cause clogging of pore spaces in the borehole

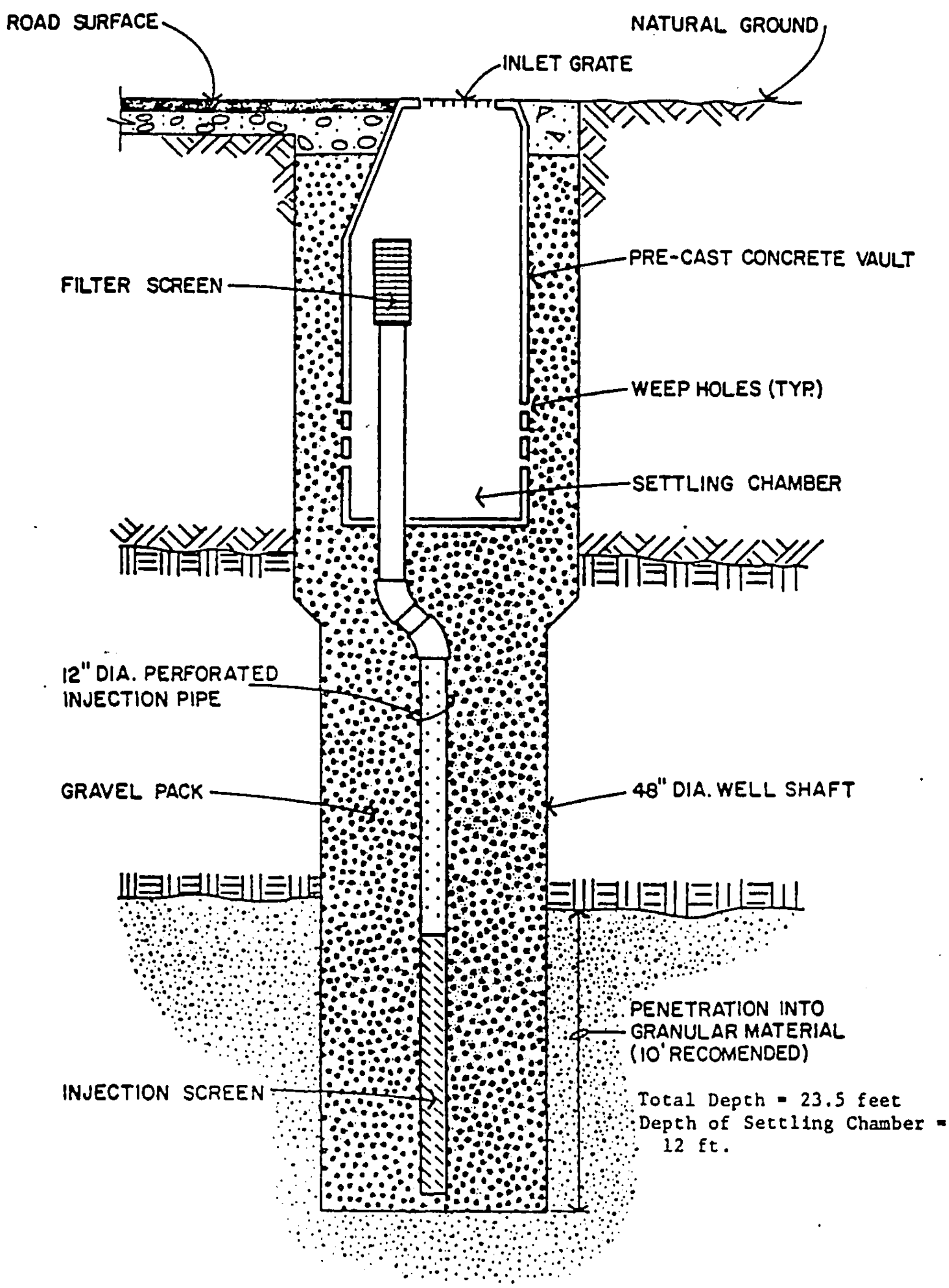


FIGURE 2. TYPICAL DRY WELL DESIGN

wall. Some clogging still occurs over time, however, and the intake capacity of the well may be reduced as a result. For purposes of designing a layout of a dry well detention/retention system in Southern Arizona, a local manufacturer estimates an average sustainable intake capacity of approximately 0.5 cubic feet per second (cfs) (McGuckin Drilling, Inc., 1984). The design service area, the area from which the dry well is expected to receive surface runoff, is approximately one paved acre.

PREVIOUS INVESTIGATIONS

Literature published to date on the use of dry wells as an aid to storm water drainage consists mostly of site-specific studies aimed at determining their potential impact on ground water quality.

Allen (1970) proposed a system of cased drainage shafts as an aid to storm water runoff drainage in a karstic locality in northwestern Alabama. The plan was shelved at the recommendation of the state geologist, on the grounds that use of the shafts would pose a threat to local ground water supplies. A subsequent study of the same area (Geological Survey of Alabama, 1986) described the hydrogeology of the area and documented the quality of runoff water routed to existing dry wells. This study also recommended discontinuing the use of dry wells on the basis of potential degradation of ground water supplies. SMC Martin, Inc. (1983) reported the results of a "first-phase" study of potential dry well impacts on ground water in the vicinity of Roanoke, Virginia to be inconclusive, and recommended a second phase long-term monitoring study of influent and ground water near drainage wells. Vadose zone deposits in this area were found to be fractured and solution channeled carbonates. The report noted, on the basis of a limited field effort, that quality of runoff appeared to be generally good, but that some potential for contamination by runoff-borne iron and lead existed. The report noted a rapid response of ground water levels to storm water recharge in the study area, where average depth to ground water was approximately 50 feet.

Wilson (1983) performed recharge tests on an experimental dry well designed to monitor tracer migration in the vadose zone and regional aquifer, characterize vadose zone flow patterns, and assess the effect of various tracers on dry well intake rates. This study provided data supporting the following conclusions:

1. Differences in permeability of vadose zone materials may cause perching and lateral flow of drainage water.

2. Suspended particulates may reduce the intake capacity of dry wells due to clogging of formation pore space near the dry well.

3. Attenuation of microorganisms, organic and inorganic tracer materials in the vadose zone may be incomplete.

Bandeen (1984) conducted a modelling study to simulate the effect of subsurface perching and lateral flow of dry well drainage water. The study concluded that, under normal design conditions, introduction of drainage water directly into the wellbore of a nearby supply well via lateral movement in the vadose zone is unlikely beyond a distance of about 100 feet.

Miller (1984) conducted a study on storm water recharge impacts on water quality of the Rathdrum-Spokane aquifer in the State of Washington. This study concluded that the use of dry wells had adversely impacted ground water quality, and drew a direct correlation between increased density of dry wells due to urban growth and decreased ground water quality. The study noted that use of grassed detention areas in conjunction with dry wells decreased the amount of contamination introduced into the ground water.

Schmidt (1985) conducted a water quality impact study on a commercial area using storm water recharge wells in Phoenix, Arizona. These recharge wells were not actually dry wells, as they terminated below the water table. Analyses showed that low levels of inorganic and organic contaminants were present in samples of runoff collected near the recharge well, but were not detected in ground water samples collected from a shallow monitor well located about 50 feet downgradient. Schmidt noted the danger of migration of potentially contaminated perched ground water through dry well shafts, and of the use of dry wells in commercial and industrial areas where hazardous materials are routinely used. Schmidt recommended that dry wells be terminated at least 10 feet above the water table.

SOLUTE ATTENUATION DURING DRY WELL RECHARGE

Processes that may contribute to the attenuation of dissolved constituents in the course of dry well recharge include volatilization, sorption, oxidation-reduction reactions, hydrolysis, and biodegradation. As a geochemical model or water quality study was not a designated part of this research, treatment of the geochemical aspects of pollutant attenuation was a limited part of this study. Sorption processes, involving interactions of dissolved constituents with the soil medium, are treated briefly here as they lent themselves to analysis within the limitations of a hydraulic model.

Sorption

Sorption generally describes processes whereby dissolved constituents are chemically bound to the solid phase of the soil matrix. These may include ion exchange, oxidation-reduction reactions, and complexation. Briefly, ion exchange involves the adsorption of positively charged ions at the negatively charged surfaces of colloidal particles, usually clay minerals. Sorption by ion exchange is measured by the cation exchange capacity, unique to a given soil or sediment. Ion exchange can effectively retard most positively charged metal complexes (Callahan, et.al., 1979). Oxidation-reduction reactions may involve chemical alteration of dissolved constituents through exchange of electrons with molecules of the solid phase. Complexation may involve bonding of species in solution to the solid phase, due to electrical attraction of ions of opposite charge. The soil/water partition coefficient is also a useful measure of the degree of sorption of a particular dissolved inorganic or organic compound on a specific soil substrate. Sorption in the vadose zone may significantly reduce the concentrations of dissolved constituents with sorptive properties. Sorption is an important retardation process for metals and most metal complexes, polychlorinated biphenyls (PCB's), some phenols, and most phthalate esters and polyaromatic hydrocarbons (Callahan, et.at., 1979).

Dilution

Mixing of relatively small volumes of contaminated recharge water with the untainted water of a regional aquifer system will effectively lower the concentration of contaminants in the recharge water. Direct injection of contaminated water in a small localized area will result in higher contaminant concentrations in the recharge/aquifer water mix than if the same volume of contaminated water were spread out over a large area. This process may be significant in dry well recharge, as varying degrees of subsurface spreading will occur depending on local vadose zone geology. Recharge into uniform materials of higher permeability will result in lower dilution by subsurface mixing than recharge into a layered system, where water is spread by lateral flow over a large area. Cross sectional area of the plume of recharged water may serve as an indication of the degree of dilution by mixing with waters of the regional aquifer likely to occur in a given recharge situation.

SOIL SURFACE AREA

Sorption processes are generally associated with exposure of dissolved constituents to soil particle surfaces. One general means of comparing the relative degree of solute attenuation by sorption available to a hydraulic model is the comparison of the surface area of soil particles to which recharged water is exposed. White (1979) correlated soil particle surface area with the degree of available surface free energy. Surface free energy is converted to work in the process of surface adsorption. Soil surface area available for reaction with recharge water can be estimated from the volume of vadose zone sediments within the drainage plume. This method was employed as part of the analysis of the simulation results in this study.

Specific surface is commonly defined as the surface area of soil particles per unit weight of soil, usually expressed as square meters per gram (m^2/g). Data on grain-size distribution for the case study soil materials were combined with data on specific surface for various grain-type fractions to calculate representative specific surface values for each soil material. Estimates of soil volumes within the drainage plume were combined with data on bulk density and specific surface for the case study soil materials to calculate total surface area of soil particles to which recharge water was exposed.

PROGRAM UNSAT 2

The variably saturated flow model Unsat 2, developed by S.P. Neuman, currently Professor of Hydrology and Water Resources at the University of Arizona, was used in the computer simulations. Unsat 2 was developed to simulate variably saturated water flow in soils, allowing for water uptake by plant roots. The program as originally developed is documented in Neuman et al. (1974). The theoretical basis of the model has been described in a series of papers by Neuman (1973 and 1975) and Neuman et al. (1974). Various applications have been reported by Feddes et al. (1974), Kroszynski and Dagan (1975), Wei and Shieh (1979), Zaslavsky and Sinai (1981) and others. A more recent documentation including example applications has been published by Davis and Neuman (1983) in association with U.S. Nuclear Regulatory Commission.

Unsat 2 uses the Galerkin approach to the finite element method to simulate variably saturated flow in porous media. The model can simulate horizontal or vertical plane flow, or radially symmetrical flow. The model can treat multiple soil types, and allows for anisotropy. The model does not allow for the effects of hysteresis, treating pressure head as a single-valued function of water content. The main components of input to the program include:

1. A delineation of the flow region (finite element grid).

2. Saturated and unsaturated hydraulic properties of each soil type within the flow region (pressure head, relative hydraulic conductivity and specific moisture capacity) as functions of water content; saturated hydraulic conductivity and effective porosity.

3. Initial conditions (distribution of pressure or total hydraulic head in the flow region).

4. Boundary conditions (prescribed head or flux).

5. Time step information, including initial time step, time step multiplier, total simulation time and convergence criterion.

The program solves a form of the Richards (1931) equation given as

$$\frac{\partial}{\partial x_i}\left[K^r(\psi)\,K^s_{ij}\,\frac{\partial\psi}{\partial x_j} + K^r(\psi)K^s_{i3}\right] - [C(\psi) + \beta S_s]\,\frac{\partial\psi}{\partial t} + S = 0$$

i,j = principal directions of hydraulic conductivity tensor (i,j = 1 through 3 where 3 denotes vertical direction)
K^r = relative conductivity
K^s = saturated conductivity
C = specific moisture capacity
β = 1 for saturated flow
0 for unsaturated flow
S_s = specific storage
S = sink term

To solve the Richards equation by the finite element method, the unsaturated hydraulic properties in each element must be known at the beginning and end of each time increment. These properties are actually known only at the beginning of the time increment. As time progresses, the pressure at each node changes which in turn causes a change in the soil parameters of hydraulic conductivity, water content and specific moisture capacity. These changes cannot be predicted without knowing the pressures at all the nodes at the end of the time increment, yet the pressures cannot be computed without knowing how the soil parameters will change with time. This type of equation calls for an iterative method of solution. To solve for the unknowns, at the end of each time increment, the soil parameters are updated to reflect the most recent pressure calculations at all the nodes, and the solution for the time increment is repeated with these updated values. This process is repeated for each time increment until the previous and updated pressure values differ by not more than a user-specified tolerance. The iterative process therefore provides an approximate solution. A smaller tolerance implies a more accurate solution, but also implies more computing time and its associated expense.

The program output consists mainly of the pressure or total head distribution over the grid nodes at the end of each time step where a convergent solution was achieved, accompanied by flow into or out of the region at each node. Also included are the maximum head change at any node in the grid for each iteration of the solution, the cumulative flow into or out of the system and the distribution of specific moisture capacity at the unsaturated nodes of the grid. As the solution progresses through time, the pressure distribution in the flow region and its associated moisture content distribution may be saved on separate files for use in creating contour plots. These contour plots give a graphic representation of the distribution of flow from a dry well at various stages of storm water recharge events and intermittent drainage.

UNSAT 2 was originally developed for use on an IBM mainframe computer and was later adapted to CDC and Cyber computers. The program is written in the ANSI standard Fortran 77 language and is therefore easy to modify for use on other computers. These simulations were performed on the Cyber 175 computer at the University of Arizona.

METHOD OF INVESTIGATION

GEOLOGIC SETTING

Regional Geology

The WRRC Field Laboratory is located in the central portion of the Tucson Basin, an alluvial basin surrounded by mountains of largely igneous and metamorphic composition. The Tucson Basin is included in the Desert Region of the Basin and Range physiographic province. As in most of the Basin and Range, major landforms are controlled by normal faulting resulting in abrupt changes in relief, and accompanied by thick sequences of sedimentary deposits. The Tucson Basin is bounded to the north and northeast by the Santa Catalina Mountains, to the northwest by the Tortolita Mountains, to the east by the Tanque Verde and Rincon Mountains, to the south and southeast by the Santa Rita and Empire Mountains, respectively, and to the west and southwest by the Tucson and Sierrita Mountains, respectively. The Tortolita, Santa Catalina, Tanque Verde and Rincon Mountains form a crystalline complex of banded granitic gneiss, granite, and some highly cemented sedimentary rocks (Gass, 1977). These rocks vary in age from Pre-Cambrian to late Tertiary. The Sierrita, Santa Rita and Empire Mountains are composed primarily of Pre-Cambrian to late Tertiary sedimentary, metamorphic and intrusive igneous rocks with some extrusive igneous rocks. The Tucson Mountains bordering the west side of the Basin are of volcanic composition, made up largely of rhyolitic and andesitic flows of Mesozoic and Tertiary age. Minor distributions of sedimentary rocks cover volcanics in localized areas, and some extrusive basalt formations occur.

The sedimentary units of the Tucson Basin, formed from uplift and erosion of the surrounding mountain ranges beginning in the Tertiary period, have been described by Pashley (1966), Davidson (1973) and Gass (1977). The major units, from oldest to youngest, comprise the Pantano Formation, Tinaja Beds, Fort Lowell Formation and surficial deposits. The Pantano is of Oligocene age and composed mainly of silty sandstone and gravel (Davidson, 1973). The Pantano reaches depths of 1,500 feet or more toward the center of the Basin. The Pantano is unconformably overlain by the Miocene to Pliocene aged Tinaja Beds, formed as a result of internal basin drainage and composed mainly of gravel, sand, clayey silt and mudstone. Sediments of the upper Tinaja beds correspond with the Rillito beds described by Pashley (1966), observed to comprise the older Tertiary sediments below a depth of about 80 feet at the WRRC field laboratory (Osborne, 1969). The Fort Lowell Formation unconformably overlies the Tinaja Beds, and represents a younger unit formed in the internal drainage basin. The Fort Lowell is of early Pleistocene Age, and grades from gravel near the edges of the Basin to clayey silt at the center. Sediments of the Fort Lowell Formation are probably correlative with Basin fill described by Pashley (1966), observed to extend from a depth of about 30 to 80 feet below land surface at the WRRC field laboratory (Osborne, 1969). Relatively thin surface deposits were unconformably laid down on the erosional surfaces of underlying units beginning in the middle Pleistocene. These deposits are mostly gravel and gravelly sand and, locally, sand to sandy silt, and include alluvial fan, sheetflow and stream-channel deposits (Davidson, 1973). These sediments represent the upper 25-30 feet of the vadose zone at the WRRC field laboratory site (Osborne, 1969).

Site Conditions

The WRRC field laboratory is located immediately west of the Miracle Mile interchange of Interstate 10 in Tucson, Arizona. The major surface soil types at the WRRC field laboratory are sandy loams of the Comoro and Vinton-Anthony Groups (U.S. Department of Agriculture, 1972). The Vinton-Anthony sandy loam predominates in the vicinity of the experimental recharge area where the dry well is located. The unit is described as a sandy loam from 8 to 12 inches deep, and a loamy, fine to very fine sand extending to approximately 60 inches. This description is in agreement with that for soils encountered in this interval during construction of the dry well. A lithologic log compiled during drilling of the borehole describes the upper three feet of soil as a silty sand/ sandy silt, including gravel and small cobbles from three to eight feet **(Table 1)**. Wilson (1983) described the sediments adjoining the lower compartment of the dry well as highly permeable and capable of transmitting water rapidly away from the well.

Dumeyer (1966) described the vadose zone sediments at the WRRC field site as a series of alternating beds of varying sand, silt, clay and gravel composition. On the basis of analysis of lithologic and

TABLE 1

LITHOLOGIC LOG
WRRC DRY WELL
(After Wilson, 1983)

DEPTH INTERVAL (feet)	DESCRIPTION OF MATERIAL	
0 - 3	SILTY SAND/ SANDY SILT:	
3 - 8	SILTY SAND:	With gravel and small cobbles.
8 - 10	SILTY COARSE SAND:	Gravel and cobbles to 5 inches. Some sandy clay lenses.
10 - 17	GRAVELLY MEDIUM SAND:	Some cobbles to 6 inches.
17 - 19	CLAYEY SAND & GRAVEL:	With cobbles to 8 inches.
19 - 20	SILTY SAND:	With gravel and cobbles.
20 - 20.5	SANDY GRAVELLY SILT:	With cobbles.
20.5 - 23.5	SILTY SAND:	Gravel and cobbles.

TOTAL DEPTH OF BOREHOLE: 23.5 FEET

geophysical logs collected from boreholes drilled at the site, Dumeyer divided the vadose zone deposits into three major units: Quaternary alluvium nearest the surface, underlain by Quaternary Basin Fill, which is in turn underlain by Tertiary Sediments. The Quaternary Alluvium unit is described as being about 30 feet thick with fair-sorted layers of sandy silt in the upper half and poor to fair sorted sandy gravel in the lower half (Dumeyer, 1966). Drilling time logs for wells at the site show water loss and decreased drilling rate over about the lower 10 feet of the unit, indicating a bed of permeable gravel and cobbles. The bottom of the unit is described as an inferred disconformable contact with the Basin Fill. The lower (highly permeable) bed may represent a basal conglomerate from the Quaternary Basin Fill formed on an erosion surface. Grain-size analyses performed by Osborne (1969) during construction of a recharge well and several monitor wells at the site indicate that the materials in the upper 30 feet are predominantly sands and gravels. The percentage of gravel increased in these wells to a maximum at a depth of about 30 to 35 feet, at which point the percentage decreased.

Dumeyer (1966) described the Quaternary Basin Fill as extending from approximately 30 to 80 feet below land surface. The material in this zone is described as beds of fines, silt and clay, sand and gravel, with an increase of fines with depth (Osborne, 1969). Overall median grain diameter decreased from 30 to 80 feet. Sand and gravel grains are composed of volcanic particles and Catalina Gneiss. The unit is described by Dumeyer as representing a period of deposition when drainage carried outwash into the area from both the Tucson and Catalina Mountains.

The contact with the older Tertiary sediments, the lowest unit identified, exists at about 80 feet, where the lithologic composition of the grains becomes entirely volcanic, with no Catalina Gneiss present (Dumeyer, 1966). The percent of silt and clay is higher overall in this unit than in the overlying ones. Osborne indicates that a moderate to high degree of cementation may have been present in the unit. Median grain diameter in lithologic samples collected during drilling a recharge well near the location of the test dry well increased at depths of 105 to 110 feet, from 118 to 121 feet, from 135 to 138 feet, and from 148 to 150 feet (Osborne, 1969). Neither the Dumeyer or Osborne studies specifies a depth to the bottom of the Tertiary sediments.

SITE FACILITIES

The WRRC Field Laboratory is equipped with an assortment of vadose zone monitor wells, neutron logging holes, ground water observation wells, a large supply well, a test recharge well, a recharge pit and a test dry well **(Figure 3)**. The site includes a water chemistry laboratory and shop facilities. The test dry well is situated in the center of a group of neutron moisture logging holes, access wells 6, 7, 9 and 10. These holes, cased with 2-inch seamless steel to about 100 feet, were used to monitor vadose zone moisture content during recharge tests on the dry well. Observation wells A, B, and C are cased with 12-inch

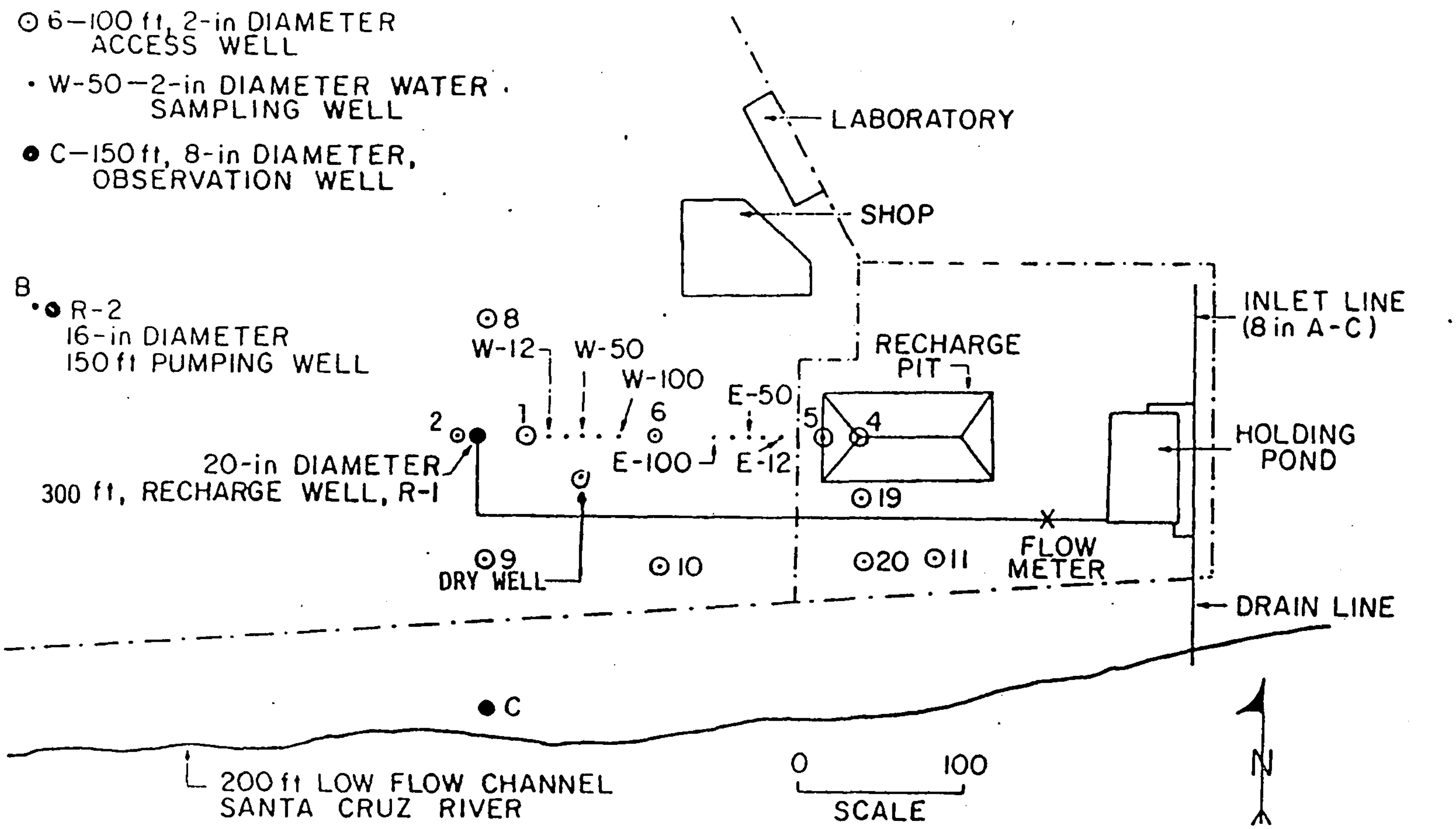


FIGURE 3. THE UNIVERSITY OF ARIZONA WATER RESOURCES RESEARCH CENTER ARTIFICIAL RECHARGE FACILITIES

stainless steel to a depth of 150 feet, and perforated from 15 to about 150 feet. The 20-inch diameter, 300-foot deep supply well R-1 is located to the northwest of the dry well, and was used to supply water for recharge tests on the dry well. A series of ten 2-inch PVC galvanized steel cased vadose zone monitor wells north of the dry well have been used to obtain water samples during recharge tests performed on the dry well (Wilson, 1983).

DERIVATION OF SOIL HYDRAULIC PROPERTIES

Performance of the computer simulations required the input of the principal parameters governing unsaturated flow, including soil water pressure head, relative hydraulic conductivity, and specific moisture capacity as functions of volumetric soil water content; soil porosity and saturated hydraulic conductivity. Soil moisture and hydraulic properties for Material 1 were determined using a combination of field data, laboratory analysis and analytical methods. Data collected during previous studies of vadose zone hydrology in the vicinity of the Tucson Basin (Coelho, 1974 and Guma'a, 1978) were used to determine the soil moisture hydraulic properties for Materials 2 and 3.

A. SOIL MATERIAL 1

A sample of the soil material surrounding the lower compartment of the WRRC experimental dry well, designated as Material 1 for the case study simulations, was retrieved from drill cuttings collected during construction of the dry well. This soil material constitutes a gravelly, sandy loam.

Soil Moisture Retention

Soil water under partially saturated conditions is subjected to the capillary suction forces of the soil pores. This suction can be expressed in terms of negative pressure head, and varies with soil moisture content in a manner unique to a given soil. The relationship between soil water suction (or negative pressure) head and volumetric soil moisture content describes the soil moisture retention curve. The soil moisture retention curve for the Material 1 soil sample was derived from an analysis performed at the University of Arizona Soils and Water Testing Laboratory. The analysis involves a porous plate assembly attached to an air-tight sampling chamber to partially desaturate a soil sample by applying a known pressure. This method is commonly used in soil science applications (Black et al., 1965). The sample chamber was filled with a known volume of soil and saturated with water. Soil grains with a diameter of over 2 mm were sieved from the sample for the analysis. A known suction was then applied to the sample on the porous plate, whereby a volume of water was drained from the sample. After the sample had reached a state of equilibrium under the given pressure, it was removed and weighed. The sample was then oven-dried at a temperature of 104°F for 24 hours, evaporating all of the soil water in the sample. The dried sample was then weighed again. The difference in wet and dry weights for the sample yielded the exact weight of the water in the sample under the specified suction. Conversion of this weight to volume allowed the calculation of volumetric water content for the sample under the specified suction, yielding a point on the soil moisture retention curve. This process was performed six times using pressures of 0.05, 0.1, 0.33, 1.0, 5.0, and 15.0 bars of suction, respectively. This combination of points describes the shape of the moisture retention curve.

Saturated Moisture Content, Porosity and Initial Moisture Content

The matrix porosity, or void ratio, of Material 1 was estimated from the saturated volumetric moisture content, saturated moisture content (θ_S) of the soil surrounding the dry well. The θ_S was measured from neutron moisture logs from 2-inch diameter access holes near the dry well during recharge experiments performed on the dry well and nearby recharge well (Wilson, 1983; **Figure 3**). A representative value for the porosity was obtained by averaging the measured θ_S values from more than a hundred moisture logs obtained from access holes 6, 7, 9, and 10 (Courtesy of University of Arizona Water Resources Research Center). The resulting θ_S and porosity value was about 37 percent, or described as a volume fraction, 0.37.

In addition, an initial, or background, volumetric moisture content of the vadose zone is required for the computer simulations. A representative value for initial moisture content was obtained by averaging background values measured by neutron logs, as for porosity. The resulting value used in the Case 1 simulation was 13 percent, or 0.13 as a volume fraction.

Relative Hydraulic Conductivity

Van Genuchten (1978), presented a method for predicting the hydraulic conductivity of unsaturated soils, based on a knowledge of the $\psi(\theta)$ relationship. Van Genuchten built on the previous theory of Mualem (1976). Mualem's method allowed for prediction of the $K(\psi)$ curve based on an analytical expression developed for the soil moisture retention curve. Van Genuchten utilized the fact that the $\psi(\theta)$ relationship is a continuous function with a continuous slope, to develop another equation for relative hydraulic conductivity as a function of pressure head or water content. Results of calculations of the $K(\psi)$ functions for a variety of soils using the van Genuchten method have been shown to compare very well with experimental results for the same soils (van Genuchten, 1980). This method presents an attractive alternative to expensive and time consuming field and laboratory methods.

Mualem (1976) derived the following equation for K_r, the relative hydraulic conductivity, defined as

(1) $K_r(\theta) = K(\theta)/K_s$

where subscript s denotes saturation, from the soil-moisture retention curve, described by $\psi(x)$:

(2) $$K_r = \Theta^{1/2}\left[\int_0^{\Theta} \frac{1}{\psi(x)}\,dx \Big/ \int_0^{1} \frac{1}{\psi(x)}\,dx\right]^2$$

Here ψ denotes suction head, expressed as a function of dimensionless water content, Θ:

(3) $$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r}$$

θ_r = residual moisture content

θ_s = saturated moisture content

Van Genuchten provides the general form of a $\Theta(\psi)$ function which serves as a solution to (2) above:

(4) $$\Theta = \left[\frac{1}{1 + (\alpha\psi)^n}\right]^m$$

From equations (2) - (4), van Genuchten derives a simple, closed-form expression for $K_r(\Theta)$, given as:

(5) $$K_r(\Theta) = \Theta^{1/2}\left[1 - (1 - \Theta^{1/m})^m\right]^2$$

$m = 1 - 1/n$

$0 < m < 1$

The parameters m, n, and α must be estimated from soil moisture retention data. For S_p, taken as the slope $d\psi/d\theta$ at a point P located half way between the values of θ_r and θ_s on the $\psi(\theta)$ curve, m can be estimated using the expressions

$$(6) \quad m = \begin{cases} 1 - \exp(-0.8\, S_p) & 0 < S_p \leq 1 \\ 1 - \dfrac{0.5755}{S_p} + \dfrac{0.1}{S_p^{\,2}} + \dfrac{0.025}{S_p^{\,3}} & S_p > 1 \end{cases}$$

The parameters n and α can be evaluated from

$$(7) \qquad n = 1/(1-m)$$

$$(8) \qquad \alpha = \frac{1}{\psi_p}\left(2^{1/m} - 1\right)^{1-m}$$

assuming ψ at point P is known.

The computer program SOHYP developed by van Genuchten (1978) incorporates the above methodology to yield an output containing a tabulation of corresponding values of θ, ψ, and K_r. The program requires for input values for θ_r, θ_s, K_s, estimates of m, n, and α, and a tabulation of corresponding θ and ψ values.

For Material 1, the already established ψ vs. θ curve was converted to tabular form for input into the model. The value of $\theta_s = 0.37$ was equated with the estimated soil matrix porosity. Using the methods of van Genuchten described above, estimates of the remaining parameters were taken as $m = 0.285$, $n = 1.4$, and $\alpha = 2.75$. A table of values describing the K_r vs. θ curve was generated and plotted (Figure 4).

Adjustment of Soil Hydraulic Parameter Curves for Gravel Fraction

The laboratory technique for determining the soil moisture retention curve necessitated sieving the gravel fraction of the soil bulk from the samples. As a result, the curve was derived to represent only the non-gravel, or "soil" fraction of the material. The effect of the gravel fraction on the hydraulic properties of the "bulk" (i.e. including the gravel fraction) medium is non-negligible, and should be accounted for. As the overall void space in a given volume of bulk material is lower for an equal volume of soil, the volumetric water content at a given pressure head will be lower for the bulk medium. By the same reasoning, the hydraulic conductivity at a given pressure head will be lower. A method of adjusting the soil moisture release and relative hydraulic conductivity curves derived for the soil fraction, for the effects of the gravel fraction as discussed by Bouwer and Rice (1984), was employed in this study. The values of non-gravel fraction volumetric moisture content, θ_{ng}, corresponding to each value of pressure head in the soil moisture release curve were adjusted for bulk moisture content, θ_b, using the relation

$$\theta_b = (1 - V_r)\,\theta_{ng}$$

θ_b = bulk moisture content
V_r = volume fraction of gravel (grains 2.0 mm or greater)
θ_{ng} = non-gravel fraction soil moisture content

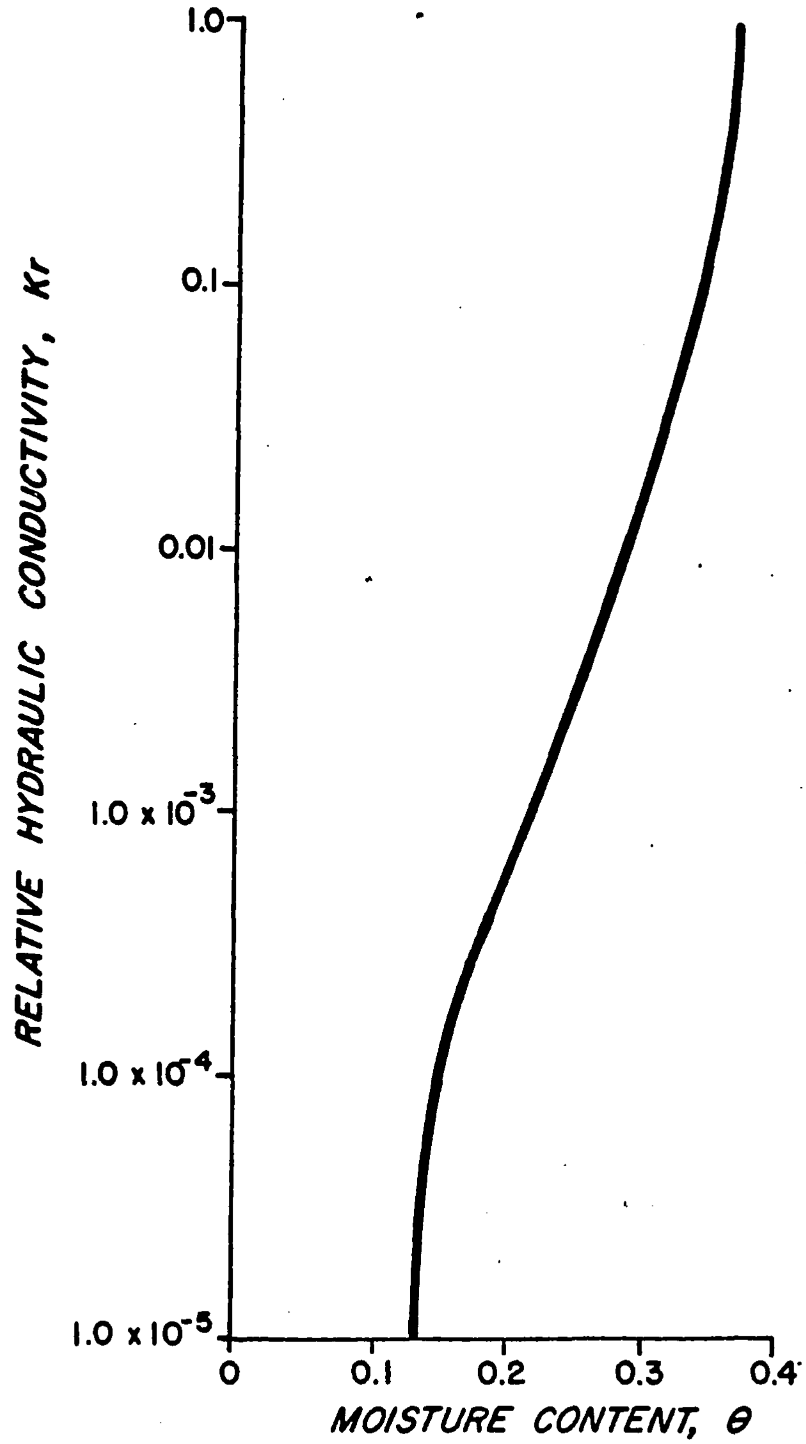


FIGURE 4. RELATIVE HYDRAULIC CONDUCTIVITY CURVE
MATERIAL 1 NON-GRAVEL FRACTION

The value of V_r for material 1, as derived from the sieve analysis, was 0.184. The complete table of values of ψ vs. θ_{ng} (soil only) was converted to ψ vs. θ_b (soil bulk) and plotted (Figure 5).

The adjustment of the soil relative hydraulic conductivity as a function of water content curve to that for the soil bulk is more involved. The method is described briefly as follows, and illustrated in Figure 6. First, a curve of hydraulic conductivity (K) vs. pressure head (ψ) for the soil matrix was derived using the relative hydraulic conductivity and pressure head as functions of water content. This curve is plotted on a log-log scale. Next, the value of saturated hydraulic conductivity for the bulk was determined. This was accomplished by using the Unsat 2 model in conjunction with data on intake rate obtained during recharge tests on the WRRC dry well, as explained in a later section. The curve of K vs. ψ is then shifted down along the vertical axis until the value of soil saturated hydraulic conductivity (K_s) is aligned with bulk saturated hydraulic conductivity. The curve remaining below the K_s of the bulk constitutes the K vs. ψ curve for the bulk. A bulk relative hydraulic conductivity vs. water content curve was then constructed using the bulk soil moisture retention curve (Figure 7).

Specific Moisture Capacity

Specific moisture capacity is defined as the change in soil water content per change in soil water suction. The curve of specific

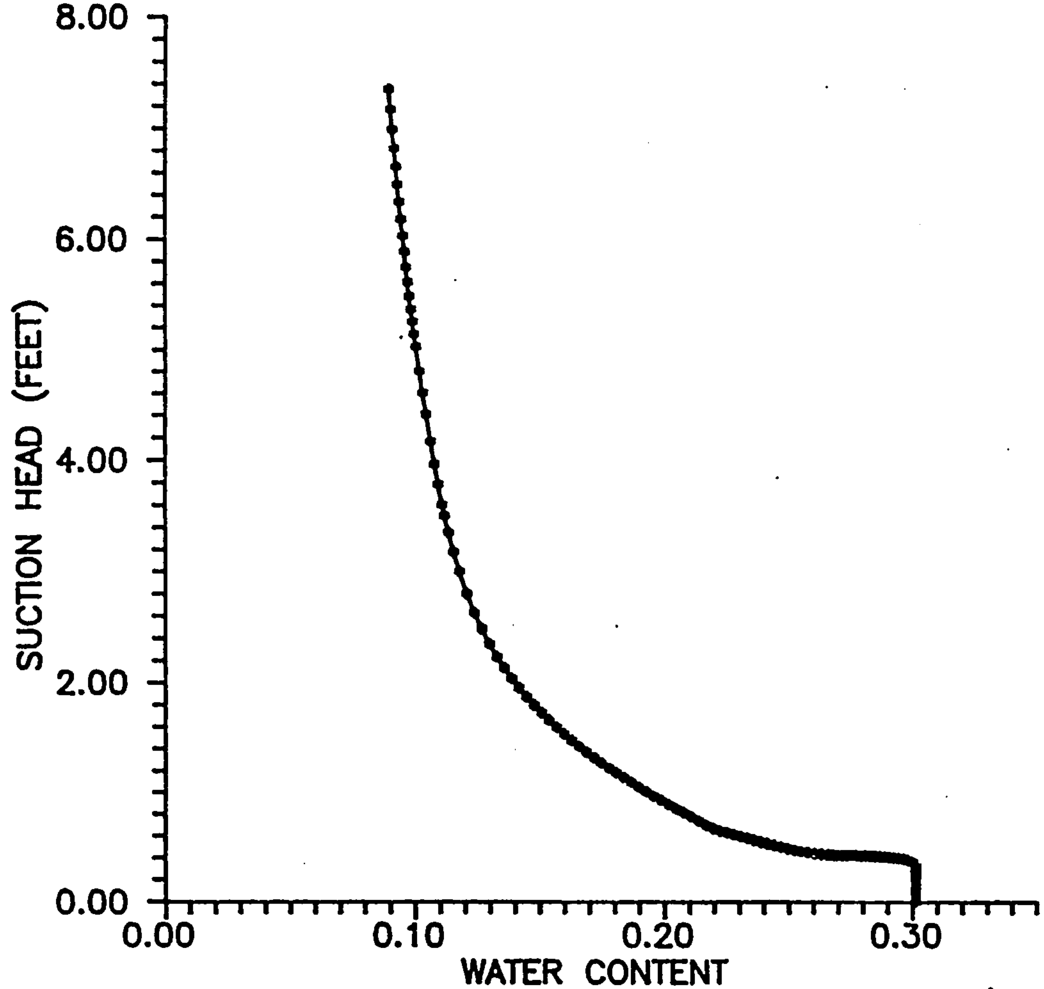


FIGURE 5. SOIL MOISTURE RETENTION CURVE, MATERIAL 1

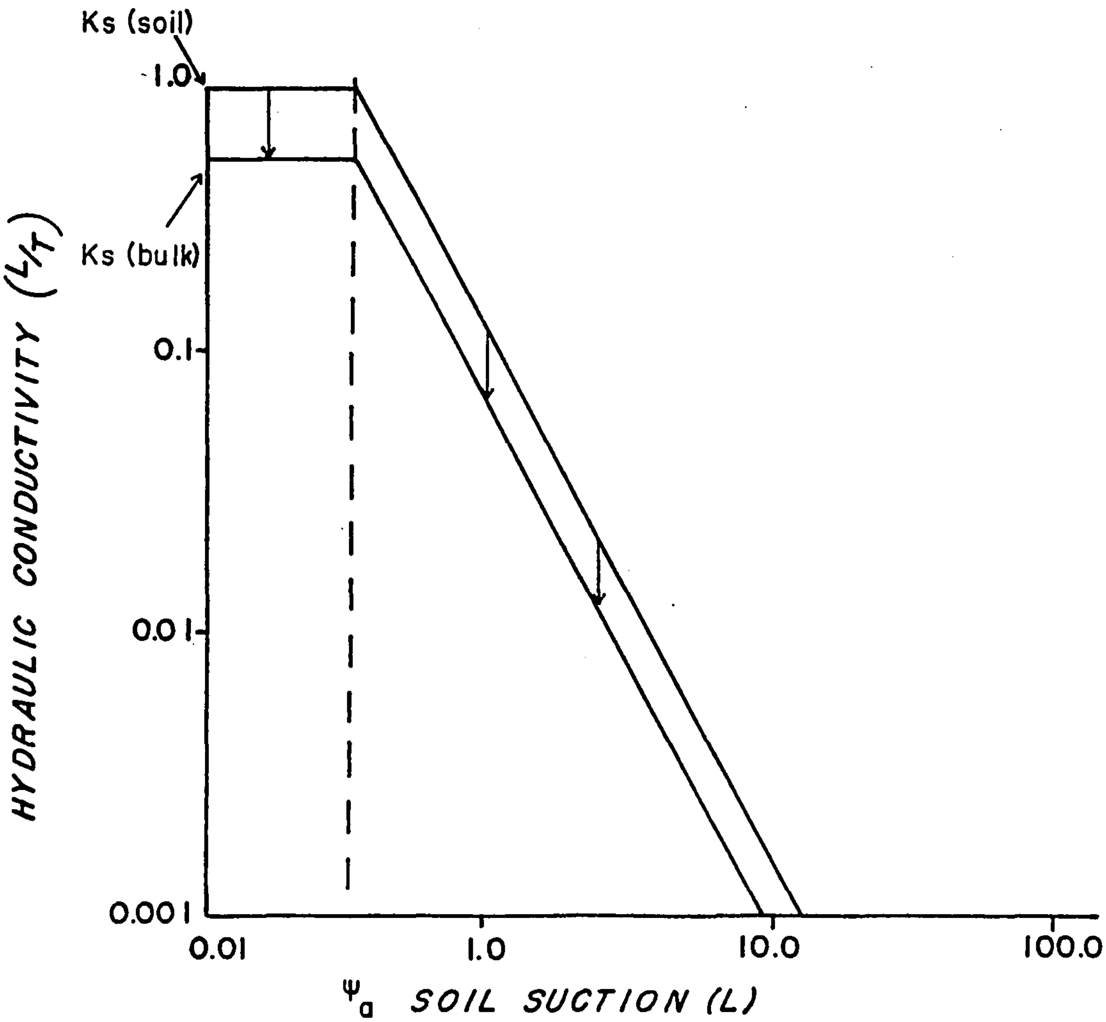


FIGURE 6. ADJUSTMENT OF K(ψ) CURVE FOR GRAVEL FRACTION

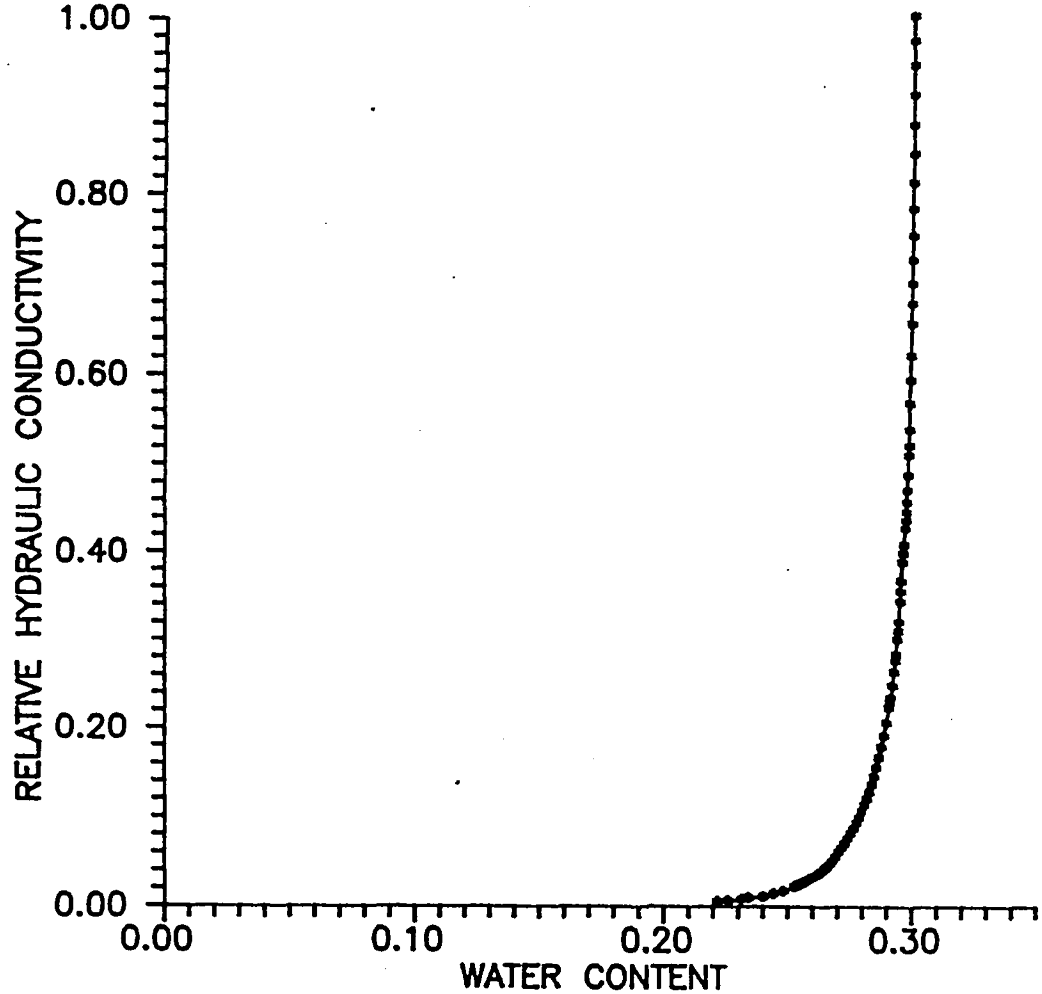


FIGURE 7. RELATIVE HYDRAULIC CONDUCTIVITY CURVE, MATERIAL 1

moisture capacity versus suction is derived from the soil moisture retention curve in the Unsat 2 computer code using linear interpolation. A cubic spline curve-fitting process was used to smooth the specific moisture capacity curve **(Figure 8)**.

Saturated Hydraulic Conductivity

The value of saturated hydraulic conductivity (K_s) for Material 1 was estimated using data obtained during recharge tests on the dry well in conjunction with the Unsat 2 model. A series of recharge tests was performed on the WRRC test dry well in conjunction with a study done by Wilson (1983) that examined the fate of tracers introduced into the dry well recharge water. During these tests, water was pumped from nearby well R-1 and routed into the dry well shaft via a conduit pipe. The rate of discharge from the supply well and recharge into the dry well was varied between the recharge tests, but kept constant during each individual test. Each test lasted several hours. Water level measurements were taken inside the dry well using an electric sounder lowered into the injection tubing. Water levels rose steadily during the first minutes of a recharge test, but maintained an approximately constant level after about one hour of recharge. For each test, a constant level of borehole head was be associated with a constant rate of recharge. For the fifth recharge test, a constant recharge rate of about 207 gallons per minute (gpm) produced a relatively constant borehole head of about 6.5 feet above the bottom of the dry well shaft. Borehole head measurements varied between 6.4 and 6.6 feet for this test. The

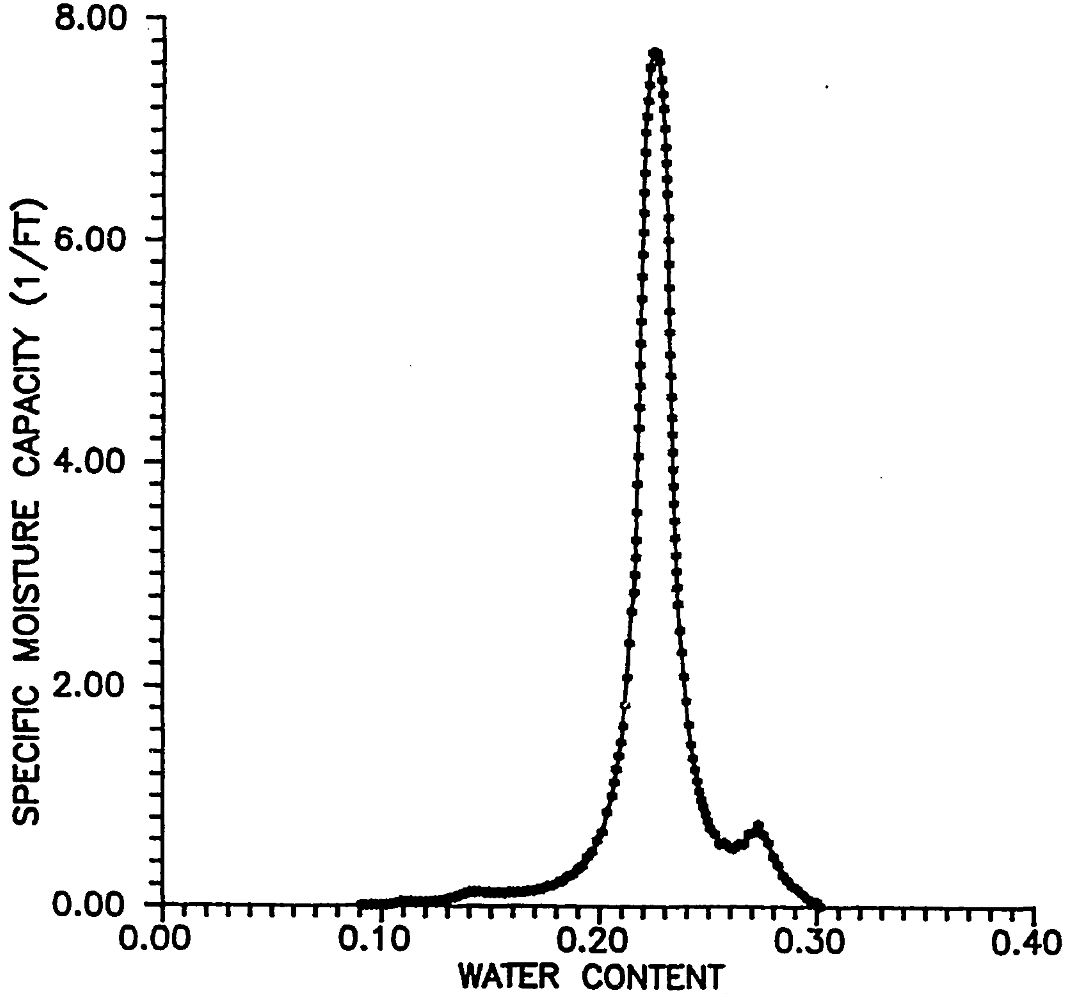


FIGURE 8. SPECIFIC MOISTURE CAPACITY CURVE, MATERIAL 1

recharge rate of about 207 gpm for the fifth test most closely approximated the manufacturer's design long-term flow rate of 0.5 cfs (about 224 gpm).

A series of calibration runs was performed with the Unsat 2 model using the Material 1 hydraulic property curves described in the preceding sections. By specifying the observed constant borehole head as a boundary condition of the borehole wall in the model, the value for K_s was varied between runs, and the resulting constant intake rate observed. For each calibration run, a period of between 0.5 and 0.75 hours was required before the intake rate stabilized to a constant value. After this point, the intake rate varied between 0.002 to 0.003 cfs or about 0.9 to 1.3 gpm. The value for K_s was varied between runs in this manner until the correct intake rate of 207 gpm, corresponding with the field-observed constant borehole head of 6.5 feet, was observed in the model results. The resulting calibrated value of K_s for Material 1 was 6.7 feet per hour.

B. SOIL MATERIAL 2

Direct measurement of soil hydraulic properties for the soil materials below a depth of 30 feet at the dry well site were not obtained during the course of this study. Instead, these data were derived from previous studies done at the WRRC site and a separate research site located northwest of Tucson. These studies were performed by Osborne (1969) and Coelho (1974). The Coelho study was performed at the University of Arizona Agricultural Experiment Station, located near Marana, Arizona, 29 kilometers northwest of Tucson. This study involved analyzing the soil hydraulic properties from 180 soil samples collected from various depths in order to statistically characterize their spatial variability. The soil hydraulic properties for the Material 2 soil used in this study were estimated by first establishing a representative grain-size distribution for the zone below 30 feet and above the water table at the WRRC site from the Osborne (1969) data, and equating them to the hydraulic properties derived by Coelho (1974) for a soil of like composition. The grain-size distribution of the Material 2 soil zone was estimated as the mean of the range of measured values for gravel, sand, and clay and silt (fines) for samples from this zone. Clay and silt comprised an average of about 34 percent of the non-gravel fraction by weight of the samples, and sand comprised an average of about 66 percent by weight. These values approximately corresponded to a sample from the Coelho study having a content of about 36 percent of the non-gravel fraction by weight of fines and about 64 percent by weight of sand. The average volumetric gravel content for the Material 2 zone was about 30 percent. As such, Material 2, as Material 1, was classified as a gravelly sandy loam. Laboratory analysis of the soil samples was performed to determine the volumetric moisture content at nine values of applied suction varying between zero and 15 bars. The method used was similar to the one described for the Material 1 analysis. The Bouwer method of adjustment of the resulting points describing the soil moisture release curve was used in the same manner as described for

Material 1. The adjusted curve was smoothed using the cubic spline curve-fitting routine **(Figure 9)**.

The van Genuchten analytical model was utilized to derive the relative hydraulic conductivity curve. This curve was also adjusted for the gravel fraction using the method of Bouwer (1984) **(Figure 10)**. The specific moisture capacity curve was derived initially using the Unsat 2 model, and smoothed using the spline routine **(Figure 11)**.

Saturated hydraulic conductivity for the Material 2 soil sample was estimated as part of the Coelho (1974) study using the falling head permeameter method. The resulting value was reported as 0.29 feet per hour.

Though Material 1 and Material 2 fall under the same textural description of gravelly sandy loam, the two soils have widely differing values of K_S. Though overall textural composition of a soil provides a general indication of the likely range of its K_S, other factors such as packing, angularity, and representative diameter of the soil grains combine to determine the magnitude of K_S. Freeze and Cherry present saturated hydraulic conductivity as

$$K = \frac{Cd^2 \rho g}{\mu}$$

where ρ is fluid density, d is grain diameter, g is the gravitational acceleration constant, μ is dynamic viscosity, and C is a proportionality constant taking into account the effects of sorting, sphericity,

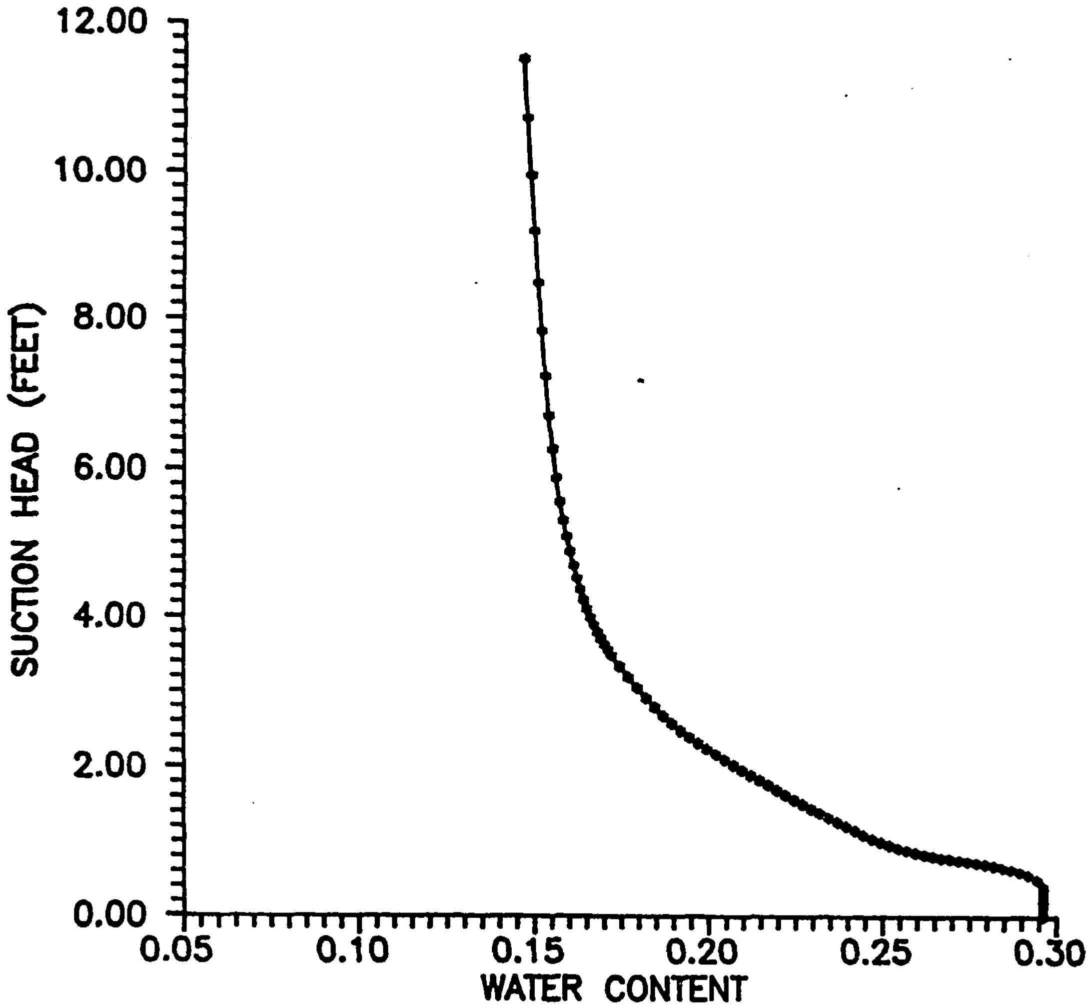


FIGURE 9. MOISTURE RETENTION CURVE, MATERIAL 2

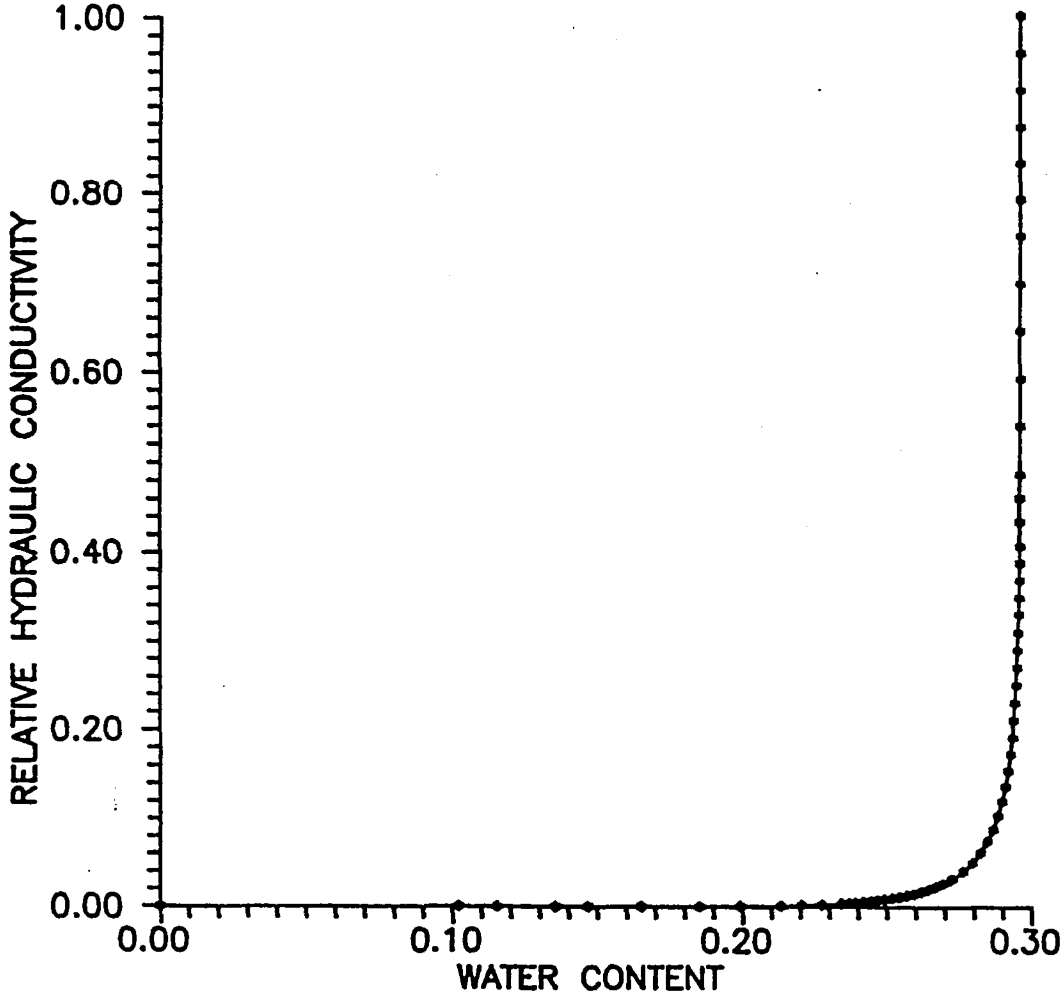


FIGURE 10. RELATIVE HYDRAULIC CONDUCTIVITY CURVE, MATERIAL 2

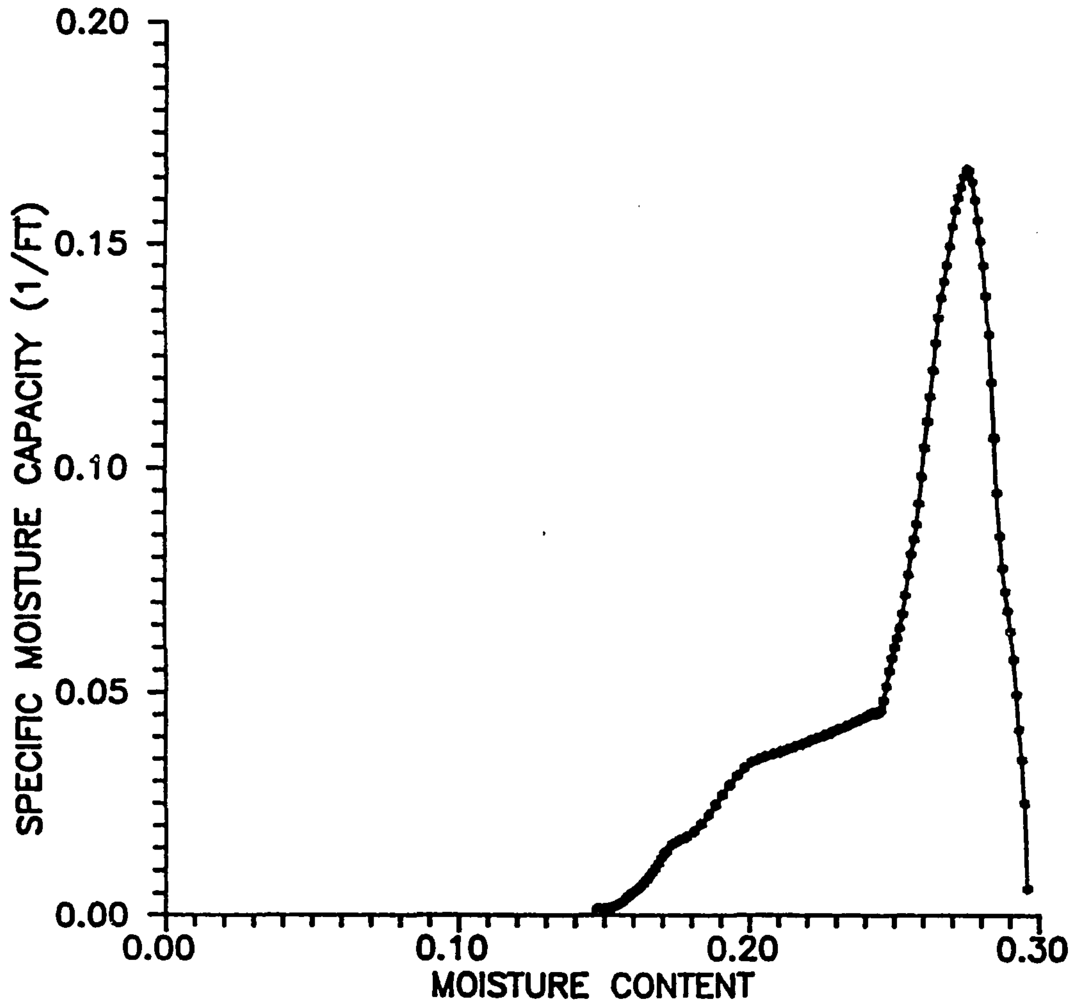


FIGURE 11. SPECIFIC MOISTURE CAPACITY CURVE, MATERIAL 2

and roundness of grains, and their packing (Freeze and Cherry, 1979). The predictive equation of Kozeny and Carmen (Bear, 1972) takes the form

$$K = \left(\frac{\rho g}{\mu}\right) \left[\frac{n^3}{(1 - n)^2}\right] \left(\frac{d_m{}^2}{180}\right)$$

where n is medium porosity and d_m is the mean particle size. In the Kozeny-Carmen equation, the effects of the proportionality constant C are presented as a function of the porosity n. For soils with approximately equal porosity, such as Material 1 and 2, differences in mean grain diameter have the primary influence on differences in K_s. Results of the grain size analysis of Osborne (1969) for the well R-1 cuttings show that decreased median grain size in the Material 2 zone relative to the Material 1 zone may account for more than a factor of 10 decrease in K_s for Material 2, using the Kozeny-Carmen equation. The reduced K_s for Material 2 is therefore probably mostly attributable to reduced overall grain size.

Initial moisture content for the Material 2 soil zone was averaged from a sample of neutron moisture logs taken from access holes near the dry well site, as for Material 1. The averaged value for volumetric water content in this zone was determined to be about 19 percent.

C. SOIL MATERIAL 3

A third soil material representative of subsurface conditions found in the Tucson Basin was selected to represent the soil zone below 30 feet and above the water table for the Case 3 simulation. In Case 1, this zone was represented by soil Material 1, having relatively high permeability. In Case 2, this zone was represented by the relatively low permeability Material 2. The Case 3 simulation was performed to simulate subsurface conditions resulting from dry well recharge into a soil of moderate permeability. The hydraulic properties of this soil material were obtained from a study by Guma'a (1978) similar to that of Coelho (1974). The hydraulic properties of this sandy loam type soil were adapted for use in the Unsat 2 model by Wilson et. al. (1986). The moisture release, relative hydraulic conductivity, and specific moisture capacity curves prepared in that study were incorporated into the Case 3 simulation **(Figures 12, 13, and 14)**. The saturated hydraulic conductivity was determined as 1.0 feet per hour, and the background moisture content of the soil, which was equated with field capacity, was determined as about 22 percent.

Hysteresis and Anisotropy

The tendency of the moisture retention properties of a soil to vary depending on whether the soil is undergoing wetting or drying, commonly known as hysteresis, is not treated in the Unsat 2 model. The model accepts only a single table of values from which it correlates water content, soil suction head, relative hydraulic conductivity and specific moisture capacity. As such, only one set of curves for these parameters was used in these simulations. Due to the particular method used to derive the soil moisture release curves for Materials 1, 2 and 3 in the

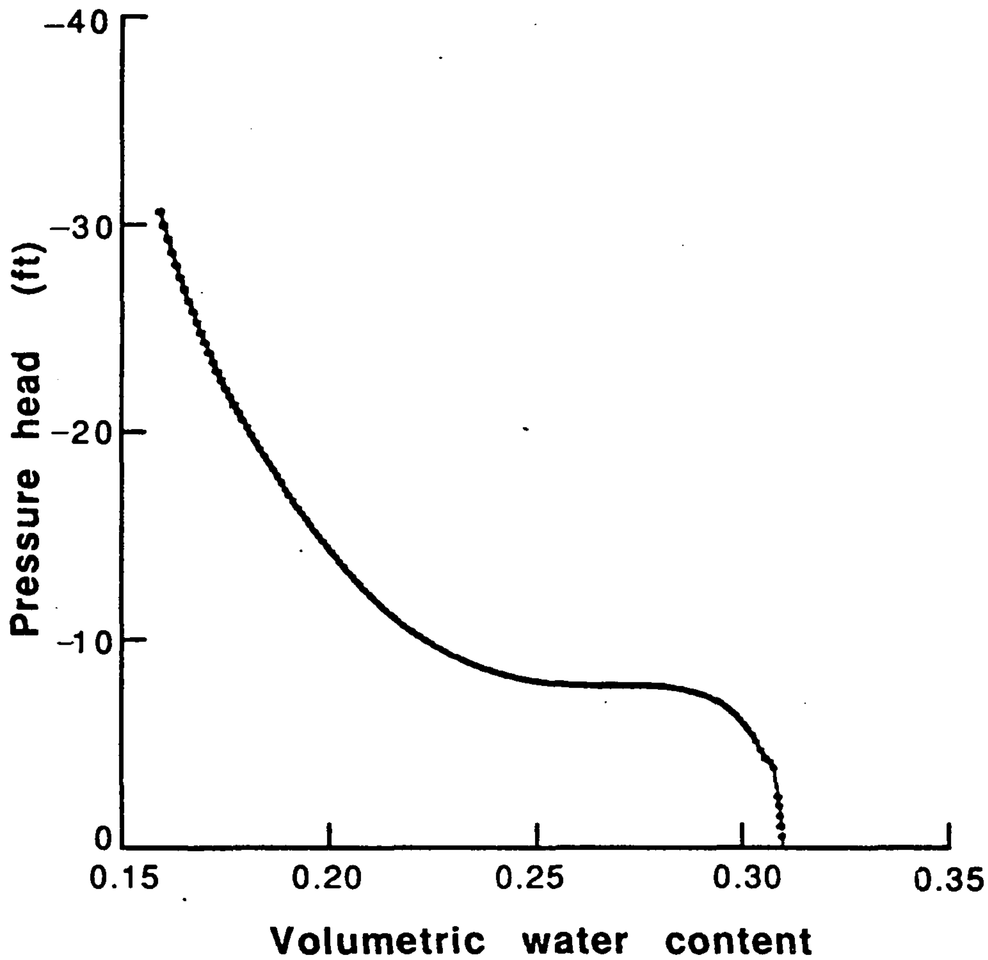


FIGURE 12. MOISTURE RETENTION CURVE, MATERIAL 3 (After Wilson, et.al., 1986)

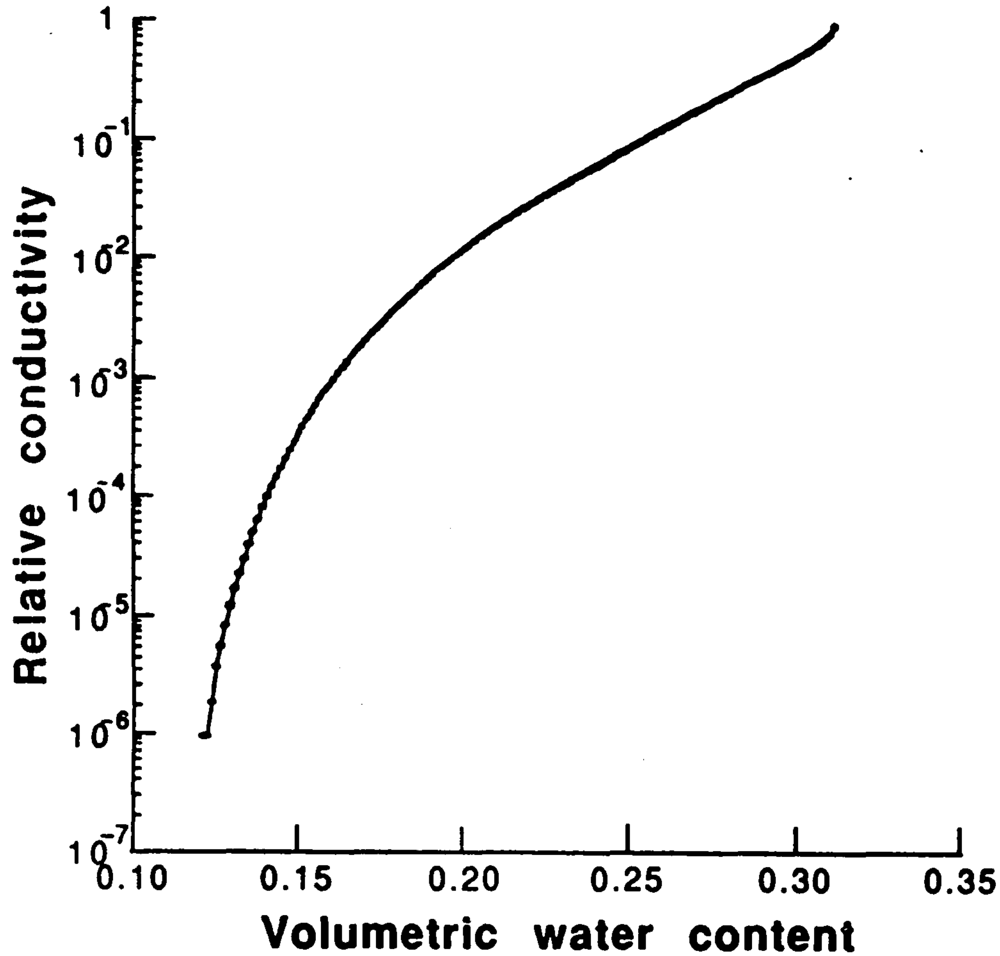


FIGURE 13. RELATIVE HYDRAULIC CONDUCTIVITY CURVE, MATERIAL 3 (After Wilson, et.al., 1986)

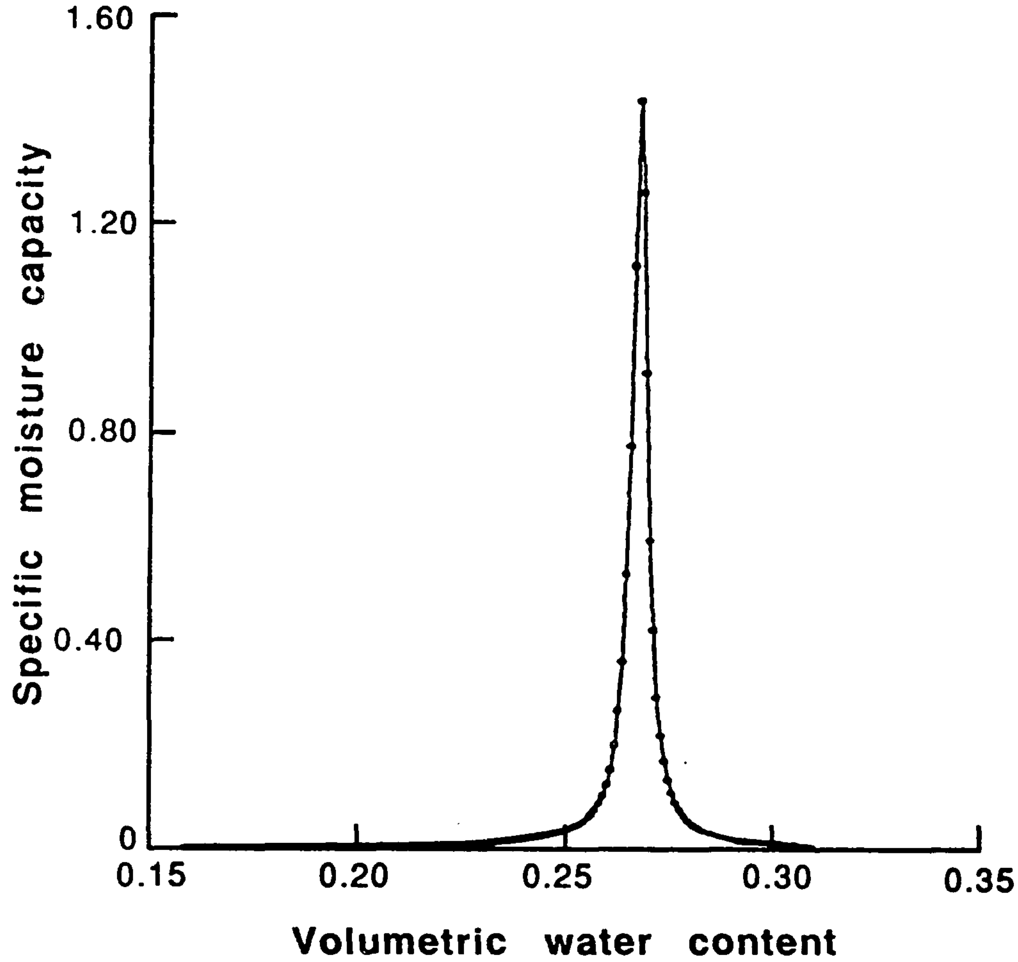


FIGURE 14. SPECIFIC MOISTURE CAPACITY CURVE, MATERIAL 3 (After Wilson, et.al., 1986)

laboratory, these curves represent the "first drying", or "de-saturation" leg of the soil moisture release curve.

This means that the model, which addresses both wetting and drying conditions in the simulations, is best suited to the latter. Soil moisture content, and hence relative hydraulic conductivity and percolation rate, will be somewhat overestimated for wetting conditions. Simulation of dry well recharge under hysteretic conditions would require knowledge of the soil moisture retention curve under wetting conditions, and a variably saturated flow model that accommodates hysteresis. As of the time of this study, such a model was not available, and modeling the effects of hysteresis was precluded from the scope of this study.

In a layered alluvial basin, some variability between horizontal and vertical hydraulic conductivity, or anisotropy, may exist. As a result, water under the same hydraulic gradient may travel faster in the horizontal direction than the vertical direction. The values of saturated hydraulic conductivity used for the soil materials in this study should not be considered estimates of the vertical or horizontal hydraulic conductivity, but as a "resultant" hydraulic conductivity lying somewhere between the vertical and horizontal. In the case of Material 1, the recharge tests from which K_s was estimated were measuring overall intake rate of the dry well, with no differentiation between vertical and horizontal components of subsurface flow. Due to the unavailability of in-situ data or undisturbed soil samples from the study site, the effects of anisotropy within specific soil material zones was not addressed within the scope of this research. However, in considering layered subsurface conditions, the Case 2 and Case 3 simulations illustrate regionally anisotropic flow in the subsurface, due to the differing hydraulic properties of the individual layers. This phenomenon, described as "layered heterogeneity" (Freeze and Cherry, 1979), will result in anisotropic flow conditions for the system as a whole.

SOIL SPECIFIC SURFACE

Soil specific surface is a measure of the surface area of soil particles per unit weight. For the purpose of comparing soil surface area available for sorption of recharge-borne solutes, estimates of the maximum soil surface area contained within a common degree of saturation were formulated for each of the case studies. First, a common degree of saturation upon which to base the comparison was established. The soil within the zone of near-saturation during a recharge event lends itself to volume calculations, as this zone is usually well-defined. However, surface adsorption processes may be more effective in the unsaturated capillary fringe of the recharge plume, due to a higher ratio of soil surface area per weight of water. The zone of 80 percent saturation was chosen as the basis of comparison of soil surface area because: A) the zone was well-defined for each case, and B) it incorporated part of the capillary fringe of the recharge plume.

White (1979) presented representative specific surface values for various alluvial aquifer soil materials from adsorption experiments **(Table 2)**. These experiments involved estimating specific surface according to the amount of nitrogen gas or ethylene glycol adsorbed on particle surfaces for various mineral types and grain-size classes. To utilize these values of specific surface, the approximate grain-size breakdown of each of the three soil materials used in the modeling study was used to calculate a weighted-average specific surface for each soil material. Using bulk density values representative of each soil material, the total weight of each soil material was calculated based on an estimated volume of soil within the zone of 80 percent saturation. Finally, these values of weight were used to estimate the total soil surface area within the zone of 80 percent saturation at its maximum extent for each simulation.

Bulk Density and Grain-Size Distribution

Values of dry bulk density for each of the three soil materials were determined directly or obtained from data reported in the literature. Grain-size distributions for the three soil types were obtained from field data presented by Osborne (1969), Coelho (1974) and Guma'a (1978).

MATERIAL 1: The approximate grain-size distribution for Material 1 was obtained from sieve analyses performed on cuttings collected from the upper 30 feet of borehole for recharge well R-1, located about

TABLE 2

SPECIFIC SURFACE VALUES FOR ALLUVIAL AQUIFER SOIL MATERIALS (After White, 1979)

MINERAL OR SIZE CLASS	SPECIFIC SURFACE (m^2/g)
Coarse sand	0.01
Fine sand	0.1
Silt	1.0
Kaolinites	5 - 100
Hydrous micas (illites)	100 - 200
Vermiculites and mixed layer minerals	300 - 500
Montmorillonite	700 - 800
Amorphous and hydrated iron and aluminum oxides	100 - 300

65 feet northwest of the dry well. The grain-size distribution for this soil zone was observed to be approximately 30 percent gravel, 45 percent sand, and 25 percent fines (silt and clay) by weight (Osborne, 1969). The distribution of grain-size within the fines was estimated based on field data presented by the U.S. Department of Agriculture (1972) for Pima County soils of similar grain-size distribution and bulk density. These data indicated that fines were distributed approximately evenly between silt and clay sized particles. Estimated breakdown of the clay fraction into various clay-sized particles and mineral types was based on data collected in Pima County by Hendricks (1987) **(Table 3)**. These estimates were used to define the clay mineral distribution for soil Material 1 **(Table 4)**. The category of "miscellaneous clay particles" represents various mineral types, such as quartz and feldspar, resulting from weathering of larger particles. The grain-size data were combined to provide a complete grain-size breakdown for Material 1 **(Table 5)**.

The sample of Material 1 soil obtained from the dry well cuttings was dried and sieved to separate particles greater than 2 millimeters, designated as the gravel fraction. Direct measurements of known volumes of Material 1 soil sample were made before and after sieving. These data were used to calculate a dry bulk density value for the bulk soil, determined as 1.48 grams per cubic centimeter (g/cc) **(Table 6)**.

MATERIAL 2: Grain-size distribution for the non-gravel fraction of soil Material 2 was reported as 64.1 percent sand, 17.5 percent silt, and 18.4 percent clay by weight (Coelho, 1974). The gravel fraction for

TABLE 3

REPRESENTATIVE DISTRIBUTION OF CLAY MINERAL TYPES FOR THE TUCSON BASIN
(After Hendricks, 1987)

CLAY MINERAL GROUP	PERCENTAGE BY WEIGHT
Montmorillonite	40
Illite	30
Kaolinite	20
Miscellaneous clay particles	5
Assorted amorphous and hydrated metal oxides	5

TABLE 4

ESTIMATED CLAY MINERAL DISTRIBUTION
FOR SOIL MATERIALS

	PERCENT OF MINERAL TYPE BY WEIGHT		
	..........SOIL MATERIAL..........		
CLAY MINERAL GROUP	1	2	3
Montmorillonite	5.00	4.48	4.76
Amorphous and hydrated metal oxides	0.625	0.56	0.595
Illite	3.75	3.36	3.57
Kaolinite	2.50	2.24	2.38
Miscellaneous clay particles	0.625	0.560	0.595

TABLE 5

ESTIMATED GRAIN-SIZE DISTRIBUTION FOR
CASE STUDY SOIL MATERIALS

MINERAL TYPE/PARTICLE SIZE	PERCENT OF MINERAL TYPE BY WEIGHT ....SOIL MATERIAL.......... 1	 2	 3
Gravel	30	39	NA
Sand	45	39	58.7
Silt	12.5	10.8	29.4
Clay	12.5	11.2	11.9
Montmorillonite	5.00	4.48	4.76
Amorphous and hydrated metal oxides	.625	.56	.595
Illites	3.75	3.36	3.57
Kaolinites	2.50	2.24	2.38
Miscellaneous clay particles	.625	.56	.595

NA = Not available

TABLE 6

BULK DENSITY OF CASE STUDY SOIL MATERIALS

Soil Type (Material)	Bulk Density (g/cm^3)
Gravelly Sandy Loam (Material 1)	1.48
Gravelly Sandy Loam (Material 2)	1.86
Sandy Loam (Material 3)	1.33

the Material 2 soil zone was estimated from the well R-1 cuttings sieve analysis as about 39 percent (Osborne, 1969). These data were combined with the data from Hendricks (1987) on clay particle distribution to obtain the complete Material 2 grain-size breakdown **(Table 5).**

Laboratory tests similar to those performed on Material 1 were performed on the Material 2 soil sample for soil moisture retention data, requiring separation of soil particles over 2 millimeters in diameter. Bulk density of the non-gravel fraction was reported as 1.52 g/cc (Coelho, 1978). Bulk density of the overall soil bulk was determined as 1.86 g/cc **(Table 6).**

MATERIAL 3: Grain-size distribution of soil Material 3 as reported by Guma'a (1978) was 58.7 percent sand, 29.4 percent silt, and 11.9 percent clay by weight. The clay fraction was further broken down according to the distribution reported by Hendricks (1987), to create an estimate of the complete grain-size distribution for Material 3 **(Table 5).** Dry bulk density for Material 3 was reported as 1.33 g/cc **(Table 6).**

Weighted Specific Surface

Values of specific surface representative of each soil material were derived by using the values presented by White (1979), weighted according to grain-size distribution. In the absence of grain diameter measurements within a specific particle size class (e.g. sand, silt, or clay), the mean value of the range specified was used.

A representative value of specific surface for the gravel fraction was determined separately. In general, specific surface varies inversely with grain-size diameter. As such, specific surface for the gravel fraction takes on a relatively minor weight in the overall specific surface for the soil material. Hillel (1971) defines specific surface (Am) in terms of the total surface (As) area to mass (Ms) ratio:

$$Am = \frac{As}{Ms}$$

Assuming that on the average the shape of the grains is best approximated by a spheroid, the specific surface is given as:

$$Am = \frac{6}{\rho_p d}$$

where ρ_p is the particle density and d is the grain diamter. Particle density for the predominantly granitic gravel was taken as 2.65 g/cm^3. Maximum particle diameter observed in cobbles from the soil sample was about 10 centimeters. Minimum grain diameter included in this fraction was 2mm, or 0.2 cm. Particle diameters for the gravel fraction appeared to be approximately normally distributed over this range. The range of specific surface calculated for the gravel fraction was 1.1×10^{-3} m^2/g to 2.20×10^{-5} m^2/g. The mean of approximately 5.7×10^{-4} m^2/g was used as the representative specific surface value for the gravel fraction.

The weighted specific surface values corresponding to each grain-size class were summed for each soil material to obtain a representative specific surface for the bulk of each material (Table 7). It is evident from these values that the clay content of the soil, and the percentage

TABLE 7

WEIGHTED SPECIFIC SURFACE ESTIMATES FOR CASE STUDY SOIL MATERIALS

MINERAL TYPE/GRAIN SIZE	WEIGHTED SPECIFIC SURFACE (M^2/G) SOIL MATERIAL 1	2	3
Montmorillonite	37.500	33.600	35.700
Illite	5.625	5.040	5.355
Kaolinite	1.312	1.176	1.249
Amorphous Metal Oxides	1.250	1.120	1.190
Miscellaneous Clay Particles	0.019	0.017	0.018
Silt	0.125	0.107	0.294
Sand	0.045	0.039	0.059
Gravel	1.71×10^{-4}	2.22×10^{-4}	NA
Total Weighted Specific Surface	45.876	41.099	43.865

NA - Not available

of montmorillonite in particular, has the largest influence on the specific surface.

COMPUTER SIMULATIONS

Finite Element Grid

The condition of radial symmetry of flow about the dry well was assumed in the computer simulations. Using the option of axisymmetric flow in Unsat 2, the finite element grid was constructed to represent a "slice" of the flow region extending radially outward from the center of the dry well, which was modeled as an open borehole at the upper left portion of the grid **(Figure 15)**. Nodal spacing was made fine in areas where large hydraulic head gradients were expected, such as near the borehole, and coarser where hydraulic head gradients were expected to be smaller, farther away from the borehole. Horizontal grid spacing varied from less than a foot in the immediate vicinity of the borehole to five feet at the outer reaches of the grid. Vertical grid spacing varied from 0.5 feet immediately below the borehole to 16 feet in the upper vadose zone far from the borehole, where no flow of drainage water was expected to take place. The total vertical distance covered by the grid was 110 feet. The total radial distance from the center of the dry well was 100 feet. The grid included a total of 1,415 nodes and 1,352 elements. These dimensions were found by trial and error to constitute the largest grid that could be accommodated by the CDC Cyber 175 computer with which the simulations were performed.

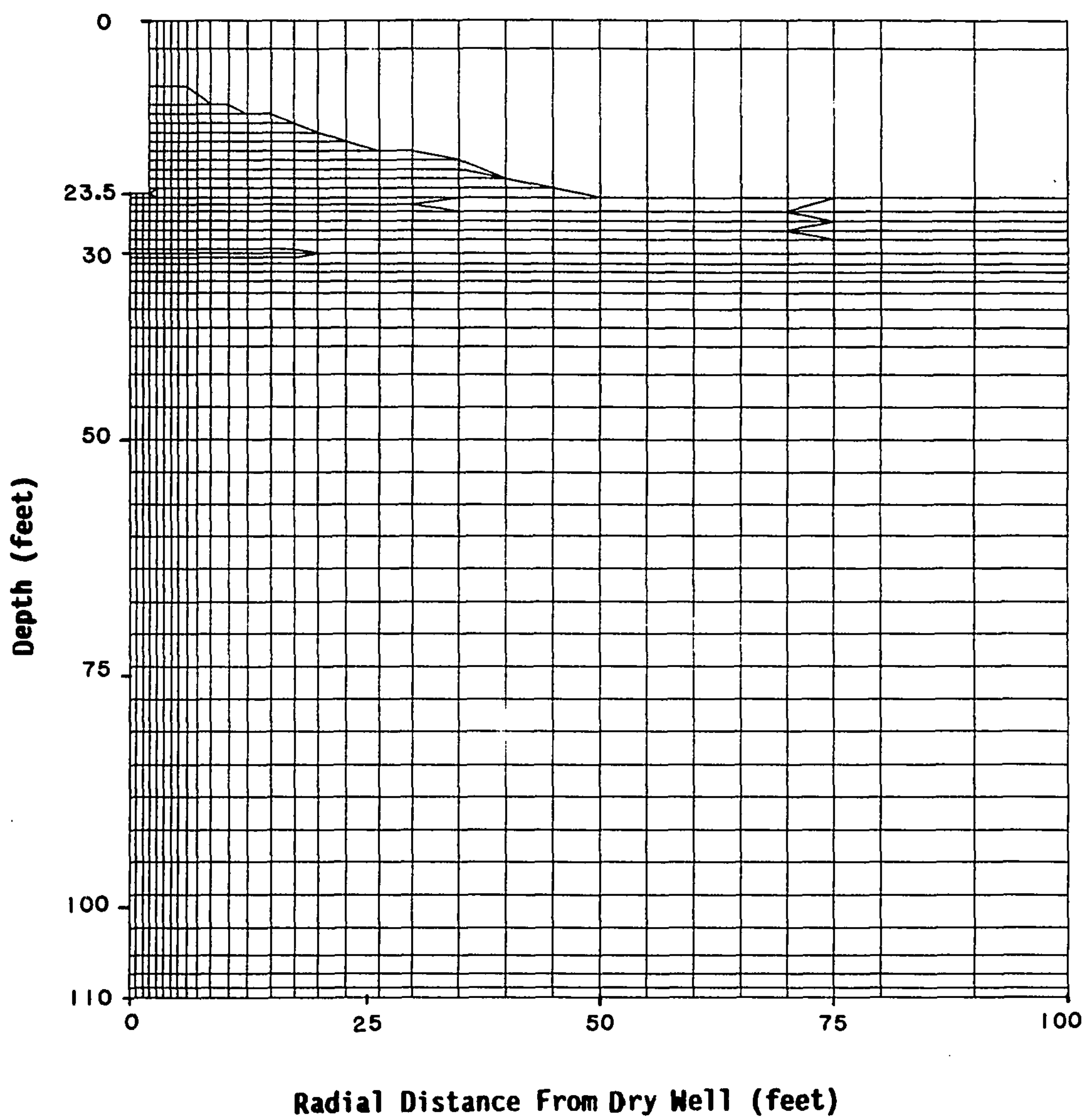


FIGURE 15. FINITE ELEMENT GRID

Boundary Conditions

Boundary conditions for the grid were made up entirely of no flow (prescribed flux of zero) and prescribed head boundaries. The left side of the grid, coinciding with the vertical axis of the dry well, was treated as a no-flow boundary, as it occupies a stream line. A prescribed head boundary condition was used along the borehole wall and base to control the rate of discharge from the borehole. Though discharge into the flow region may be most directly simulated by a prescribed flux boundary condition, the prescribed head boundary proved more conducive to a convergent solution. The basal and perimeter boundaries were specified as zero-flux boundaries, and the top (land surface) boundary was set to a prescribed head equal to the background pressure head, or initial condition, of that part of the vadose zone. The initial conditions for each soil material used in the three simulations are summarized in **Table 8.**

Time Step and Convergence Criteria

The error criterion for closure, defined as the maximum change in pressure head at any node in the grid between successive iterations, determines the accuracy of the finite element solution of the Richards equation, and the number of iterations required to reach a solution of this accuracy. Choice of the closure criterion involves a trade-off between computer time and cost, and accuracy of results. The error criterion used in these simulations was 0.1 feet.

TABLE 8

INITIAL MOISTURE CONDITIONS FOR CASE STUDY SOIL MATERIALS

SOIL MATERIAL	INITIAL MOISTURE CONTENT (vol/vol)	SUCTION HEAD (ft)
1	0.13	2.3
2	0.19	2.4
3	0.22	11.04

To achieve a solution under this error criterion, small time steps had to be used in the initial phase of a recharge event. The time step used near the beginning of a recharge event was about 10 seconds. Time steps increased in length throughout the course of a simulation through the user-specified multiplier. The length of each successive time step is determined as the product of the multiplier and the previous time step. If a given time step is too large, the program fails to achieve a solution within the accuracy of the convergence criterion, and the computed pressure heads are dumped to tape. The simulation may then be resumed with a smaller time step using a restart feature in the program. The time step multiplier used during a recharge event was between 1.07 and 1.1. Time steps near the end of a recharge event were usually between 5 and 10 minutes. During the drainage period following recharge events, the time step multiplier was increased to between 1.2 and 1.5. Time steps at the beginning of a drainage period were usually less than a minute, due to the rapid change of pressure head near the borehole, resulting from removal of the recharge source. Time steps near the end of a prolonged drainage period increased to several hours in length.

Simulation of Recharge to Dry Well

The scope of work under which this research was performed called for simulation of recharge of runoff to a dry well from two successive precipitation events representative of Tucson Basin conditions. Specifically, the recharge/runoff events were stipulated as 5-year, 1-hour storms separated by a 24-hour lag period, such as might occur during the summer monsoon season. Certain manufacturer's design criteria were considered in estimating the volume and rate of recharge associated with these precipitation events. According to a local manufacturer, a typical dry well installed in Arizona is designed to service one paved acre of urban watershed, and is expected to maintain an approximate long-term recharge rate of 0.5 cubic feet per second (cfs) over the effective life of the dry well (McGuckin Drilling, Inc., 1984). Sustainable recharge rate is usually greater for a new dry well, decreasing somewhat over time as runoff-borne suspended and dissolved constituents clog formation pore space near the borehole. The estimated sustainable recharge rate of 0.5 cfs was specified as a representative design value for urban dry wells in Arizona.

The volume of runoff expected from a design watershed of one-paved acre from a 5-year, 1-hour storm in Southwest Arizona was estimated using a runoff hydrograph method developed for use in Pima County Arizona (Zeller, 1979). Essential parameters for the hydrograph were: 1.0 acre watershed area, 5-year rainfall recurrence interval, 100 percent imperviousness, watercourse length of 210 feet, average slope of 0.01 and basin factor of 0.022. The total volume of runoff estimated by this method was approximately 5,000 cubic feet for each storm. A detailed description of the hydrograph method is presented in Zeller (1979).

Recharge of runoff to the dry well was simulated by prescribing a pressure head for nodes along the dry well's borehole wall that induced an intake rate consistent with the manufacturer's projected long-term design flow rate of 0.5 cfs. For the Case 1 simulation, a constant borehole head of about 7.0 feet produced a consistent intake rate of 0.5 cfs. For the Case 2 and 3 simulations, the borehole head was varied between 7.0 and 9.5 feet for a similar result. The borehole head in Cases 2 and 3 was somewhat higher due to the effects of back-pressure from mounding at the layer transition 30 feet below land surface.

RESULTS

Output data from Unsat 2, in the form of the nodal distribution of pressure head, was written to a separate plot file, specifying x and y coordinates and value of pressure head for each node at a specified point in time. A utility program was prepared to calculate the degree of saturation, defined as the ratio of observed water content to saturated water content at each node. This was done by relating the computed pressure head to moisture content, as specified by the soil moisture retention curve. A two-dimensional contouring program was used to plot contours of equal saturation over the flow region, depicted by a radial "slice" of infinitesimal width centered about the longitudinal axis of the dry well **(Figures 16 through 33)**. The highest value of saturation contoured was 95 percent. Much of the zone of the drainage plume upgradient of the wetting front was in a state of between 99 percent and 100 percent saturation, so that the zone of 100 percent saturation was small compared with the zone of 95 percent saturation. The 95 percent saturation level best illustrates the profile of the zone of the drainage plume close to saturation.

CASE 1 SIMULATION

Storm 1

Figures 16a through 17b illustrate the saturation profile of the flow region for the period of storm water recharge from the first

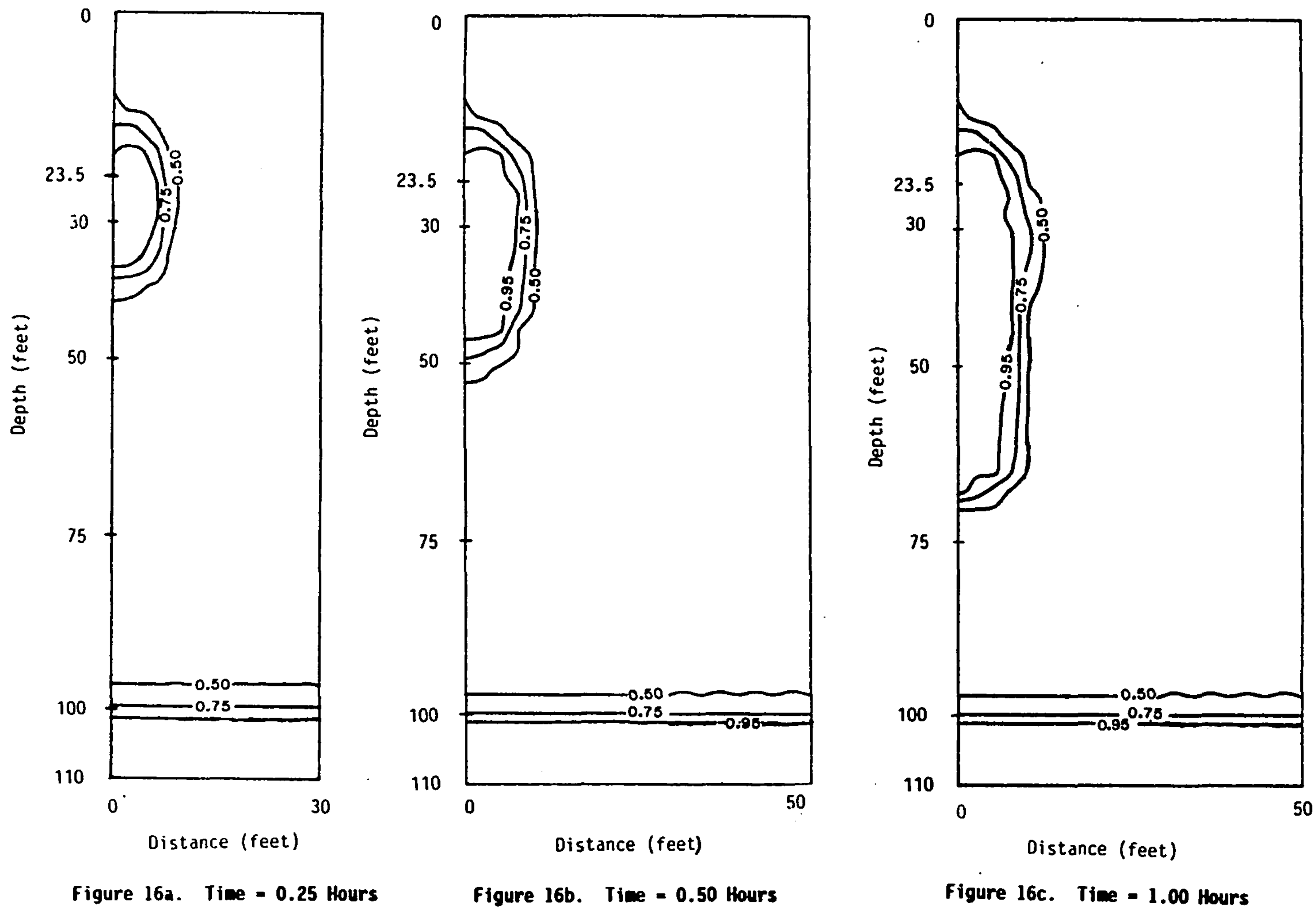


FIGURE 16. DEGREE OF SATURATION: CASE 1, STORM 1

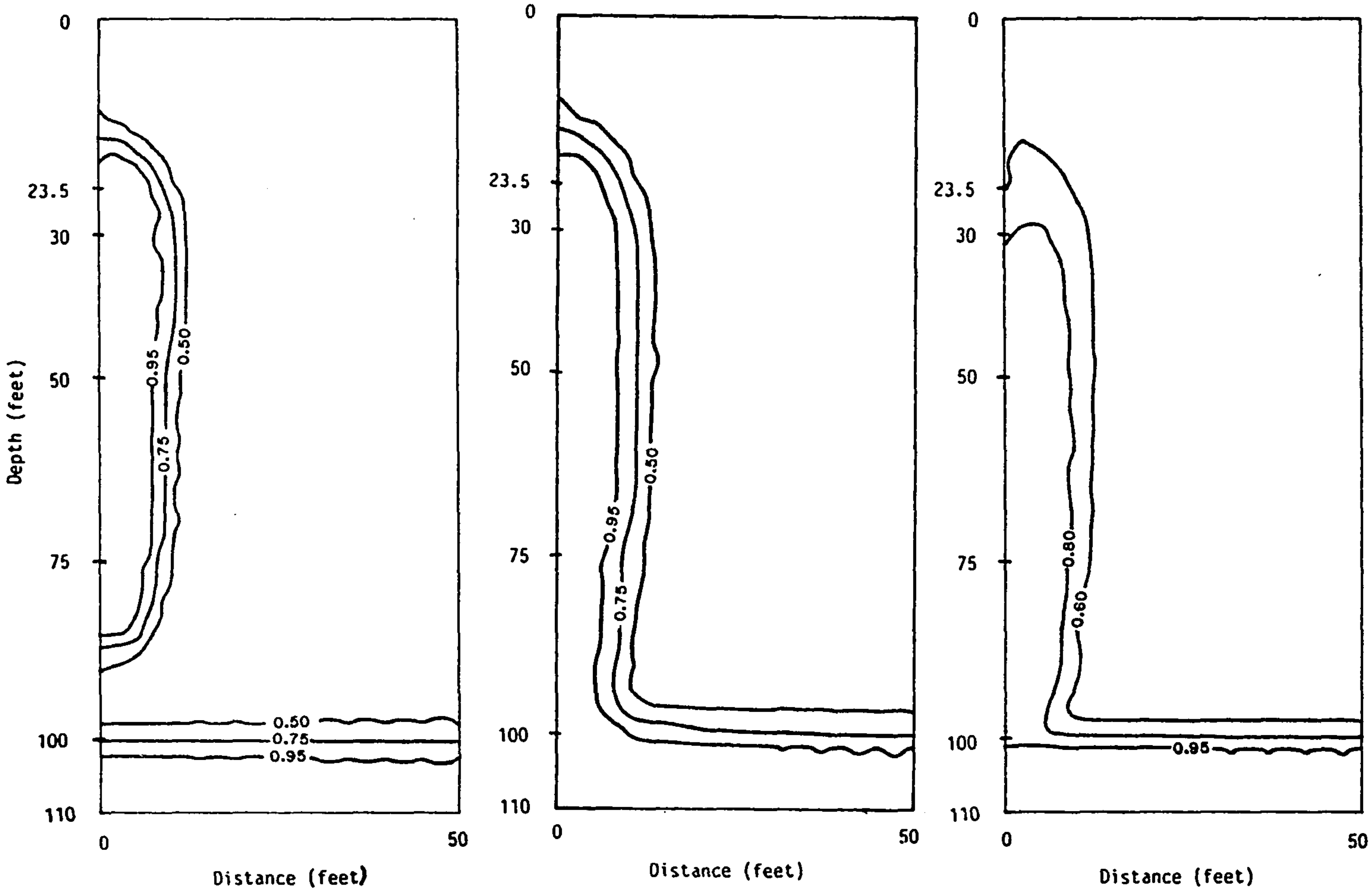


Figure 17a. Time = 1.50 Hours

Figure 17b. Time = 2.72 Hours

Figure 17c. Time = 4.72 Hours

FIGURE 17. DEGREE OF SATURATION: CASE 1, STORM 1 AND POST-STORM 1 DRAINAGE PERIOD

5-year, 1-hour storm. Recharge of the entire volume of runoff from the storm, 5,000 cubic feet, lasted about 2.72 hours, draining at the design rate of about 0.5 cfs. Over the first 0.25 hours of discharge, the 95 percent saturated portion of the drainage plume advanced 12.8 feet from the bottom of the dry well at 23.5 feet below land surface (bls), for an average rate of 51.2 feet per hour **(Table 9)**. Between 1.0 and 1.5 hours, this average rate decreased to about 34.4 feet per hour (ft/hr). For an approximately constant borehole head, average vertical velocity generally decreased with time. The perturbation in this pattern observed between the times of 0.5 and 1.0 hours, where average vertical velocity increased, is probably due to an increase in the specified borehole head during this period, which was required to maintain a constant influx rate of about 0.5 cfs. The 95 percent saturated portion of the drainage plume reached the water table at 100 feet bls between 1.5 and 2.5 hours.

The maximum horizontal extent of the 95 percent saturated portion of the drainage plume during the first storm was observed to be about 8.5 feet from the longitudinal axis of the dry well **(Figure 17b)**. The relatively high vertical velocity and low extent of radial movement of drainage water are a result of the relatively high saturated hydraulic conductivity of 6.7 ft/hr specified over the vadose zone.

TABLE 9

AVERAGE VELOCITY OF ZONE OF 95 PERCENT SATURATION: CASE 1, STORM 1

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	AVERAGE VERTICAL VELOCITY (ft/hr)
0	0.25	12.8	51.2
0.25	0.50	9.9	39.6
0.50	1.0	21.4	42.8
1.0	1.5	17.2	34.4

24-Hour Drainage Period

A 24-hour lag period between simulated storm run-off events allowed for redistribution of soil moisture following the first storm **(Figures 17c through 19a)**. The zone above 95 percent saturation was completely dissipated by a time of 2.0 hours following the completion of recharge from the first storm, or 4.72 hours total time. As such, the continued downward movement of vadose zone bound water was illustrated by the downward progression of the 80 percent saturation zone. Between the times of 2.0 and 15.0 hours of drainage, (corresponding to 4.72 and 17.72 hours of total simulation time, respectively) this rate remained relatively constant, varying between 4.4 ft/hr and 4.8 ft/hr **(Table 10)**. During the final 9.0 hours of drainage, this average rate decreased to about 0.8 ft/hr.

Storm 2

Drainage water introduced into the dry well from a second 5-year, 1-hour storm propagated through the vadose zone more rapidly than during the first, due to higher levels of background moisture content in the vadose zone sediments beneath the dry well **(Figures 19b through 20a)**. Moisture retained in the soil from the first storm resulted in higher values of relative hydraulic conductivity. The rate of vertical movement for the zone of 95 percent saturation over the first 0.25 hours of recharge from the second storm was about 106 ft/hr **(Figure 19b)**. This zone of saturation reached the water table by a time of 0.75 hours of

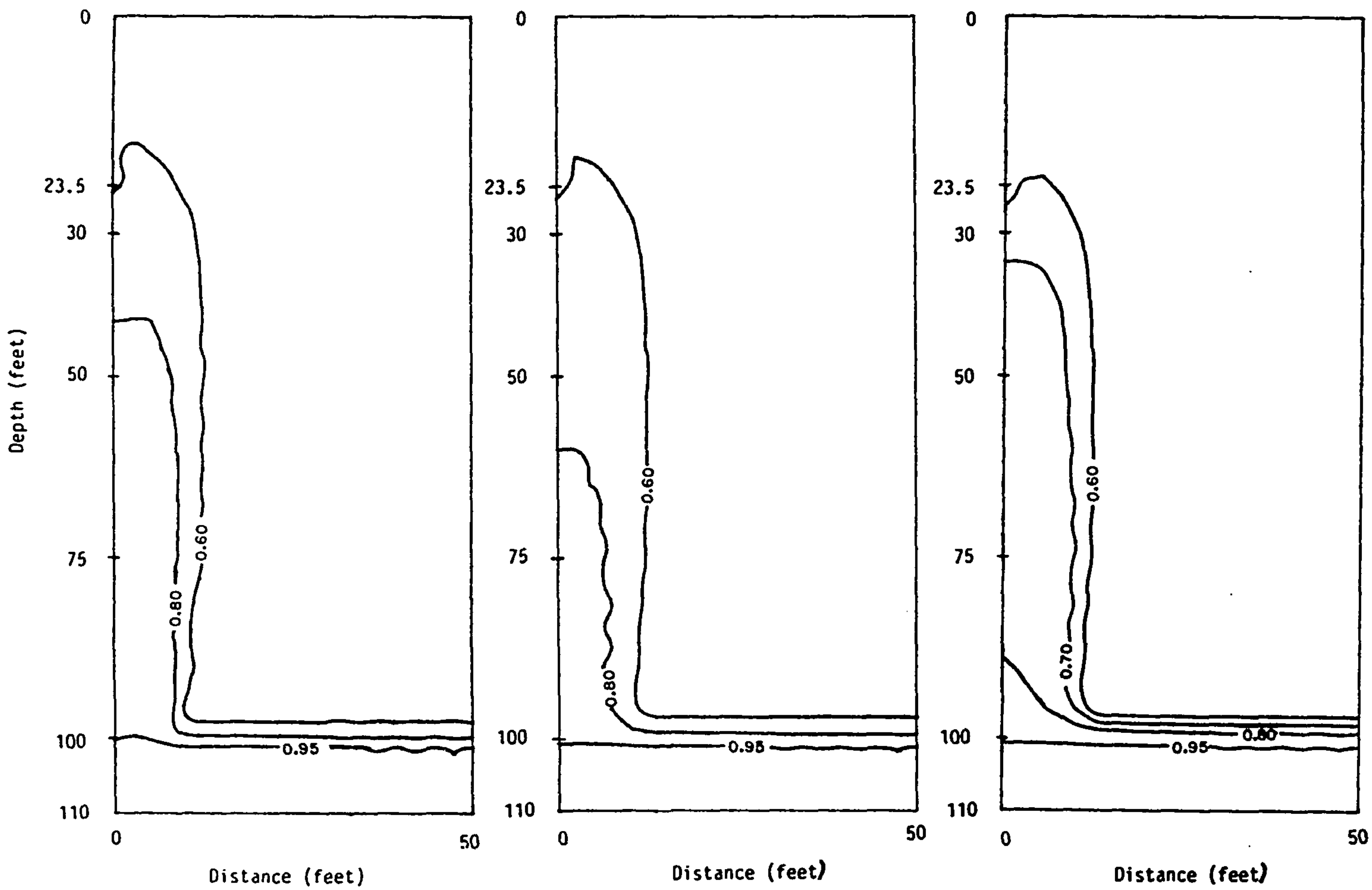


Figure 18a. Time = 7.72 Hours

Figure 18b. Time = 11.72 Hours

Figure 18c. Time = 17.72 Hours

FIGURE 18. DEGREE OF SATURATION: CASE 1, POST-STORM 1 DRAINAGE PERIOD

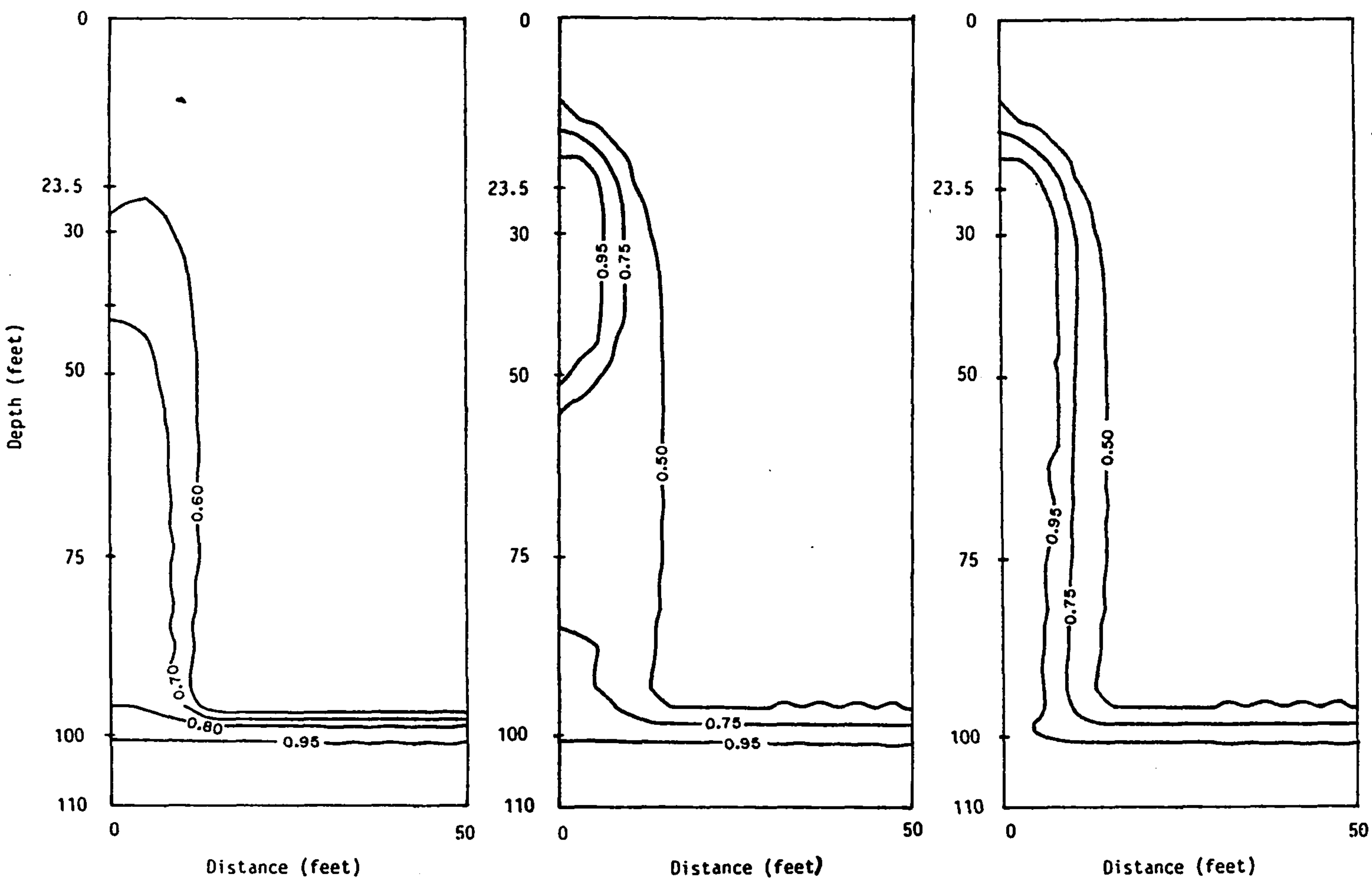


Figure 19a. Time = 26.72 Hours

Figure 19b. Time = 26.97 Hours

Figure 19c. Time = 27.47 Hours

FIGURE 19. DEGREE OF SATURATION: CASE 1, POST-STORM 1 DRAINAGE PERIOD AND STORM 2

TABLE 10

AVERAGE VELOCITY OF ZONE OF 80 PERCENT SATURATION: CASE 1 SIMULATION, 24-HOUR DRAINAGE PERIOD

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	AVERAGE VERTICAL VELOCITY (ft/hr)
4.72	7.72	13.8	4.6
7.72	11.72	17.6	4.4
11.72	17.72	28.9	4.8
17.72	26.72	7.3	0.8

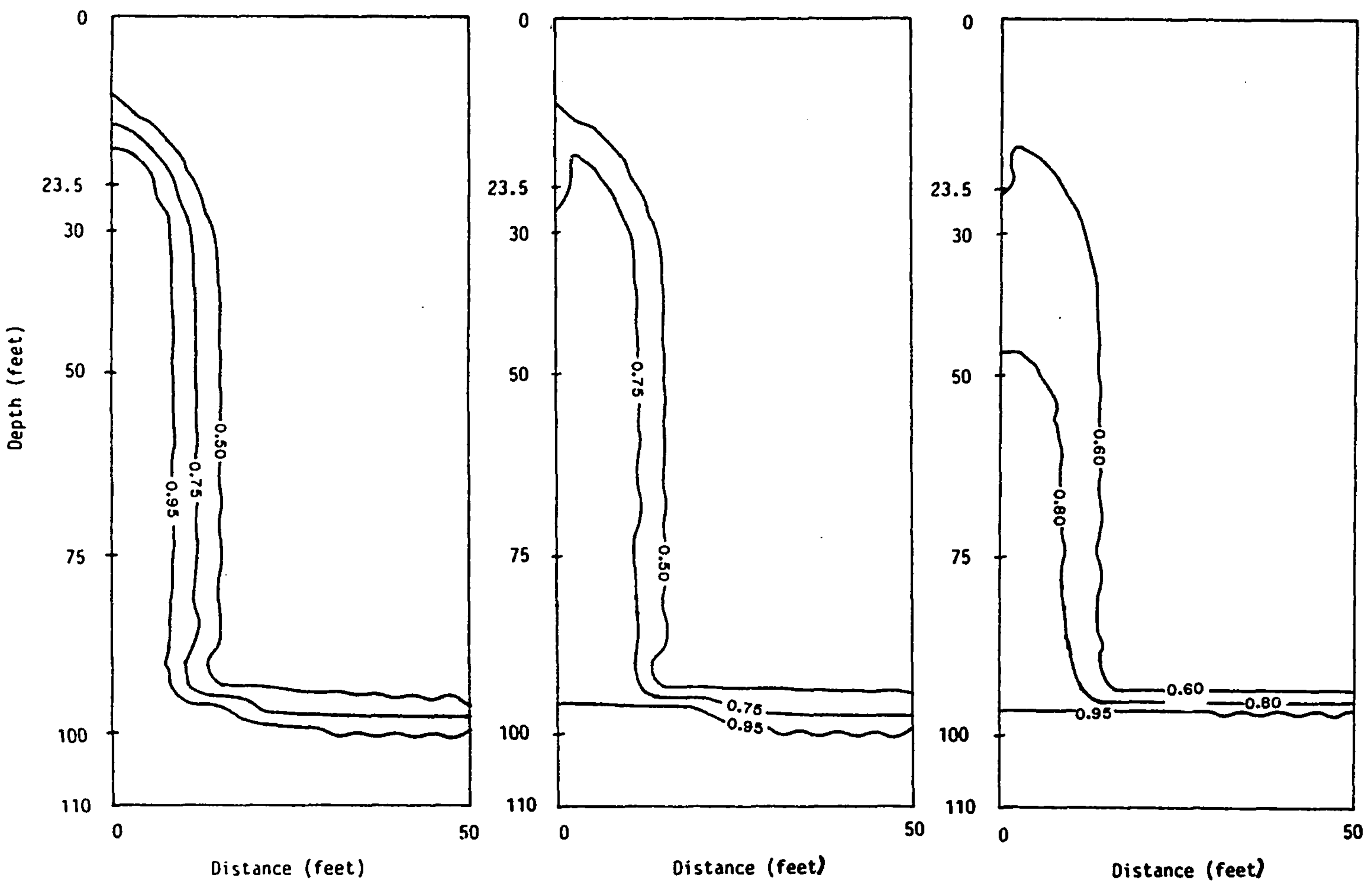


Figure 20a. Time = 29.38 Hours

Figure 20b. Time = 29.88 Hours

Figure 20c. Time = 35.38 Hours

FIGURE 20. DEGREE OF SATURATION: CASE 1, STORM 2 AND POST-STORM 2 DRAINAGE PERIOD

drainage, or by 27.47 hours total time **(Figure 19c)**. Soil moisture redistribution following the second storm proceeded at a relatively stable rate, as in the period immediately following the first storm. The 80 percent saturated portion of the drainage plume proceeded at a rate of about 4.2 ft/hr over the period of 0.5 to 6.0 hours of drainage, or 29.88 to 35.38 hours total time **(Figure 20c)**. This average rate increased to about 5.0 ft/hr over the following 6-hour period **(Figure 21a)**. By a time of 24 hours following the second storm drainage (corresponding to 53.38 hours total time), the zone of 80 percent saturation was completely assimilated into the regional groundwater system. The maximum radial extent of drainage water during the second storm was again about 8.5 feet from the center of the dry well.

CASE 2 SIMULATION

The second case study introduced a significant radial, or horizontal component of flow into the vadose zone beneath the dry well, as a result of the layered subsurface condition. The transition at a depth of 30 feet from the overlying material with a K_S of 6.7 ft/hr. to the underlying material with a K_S of about 0.29 ft/hr induced a temporary perching of drainage water at the layer transition **(Figure 22a through 22c)**. The results of the second case study are distinguished by perching and lateral spreading of drainage water at the layer transition, and a much slower infiltration velocity through the Material 2 soil.

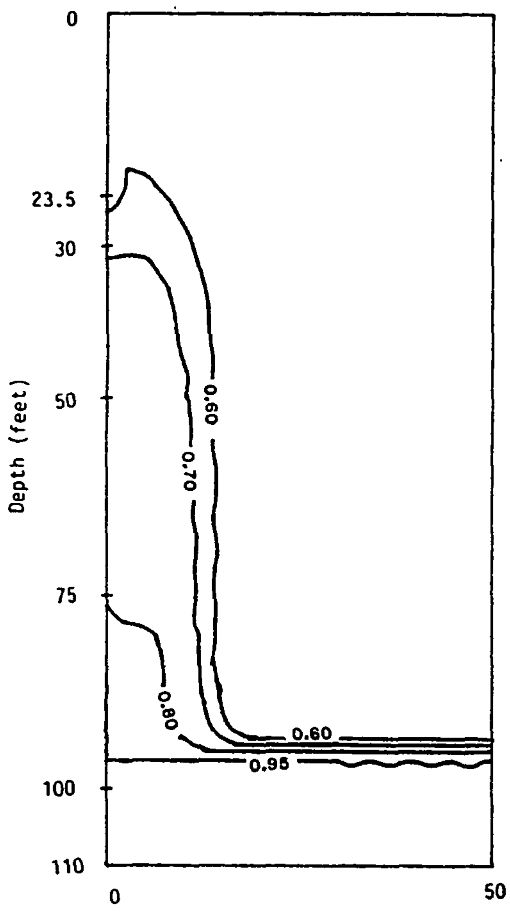


Figure 21a. Time = 41.38 Hours

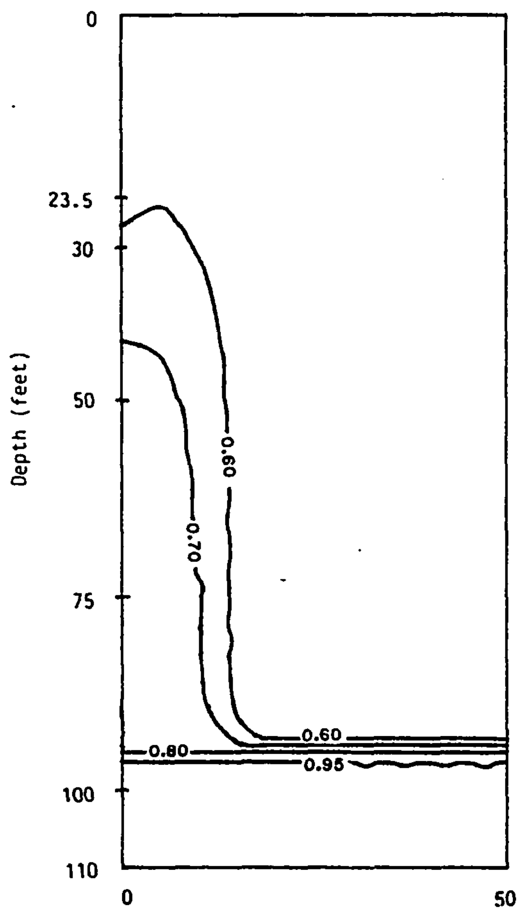


Figure 21b. Time = 53.38 Hours

FIGURE 21. DEGREE OF SATURATION: CASE 1, POST-STORM 2 DRAINAGE PERIOD

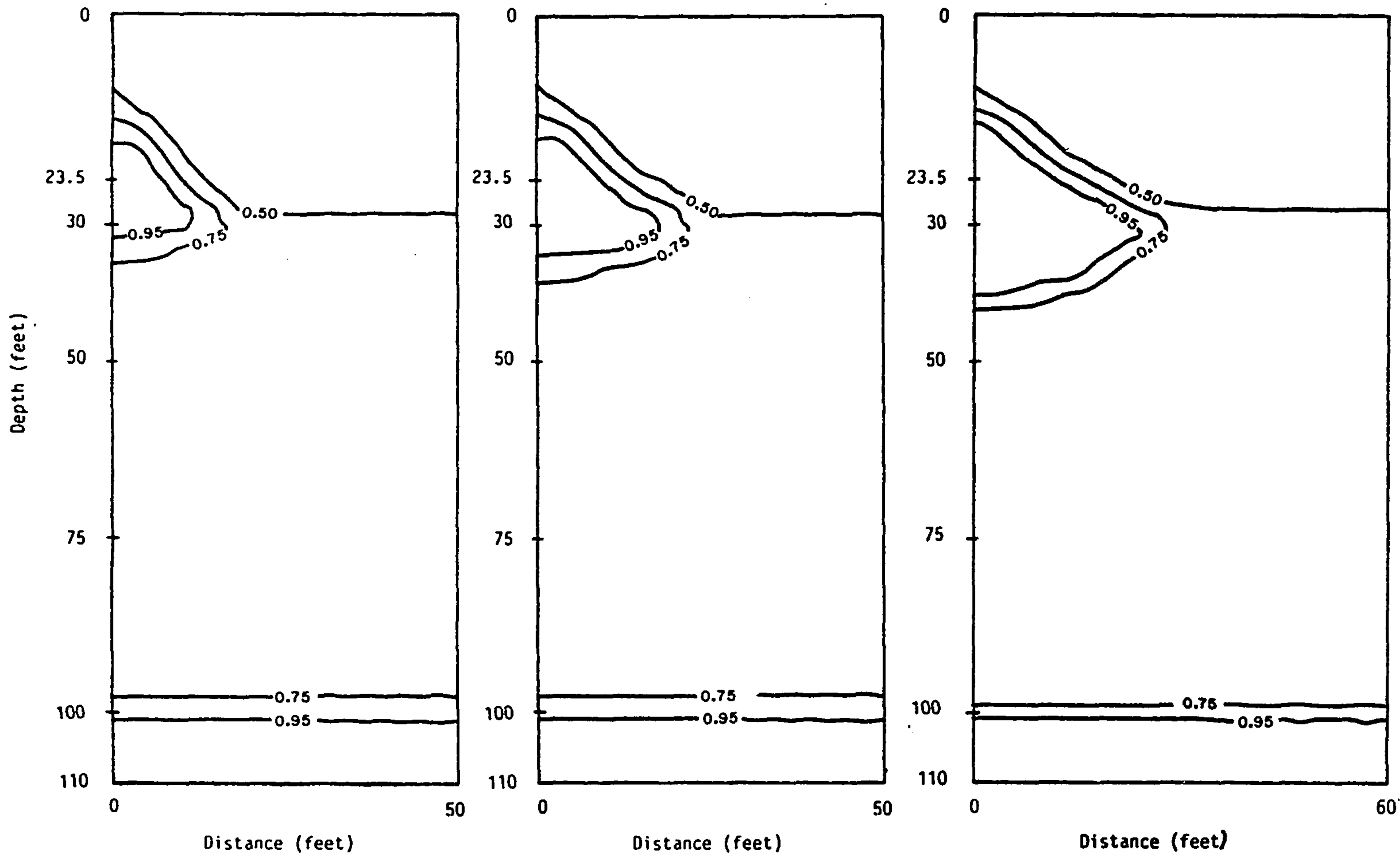


Figure 22a. Time = 0.50 Hours

Figure 22b. Time = 1.00 Hours

Figure 22c. Time = 2.00 Hours

FIGURE 22. DEGREE OF SATURATION: CASE 2, STORM 1

Storm 1

The average vertical velocity of the 95 percent saturated part of the drainage plume was about 16.4 ft/hr over the first 0.5 hours of the first recharge event **(Table 11)**. During this time water moved primarily through soils above 30 feet bls, composed of Material 1 soil type. From this time to 2.83 hours of recharge, the total time required to recharge 5,000 cubic feet of runoff, average vertical flow velocities ranged between 3.3 and 5.7 ft/hr **(Figure 22a through 23a; Table 11)**. Vertical flow velocity increased slightly over the period 1.0 to 2.0 hours of recharge, as borehole head was increased to maintain a constant inflow rate at about 0.5 cfs. Horizontal flow velocities were at a maximum along the layer interface at a depth of 30 feet. The average horizontal flow velocities predicted by the model decreased steadily over the course of the first recharge event. Horizontal velocities averaged about 19.2 ft/hr over the first 0.5 hours of recharge and decreased to an average of 3.6 ft/hr over the final 0.83 hours **(Table 11)**. Wilson (1983) estimated horizontal flow velocities along a layer interface during a recharge test on the WRRC test dry well. This was accomplished by monitoring the arrival of the recharge plume at various 2-inch steel-cased access holes using a neutron moisture probe. Estimates of horizontal flow velocity made at five access holes, located from 51.5 feet to 128.3 feet away from the dry well, ranged from 10.8 ft/hr to 31.4 ft/hr, and averaged 19.2 ft/hr. Overall, these horizontal flow velocities are higher than velocities derived from the model results. These higher velocities are probably mostly due to the higher borehole

TABLE 11

AVERAGE VELOCITY OF ZONE OF 95 PERCENT SATURATION: CASE 2, STORM 1

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	HORIZONTAL DISPLACEMENT (ft)	AVERAGE VERTICAL VELOCITY (ft/hr)	AVERAGE HORIZONTAL VELOCITY (ft/hr)
0	0.5	8.2	9.6	16.4	19.2
0.5	1.0	2.6	5.8	5.2	11.6
1.0	2.0	5.7	6.5	5.7	6.5
2.0	2.83	2.7	3.0	3.3	3.6

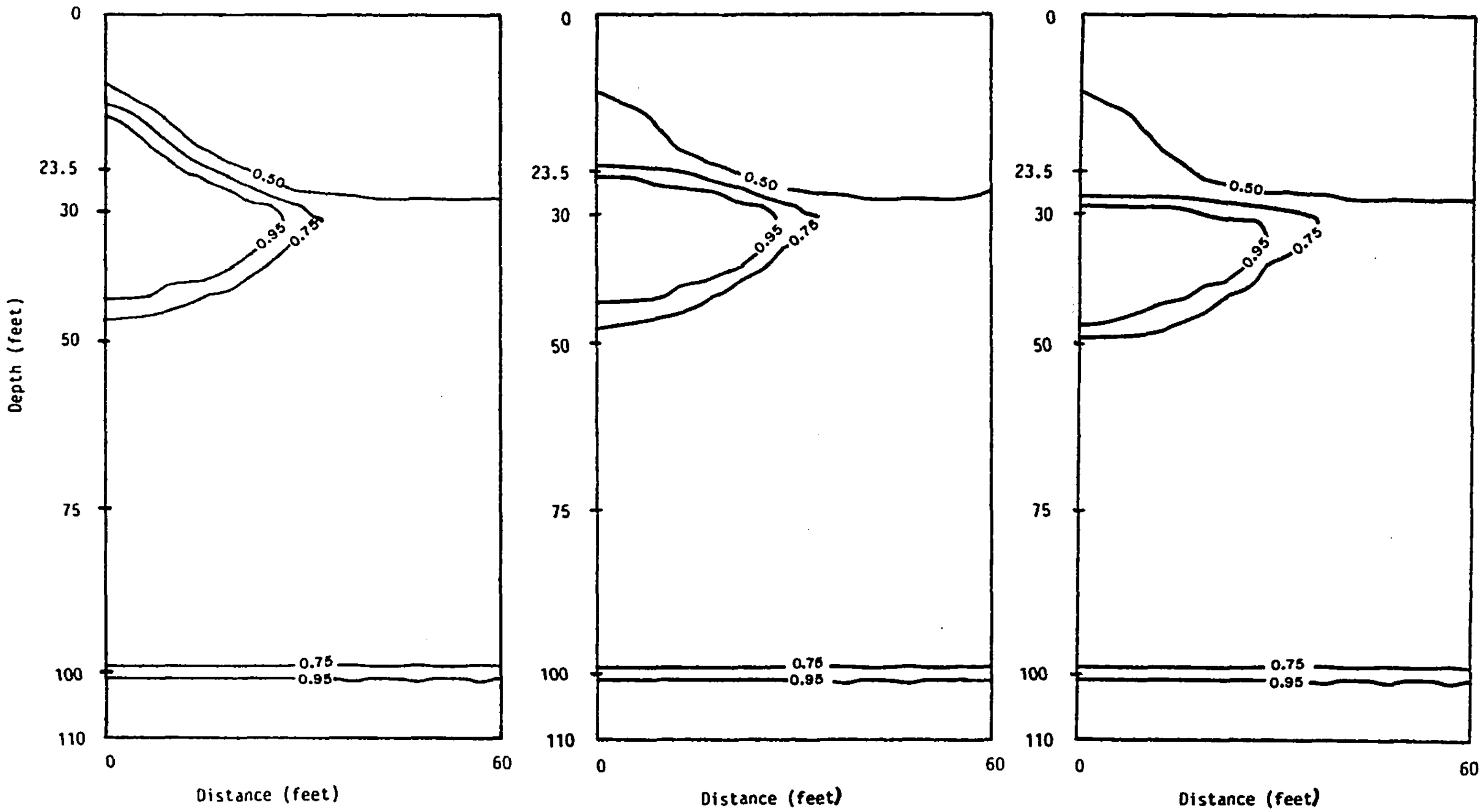


Figure 23a. Time = 2.83 Hours

Figure 23b. Time = 3.08 Hours

Figure 23c. Time = 3.83 Hours

FIGURE 23. DEGREE OF SATURATION: CASE 2, STORM 1 AND POST-STORM 1 DRAINAGE PERIOD

head sustained during the field test. Borehole head averaged about 13.2 feet over the 8.3 hour duration of the test. The borehole head was relatively constant over the length of the test, with a standard deviation of 0.48 feet about the mean. Borehole head used in the computer simulation for Case 2 varied between 7.0 feet and 9.0 feet during the first recharge event. Borehole head was varied between different time intervals in the simulated recharge event as required to maintain a constant inflow rate of 0.5 cfs. The maximum lateral extent of the 95 percent saturated part of the drainage plume during recharge from the first storm was about 26.8 feet **(Figure 23a)**.

24-Hour Drainage Period

The downward percolation of the plume of drainage water following the first storm is best illustrated by the progress of the 90 percent saturation zone. The zone of 95 percent saturation dissipated by a time of 4.0 hours of drainage from the first storm **(Figure 24a)**. Average vertical velocity of the 90 percent saturated portion of the plume remained constant at about 0.2 ft/hr over the 24-hour lag period, from 2.83 hours to 26.83 hours total time **(Figures 24b through 25a; Table 12)**.

Storm 2

As in Case 1, average vertical velocities of the 95 percent saturation level were higher during recharge of water from the second storm

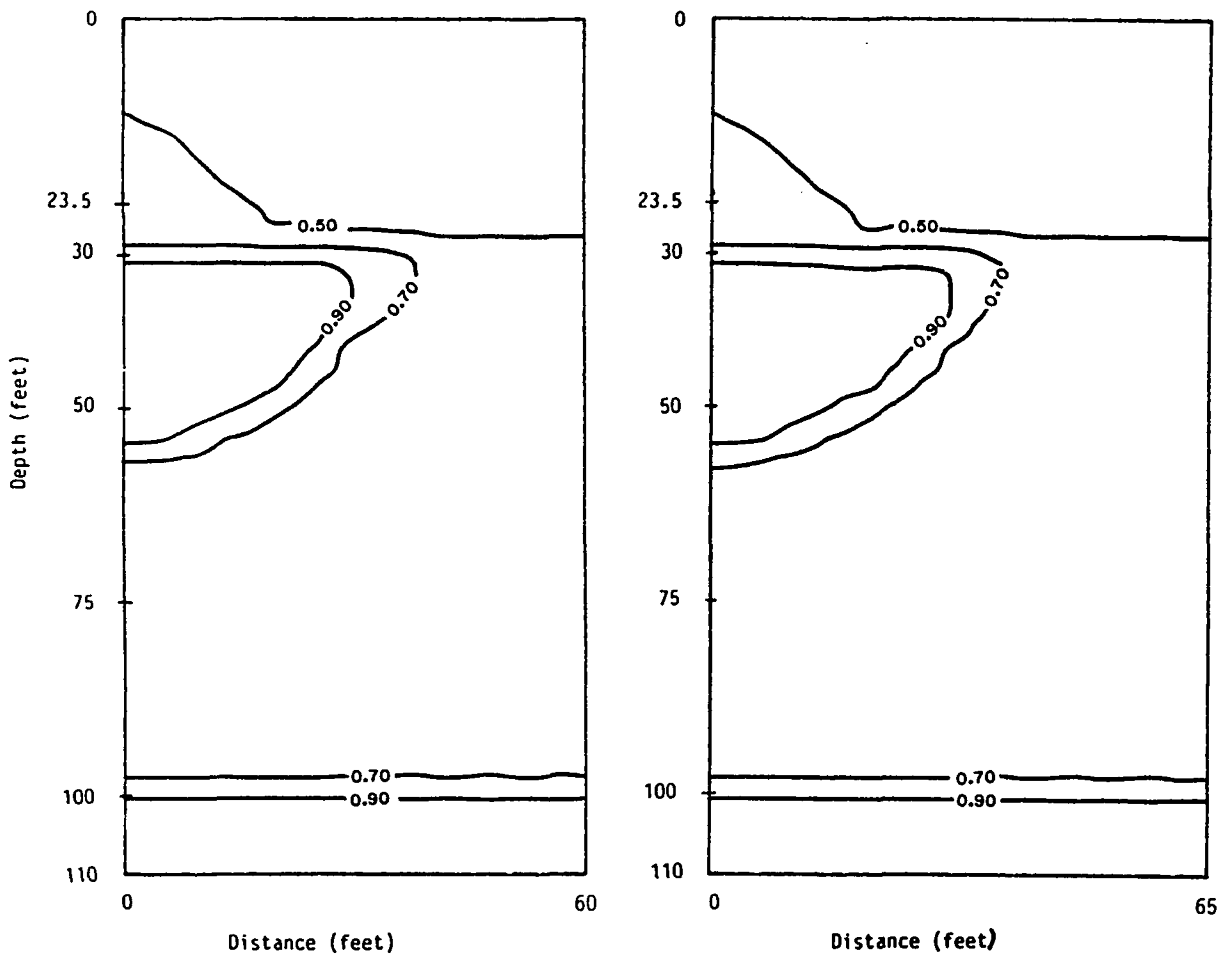


Figure 24a. Time = 6.83 Hours

Figure 24b. Time = 8.33 Hours

FIGURE 24. DEGREE OF SATURATION: CASE 2, POST-STORM 1 DRAINAGE PERIOD

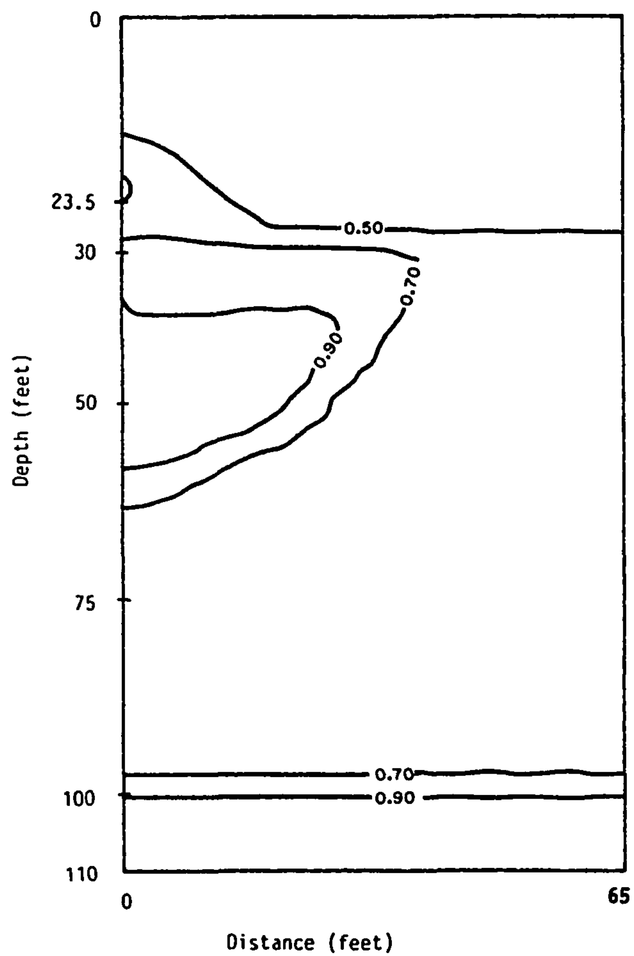


Figure 25a. Time = 26.83 Hours

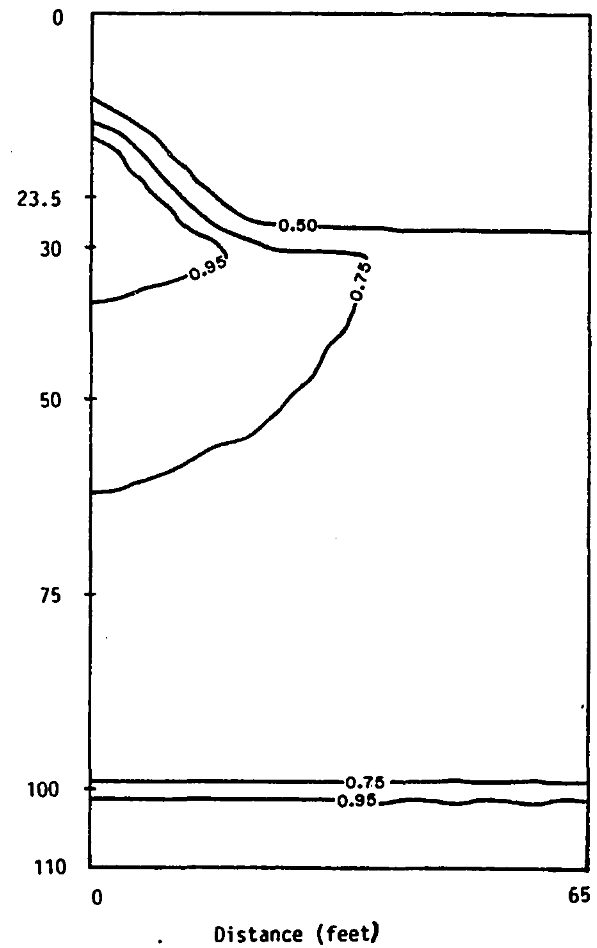


Figure 25b. Time = 27.33 Hours

FIGURE 25. DEGREE OF SATURATION: CASE 2, POST-STORM 1 DRAINAGE PERIOD AND STORM 2

TABLE 12

AVERAGE VELOCITY OF ZONE OF 90 PERCENT SATURATION: CASE 2, 24-HOUR DRAINAGE PERIOD

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	AVERAGE VERTICAL VELOCITY (ft/hr)
6.83	8.33	0.5	0.2
8.33	26.83	3.7	0.2

than from the first, due to higher levels of initial moisture content in the soil. Vertical velocity was highest over the first 0.5 hours of recharge, where it averaged about 27.4 ft/hr **(Table 13)**. Average vertical velocity decreased to about 10.9 ft/hr between 0.5 and 1.25 hours of recharge, and varied between 8.7 and 8.8 ft/hr for the duration of the recharge event. Horizontal flow velocity of the zone of 95 percent saturation decreased from 31.4 ft/hr over the first 0.5 hours of recharge to about 12.5 ft/hr over the period 0.5 to 1.25 hours, corresponding to 27.33 hours to 28.08 hours total time **(Figures 25b and 26a)**. Horizontal flow velocity varied between about 4.2 ft/hr and 4.3 ft/hr for the duration of the recharge event **(Table 13)**. The maximum lateral extent of the 95 percent saturated part of the drainage plume during recharge from the second storm was about 32.5 feet **(Figure 27a)**.

Post-Storm 2 Drainage

The zone of 95 percent saturation continued to decrease in average vertical flow velocity during the first 0.5 hours of drainage following recharge of water from the second storm (corresponding to 29.46 to 29.96 hours total time), to about 5.5 ft/hr during this period **(Figure 27b)**. Continued advance of the drainage plume is shown by movement of the zone of 90 percent saturation. Percolation progressed slowly between 9 and 130 hours of drainage (38.46 and 159.46 hours total time), varying between 0.1 and 0.3 ft/hr **(Table 14; Figures 28a through 29a)**. The 90 percent saturated portion of the plume dissipated by a time of 262 hours of drainage, or 291.46 hours total time. Based on an

TABLE 13

AVERAGE VELOCITY OF ZONE OF 95 PERCENT SATURATION:
CASE 2, STORM 2

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	HORIZONTAL DISPLACEMENT (ft/hr)	AVERAGE VERTICAL VELOCITY (ft/hr)	AVERAGE HORIZONTAL VELOCITY (ft/hr)
0	0.5	13.7	15.7	27.4	31.4
0.5	1.25	8.2	9.4	10.9	12.5
1.25	1.75	4.4	2.1	8.8	4.2
1.75	2.63	7.7	3.8	8.7	4.3

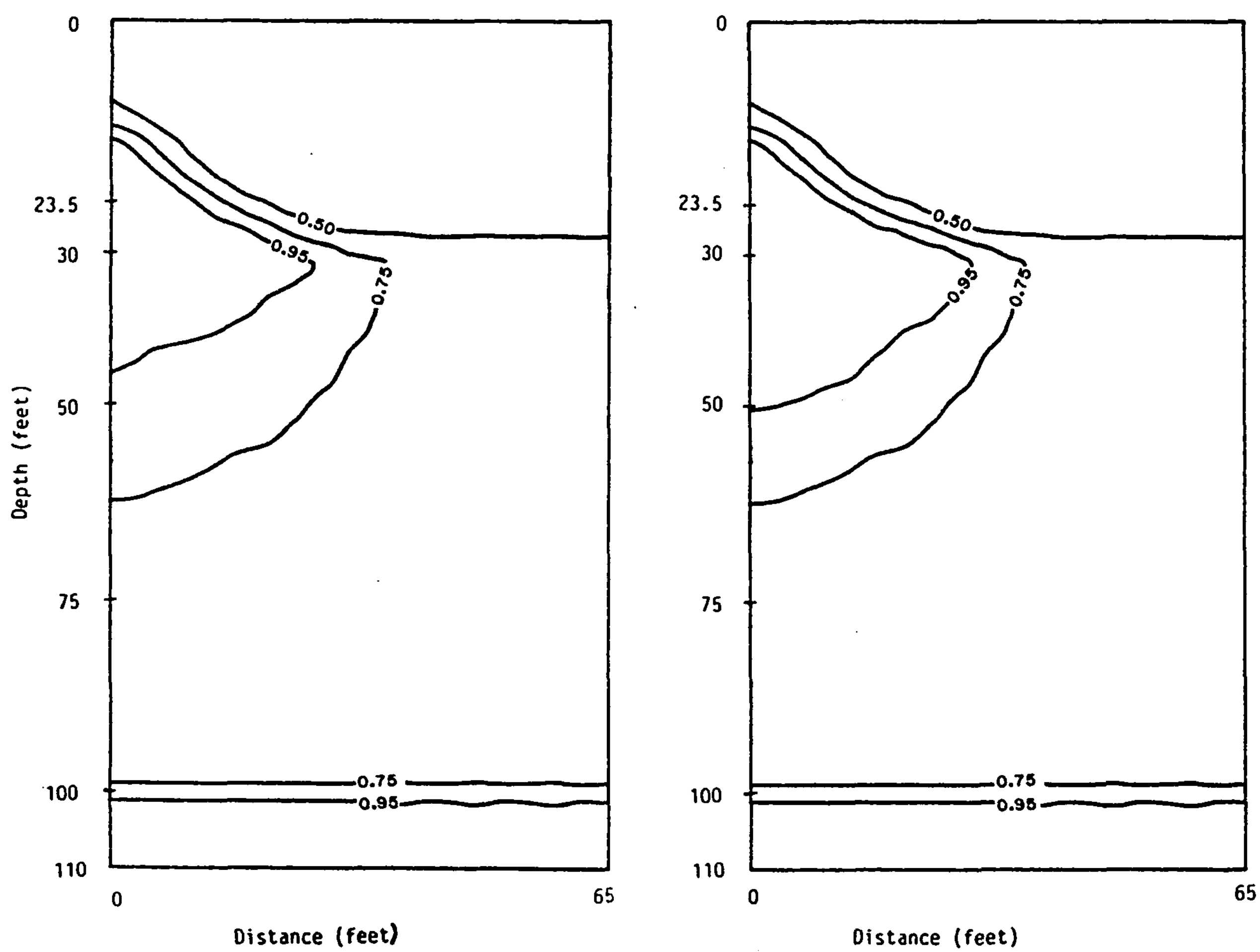


Figure 26a. Time = 28.08 Hours

Figure 26b. Time = 28.58 Hours

FIGURE 26. DEGREE OF SATURATION: CASE 2, STORM 2

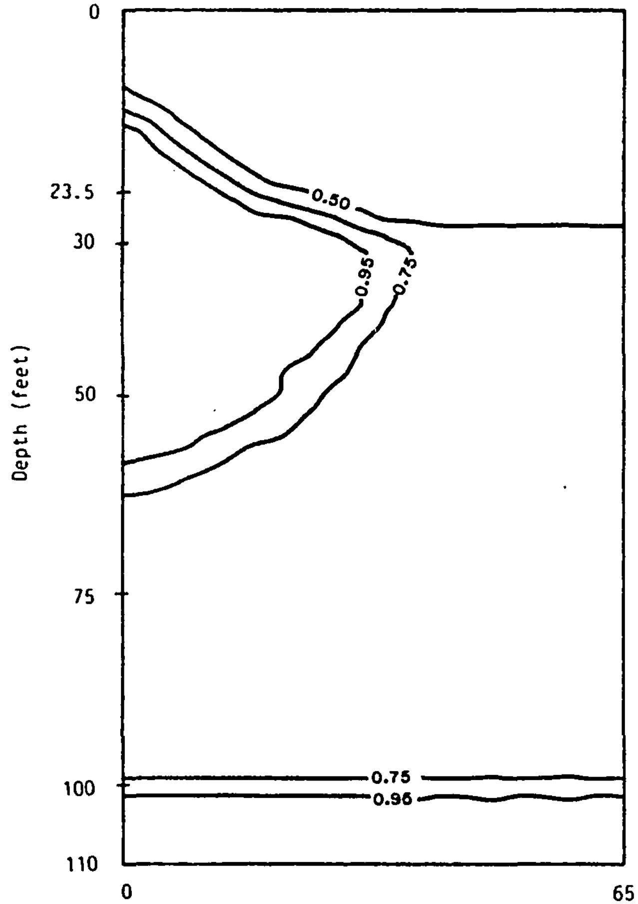


Figure 27a. Time = 29.46 Hours

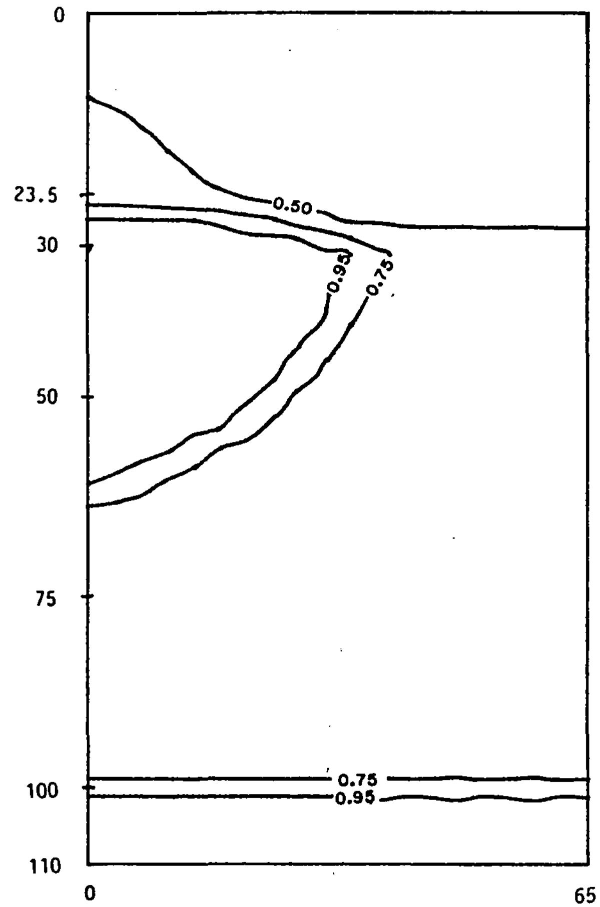


Figure 27b. Time = 29.96 Hours

FIGURE 27. DEGREE OF SATURATION: CASE 2, STORM 2 AND POST-STORM 2 DRAINAGE PERIOD

TABLE 14

AVERAGE VELOCITY OF ZONE OF 90 PERCENT SATURATION: CASE 2, POST STORM 2 DRAINAGE

t_1 (hrs)	t_2 (hrs)	VERTICAL DISPLACEMENT (ft)	AVERAGE VERTICAL VELOCITY (ft/hr)
9.0	20.0	3.5	0.3
20.0	130.0	12.1	0.1

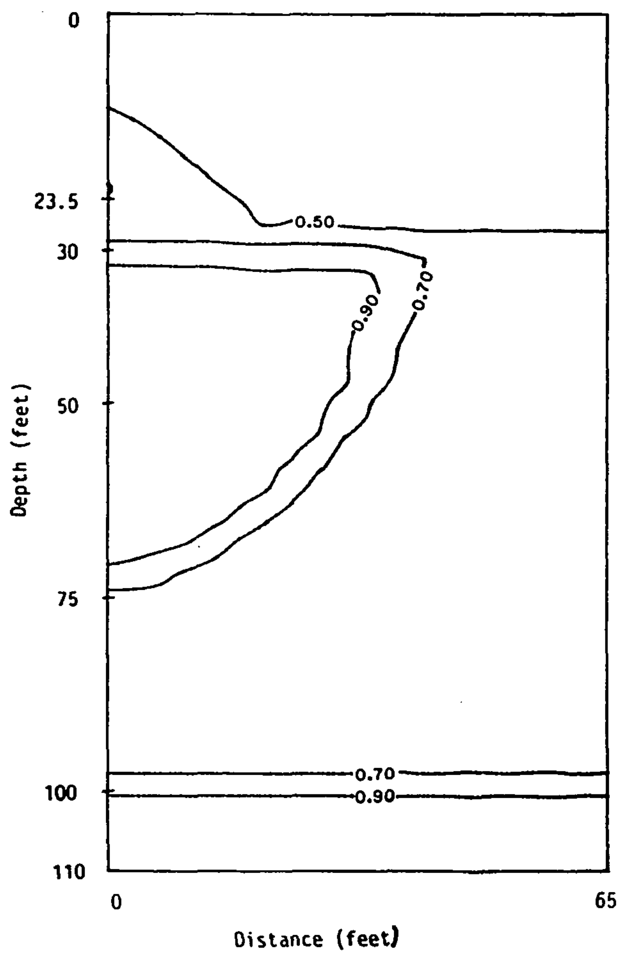


Figure 28a. Time = 38.46 Hours

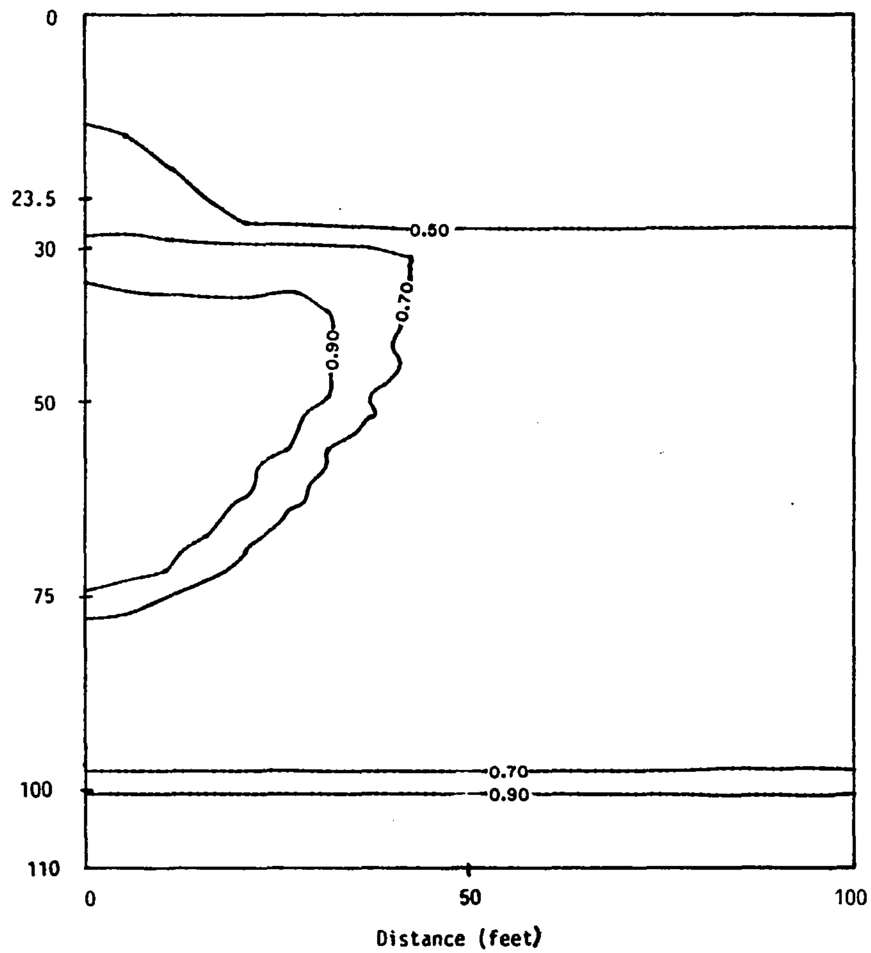


Figure 28b. Time = 49.46 Hours

FIGURE 28. DEGREE OF SATURATION: CASE 2, POST-STORM 2 DRAINAGE PERIOD

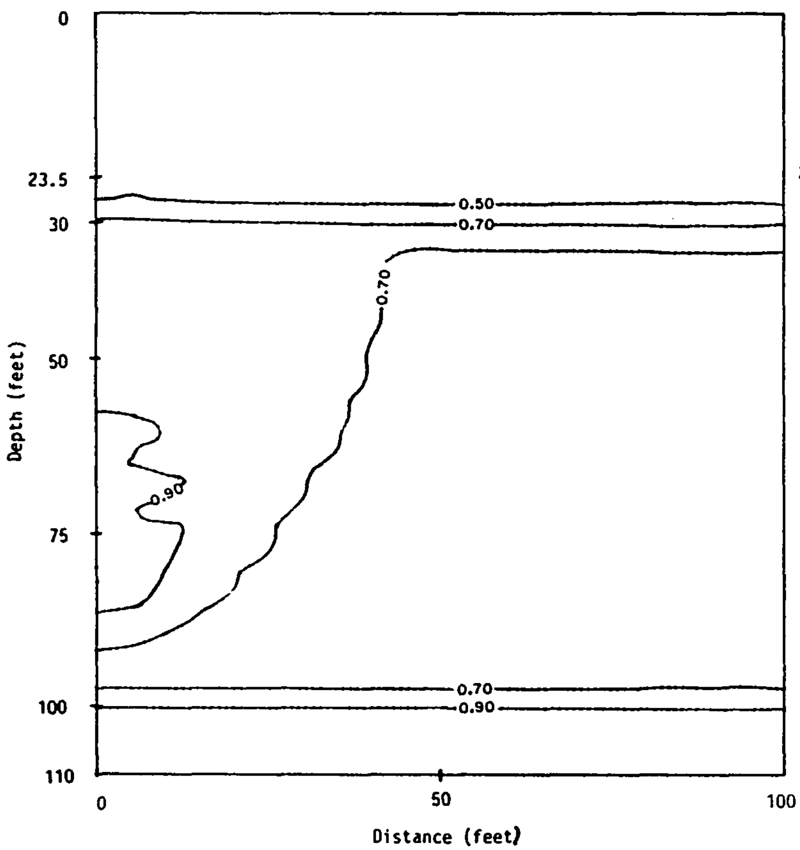


Figure 29a. Time = 159.46 Hours

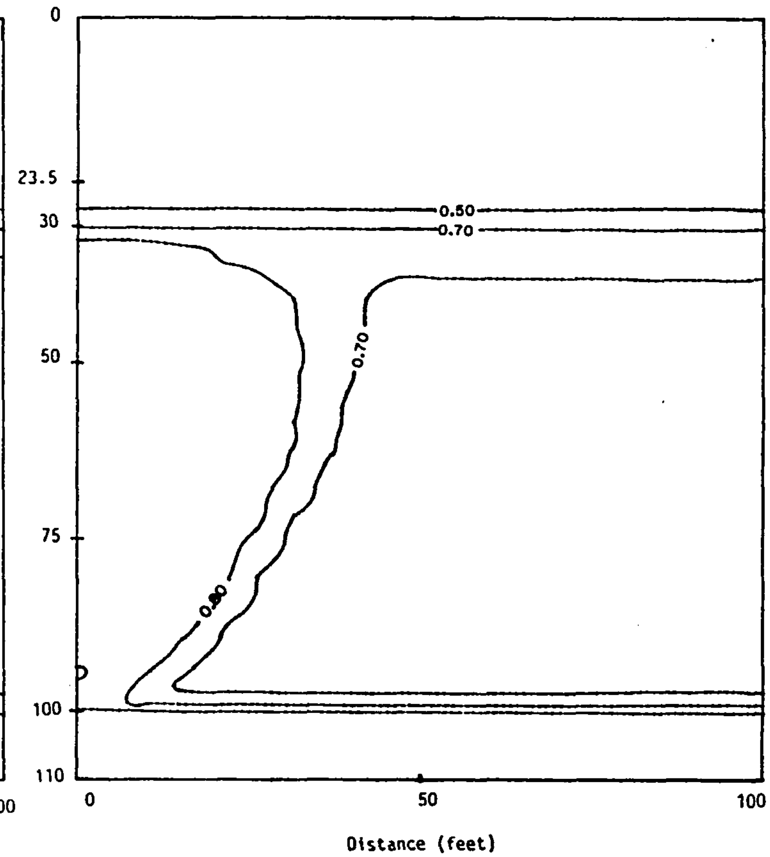


Figure 29b. Time = 291.46 Hours

FIGURE 29. DEGREE OF SATURATION: CASE 2, POST-STORM 2 DRAINAGE PERIOD

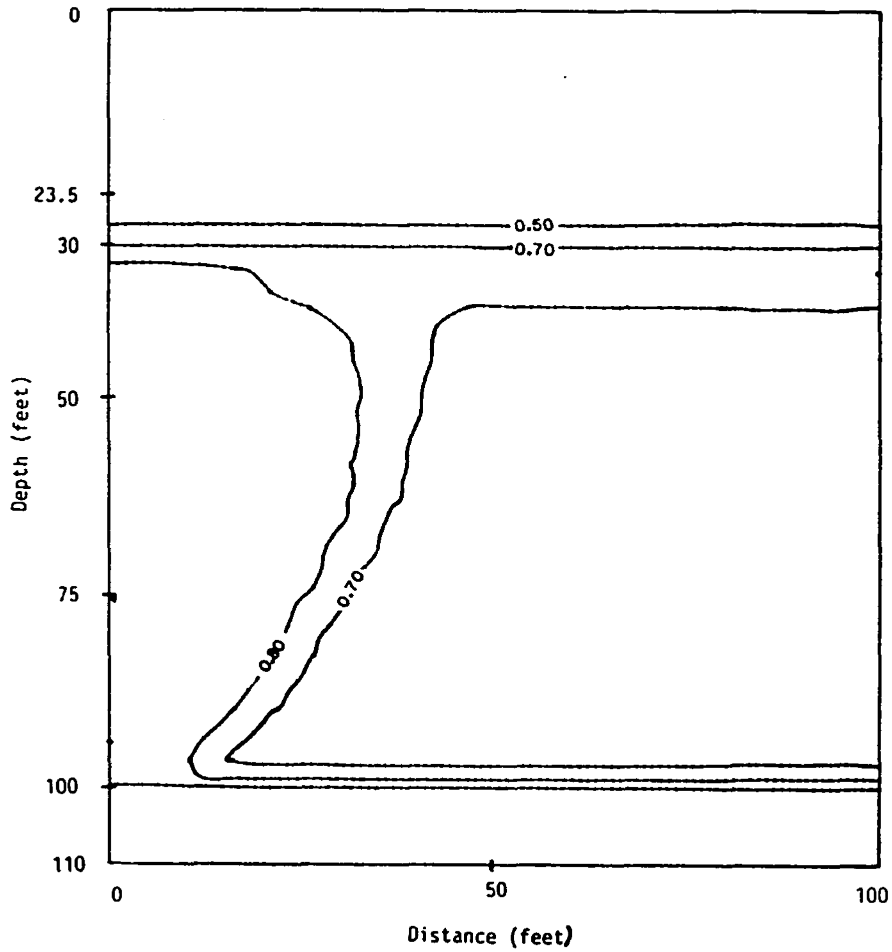
average vertical groundwater velocity of 0.2 ft/hr for the drainage plume, the zone of 80 percent saturation is estimated to contact the water table at about 187 hours of drainage following the second storm. The configuration of the 80 percent saturated part of the drainage plume changed very little over the 24-hour period between 262 and 286 hours of drainage (corresponding to 291.46 and 315.46 hours total time), an indication that moisture redistribution continued very slowly beyond this point in time **(Figures 29b and 30)**. No measurable additional horizontal movement took place during the period of drainage following the second storm.

CASE 3 SIMULATION

The third case study provided an intermediate case in contrast to Cases 1 and 2. The saturated hydraulic conductivity of the Material 2 soil zone was less than that of Case 1 and greater than in Case 2, at a value of 1.0 ft/hr. Accordingly, the degree of perching at the layer transition was lesser than in Case 2.

Storm 1

The 95 percent saturated part of the drainage plume moved downward at an average rate of about 29.4 ft/hr over the first 0.5 hours of drainage **(Table 15; Figure 31a)**. The rate of average vertical velocity decreased over the period 0.5 to 2.75 hours, varying between about 9.2 ft/hr and 11.8 ft/hr. Horizontal flow velocity of the zone of



Time = 315.46 Hours

FIGURE 30. DEGREE OF SATURATION: CASE 2, POST-STORM 2 DRAINAGE PERIOD

TABLE 15

AVERAGE VELOCITY OF ZONE OF 95 PERCENT SATURATION: CASE 3, STORM 1

t (hrs)	t (hrs)	VERTICAL DISPLACEMENT (ft)	HORIZONTAL DISPLACEMENT (ft/hr)	AVERAGE VERTICAL VELOCITY (ft/hr)	AVERAGE HORIZONTAL VELOCITY (ft/hr)
0	0.5	14.7	10.1	29.4	20.2
0.5	1.167	7.9	7.6	11.8	11.4
1.167	2.0	7.7	2.3	9.2	2.8
2.0	2.75	7.3	1.4	9.7	1.9

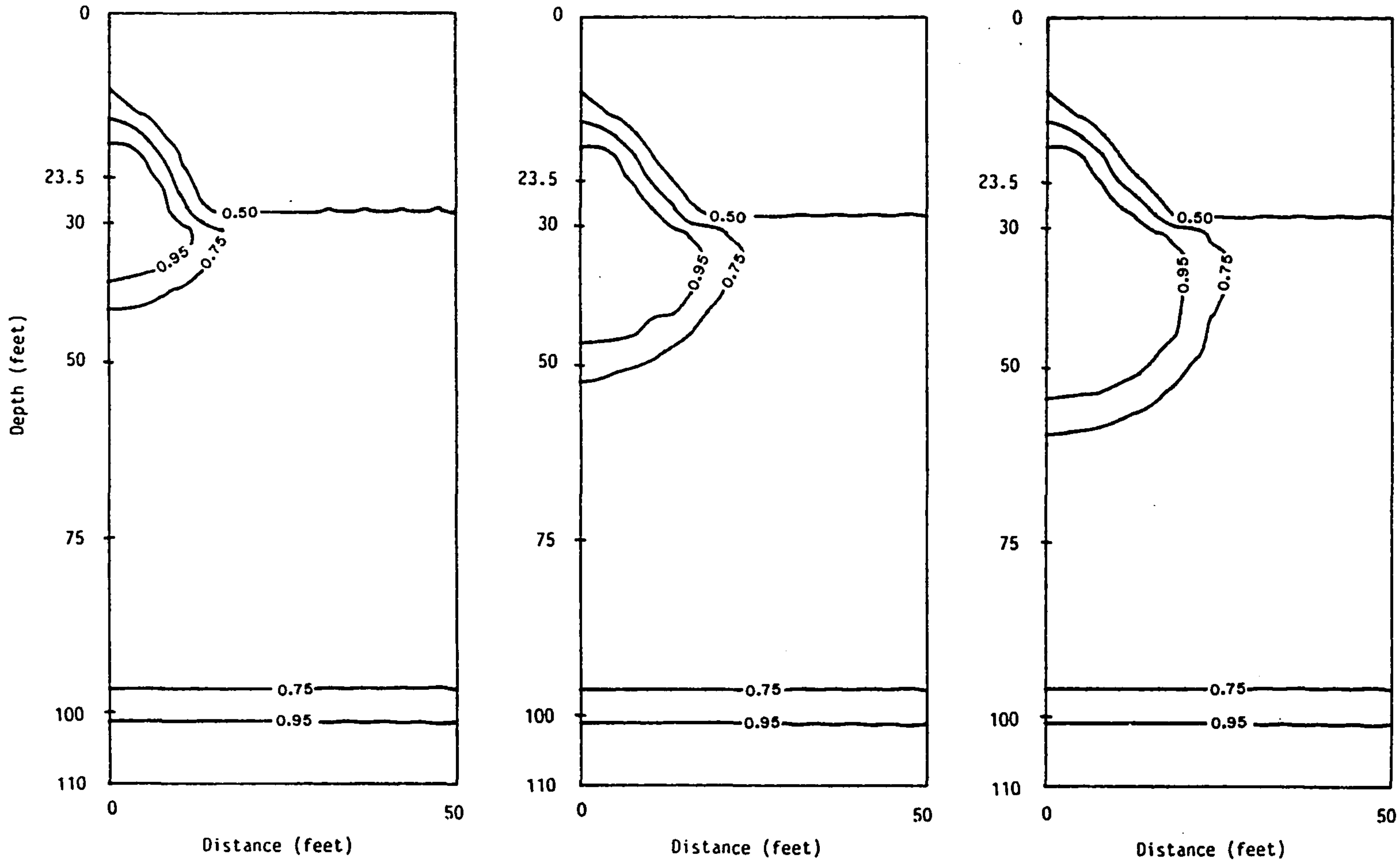


Figure 31a. Time = 0.50 Hours

Figure 31b. Time = 1.167 Hours

Figure 31c. Time = 2.00 Hours

FIGURE 31. DEGREE OF SATURATION: CASE 3, STORM 1

95 percent saturation decreased steadily over the period of recharge, from a maximum of about 20.2 ft/hr over the first 0.5 hours, to about 1.9 ft/hr over the last 0.75 hours of recharge **(Figures 31b through 32c)**. The maximum horizontal extent of the 95 percent saturated part of the drainage plume was about 20 feet **(Figure 32a)**.

24-Hour Drainage Period

The 95 percent saturated part of the drainage plume continued to decrease in velocity upon ceasing recharge, to about 2.75 ft/hr over the first 0.5 hours of drainage following the first storm, corresponding to 2.75 to 3.25 hours total time **(Figure 32b)**. The 90 percent saturated part of the plume contacted the water table by a time of 6.25 hours of drainage, or 9.00 hours total time **(Figure 33a)**.

An anomalous rise in the water table was observed during the latter part of the 24-hour drainage period **(Figures 33b and 33c)**. This may have been due to numerical instability of the model in the zone of the capillary fringe, due to the high specified background suction head of about 11 feet. Due to the persistence of this anomalous condition, simulation of recharge from a second storm event was not performed.

COMPARISON OF SOIL SURFACE AREA MEASUREMENTS

The volume of vadose zone material contained within the zone of 80 percent saturation at its maximum extent was estimated for each of

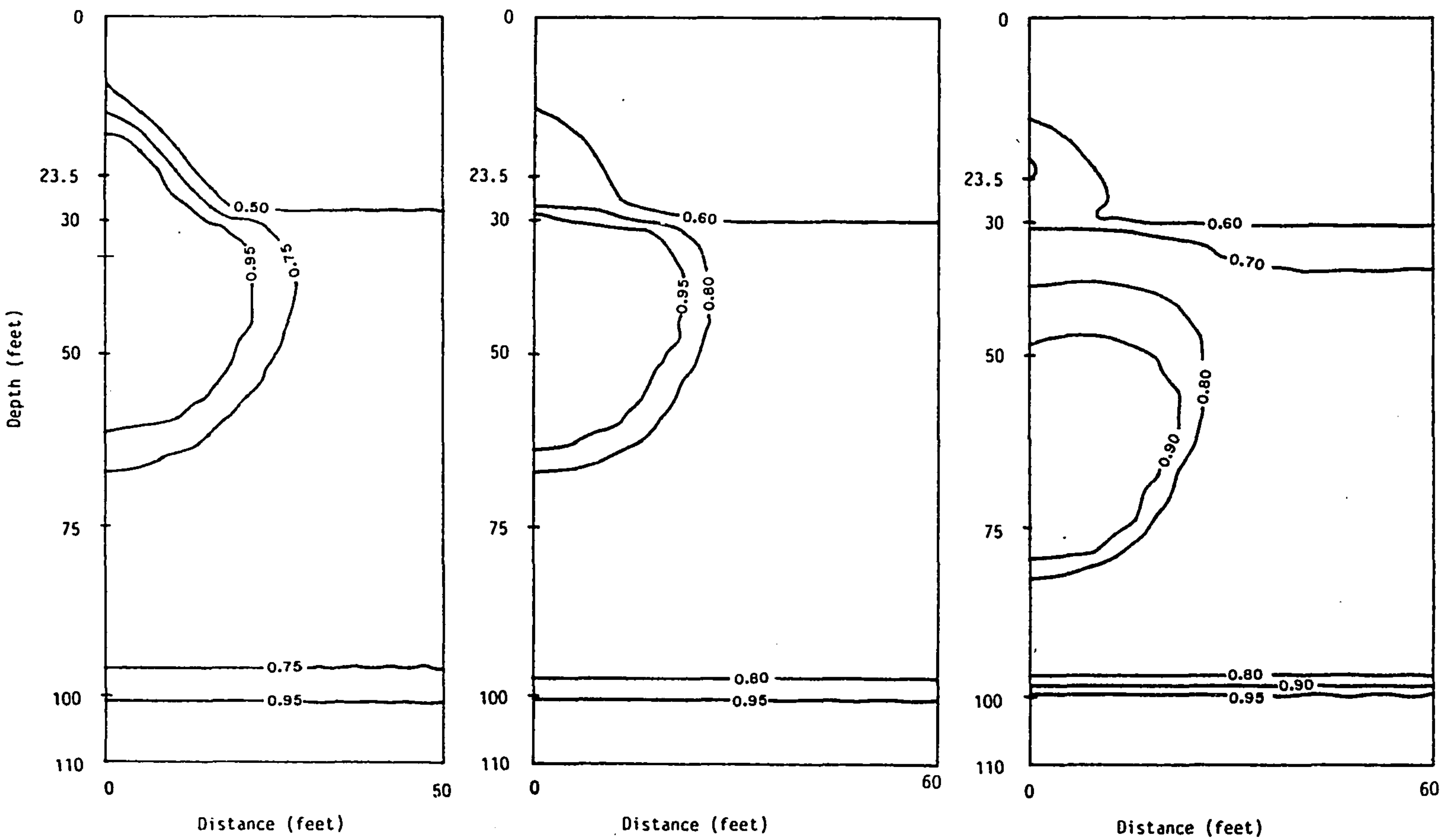


Figure 32a. Time = 2.75 Hours

Figure 32b. Time = 3.25 Hours

Figure 32c. Time = 5.50 Hours

FIGURE 32. DEGREE OF SATURATION: CASE 3, STORM 1 AND POST-STORM 1 DRAINAGE PERIOD

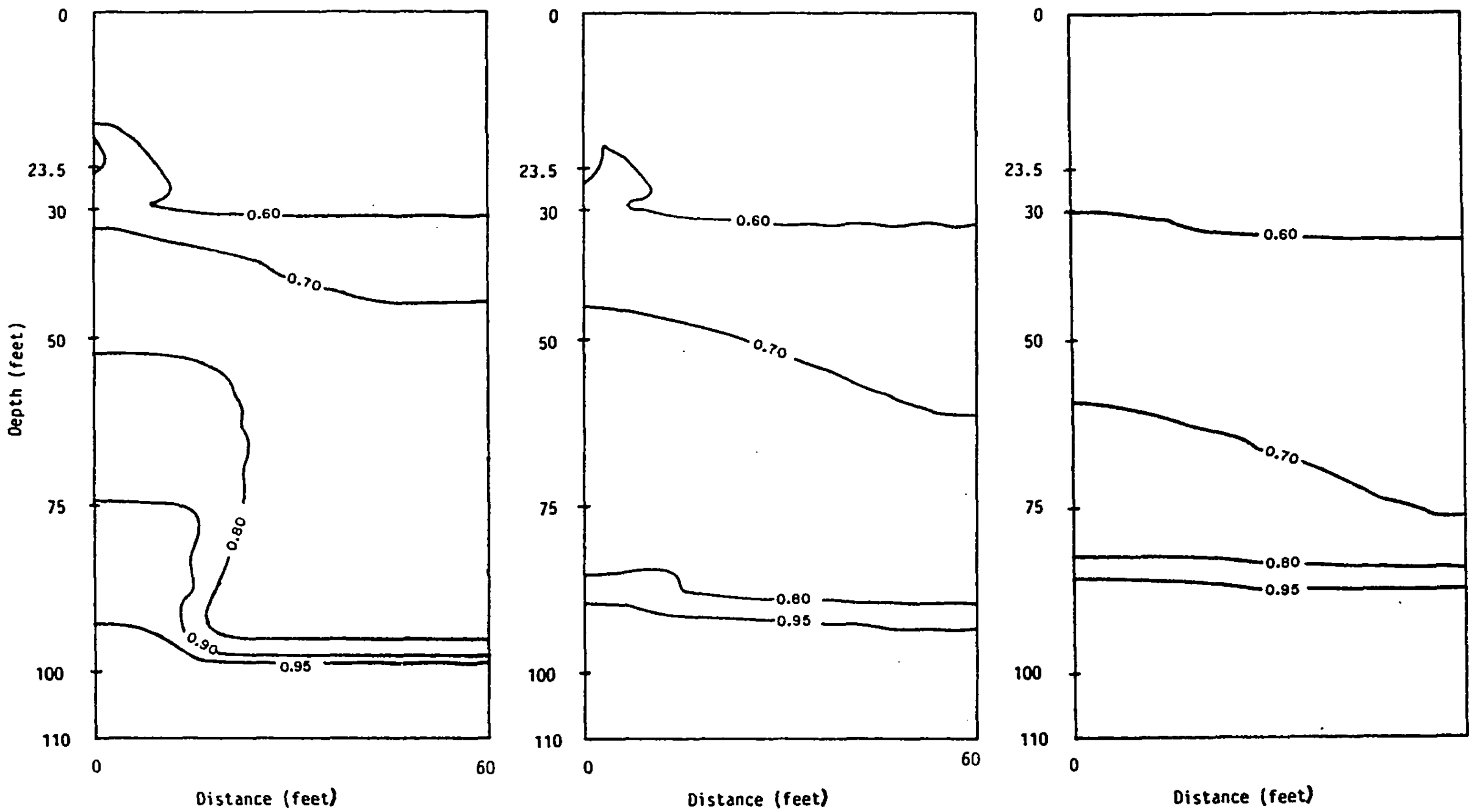


Figure 33a. Time = 9.00 Hours

Figure 33b. Time = 17.00 Hours

Figure 33c. Time = 24.00 Hours

FIGURE 33. DEGREE OF SATURATION: CASE 3, POST-STORM 1 DRAINAGE PERIOD

the case study simulations. Using the estimated volume in conjunction with the representative bulk density for each material, an estimate of the total weight of soil material within the zone of 80 percent saturation was obtained. Using this value in conjunction with the representative specific surface value for each soil material, an estimate of total soil surface area for each case was calculated.

Volume calculations were made by approximating the shape of the 80 percent saturated part of the drainage plume with simplified geometric shapes, such as right circular cones and cylinders, and combinations and portions thereof. In Case 1, the total volume was estimated by representing the area of saturation as a right circular cone of radius 10.5 feet, the maximum extent of the zone of 80 percent saturation during the simulation. The height of the cone was taken as the distance between the top of the zone of saturation by the dry well borehole and the water table, 83.5 feet (**Figure 17b**). In Case 2, the saturated volume was estimated as a combination of right circular cones, one above the layer transition at 30 feet and one below. The cone below 30 feet was truncated where the zone of saturation contacted the water table (**Figure 30**). The zone of 80 percent saturation for Case 3 was depicted in a similar manner as Case 2, but representing the zone from a depth of 30 feet to the water table as a cylinder (**Figure 32a through 33b**). The estimated volumes and parameters used in their calculation are shown in **Table 16**.

TABLE 16

ESTIMATED SOIL VOLUMES WITHIN ZONE OF 80 PERCENT SATURATION

CASE	REGION	APPROXIMATE SHAPE	HEIGHT (feet)	RADIUS (feet)	VOLUME ($feet^3$)
1	Vadose zone above water table	Cylinder	83.5	10.5	28,921
2	Zone above 30 feet bls	Cone	14.5	30	13,666
	Zone below 30 feet bls and above water table	Truncated cone	70	47 (top) 11 (base)	210,032
3	Zone above 30 feet bls	Cone	14.75	16.3	4,104
	Zone below 30 feet bls	Cylinder	70	20.5	92,418

Finally, estimated volumes of vadose zone soil material in the zone of 80 percent saturation were multiplied by values of bulk density for the soil materials to compute the maximum soil surface area in the zone of saturation for the given simulation. These values were compared as a measure of the relative degree of sorption likely to take place in each case. The resulting values are shown in **Table 17**.

TABLE 17

ESTIMATED SURFACE AREA OF
ZONE OF 80 PERCENT SATURATION

CASE	MATERIAL	SOIL SURFACE AREA (m^2)	TOTAL SOIL SURFACE AREA (m^2)
1	1	5.56×10^{10}	5.56×10^{10}
2	1	2.63×10^{10}	
	2	4.55×10^{11}	4.81×10^{11}
3	1	7.89×10^{9}	
	3	1.53×10^{11}	1.61×10^{11}

Total Soil Surface Area Ratio: Case 1: Case 2: Case 3 = 1:8.6:2.9

SUMMARY OF RESULTS

CASE 1

Vertical flow velocities of the simulated dry well recharge at a constant rate of approximately 0.5 cfs decreased over the period of recharge from a maximum at the start of recharge to a minimum at the termination of recharge. Small deviations from decreasing vertical velocity with time resulted from small variations in borehole head. Vertical flow velocities of the zone of 95 percent saturation varied from 51.2 ft/hr to 34.4 ft/hr during the first recharge event. Vertical flow velocities were generally higher during the second recharge event, due to higher relative hydraulic conductivity. The 95 percent saturated part of the recharge plume reached the water table between 1.5 and 2.5 hours into the first recharge event. Vertical flow velocity of the zone of 95 percent saturation for the second recharge event exceeded twice that of the first recharge event for the first 15 minutes of recharge. The vertical velocity of the zone of 80 percent saturation during the 24-hour lag period decreased from an average of 4.6 ft/hr over the first 3 hours to one of 0.8 ft/hr during the final 9 hours. The maximum horizontal extent, or radius, from the center of the dry well of the zone of 95 percent saturation for the first case was approximately 8.5 feet **(Table 18)**. The estimated maximum soil surface area within the zone of 80 percent saturation for the first case was 5.56×10^{10} square meters.

TABLE 18

MAXIMUM RADIUS OF ZONE OF 95 PERCENT SATURATION
DURING DRY WELL RECHARGE

CASE NUMBER	RADIUS (feet)
1	8.5
2	32.5
3	20.0

CASE 2

Vertical flow velocities for the zone of 95 percent saturation during recharge similarly decreased with time for the second case study. Vertical flow velocities were at a maximum during the first 0.5 hours of recharge, when flow took place primarily through Material 1 sediments. Average vertical flow velocity for the first 0.5 hours of recharge was approximately 16.4 ft/hr for the first recharge event, increasing to 27.4 ft/hr for the second recharge event. Average vertical flow velocities decreased to 3.3 ft/hr for the final 0.83 hours of the first recharge event, and to 8.7 ft/hr for the final 0.88 hours of the second storm event. Flow velocities for the zone of 95 percent saturation moving horizontally through Material 1 at the layer transition decreased steadily over the duration of the recharge events. Average horizontal flow rates decreased from 19.2 ft/hr to 3.6 ft/hr during the first recharge event, and from 31.4 ft/hr to 4.3 ft/hr during the second recharge event. Vertical flow velocity of the zone of 80 percent saturation averaged approximately 0.20 ft/hr during the intermediate 24-hour lag period. This velocity varied between about 0.3 ft/hr and 0.1 ft/hr over the 130 hour period following the second recharge event. The maximum radial extent of the 95 percent saturated part of the drainage plume during the first recharge event was 26.8 feet, increasing to 32.5 feet during the second recharge event. The 80 percent saturated part of the recharge water plume was estimated to contact the water table after 187 hours of drainage following the second recharge event. The estimated maximum soil surface area within the zone of 80 percent saturation was 4.81 x 10^{11} square meters.

CASE 3

Vertical flow velocities of the zone of 95 percent saturation during the first recharge event were less than those of Case 1 and greater than those of Case 2 for corresponding time periods. Average vertical flow velocity was at a maximum of 29.4 ft/hr over the first 0.5 hours of recharge, decreasing to 9.2 ft/hr between 1.167 hours and 2.0 hours. Average vertical flow velocity increased to 9.7 ft/hr between 2.0 and 2.75 hours due to a slight increase in borehole hydraulic head. The 90 percent saturated part of the drainage plume contacted the water table between 2.75 and 6.25 hours of drainage following the first recharge event. Average horizontal flow velocity along the layer transition was less than in Case 2, due to increased percolation into the Material 3 sediments. Average horizontal velocity of the 95 percent saturated part of the plume decreased steadily with time, from a maximum of 20.2 ft/hr over the first 0.5 hours, to 1.9 ft/hr between 2.0 and 2.75 hours of recharge. The maximum radial extent of the zone of 95 percent saturation was approximately 20.0 feet. The estimated maximum soil surface area included in the zone of 80 percent saturation was 1.61 x 10^{11} square meters. The ratio of soil surface area among the three cases relative to the first case was 1.0 for Case 1, 8.6 for Case 2, and 2.9 for Case 3.

DISCUSSION OF RESULTS

CASE 1 SIMULATION

The results of the Case 1 simulation indicate that water recharged into dry wells constructed in soil materials of uniformly high permeability will be characterized by rapid percolation through the vadose zone and undergo a relatively low degree of attenuation of contaminants. Coarse-grained deposits are generally characterized by high hydraulic conductivity and low clay content. As such, transmission time to the water table, the volume of the recharge plume, and total soil surface area available for attenuation will be relatively small. Also, due to the small radius of the recharge plume, dilution of recharged water by mixing with the regional aquifer will be relatively low.

The small radius of the recharge plume and low degree of horizontal movement make it less likely for recharged water to migrate to within the influence of a nearby pumping well, relative to Cases 2 and 3.

CASE 2 SIMULATION

The results of the Case 2 simulation indicate that water recharged into dry wells constructed in vadose zone deposits above layers of relatively low permeability will be characterized by relatively slow percolation through the vadose zone, a relatively high degree of lateral migration, and will undergo a relatively high degree of attenuation.

The presence of low-permeability soil horizons will aid attenuation of contaminants by: A) inducing lateral flow of recharged water, increasing the radius and overall volume of the recharge plume, B) increasing overall soil surface area within the recharge plume through their high-specific surface clay fractions, and C) increasing contact time of recharge water with soil particle surfaces. Also, due to the large radius of the recharge plume, dilution of recharged water by mixing with the regional aquifer will be relatively high.

The large radius of the recharge plume and high degree of horizontal movement make it more likely for recharged water to migrate to within the influence of a nearby pumping well, relative to Cases 1 and 3.

CASE 3 SIMULATION

The results of the Case 3 simulation indicate that water recharged into dry wells constructed in vadose zone deposits with layers of moderate permeability will be characterized by percolation rate, and degree of lateral migration and attenuation intermediate in comparison with Cases 1 and 2. The permeability and clay content of the soil horizons will mainly govern the rate of percolation, degree of horizontal migration and soil surface area within the recharge plume relative to Cases 1 and 2.

CONCLUSIONS

1. Recharge of water into dry wells constructed in uniform, highly permeable vadose zone deposits will be generally characterized by rapid percolation, low attenuation of contaminants, low dilution with groundwater, and a low component of horizontal flow, relative to recharge into a layered vadose zone.

2. Recharge of water into dry wells constructed above layered, low permeability vadose zone deposits will be characterized by slow percolation, high attenuation of contaminants, high duration of contact with vadose zone soil, high dilution and a high component of horizontal flow, relative to recharge into uniform soils of high permeability.

3. Overall soil clay content, especially that of montmorillonite, holds the greatest control on the sorptive surface area of the soil matrix.

4. Hydraulic conductivity holds the greatest control on the rate of percolation and ratio of vertical to horizontal components of flow.

5. Increased initial moisture content in the vadose zone will lead to increased percolation rates of recharged water.

RECOMMENDATIONS

A. DRY WELL USE

1. Dry wells should not be placed in areas where subsurface conditions are characterized by uniform, highly permeable soil materials between the dry well and the water table. Drainage via dry wells into this type of subsurface environment is conducive to low attenuation of water-borne pollutants in the vadose zone and low dilution by mixing with the regional aquifer.

2. Dry wells should be placed in areas where subsurface conditions are characterized by layered soil materials, some of which are characterized by low hydraulic conductivity and high clay content. Drainage via dry wells into this type of subsurface environment allows for a high degree of attenuation of water-borne pollutants in the vadose zone, and a high degree of dilution by mixing with the regional aquifer. However, care should be taken to locate the dry well sufficiently far from any active water supply wells.

3. A test borehole completed to the water table should be required at all proposed dry well locations. Cuttings from the borehole should be logged for lithologic classification and grain-size distribution. These data may be used to estimate the relative degree of attenuation, dilution, and horizontal migration of recharged water among proposed locations.

B. ADDITIONAL RESEARCH

1. Additional research should be conducted to characterize vertical to horizontal anisotropy in Pima County soils. Dry well recharge and other recharge/infiltration studies may be carried out with greater accuracy using this data.

2. Additional research should be conducted to determine moisture retention and relative hydraulic conductivity properties under both wetting and drying conditions for a wide distribution of soil types. This data would help to quantify the variability in soil water content and percolation rates between the two conditions.

3. Research should continue on incorporation of hysteresis effects and solute transport phenomena in variably saturated groundwater flow models.

4. Additional field studies should be conducted to better characterize the fate of recharge-borne solutes, and the mechanisms of attenuation in the vadose zone.

REFERENCES CITED

Allen, R.W., 1970. **Muscle Shoals, Alabama - Evaluation of Subsurface Drainage System Within Area Subject to Flooding.** Tennessee Valley Authority, Division of Water Control Planning, Knoxville, Tennessee.

Bandeen, R. F., 1984. **Case Study Simulations of Dry Well Recharge in the Tucson Basin.** Completion Report to Pima County Department of Transportation and Flood Control District.

Black, C.A., Evans, D.D., White, J.L., Ensminger, L.E., Clark, F.E., eds. 1965. **Methods of Soil Analysis.** No. 9, Am. Soc. Agron., Madison, Wisconsin.

Bouwer, H., and R. C. Rice, 1984. **"Hydraulic Properties of Stony Vadose Zones,"** Ground Water, Vol. 22, No. 6.

Callahan, M., Slimak, M., Gable, N., May, I., Fowler, C., Freed, J., Jennings, R., Durfee, R., Whitmore, F., Maestri, B., Maybe, W., Holt, B., Gould, C., 1979. **Water Related Fate of 129 Priority Pollutants.** U.S. Environmental Protection Agency, Office of Water and Waste Management; EPA-440/4-79-029.

Camp, Dresser, and McKee, Inc., 1983. **Analysis of Dry Wells as a Drainage Alternative, City of Chandler, Arizona.** Project Completion Report 82-313.

Coelho, M. A., 1974. **Spatial Variability of Water Related Soil Physical Properties.** Ph.D. Dissertation, University of Arizona, Tucson, Arizona.

Davidson, E.S., 1973. **Geohydrology and Water Resources of the Tucson Basin, Arizona.** U. S. Geological Survey Water Supply Paper 1939-E.

Davis, L. A., and S. P. Neuman, 1983. **Documentation and User's Guide: UNSAT 2 - Variably Saturated Flow Model.** Water, Waste and Land, Inc. Prepared for the U.S. Nuclear Regulatory Commission, NUREG/CR-3990, WWL/TM-1791-1.

Dumeyer, J. M., 1966. **Stratigraphy and Hydrogeology of the Water Resources Research Center Recharge Area.** Unpublished Report, University of Arizona, Department of Hydrology, Tucson, Arizona.

Feddes, R. A., E. Bresler, and S. P. Neuman, 1974. **Field Test of a Modified Numerical Model for Water Uptake by Root Systems.** Water Resources Research, 10(6): pp. 1199-1206.

Gass, T.E., 1977. **Subsurface Geology of the Santa Cruz Well Field, Pima County, Arizona.** Unpublished Master's Thesis, University of Arizona, Tucson, Arizona.

Geological Survey of Alabama, 1986. **Evaluation of Storm-Water Drainage (Class V) Wells, Muscle Shoals Area, Alabama.** Tuscaloosa, Alabama.

Guma'a, G. S., 1978. **Spatial Variability of In-Situ Available Water.** Unpublished Ph.D. Dissertation, University of Arizona, Tucson, Arizona.

Hem, John D., 1985. **Study and Interpretation of the Chemical Characteristics of Natural Water.** United States Geological Survey Water Supply Paper 2254, Third Edition. U.S. Government Printing Office, Washington, D.C.

Kirkham, D., and W. Powers, 1984. **Advanced Soil Physics.** R. E. Krieger Company, Malabar, Florida, 1984.

Kroszynski, U. I., and G. Dagan, 1975. **Pumping in Unconfined Aquifers: the Influence of the Unsaturated Zone.** Water Resources Research, 11(3): pp. 479-490.

McGuckin Drilling, Inc., Phoenix, Arizona 1984. Personal communication.

Miller, S., 1984. **The Impact of Stormwater Runoff on Groundwater Quality and Potential Mitigation.** Conference on Protection an Management of Aquifers with Emphasis on the Rathdrum-Spokane Aquifer, Spokane, Washington.

Mualem, Y., 1976. **A New Model for Predicting the Hydraulic Conductivity of Unsaturated Porous Media.** Water Resources Research, 12:513-522.

Neuman, S.P., 1973. **Saturated-Unsaturated Seepage by Finite Elements.** Proc. ASCE, J. Hydraulics Division, Vol. 99, No. HY12, pp. 2233-2250.

Neuman, S. P., R. A. Feddes, and E. Bresler, 1974. **Finite Element Simulation of Flow in Saturated-Unsaturated Soils Considering Water Uptake by Plants.** Development of Methods, Tools and Solutions for Unsaturated Flow. Third Annual Report. Technion, Haifa, Israel.

Neuman, S. P., 1975. **Galerkin Approach to Saturated-Unsaturated Flow in Porous Media.** Chapter 10 in Finite Elements in Fluids. Vol. I: Viscous Flow and Hydrodynamics. Edited by R. H. Gallagher, J. T. Oden, and O. C. Zienkiewicz. John Wiley and Sons, London. pp. 201-217.

Osborne, P. S., 1969. **Analysis of Well Losses Pertaining to Artificial Recharge.** Master's thesis, University of Arizona, Tucson, Arizona.

Pashley, E.F., Jr., 1966. **Structure and Stratigraphy of the Central, Northern and Western Parts of the Tucson Basin.** Unpublished Ph.D. Dissertation, University of Arizona, Tucson, Arizona.

Philip, J.F., 1957. **The Theory of Infiltration - the Profile at Infinity.** Soil Science, 83:435-448.

Richards, L. A., 1931. **Capillary Conduction of Liquids in Porous Mediums.** Physics, 1, pp. 318-333.

Schmidt, K.D., 1985. **Results of Dry Well Monitoring Project for a Commercial Site in the Phoenix Urban Area.** Completion Report for Maricopa Association of Governments, Phoenix, Arizona.

SMC Martin, Inc., 1983. **Preliminary Assessment of the Impact of Stormwater Drainage Wells on Ground-Water Quality, Roanoke, Virginia.** Technical Completion Report 68-01-6288, Task #2, Valley Forge, Pennsylvania.

Stumm, W., and Morgan, J., 1981. **Aquatic Chemistry.** Wiley and Sons, New York, N.Y. Second edition.

U.S. Department of Agriculture, Soil Conservation Service, 1972. **Soil Survey of Tucson-Avra Valley Area, Arizona.** U.S. Government Printing Office, Washington, D.C.

U.S. Department of Agriculture, Soil Conservation Service and Forest Service, 1979. **Soil Survey of Santa Cruz and Parts of Cochise and Pima Counties, Arizona.** U.S. Government Printing Office, Washington, D.C.

U.S. Department of Interior, 1977. **Groundwater Manual.** Water Resources Technical Publication. U.S. Government Printing Office, Washington, D.C.

Van Genuchten, M., Th., 1978. **Calculating Unsaturated Hydraulic Conductivity with a Closed-form Analytical Model.** U. S. Salinity Laboratory, U. S. Department of Agriculture, Riverside, California.

Van Genuchten, M., Th., 1980. **A Closed-Form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils.** Soil Science, Soc. Am. Jour. 44:892-898.

Wei, C. Y., and W. Y. Shieh, 1979. **Transient Seepage Analysis of Guri Dam.** Jour. Tech. Council ASCE 105(TCI), pp. 135-147.

White, R. E., 1979. **Introduction to Principles and Practice of Soil Science.** Wiley and Sons, New York.

Wilson, L. G., Neuman, S.P., Guzman, A.G., Leo, T.P., Osborn, M.D., and Worthington, M., 1986. **Effects of Channel Stabilization in Tucson Stream Reaches on Infiltration and Ground Water Recharge.** Draft final report to Pima County Department of Transportation and Flood Control District.

Wilson, L.G., 1983. **A Case Study of Dry Well Recharge.** U.S. Department of Interior, Research Project Technical Completion Report A-114-ARIZ, University of Arizona Water Resources Research Center, Tucson, Arizona.

Zaslavski, D. and G. Sinai, 1981. **Surface Hydrology: In-surface Transient Flow.** Jour. Hydr. Div. ASCE 107 (HYI), pp. 65-93.

Zeller, M.E., 1979. **Hydrology Manual for Engineering Design and Floodplain Management within Pima County Arizona.** Floodplain Management Section, Pima County Department of Transportation and Flood Control District.