Stormwater Retention Pond Infiltration Analyses in Uncofined Aquifers

Prepared for: Southwest Florida Water Management District
2379 Broad Street
Brooksville, Florida 34601
Telephone (904) 796-7211

Prepared by: Nicolas E. Andreyev, P.E. and Lee P. Wiseman, E.I.
Jammal & Associates, Inc.
1675 Lee Road
Winter Park, Florida 32789
Telephone (407) 645-5560

March 1989
Revised June 1989

Acknowledgements

We would like to thank Dr. Martin Wanielista, of the University of Central Florida and Dr. Sayed Sayed of Jammal & Associates, Inc. for their review of the technical content presented in this report. We would also like to extend our thanks to Mr. Jim Jammal, President of Jammal & Associates, Inc., for his review of the manuscript. We would like to thank Mr. Scott Hayes for his production direction of the final report. We would especially like to thank all personnel from Southwest Florida Water Management District for their review of the report, their technical input and their patience and cooperation.

Limited Use Statement

Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers

Table of Contents

Table of Contents (Continued)

CHAPTER 1

Introduction

Chapter 1
Introduction

Background

Due to the sensitive nature of Florida's environment (numerous lakes, wetlands and high groundwater table conditions) and the need to control localized flooding in urbanized areas, stormwater retention and detention ponds are frequently used. A stormwater detention pond is generally used to temporarily store and subsequently discharge collected stormwater runoff from a developed site at a rate less than or equal to the predevelopment discharge rate. A stormwater retention pond usually receives a minimum first flush (first 1/2-inch and up to first 1-inch) of stormwater runoff generated by the contributing area then discharges this volume through on-site soil infiltration.

The main purpose of a stormwater detention pond is to control localized flooding, while a stormwater retention pond is often used to collect and treat "the first flush" of pollutants in the stormwater runoff. In many instances, a single pond incorporates both design features of detention and retention. In this report, only the stormwater retention pond infiltration analysis will be presented and surface discharge stormwater detention ponds will not be elaborated further.

Several types of retention ponds are being used in Florida, including partial retention systems (retention with detention capacity and off-line retention), dry retention or wet retention, 100% retention (closed watersheds) and existing natural depressions. Establishment of the geologic and hydrogeologic setting of the site and estimating appropriate analytical parameters are essential for design of an effective surface water management system.

In recent years, stormwater runoff has been recognized as a source of surface water pollution. Local regulatory agencies have responded to this concern by urging better design and construction of retention ponds. The design strategy of stormwater retention ponds is based on the premise that the most highly polluted stormwater will occur at the beginning of a storm. It is assumed that after the first 1/2-inch to 1-inch of rain has fallen, the contributing polluted surfaces and storm sewers have been sufficiently washed and subsequent runoff will be relatively clean. The partial retention pond is designed to hold the runoff resulting from the first 1/2-inch to 1-inch of rain that falls during any storm. Runoff exceeding the first flush retention volume is not retained and is discharged to a detention pond or is directed off-site. The retention volume held in the basin is then disposed of by evaporation and infiltration into the shallow groundwater aquifer.

For more effective pollution control systems, retention ponds (off-line treatment) are provided with a diversion structure that bypasses runoff water downstream of the pond after the water level in the pond rises to the design level and the required first flush of polluted water is stored. It is expected at that time that the most highly polluted water will infiltrate into the ground and be removed of pollutants by biological, chemical and filtration processes, as it infiltrates into the natural soil. Because approximately 90% of storms in Florida have less than 1-inch of rainfall, such a pond will retain and dispose of a high percentage of the yearly rainfall. In addition to the water quality benefit, this process reduces the total volume of runoff from the catchment and increases the amount of groundwater recharge. However, if at the beginning of the storm a retention pond contains runoff from the preceding storm, its effectiveness will be reduced. If the purpose is solely to reduce water pollution, then the antecedent storm problem may not be so serious. During periods when the antecedent storm was so recent that water remains in the pond and the "first flush" theory is correct, then there should not be much upstream pollution to accumulate in the runoff from the second storm. Therefore, it is important to determine how soon after the design storm the retention basin will again be empty and thus, be capable of operating at the intended design capacity.

In areas where the surface soil deposits consist of highly permeable material, it is possible to construct retention ponds to dissipate all collected stormwater through soil infiltration only. This practice is becoming increasingly common in developments located within closed watersheds. Often, such ponds are constructed in response to reg[illegible]ns on the amount of stormwater that can be discharged from a newly developed area or the lack of positive surface outfall. This type of pond is characterized by the expectation that all of the stormwater will be disposed of by infiltration to the effective shallow aquifer. The pond must have enough direct storage capacity to absorb the difference between the runoff inflow and the outflow due to infiltration. If the capacity is not sufficient, there will be either overflow, flooding and/or surcharge of the storm sewer. In order to design an effective retention pond system, the inflow hydrograph volume versus infiltration hydrograph volume and direct storage volume must be balanced for a design storm event.

In this report, a stormwater retention pond is defined as a man-made or natural depression into which stormwater runoff is directed for temporary storage with the expectation of disposal by infiltration into the effective shallow groundwater aquifer. The basic feature of such a pond is the transient nature of both the inflow into the pond and the outflow (infiltration). If a retention pond is to be effective it must have sufficient capacity available through both direct storage and infiltration at the beginning of a design storm event.

The required size of the pond depends on the quantity and the rate of inflow to the pond as well as the rate of infiltration during a storm event, given the antecedent conditions of the receiving aquifer. Under favorable conditions, the infiltration rate from the pond can be high enough to significantly reduce the required size of the pond. Another important design criteria presented in this report is the time of retention volume recovery of the pond. The retention volume recovery time, as defined in this report, is the time it takes for the retained stormwater to completely dissipate from the pond after the design storm event.

The continuous increase of property value has encouraged developers to optimize land utilization through the reduction of pond size. As a result, the presence of numerous undersized stormwater retention ponds in Florida has forced the regulatory agencies to require a more detailed engineering evaluation and analysis to ensure the design and construction of effective stormwater retention ponds.

Purpose

Due to the transient nature of stormwater inflow and infiltration rate of stormwater retention pond systems, a relatively complex analytical approach is required to size an effective, optimum stormwater retention pond. The primary purpose of this report is to provide a systematic approach to estimate the infiltration capacity of a stormwater retention pond during and following a storm event. The analysis is limited to shallow unconfined aquifer conditions with groundwater table at shallow depths (i.e., less than or equal to the pond width). The analytical approach assumes that the aquifer can be characterized as uniform, homogeneous, areally extensive and horizontal. Using the analytical approach documented in this report, it will be possible for a designer to follow step-by-step procedures to characterize the aquifer system, select appropriate hydraulic parameters and to estimate rate and volume of stormwater infiltration from a rectangular or circular pond system.

Scope

The scope of work was to review available literature relative to unsaturated and saturated groundwater flow below recharging basins andto develop a reliable analytical approach to estimate infiltration capacity of stormwater retention ponds. More specifically, the scope of service included:

1. Review of pertinent published literature.
2. Review of pertinent data collected by Jammal & Associates, Inc.
3. Selection of a suitable groundwater flow model to simulate retention pond recharge and volume recovery.
4. Based on a literature review, select suitable methods of analysis to estimate infiltration during unsaturated and saturated flow conditions.
5. Using a three-dimensional groundwater flow computer model, generated a series of dimensionless curves for use in estimating infiltration from stormwater retention ponds during saturated flow.
6. Development of guidelines for planning a field investigation program and presentation of a series of available methods of field and laboratory testing.
7. Presentation of example problems for the evaluation and design of stormwater

retention ponds.

8. Development of guidelines for construction and maintenance of stormwater retention ponds.
9. Preparation of this report to be used as a guide for permitting evaluators and the designer and as a source of referral for stormwater retention pond infiltration analyses.

CHAPTER 2

Literature Survey

Chapter 2
Literature Survey

The primary intent of the literature survey was to collect and review published literature relative to transient unsaturated and saturated groundwater flow in unconfined aquifers. A computer data base called "LINE", associated with the University of Central Florida Library, was used in addition to direct literature search methods. The following brief summary of relevant data is the result of our literature search and review.

SUMMARY

Unsaturated Flow

Whisler & Bouwer (1970) conducted a comprehensive review of unsaturated flow infiltration equations developed by various researchers. They compared the infiltration equations of Green and Ampt, Youngs and Ligon, et.al. and found that the input parameters are easy to measure and the solutions give reasonably accurate results. For the more complex models, such as developed by Philips (1957) and numerical analyses, the parameters are functions which must be analyzed statistically from known drainage behavior or evaluated experimentally for a complete water content range. Numerical methods yield results closer to field observations than the simpler models but input data for the numerical methods are not readily available. In addition, high-speed computers are needed for numerical methods. Whisler & Bouwer found that the Green and Ampt and Youngs equations are the most useable for infiltration analyses and yield relatively good results. However, these equations do not yield good water content or hydraulic head profiles.

Other authors that have evaluated infiltration (unsaturated groundwater flow) equations are Morel-Seytoux and Khanji (1974). They demonstrated that the empirical parameters of Green and Ampt have a precise physical meaning. The functional relationship between effective capillary head and the initial water content was developed.

In addition to the empirical equations of infiltration, several field methods have been developed. One such method involves the use of double ring infiltrometers. The feasibility of using this method for field measurement of infiltration was evaluated by Johnson (1963) and Beaver, Hartman and Wanielista (1977). The experimental data can be compared with Horton's equation to predict infiltration at any time. This field measurement of infiltration does not scale well to full scale stormwater retention systems due to lateral diversions of flow. However, the results of a double ring infiltrometer test can be used to estimate the initial vertical hydraulic conductivity within an unconfined aquifer which can then be used for the initial unsaturated infiltration analysis.

Saturated Flow

Methods to evaluate saturated infiltration and shallow groundwater behavior as a result of recharge to an unconfined aquifer has been the subject of numerous research papers. A summary of the infiltration equations, methods for solving the infiltration equations and the results of our research are listed in Table 1.

Groundwater recharge from a rectangular basin of infinite length was explored by Marino (1974b); Hantush (1967); and Rai and Singh (1980). Hantush considered uniform hydraulic conductivity, uniform effective storage and uniform recharge in his flow equations. He presented the original equations for groundwater mounding beneath rectangular basins. The equations were applicable when the recharge basins were outside the influence of aquifer boundaries and the groundwater mound did not exceed 50% of the initial saturated aquifer thickness.

Methods describing groundwater recharge from a rectangular basin of finite length were also investigated by Brock (1976) and Marino (1975b). Both models considered two-dimensional flow only. Brock compared Dupuit-Forchheimer's theory with potential theory for various recharge conditions and found that Dupuit-Forchheimer's theory approximates shallow percolation. However, for deep percolation theory, Poisson's equation, must be used. Brock's research (1976) considered hydraulic conductivity and recharge to be uniform, however, effective storage was spatially variable. Marino (1975b) developed a non-linear partial differential equation to describe groundwater recharge from a rectangular basin considering uniform hydraulic conductivity, uniform effective storage and uniform recharge. Upon comparison with Hantush's equations, he found that his equations yield higher groundwater mound heads.

Groundwater recharge from circular basins was investigated by several researchers, including Glover (1964), Marino (1975a) and Ortiz, Zachmann, McWhorter and Sunada (1979). Glover was the first researcher to describe mound geometry as a result of transient recharge below a circular basin. He considered uniform storage coefficient, hydraulic conductivity and recharge rate. The flow equation in the latter two research studies is a non-linear partial differential equation.Hydraulic conductivity and recharge were uniform in both equations. However, effective storage was horizontally variable in the research by Ortiz, et. al. (1979) whereas Marino (1975a) considered uniform effective storage. Marino compared the results of his equation with published results by Hunt (1971) and Hantush (1967) for the same conditions. Marino found that his equations yield heads less than those predicted by Hunt but greater than those predicted by Hantush.

Ortiz et al. (1979) developed groundwater mounding equations for different zones of the aquifer system, to account for spatial variation of effective storage coefficient. These equations were compared with Glover's equations and other numerical methods and indicated good correlation. The general idea of this research was the basis for our selected method of saturated infiltration analysis presented in this report.

Rai and Singh (1980) applied Boussinesq's equation for uniform effective storage and time varying recharge. They looked at varying the effects of effective storage and recharge on groundwater mound growth. They found that the higher the effective storage the less significant is the mounding effect.

Numerous authors investigated groundwater recharge adjacent to aquifer boundaries. Amar (1975) compared his model with the Dupuit-Forchheimer equation and experimental data. He found that his model correlates well with experimental data but not with the Dupuit-Forchheimer equation except under loading conditions. He also found a discrepancy between mathematical approximations and experimental data to be due to capillary effects. Cabrera and Mathey (1984) compared their groundwater mounding equations with available analytical solutions and other numerical methods and showed good agreement between all three. They found, however, that computer times for their model are longer than the time consumed by other numerical models. Other researchers, Latinopoulous (1981) and Rai and Singh (1981) examined the effects of rain recharge, hydraulic conductivity and effective storage on groundwater mound height. Rao and Sarma (1984) developed a flow equation to simulate localized groundwater recharge to a finite aquifer adjacent to an impermeable wall and a constant head stream. They compared their model with the method of images, experimental data and numerical solutions and obtained excellent correlation.

Groundwater recharge from various shaped recharge basins in two dimensions was explored by Youngs (1980). He used Boussinesq's potential function (an approximation to Poisson's equation) and considered a vertical variation of hydraulic conductivity, uniform effective storage and recharge. Youngs found that the derived potential function calculates reliable estimates of infiltration rates but incorrect groundwater table heights.

Groundwater recharge from wells was investigated by Karamouzis and Terzidis (1984). Hunt (1973) considered groundwater recharge from axisymmetric reservoirs. Tinsley (1974) investigated groundwater recharge adjacent to aquifer boundary in two dimensions. He developed a model to predict increased groundwater flow to a stream during and following a storm. Tinsley compared results with experimental data obtained from the Hele-Shaw field model and examined effects of initial slope of groundwater table on groundwater flow rate and found the correlation between the two models decreases with increasing groundwater slope.

Of the saturated flow models that have been developed to date, the computer model titled "A Three-Dimensional Finite Difference Groundwater Flow Model", also known as MODFLOW, developed by U.S. Geological Survey (McDonald and Harbaugh, 1984) was found to be the most powerful and flexible model to incorporate the complex groundwater flow conditions of stormwater infiltration from retention ponds. This model has been approved by the U.S. Environmental Protection Agency, the Florida Department of Environmental Regulation and the Water Management Districts as an effective tool to predict groundwater flow in three-dimensional modeling. This model can be used to simulate flow in two or three dimensions and can consider uniform or variable hydraulic parameters of the aquifer systems, including the effective storage coefficient (fillable porosity), hydraulic conductivity and varying recharge rate (with time and space). In addition, the model can account for all forms of hydraulic drainage features and can simulate flow in three dimensions for any combination of unconfined, confined or semi-confined leaky aquifers.

RECOMMENDED MODELS

Based on the results of our literature review, evaluation of the model flexibilities and the practicality of obtaining the necessary hydraulic parameters to use the models, we conclude that the following equation and models would be the most practical and produce reliable results:

Unsaturated Flow Model

The Green and Ampt Equation

Saturated Flow Model

The USGS "MODFLOW" computer model using finite difference equations to solve three-dimensional groundwater flow. Using this computer model, a series of dimensionless curves were generated for stormwater retention pond analysis.

The details for using these equations and dimensionless curves are presented in the subsequent chapters of this report.

TABLE 2-1

Results of Literature Survey for Methods to Predict Shallow Groundwater Behavior and Estimate Saturated Seepage as a Result of Recharge to an Unconfined Aquifer.

Seepage Condition	Researcher(s) and Date	Seepage Equation	Method of Solving Seepage Equation	Aquifer Parameters			Comments
				k	*f*	Recharge	
Groundwater recharge from rectangular basin-finite length (two-dimensional flow)	Brock, 1976	Dupuit Forchheimer	General Analytical				Looked at shallow and deep percolation from experimental data, found Dupuit-Forchheimer theory approximates shallow percolation well. However, for deep percolation potential theory must be used.
		Potential theory (Poisson's Equation)	Finite Difference Methods	uniform	variable (space)	uniform	
	Marino, 1975b differential equation	Non-linear partial Methods	Finite Difference	uniform	uniform	uniform	Followed Dupuits assumptions. The rate of recharge was much smaller than hydraulic conductivity of underlying aquifer. Compared to Hantush's equations and yield higher mounded heads than Hantush.
Groundwater recharge from circular basin (two-dimensional flow	Marino, 1975a	Second Order non-linear partial differential equation	Finite Difference Methods	uniform	uniform	uniform	Followed Dupuit's assumptions. Compared with Hantush equations and research by Hunt (Laplaces equation). Comparison resulted in heads less than predicted by Hunt but greater than predicted by Hantush for same conditions.
	Ortiz, Zachmann, McWhorter & Sunada, 1979	Derived partial differential equations	Analogy for heat flow	uniform (space)	variable	uniform	Accounted for reduction in storage capacity caused by in-transit water and developed curves for groundwater mound growth. Developed and presented dimension-less curves for prediction of groundwater growth.
	Glover, 1964	Modified well function, w(u)	Direct solution	uniform	uniform	uniform	Presented the original equations describing groundwater mound geometry due to transient recharge.

TABLE 2-1 (continued)

Results of Literature Survey for Methods to Predict Shallow Groundwater Behavior and Estimate Saturated Seepage as a Result of Recharge to an Unconfined Aquifer.

Seepage Condition	Researcher(s) and Date	Seepage Equation	Method of Solving Seepage Equation	Aquifer Parameters k	Aquifer Parameters *f*	Aquifer Parameters Recharge	Comments
Groundwater recharge from rectangular basin-infinite length (two-dimensional flow)	Hantush, 1967	Derived partial differential equation	LaPlace transform for t Fourier Cosine transform with respect to x & y	uniform	uniform	uniform	Presented original equations for groundwater mounding beneath rectangular and circular basins. Equations applicable when outside of influence of aquifer boundaries and groundwater mound does not exceed 50% of initial saturated aquifer thickness.
	Rai & Singh, 1980	Boussinesq's equation (non-linear partial differential equation)	Method of successive approximations	uniform	uniform	variable (time)	Follows Dupuit assumptions. Applicable when groundwater moun does not exceed 50% of initial saturated thickness. Looked at varying effects of effective storage. Higher effective storage the less significant is the mounding effect.
	Ortiz, McWhorter, Sunada & Duke, 1978	Derived partial differential equations	Analogy for head flow	uniform	variable (space)	uniform	Developed equations to predict groundwater table mounding beneath recharge basin. Compared with Glover's equation and a previously verified numerical method. Good correlation was observed between all three methods.
Groundwater recharge adjacent to aquifer boundaries (two-dimensional flow)	Amar, 1975	LaPlace's equation	Finite difference methods (accelerated Liebman theorem)	uniform	variable (space)	uniform	Compared model with Dupuit-Forchheimer equation and experimental data. Found model correlated well with experimental data but not with Dupuit equation except under limited conditions. Found discrepancy between mathematical approximations and experimental data to be due to capillary effects.

TABLE 2-1 (continued)

Results of Literature Survey for Methods to Predict Shallow Groundwater Behavior and Estimate Saturated Seepage as a Result of Recharge to an Unconfined Aquifer.

Seepage Condition	Researcher(s) and Date	Seepage Equation	Method of Solving Seepage Equation	Aquifer Parameters k	Aquifer Parameters *f*	Aquifer Parameters Recharge	Comments
Groundwater recharge adjacent to aquifer boundaries (two-dimensional flow)	Cabrera & Mathey, 1984	Galerkin approximation of non-linear partial differential equations	Newton Raphson method	uniform	uniform	uniform	Compared with available analytical solutions and other numerical methods and showed good agreement. Computer times, however, are longer than time consumed by other numerical models.
	Marino, 1974a	Non-linear partial differential equation approximations	LaPlace transform and method of successive	uniform	uniform	uniform	Groundwater recharge next to a constant head reservoir. For prediction of future water levels the equations should be used with the method of successive approximations.
	Marino, 1974b	Second Order non-linear partial differential equation	LaPlace transformation	uniform	uniform	uniform	Followed Dupuits assumptions. Long strip recharged adjacent to constant head reservoir.
	Marino, 1974c	Non-linear partial differential equation	LaPlace transform and method of successive approximations	uniform	uniform	uniform or variable (time)	Groundwater recharge bounded by a constant level stream or drain on both sides of recharging area. Compared with Hele-Shaw viscous flow model and yield comparable results.
	Latinopoulos, 1981	Derived partial differential equation	LaPlace transform and Cauchy theories of residues	uniform	uniform	uniform	Examined effects of recharge duration, transmissivity and storm duration on mounding effects.
	Rai & Singh, 1981	Marino's partial differential equation	LaPlace transform and method of successive approximations	uniform	uniform	uniform	Localized transient recharge adjacent to a constant head reservoir or one side. Examined effects of varying rate of recharge, effective storage and hydraulic conductivity.

TABLE 2-1 (continued)

Results of Literature Survey for Methods to Predict Shallow Groundwater Behavior and Estimate Saturated Seepage as a Result of Recharge to an Unconfined Aquifer.

Seepage Condition	Researcher(s) and Date	Seepage Equation	Method of Solving Seepage Equation	Aquifer Parameters k	Aquifer Parameters *f*	Aquifer Parameters Recharge	Comments
Groundwater recharge adjacent to aquifer boundaries (two dimensional flow)	Rao & Sarma, 1984	Dupuit-Forchheimer for Q and Hantush equations for head	Finite Fourier transform	uniform	uniform	uniform	Localized groundwater recharge to a finite aquifer adjacent to impermeable wall and constant level stream. Compared with method of images, experimental data and numerical solutions and yielded excellent correlation.
Groundwater recharge from various shaped recharge basins (two dimensional flow)	Youngs, 1980	Girinskii's potential function (an approx. to Poisson's equation)	Numerical relaxation for head and Leibniz's theorem for Q	vertical variation (space)	uniform	uniform	Looked at water table drawdown by pumped wells and groundwater mounds from localized surface infiltration. The potential function calculates reliable estimates of seepage rates but incorrect groundwater table heights.
Groundwater recharge from wells (two dimensional flow)	Karamouzis & Terzidis, 1984	Galerkin approximation method to approximate a non-linear partial differential equation	Finite difference methods	uniform	uniform	uniform	Developed finite element model which simulates artificial recharge and flow due to pumped wells. Can account for drainage features.
Groundwater recharge from axisymmetric reservoirs	Hunt, 1973	Derived approximate 1st order saturated flow model	Direct calculation	uniform	uniform	uniform	An approximate solution for seepage from axisymmetric reservoir. Compared results with another researcher (Jeppson) and results agreed.

TABLE 2-1 (continued)

Results of Literature Survey for Methods to Predict Shallow Groundwater Behavior and Estimate Saturated Seepage as a Result of Recharge to an Unconfined Aquifer.

Seepage Condition	Researcher(s) and Date	Seepage Equation	Method of Solving Seepage Equation	Aquifer Parameters: k	Aquifer Parameters: *f*	Aquifer Parameters: Recharge	Comments
Groundwater recharge adjacent to aquifer boundaries (two-dimensional flow)	Tinsley, 1974	Boussinesq's equation	Finite difference methods	uniform	uniform	uniform	Developed a model to predict increased groundwater flow to a stream during and following a storm. Compared results with experimental results obtained from Hele-Shaw field model. Examined effects of initial slope of ground-water table on seepage rate. Correlation decreased with increasing groundwater slope.
All conditions listed above (two or three dimensional flow)	McDonald & Harbaugh, 1984	Derived 1st order non-linear partial differential equation	Finite difference methods	uniform or variable (space)	uniform or variable (space)	uniform or variable (space & time)	EPA approved model which can simulate flow in three dimensions for any combination of unconfined, confined of semi-confined leaky aquifers. Can account for all forms of drainage features and transient or steady state seepage.

CHAPTER 3

Review of Field and Laboratory Test Methods

Chapter 3

Review of Field and Laboratory Test Methods

General Considerations

One of the most important steps in the evaluation of a stormwater retention pond is determining which test methods and how many tests should be conducted per site or per system. Typically, a soil boring and some type of hydraulic conductivity measurement is conducted for each stormwater retention pond, as a minimum. The number of soil borings and hydraulic conductivity tests performed are usually based on site topography, subsurface hydrogeologic conditions, pond size and pond geometry. Judgement and experience are usually applied in the decision making process. In this report, we have developed methods for estimating the required number of borings and hydraulic conductivity tests in order to characterize the shallow aquifer system for retention pond designs. These methods should only be used as a guide and more or less tests may become necessary based on local experience and knowledge of site hydrogeologic conditions.

Soil Borings

To explore the subsurface soil and groundwater table conditions within an area proposed for a stormwater retention pond, Standard Penetration Test (SPT) borings (ASTM D-1586) or auger borings (ASTM D-1452) can be used. Standard Penetration Test borings provide a reasonable soil profile and an estimate of the relative density of the soils. However, measurement of the groundwater table depth in SPT borings is usually less accurate than in auger borings due to the drilling fluid (bentonite-mud) used during the drilling process. Power auger borings generally provide more accurate soil profiles and a better estimate of depth to the groundwater table. Therefore, a combination of SPT and auger borings in a retention pond would provide the best data to characterize the effective aquifer system.

In general, it is preferable to extend soil borings to the confining layers of the effective aquifer system. However, for small retention pond systems (<1,000 ft^2), such a requirement may not be practical or cost effective. A more appropriate method of estimating minimum soil boring depth would be to extend the boring to the confining layers or a minimum of 10 feet below proposed pond bottom. For modeling purposes confining layers should be set at the encountered elevations of poorly permeable soil layers (confining layers) or at the bottom of the test borings, if confining layers are not encountered.

When selecting the minimum number of borings, a minimum of one soil boring should be drilled to at least 10 feet below the proposed pond bottom elevation within the pond area. When more than one boring is required, the following approximate equation (empirical equation developed by Jammal & Associates, Inc.) can be applied to estimate the number of soil borings required. The approximate equation takes into consideration the average area and configuration of the proposed pond:

$$B = 1 + \sqrt{2A} + \frac{L}{(2\pi W)} \qquad \textbf{(3-1)}$$

Where:

B = number of borings required
A = average pond area in acres
L = length of pond, in feet
W = width of pond, in feet

In addition, an approximate equation to estimate the minimum number of hydraulic conductivity tests to be conducted was also developed by Jammal & Associates, Inc. and is presented below:

$$P = 1 + \frac{B}{4} \qquad \textbf{(3-2)}$$

Where:

P = number of hydraulic conductivity tests required
B = number of borings drilled

These equations are useful in determining the minimum number of tests that should be conducted. Additional tests may be required for systems located within a site which has complex hydrogeology and/or appreciable topographic relief.

Measurement Of Hydraulic Conductivity

The hydraulic conductivity can be defined as the discharge rate through a unit area under a unit hydraulic gradient.

$$K = \frac{Q}{iA} \qquad \textbf{(3-3)}$$

If the seepage rate, perpendicular flow area and hydraulic gradient are known, hydraulic conductivity can be calculated for any flow condition in a laboratory test or in the field. Likewise, for any situation where the seepage velocity is known at a point at which the hydraulic gradient and soil porosity are also known, hydraulic conductivity can be calculated. Although the hydraulic conductivity is usually constant throughout a given material, the magnitude may vary depending on several factors such as:

1. The viscosity and quality of the water
2. Grain size distribution of the soils
3. The size and shape of the soil particles
4. Density of the soil
5. Cementation of the soil
6. Degree of saturation

All of these factors strongly influence the hydraulic conductivity. The relationship between the hydraulic conductivity and these factors can be expressed by the following equation (Darcy 1856, Kozeny 1927 and Carmon 1956):

$$K = \frac{2g}{\nu C_s} D^2 \frac{e^3}{1+e} \qquad \textbf{(3-4)}$$

Where:

- K = hydraulic conductivity, ft/sec
- g = the acceleration due to gravity = 32.2 ft/s^2
- ν = the kinematic viscosity of water = (1.059 x 105 ft^2/s at 70º F)
- C_s = particle shape factor
- D = the weighted or characteristic particle diameter, feet
- e = the void ratio

The characteristic diameter D is obtained from a grain size distribution analysis using the following equation:

$$D = \frac{\Sigma Mi}{\Sigma (Mi/Di)} \qquad \textbf{(3-5)}$$

Where:

- Mi = the mass retained between two adjacent sieves
- Di = the mean diameter of the two adjacent sieves

Typical hydraulic conductivity values for granular soils and consolidated materials are summarized in **Table 3-1**. Typical values of hydraulic conductivity for various soil types in unconfined sand aquifers in Florida are presented in **Table 3-2**.

There are several direct methods of hydraulic conductivity measurement which can be performed in the laboratory or in the field. In general, field methods, if performed properly, can yield the most accurate results. Even though laboratory methods can be relatively accurate, the disturbance of the soil sample during sample collection is a major concern. In general, it is possible to obtain relatively undisturbed soil samples at shallow depths (less than 10 feet) by excavating a pit and driving thin-wall, short "Shelby tubes" by hand. Based on our experience, obtaining relatively undisturbed tube samples of sand at more than 10 feet below ground surface is typically not possible.

Laboratory Methods

There are two standard types of laboratory hydraulic conductivity measurements. The first type involves the collection of an undisturbed Shelby tube soil sample (ASTM D-1587). The sample is either collected in the horizontal or vertical direction using a Shelby tube soil sampler and transported to the laboratory for

TABLE 3-1. *Typical Values of Hydraulic Conductivity for Various Soils (Bouwer, 1978)*

TYPE OF SOIL	HYDRAULIC CONDUCTIVITY (Meters/Day) *
Clay soils (surface)	0.01 to 0.2 m/day
Deep clay beds	10^{-8} to 10^{-2} m/day
Loam soils (surface)	0.1 to 1 m/day
Fine sand	1 to 5 m/day
Medium sand	5 to 20 m/day
Coarse sand	20 to 100 m/day
Gravel	100 to 1000 m/day
Sand and gravel mixes	5 to 100 m/day
Clay, sand, and gravel mixes (till)	0.001 to 0.1 m/day
Sandstone	0.001 to 1 m/day
Carbonate rock with secondary porosity	0.001 to 1 m/day
Shale	10^{-7} m/day
Dense, solid rock	<10 to 5 m/day
Fractured or weathered rock (aquifers)	0.001 to 10 m/day
Fractured or weathered rock (core samples)	almost 0 to 300 m/day
Volcanic rock	almost 0 to 1000 m/day

* To convert to feet/day, multiply by 3.281 feet/meter

To convert to inches/hour, multiply by 1.64 $\frac{\text{inches - day}}{\text{meter - hour}}$

TABLE 3-2. *Typical Values of Permeability for Various Soil Types in Unconfined Sand Aquifers in Florida (Jammal & Associates, Inc. files)*

Type of Soil and USCS Classification	**Hydraulic Conductivity (Feet/Day) ***
Clayey fine sands and silty fine sands (SM-SC)	0.01 to 0.5
Slightly silty fine sands (SP-SM)	0.5 to 5.0
Clean fine sands (SP)	5.0 to 50.0
Medium fine sands (SP)	20.0 to 100.0

* Range of hydraulic conductivity values generally reflect the variation of percent of fine and soil density. Soil cementation also affects the hydraulic conductivity.

TABLE 3-3. *Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity*

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Laboratory Permeameter	Variable Head	Undisturbed	K_v & K_H	Das, 1982; Lambe, 1951	Good method for hydraulic conductivity measurement in any direction. Yields controlled test conditions and reliable results. Primary limitations are soil sample disturbance during collection. Measures hydraulic conductivity of one point in the effective aquifer system. Water must be distilled and de-aired.
	Variable Head	Remolded	K	Das, 1982; Lambe, 1951	Primary limitations are soil disturbance and density. Soil grain orientation may not be the same as *in-situ* conditions.
	Constant Head	Undisturbed Tube	K_v & K_H	ASTM D - 2434 Hvorslev, 1951	Good method for hydraulic conductivity measurement in any direction. Yields controlled test conditions and reliable results. Primary limitations are soil sample disturbance during collection. Measures hydraulic conductivity of one point in the effective aquifer system. Water must be distilled and de-aired.
	Constant Head	Remolded	K	Hvorslev, 1951 Lambe, 1951	Good for measuring low hydraulic conductivity soils such as clayey sands and clays. Limitations are soil disturbance, density, and soil grain orientation may not be the same as *in-situ* conditions.
Field Auger Hole (uncased)	Variable Head (pump out)	Slug	K_H	Boast & Kirkham, 1971	a.k.a. Auger Hole Method. Typical problems are found in keeping the test hole open (especially with increasing test depth). Good for measuring hydraulic conductivity close to ground surface. Applicable for tests below groundwater level only.

TABLE 3-3. *Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity (continued)*

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Field Auger Hole (uncased)	Variable Head	Slug (pump in)	$\frac{K}{b}$	SFWMD, 1987	Not a good equation for calculating true hydraulic conductivity value. The driving force for flow is only head and not hydraulic gradient. The calculated value is an approximation of leakance rather than hydraulic conductivity This method is not recommended to be used to measure hydraulic conductivity. Where applicable (South Florida), this method could be used as an empirical value to design retention ponds.
	Variable Head	Slug (pump out)	K_H	Van Bavel & Kirkham, 1948	Good for hydraulic conductivity derminations at shallow depths. Must pump from hole several times to develop soil pores along borehole sides. Limitations are the disturbance of open hole and maintaining the hole from caving.
	Variable Head	Slug (pump out)	K_H	U.S. Navy, 1974	Good for hydraulic conductivity measurements at shallow depths below groundwater. Not applicable in stratified soils. Difficult to maintain open hole with exact dimensions.
	Variable Head	Slug (pump in or out)	K_H	Hvorslev, 1951	a.k.a. Time Lag. Good for obtaining composite K_H value. Best accuracy when fully penetrating permeable soils. Limitations are the soil disturbance, developing of open hole and maintaining uniform open hole.
	Constant Head	Pump (Const Q)	$\frac{K}{b}$	SFWMD, 1988	Not a good equation for calcuating true hydraulic conductivity value. The driving force for flow is only head and not hydraulic gradient. The calculated value is an approximation of leakance rather than hydraulic conductivity.

TABLE 3-3. *Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity (continued)*

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Field Cased Hole	Variable Head (open hole below casing)	Slug (pump out)	K_m	Hvorslev	a.k.a. Time Lag. Good for measuring K_H at relatively shallow depths and K_v in permeable soils underlying restrictive or confining soils. Difficult to maintain dimensions of open hole below casing. Generally, difficult to duplicate test results.
	Variable Head	Slug (pump in or out)	K_v (casing flush with bottom of hole) K_H (open hole below casing)	U.S. Navy, 1974	If casing terminates in soil (no open hole) use to measure K_v in anisotorpic soils. If open hole section is used below casing then method measures K_H. Can also be used to measure K_v below restrictive or confining soils. Applied below groundwater only. Difficult to maintain dimensions of open hole below casing. Generally difficult to duplicate test results.
	Variable Head	Slug (pump out)	K_v	Frevert & Kirkham, 1948	a.k.a. Tube Method. Casing must fit the borehole tightly to prevent short circuiting. Also clogging of bottom of casing tends to lower calculated hydraulic conductivity value. Applied below groundwater only. Difficult to maintain dimensions of open hole below casing. Generally, difficult to duplicate test results.
	Constant Head	Pump (Const Head)	K_H	U.S.B.R., 1973 Designation E-18	a.k.a. Packer Test. Can be used above or below groundwater. Good for measuring K_H in consolidated material. Care must be taken to ensure proper sealing of casing annulas.

TABLE 3-3. *Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity (continued)*

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Field					
Cased Hole	Constant Head	Pump (Const Head)	K_m (below groundwater) K_v (above groundwater)	U.S.B.R., 1973 Designation E-18	a.k.a. open end test. Good for measuring K_v above or below groundwater level in pervious strata. Calculated permeability values are reduced by clogging in bottom of casing. Difficult to clean inside casing exactly to the bottom of casing. Difficult to duplicate test results.
	Constant Head	Pump (Const Head)	K_H	U.S.B.R., 1974 Designation E-19	a.k.a. well permeameter method. Good for measuring K_v above groundwater level.
Piezometer	Variable Head	Slug (pump in)	K_H (below groundwater)	Hvorslev, 1951	a.k.a. Time Lag. Good for measuring K_H in unconfined aquifers at any depth. Also good for calculating K_H in permeable soils underlying restrictive or confining soils. Limitations are soil disturbance during piezometer installation. Very important to develop piezometers prior to testing to minimize soil disturbance effects.
	Variable Head	Slug (pump in or out)	K_H	U.S. Navy, 1974	Good for measuring K_H at greater depths in unconfined aquifers. Also good for measuring K_H below restrictive soils. Installation into clayey soils can result in considerable soil disturbance and smearing of borehole surface with resulting calculations of lower hydraulic conductivity. Very important to develop piezometers prior to testing.

TABLE 3-3. ***Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity (continued)***

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Field					
Piezometer	Variable Head	Slug (pump out)	K_H	Frevert & Kirkham, 1948 Youngs, 1968	a.k.a. Piezometer Method. Good for measuring K_H in anisotropic soils. Limitations are soil disturbance during piezometer installation. Very important to develop piezometers prior to testing to minimize soil disturbance effects.
	Variable Head	Slug (pump out)	K_H	Bouwer & Rice, 1976	a.k.a. Slug Test Method. Good for shallow permeability measurements. Limitations are soil disturbance during piezometer installation. Very imporant to develop piezometers prior to testing to minimize soil disturbance effects.
	Variable Head	Pump (Const. Q out)	K_H	Theis, 1935	a.k.a. Recovery Method. Good method of measuring in-situ permeability. Requires pumping (development) of piezometer prior to running test. Usually used after completion of pump test. Poor or sub-standard installation is the greatest factor in producing erroneous results.
	Constant Head	Pump (Const. Q in)	K_H	Hvorslev, 1951	a.k.a. Time Lag. Good for measuring K_H in unconfined aquifers at any depth. Also good for calculating K_H in permeable soils underlying restrictire or confining soils.

TABLE 3-3. *Summary of Field and Laboratory Methods to Measure the Hydraulic Conductivity (continued)*

Methods of Analysis	Head Condition	Sample Type	Measurement	Reference(s)	Applicability and Limitations
Field					
Pump Tests Unconfined Aquifer	Variable Head	Pump (Const. Q out)	K_H	Lohman, 1972	a.k.a. Delayed yield. Can be pumped for relatively short periods (6 to 8 hours). Measures K_H average for effective aquifer thickness (anisotropy) depending on screen interval. Not as sensitive to well installation soil disturbance. The shape of the drawdown measured in the observation piezometer is used to calculate hyudraulic conductivity. This is perhaps the best method of measuring average K_H in unconfined aquifers.
Semi-Confined Aquifer	Variable Head	Pump (Const. Q out)	K_H and $\frac{K}{b}$	Lohman, 1972 Thies	Good for measuirng K_H below restrictive or confining soils. Also provides estimate of K_v through restrictive soil. This is perhaps the best method of measuring average K_H in semi-confined, leaky aquifers.
Tracers or Dyes	N/A	Slug (pump in)	K_H	Direct Calculation Cedergren, 1977	Takes long time to conduct test (Seepage velocity usually small). Observation well must be downgradient. Not practical for normal applications.
Double Ring Infiltrometer	Constant Head	Pump (Const. Q in)	K_v	ASTM D 3385 Johnson, 1963	Provides good estimate of initial infiltration at the tested surface level. Can be used to estimate *in-situ* K_v (unsaturated infiltration). Results do not apply to infiltration estimates during saturated flow. The tested surface should be compacted to the expected post-construction condition.

a.k.a.- also known as
K- hydraulic conductivity, L/T (non-directional)
K_m- Composite hydraulic conductivity (mean hydraulic conductivity), L/T
$\frac{K}{b}$- Leakance, L^2/t

testing. The sample can be analyzed using either a falling head or a constant head method (ASTM D-2434) in a laboratory permeameter. A variety of laboratory permeameters are commercially available. However, the most effective laboratory permeameter for undisturbed sandy soil samples is the type that does not require extraction of the soil samples from the Shelby tube. In such a permeameter, the Shelby tube itself is inserted into the permeameter (without soil sample extraction) and the hydraulic conductivity test is conducted. Such a permeameter was designed by Jammal & Associates, Inc. and has been successfully used to measure hydraulic conductivity of loose, sand soil samples. A schematic of this permeameter (not requiring soil sample extraction) is presented on **Figure 3-1**.

The second method of laboratory hydraulic conductivity measurement involves remolding a disturbed soil sample by compacting it to an estimated in-place density and then placing the sample in either a regular laboratory permeameter or a triaxial shear machine. The triaxial shear machine is generally used for low hydraulic conductivity soils such as silts, clayey sands and clays. This is primarily due to the ability of a triaxial shear machine to induce a high hydraulic head across a soil sample.

Regardless of the equipment used to measure hydraulic conductivity, the falling head test equation can be expressed as:

$$K = \frac{aL}{At} \ln \frac{h_f}{h_i} \qquad \textbf{(3-9)}$$

Where:

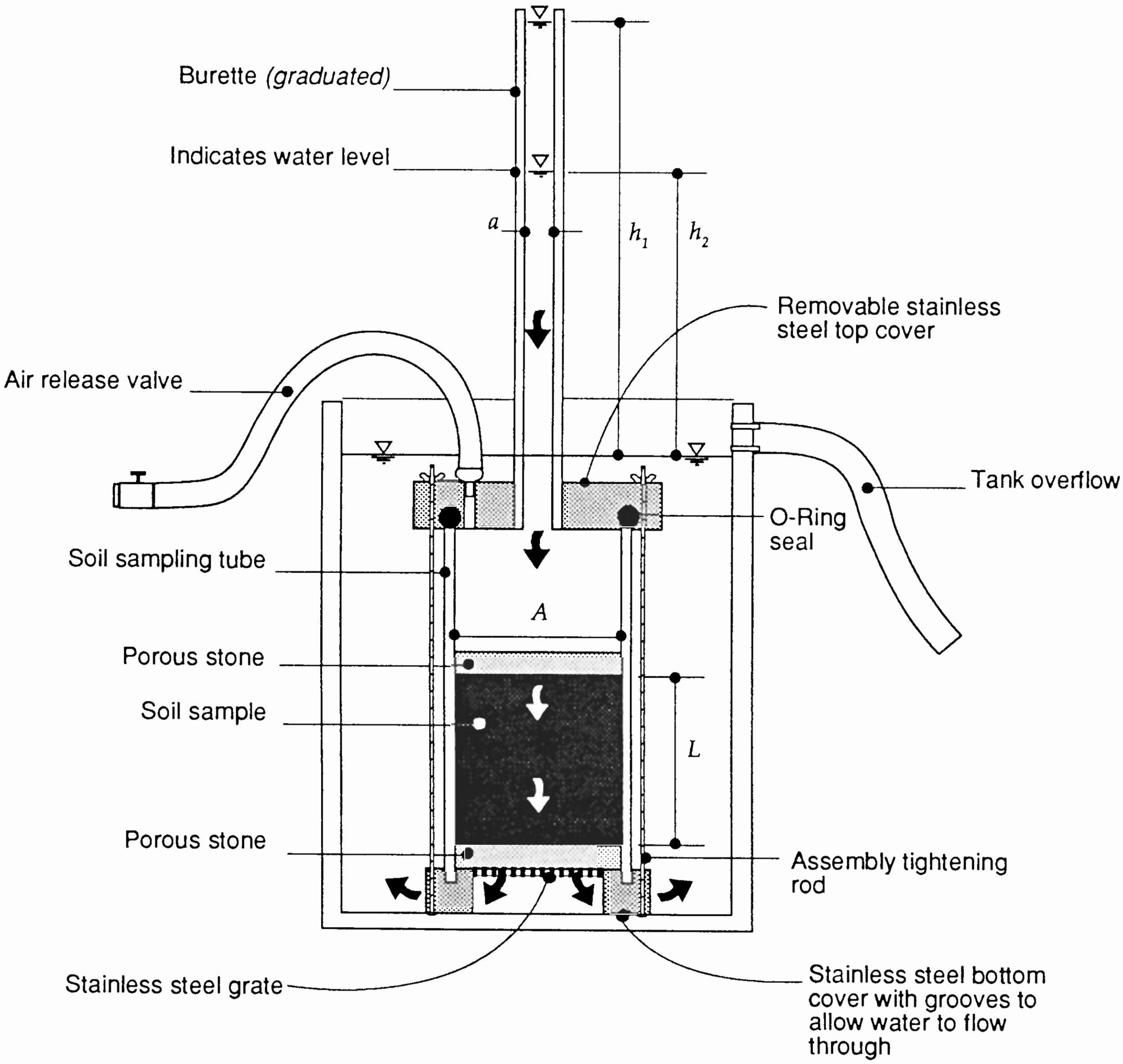
a = area of the stand pipe
L = length of the soil specimen
A = cross sectional area of the sample
h_i and h_f = initial and final head difference through the sample
t = difference in time from initial head (h_i) to final head (h_f) measurement

For the constant head method, a constant hydraulic gradient is applied through the sample and the discharge through the sample is measured. The equation for hydraulic conductivity can be written as:

$$K = \frac{VL}{Aht} \qquad \textbf{(3-10)}$$

Where:

V = volume measured after flow through the soil
L = length of the soil specimen
A = cross sectional area of the sample
h = head difference applied through the sample
t = difference in time from initial measurement to the final measurement of V



Hydraulic Conductivity Testing Equipment

(By Jammal & Associates, Inc.)

JAMMAL & ASSOCIATES, INC.

Based on our experience of hydraulic conductivity testing, we recommend that a falling head laboratory permeameter be used with undisturbed sand soil samples (Shelby tubes). For well-drained sand soils, only permeameters that do not require sample extraction should be used. Laboratory test methods, when applied to undisturbed soil samples, generally provide very consistent and reasonable results for soil samples collected at shallow depths above the groundwater table. For deeper deposits of saturated sands, it is difficult to obtain an undisturbed soil sample and, therefore, this method of hydraulic conductivity measurement becomes ineffective.

Field Methods

There are generally three types of field hydraulic conductivity tests, namely:

1. Auger hole or tube tests
2. Piezometer tests
3. Pumping tests

Auger hole or cased borehole tests generally involve drilling an auger hole to the desired depth (cased or uncased) and performing either a slug test (falling head) or a constant head test. Disturbance of auger hole walls and setting the casing at a proper depth with a good seal around the casing are the major concerns for these type of tests. Piezometer tests usually involve drilling and installing a piezometer in the drilled hole. A slug test (variable head) or a constant head test can be used to measure the hydraulic conductivity.

For the slug test, a slug of water is either added or pumped from the piezometer, the rate of water level recovery in the well is measured and the hydraulic conductivity calculated.

For the constant head test, a constant water level is maintained in the casing and the pumping rate or water recharge rate is recorded. Appropriate equations are then used to calculate hydraulic conductivity. Proper installation and development of the piezometer play a key role in the accuracy of the hydraulic conductivity measurement in piezometers.

Steady state or unsteady state pumping tests involve installing a minimum of two piezometers at some measured distance apart, one piezometer is pumped and the drawdown is measured in the other (observation) piezometer. To enhance the accuracy of this method, a second or third observation piezometer can also be installed. This method is less dependent on installation and development techniques than the piezometer hydraulic conductivity methods. In the pump test method, the shape of the drawdown curve and the magnitude of drawdown at the observation well are a function of hydraulic conductivity and the pumping rate at the pumping well.

Other field methods of permeability measurement include tracer studies and double ring infiltrometer tests. While *in-situ* tracer or dye studies can yield accurate permeability measurement, the length of the test is usually prohibitive. Even in transmissive aquifers the test can take as long as two to six months to detect the tracer in a downgradient observation well. Double ring infiltrometer tests are typically used to estimate soil infiltration or runoff potential. However, the limiting or ultimate infiltration (I_c) measured during a double ring infiltrometer test approximates the unsaturated vertical hydraulic conductivity, which can be 10 to 50 times the saturated infiltration rates that occur in retention ponds constructed in high groundwater table conditions.

Auger Hole and Piezometer Tests

Prior to conducting a slug test in an auger hole or a piezometer, the stratigraphy at the test location should be determined by drilling a soil boring. If it is desired to determine the hydraulic conductivity of the soil at or above the water table, water must be added to the piezometer instead of being pumped out of the piezometer. Two methods of analyzing permeability under these conditions have been developed by the U.S. Bureau of Reclamation (1973 and 1974). These test methods are referred to as the open-end and packer tests.

The open-end test (U.S.B.R. designation E-18) consists of installing a casing in the ground to a desired depth, carefully cleaning out the soil from the casing and adding water to the casing at a known rate to maintain a constant water level. The required data for analysis includes the hydraulic head maintained under a constant rate of flow, the diameter of the casing, and the average rate of recharge under saturated conditions (**Figure 3-2**). Hydraulic conductivity is calculated from the following equation:

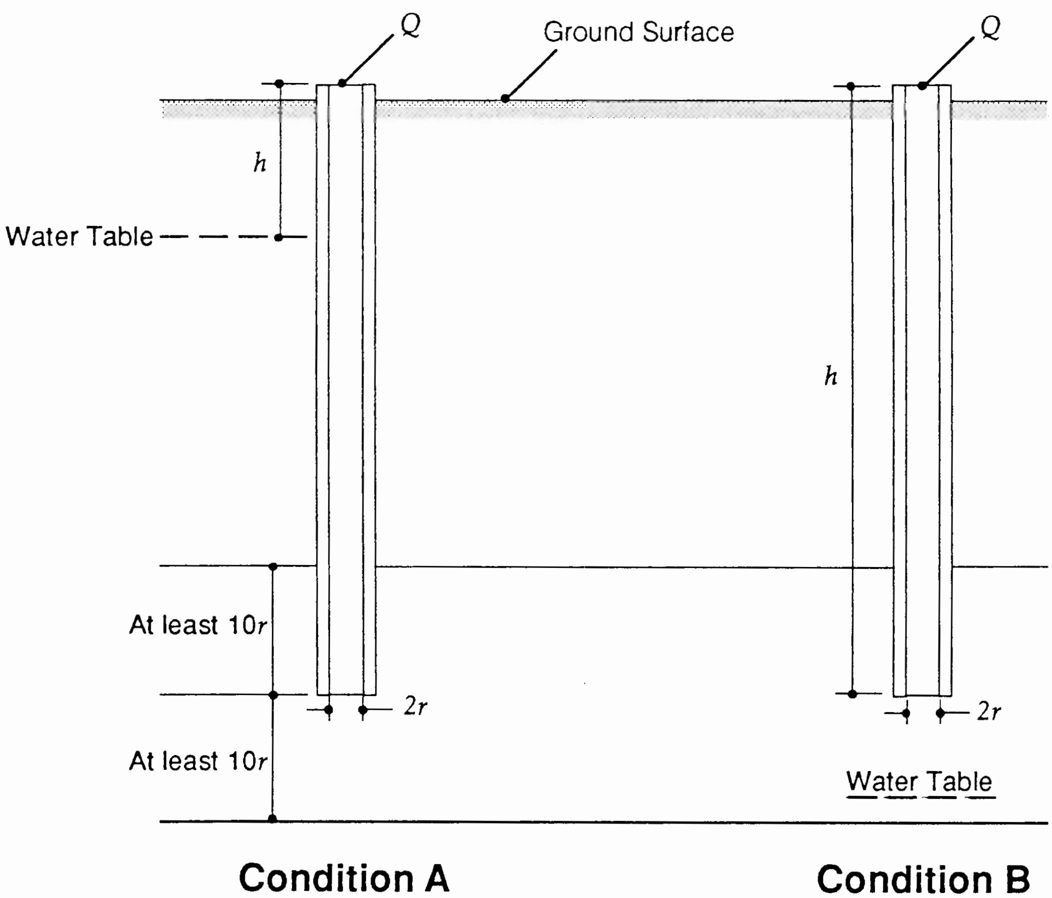
$$K = \frac{Q}{5.5rh} \qquad \textbf{(3-11)}$$

Where:

Q = flow rate at saturation
r = radius of the casing
h = the hydraulic head differential **(Figure 3-2)**

Any consistent units may be used in computing hydraulic conductivity.

When performing packer tests, it is customary to drill the hole, remove the core barrel or other tool, install the packer, perform the test, remove the packer, drill the hole deeper, re-seat the packer and test. This procedure is repeated until the desired depth has been achieved. In general, this test is conducted in consolidated material or bedrock. Considerable difficulty will be encountered in installing the packers and



$$k = \frac{Q}{5.5r\,h}$$

$$h = h(gravity) + h(pressure)$$

An Open-End Pipe Hydraulic Conductivity Test

(After U.S. Bureau of Reclamation, 1973)

JAMMAL & ASSOCIATES, INC.

Figure 3-2

conducting an acceptable test in unconsolidated sandy materials.Hydraulic conductivity is calculated from the following formulas:

$$K = \frac{q}{2\pi Lh} \ln \frac{L}{r}, \text{ for } L \geq 10r \qquad \textbf{(3-12)}$$

$$K = \frac{q}{2\pi Lh} \sinh^{-1} \frac{L}{2r}, \text{ for } 10r > L \geq r \qquad \textbf{(3-13)}$$

Where:

q = constant rate of flow into the test hole
L = length of the section of the hole being tested
r = radius of the hole tested
h = differential head

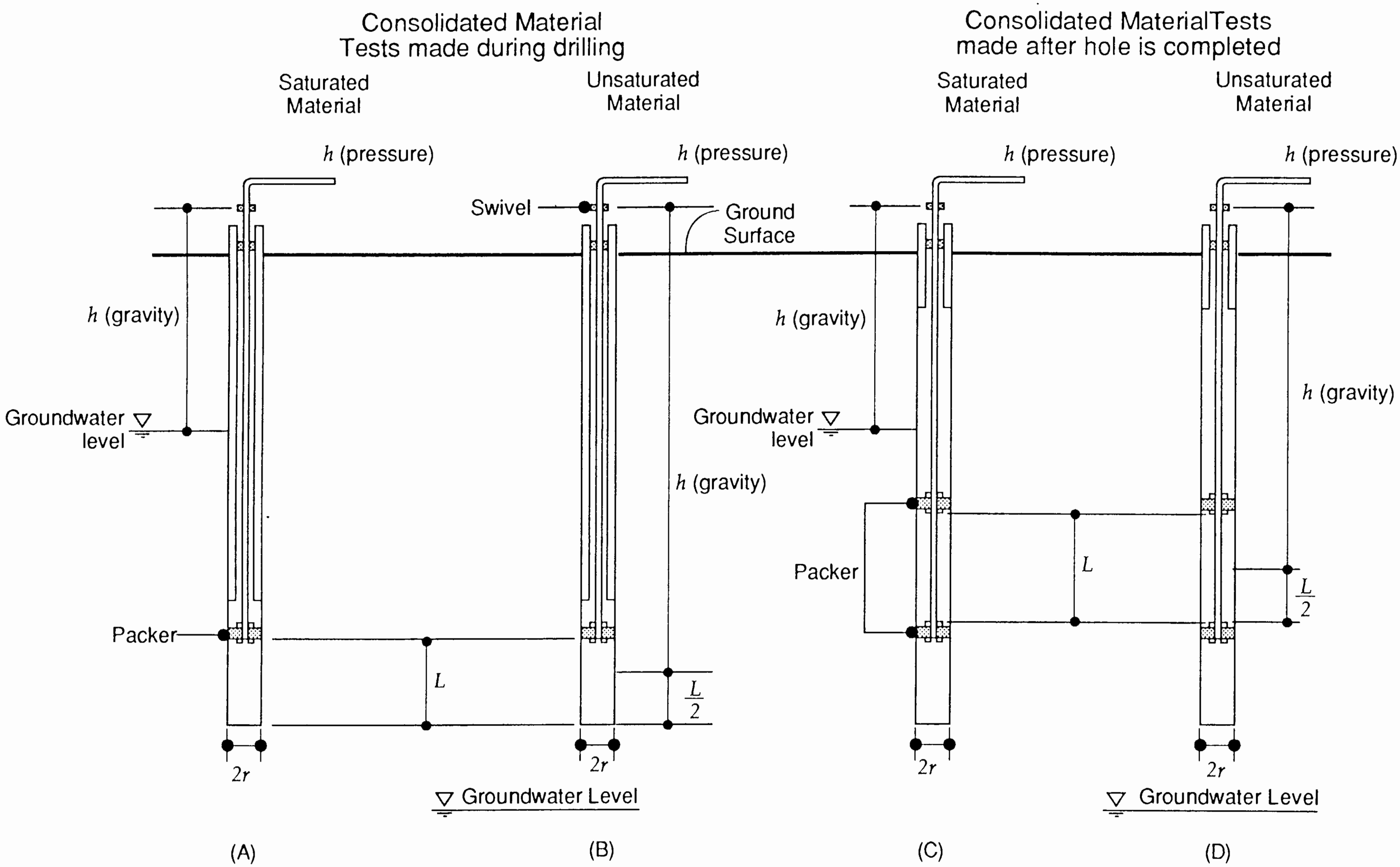
A diagram explaining the terms in these equations is presented on **Figure 3-3**.

Several other steady and non-steady flow methods are used in determining *in-situ* hydraulic conductivity. The Bureau of Reclamation (1974) also recommends a method by which the flow of water out of cased or uncased auger boreholes is measured and hydraulic conductivity is calculated. This method is known as the well permeameter method. **Figure 3-4** presents the schematics and the formulas for calculating hydraulic conductivity using the Bureau of Reclamation constant head well permeameter method. A discharge time curve and sample calculation is presented on **Figure 3-5**.

The U.S. Department of the Navy, Naval Facilities Engineering Command (1974) has standard methods of performing variable head tests to estimate the *in-situ* hydraulic conductivity by means of cased and uncased holes. **Figure 3-6** summarizes the methods of calculating hydraulic conductivity using the U.S. Department of the Navy methods.

Bouwer (1978) presented an auger hole method for field hydraulic conductivity measurement. The diagram for the test method, the hydraulic equation and the associated dimensionless parameters table are presented on **Figure 3-7**.

Hvorslev (1951) conducted studies for the U.S. Corp of Engineers, Waterways Experiment Station for measuring hydraulic conductivity from borings, cased boreholes and piezometers. Whenever a boring is drilled or a piezometer is installed, the initial water level (hydrostatic pressure) measured in the borehole/piezometer seldom reflects the true ambient water level. The groundwater must flow to or from the borehole or piezometer until the measured water level matches the ambient level. The flow of water to or from the borehole/piezometer will occur until the hydrostatic pressure gradient approaches zero and the time in which the flow occurs



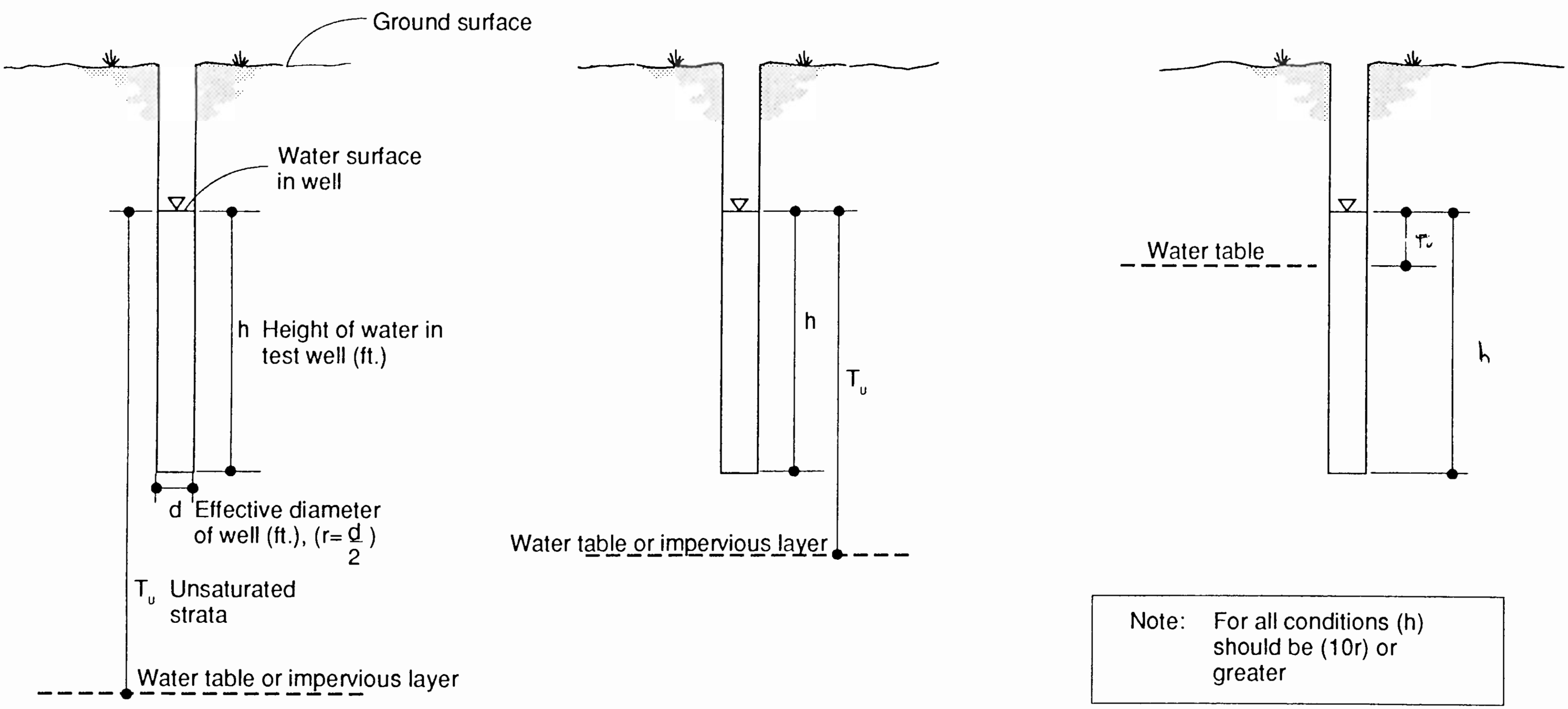
h=h (gravity) + h (pressure)

The Packer Hydraulic Conductivity Test for Soil Permeability

(U.S. Bureau of Reclamation, 1973)

JAMMAL & ASSOCIATES, INC.

Figure 3-3



Condition 1

$T_u > 3h$

$$k_{20}=525{,}600\,\frac{\left[\sinh^{-1}\left(\frac{h}{r}\right)-1\right]\frac{Q}{2\pi}}{h^2}\left(\frac{\mu_T}{\mu_{20}}\right)$$

$\left(\frac{\mu_T}{\mu_{20}}\right)$= Water viscosity correction factor

Condition 2

$h \le T_u \le 3h$

$$k_{20}=525{,}600\,\frac{\log_e\left(\frac{h}{r}\right)\frac{Q}{2\pi}}{h^2\left[\frac{1}{6}+\frac{1}{3}\left(\frac{h}{T_u}\right)^{-1}\right]}\left(\frac{\mu_T}{\mu_{20}}\right)$$

Condition 3

$T_u < h$

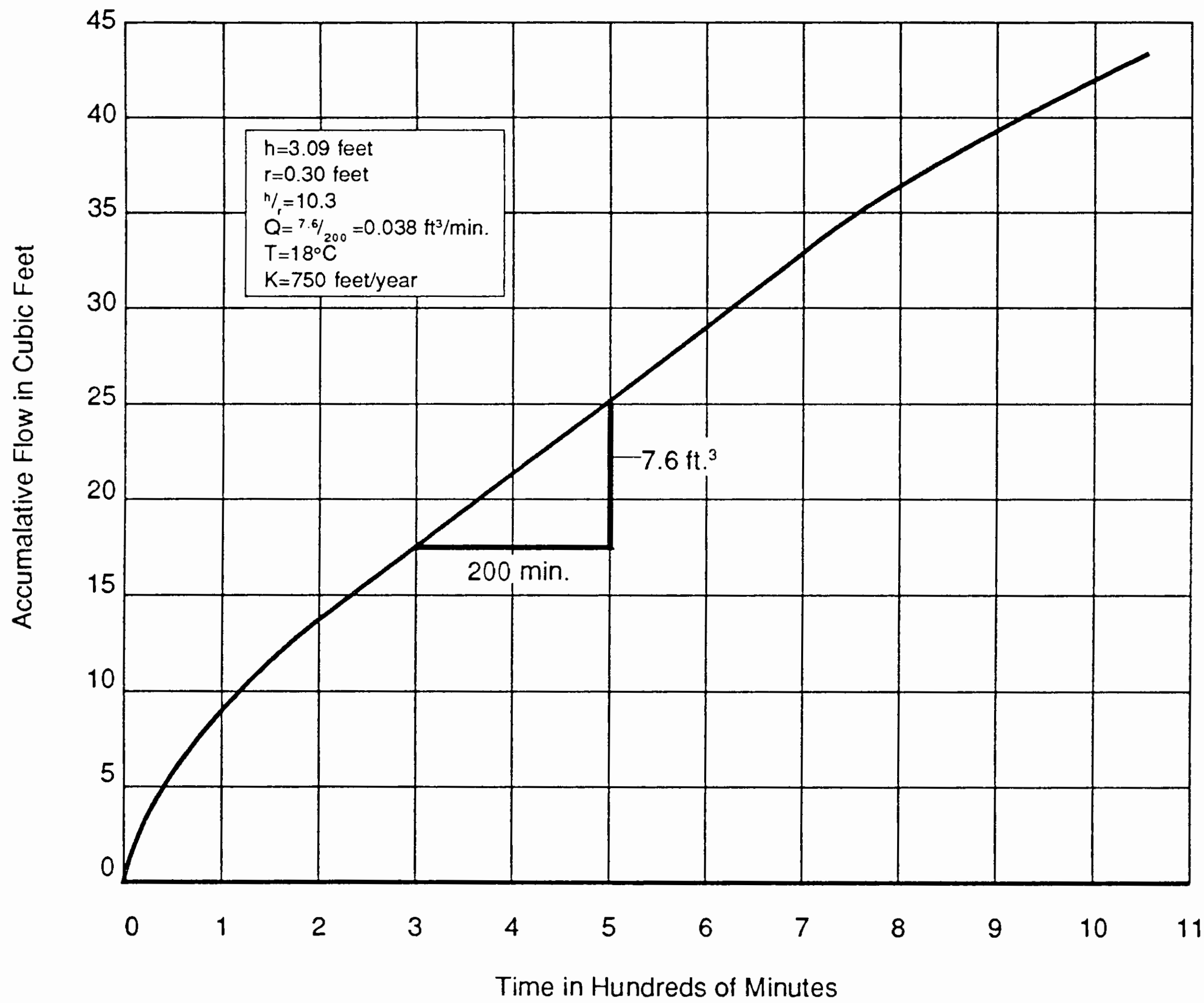
$$k_{20}=525{,}600\,\frac{\log_e\left(\frac{h}{r}\right)\frac{Q}{2\pi}}{h^2\left[\left(\frac{h}{T_u}\right)^{-1}-\frac{1}{2}\left(\frac{h}{T_u}\right)^{-2}\right]}\left(\frac{\mu_T}{\mu_{20}}\right)$$

Well Permeameter Tests

(From U.S. Bureau of Reclamation, 1974)

JAMMAL & ASSOCIATES, INC.

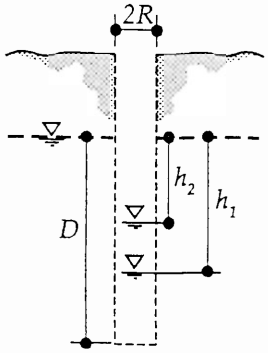
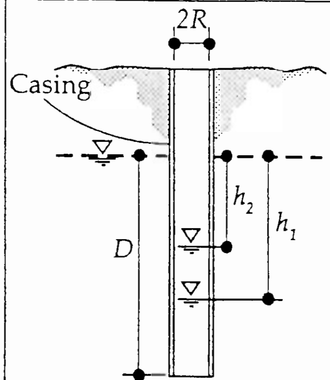
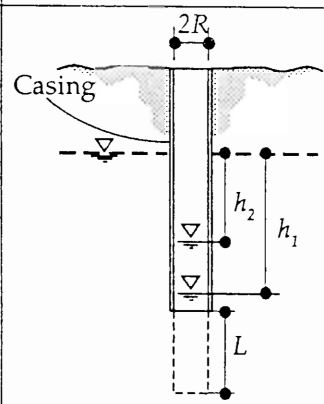
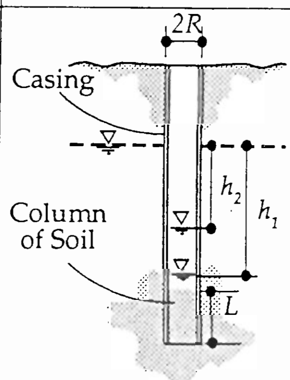
Figure 3-4



Example of Discharge Time Curve for Well Permeameter Test

(After U.S. Bureau of Reclamation,1974)

JAMMAL & ASSOCIATES, INC.

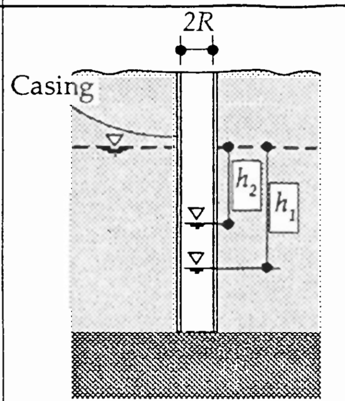
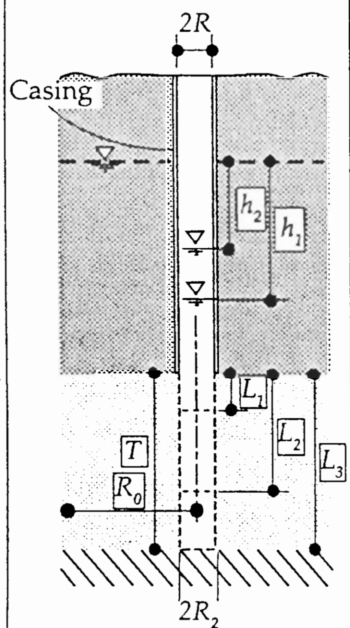
	Condition	Diagram	Shape Factor, F	Permeability, k by variable head test	Applicability
Observation well or piezometer in saturated isotropic stratum of infinite depth	a) Uncased hole		$F = 16\pi DSR$	$k_h = \frac{R}{16DS} \times \frac{(h_2-h_1)}{(t_2-t_1)}$ for $\frac{D}{R} < 50$	Simplest method for permeability determination; not applicable in stratified soils.
	b) Cased hole, soil flush with bottom		$F = \frac{11R}{2}$	$k_v = \frac{2\pi R}{11(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$ for $6 \text{ in.} \le D \le 60 \text{ in.}$	Used for permeability determination at shallow depths below the water table; may yield unreliable results in falling head test with silting of bottom of hole.
	c) Cased hole, uncased or perforated extension of length L		$F = \frac{2\pi L}{\ln(L/R)}$	$k_h = \frac{R_1^2}{2L(t_2-t_1)} \ln\left(\frac{L}{R_2}\right) \ln\left(\frac{h_1}{h_2}\right)$ for $\frac{L}{R} > 8$	Used for permeability determinations at greater depths below the water table. R_1 = Radius of casing R_2 = Radius of open hole
	d) Cased hole, column of soil inside casing to height L		$F = \frac{11\pi R^2}{2\pi R^2 + 11L}$	$k_v = \frac{2\pi R + 11L}{11(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$	Principal use is for permeability in vertical direction in anisotropic soils.

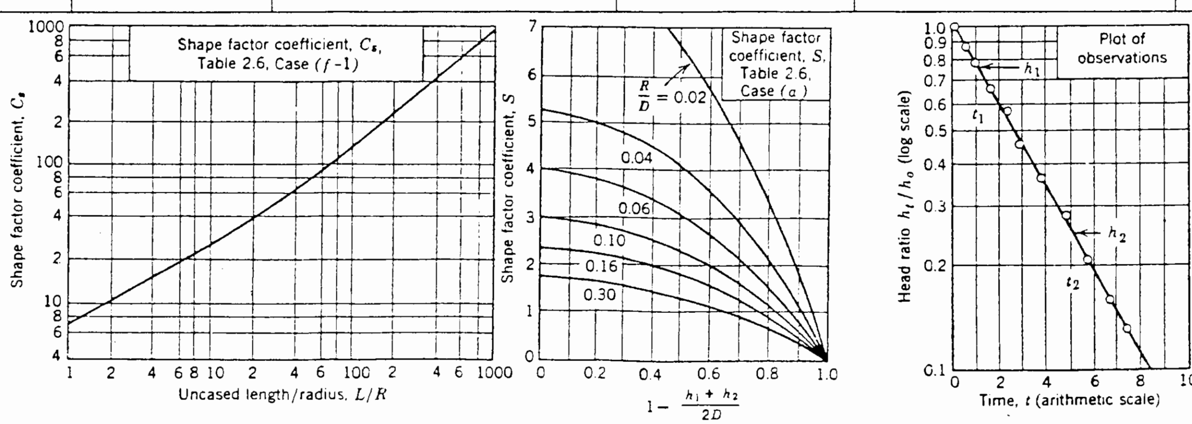
JAMMAL & ASSOCIATES, INC.

Shape Factors for Computation of Permeability from Variable Head Tests

(From U.S. Department of the Navy, Naval Facilities Engineering Command, 1974)

Figure 3-6

	Condition	Diagram	Shape Factor, F	Permeability, k by variable head test	Applicability
Observation well or piezometer in aquifer with impervious upper layer	e) Cased hole, opening flush with upper boundary of aquifer of infinite depth		$F = 4R$	$k_v = \frac{\pi R}{4(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$	Used for permeability determination when surface impervious layer is relatively thin; may yield unreliable results in falling head test with silting of bottom of hole.
	f) Cased hole, uncased or perforated extension into aquifer of finite thickness: 1) $\frac{L_1}{T} \leq 0.20$		1) $F = C_s R_1$	$k_h = \frac{\pi R_1}{C_s(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$	Used for permeability determination at depths greater than about 5 feet.
	2) $0.2 < \frac{L_2}{T} < 0.85$		2) $F = \frac{2\pi L_2}{\ln(L_2/R_2)}$	$k_h = \frac{R_1^2 \ln(L_2/R_2)}{2L_2(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$ for $\frac{L_2}{R_2} > 8$	Used for permeability determination at greater depths and for fine grained soils using porous intake point of piezometer.
	3) $\frac{L_1}{T} = 1.00$ Note: R_0 equals effective radius to source at constant head		3) $F = \frac{2\pi L_3}{\ln(R_0/R_2)}$	$k_h = \frac{R_1^2 \ln(R_0/R_2)}{2L_3(t_2-t_1)} \ln\left(\frac{h_1}{h_2}\right)$	Assume value of $\frac{R_0}{R} = 200$ for estimates unless observation wells are made to determine actual value of R_0

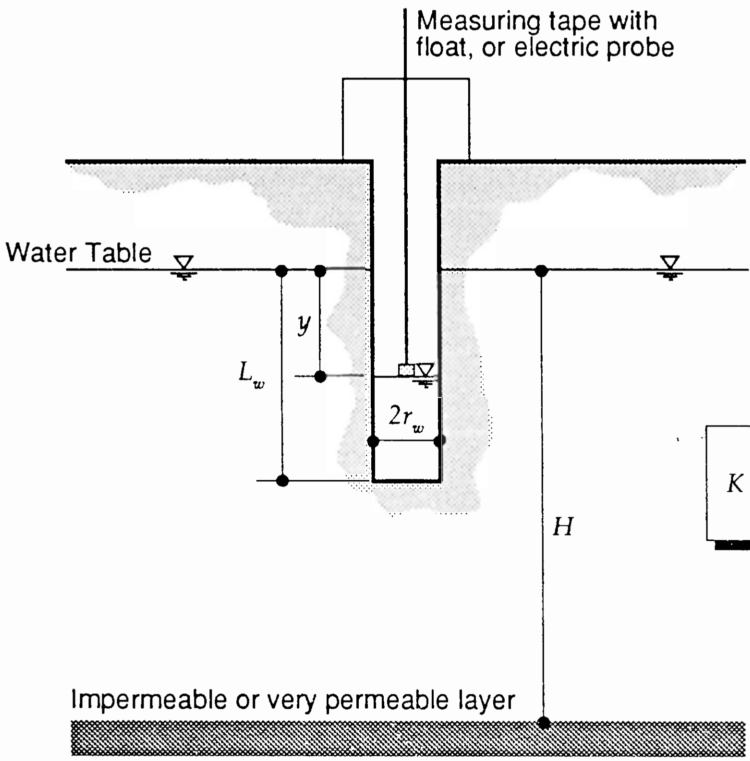


Shape Factors for Computation of Permeability from Variable Head Tests

(From U.S. Department of the Navy, Naval Facilities Engineering Command, 1974)

JAMMAL & ASSOCIATES, INC.

Figure 3-6 Continued



$$K = C_a \frac{(dy/dt)}{864}$$

.Geometry and Symbols for Auger Hole Method

		$(H - L_w)/L_w$ for impermeable layer								$H - L_w$	$(H - L_w)/L_w$ for infinetly permeable layer			
L_w/r_w	y/L_w	0	0.05	0.10	0.20	0.50	1	2	5	∞	5	2	1	0.5
1	1.00	447.00	423.00	404.00	375.00	323.00	286.00	264.00	255.00	254.00	252.00	241.00	213.00	166.00
	0.75	469.00	450.00	434.00	408.00	360.00	324.00	303.00	292.00	291.00	289.00	278.00	248.00	198.00
	0.50	555.00	537.00	522.00	497.00	449.00	411.00	386.00	380.00	379.00	377.00	359.00	324.00	264.00
2	1.00	186.00	176.00	167.00	154.00	134.00	123.00	118.00	116.00	115.00	115.00	113.00	106.00	91.00
	0.75	196.00	187.00	180.00	168.00	149.00	138.00	133.00	131.00	131.00	130.00	128.00	121.00	106.00
	0.50	234.00	225.00	218.00	207.00	138.00	175.00	169.00	167.00	167.00	166.00	164.00	156.00	139.00
5	1.00	51.90	48.60	46.20	42.80	38.70	36.90	36.10		35.80		35.50	34.60	32.40
	0.75	54.80	52.00	49.90	46.80	-	41.00	40.20		40.00		39.60	38.60	36.30
	0.50	66.10	63.40	61.30	58.10	53.90	51.90	51.00		50.70		50.30	49.20	46.60
10	1.00	18.10	16.90	16.10	15.10	14.10	13.60	13.40		13.40		13.30	13.10	12.60
	0.75	19.10	18.10	17.40	16.50	15.50	15.00	14.80		14.80		14.70	14.50	14.00
	0.50	23.30	22.30	21.50	20.60	19.50	19.00	18.80		18.70		18.60	18.40	17.80
20	1.00	5.91	5.53	5.30	5.06	4.81	4.70	4.66		4.64		4.62	4.58	4.46
	0.75	6.27	5.94	5.73	5.50	5.25	5.15	5.10		5.08		5.07	5.02	4.89
	0.50	7.67	7.34	7.12	6.88	6.60	6.48	6.43		6.41		6.39	6.34	6.19
50	1.00	1.25	1.18	1.14	1.11	1.07	1.05			1.04			1.03	1.02
	0.75	1.33	1.27	1.23	1.20	1.16	1.14			1.13			1.12	1.11
	0.50	1.64	1.57	1.54	1.50	1.46	1.44			1.43			1.42	1.39
100	1.00	0.37	0.35	0.34	0.34	0.33	0.32			0.32			0.32	0.31
	0.75	0.40	0.38	0.37	0.36	0.35	0.35			0.35			0.34	0.34
	0.50	0.49	0.47	0.46	0.45	0.44	0.44			0.44			0.43	0.43

Values of C_a for Auger Hole Underlain by Impermeable or infinetly permeable material

(Boast and Kirkham, 1971)

JAMMAL & ASSOCIATES, INC.

is referred to as "time lag". This time lag is related to the permeability of the soil and configuration of the piezometer/borehole. A basic differential equation for time lag can be written as follows:

$$\frac{dy}{z-y} = \frac{dt}{T} \qquad \textbf{(3-14)}$$

Where:

z = initial water level difference at time equals 0 (at the stop of pumping)
y = water level above the datum z at some time t
T = time lag

A diagram presenting these parameters is represented on **Figure 3-8**.

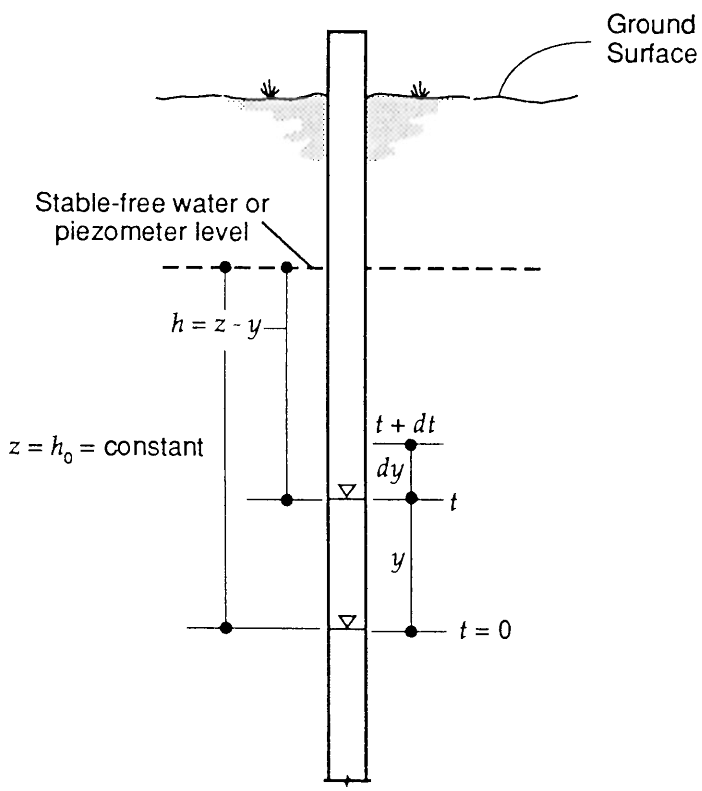
In the field, the basic time lag is determined by raising or depressing the head in the piezometer/borehole and recording the head at a number of time intervals. A plot is then made with time on an arithmetic scale and the head ratio (h/h_o) on a log scale. The basic time lag is the time at which the head ratio equals 0.37. The equalization ratio is defined as ($1-h/h_o$); thus when the head ratio is 0.37 the equalization ratio is 0.63. An equalization ratio of 0.90, which corresponds to a time lag of 2.3 x the basic time lag is considered by Hvorslev to be adequate for many practical purposes. The basic time lag T corresponds to $H = 0.37h_o$: that is,

$$\frac{\ln(h_o)}{h} = \frac{\ln(h_o)}{0.37h_o} = \ln(2.7) = 1.0 \qquad \textbf{(3-15)}$$

Figure 3-9 is a summary of formulas compiled by Hvorslev (1951) for the determination of hydraulic conductivity by constant head, variable head and basic time lag tests.

Based on our experience in field hydraulic conductivity measurements, we conclude that all these test methods are generally reasonable and provide reasonably accurate results, provided:

1. The open borehole or piezometer is constructed without significant disturbance to the surrounding soil.
2. The open borehole does not collapse and/or the walls of the borehole do not cave-in during the test.
3. If casing is installed into a borehole, the soil material in the casing needs to be removed to the exact depth (bottom of the casing or an exact known distance below the casing).
4. If a piezometer is installed, it should be sufficiently developed to mitigate the installation soil disturbance as much as possible.
5. All assumptions and conditions of the test method and equation restrictions are



Differential Equation

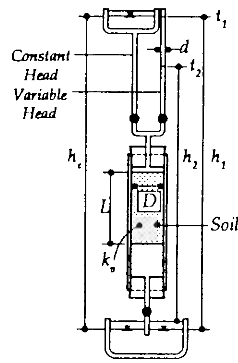
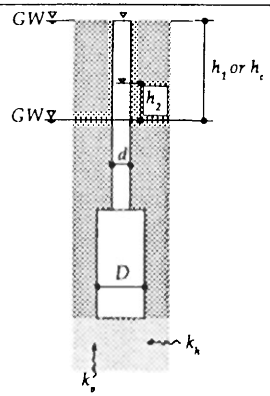
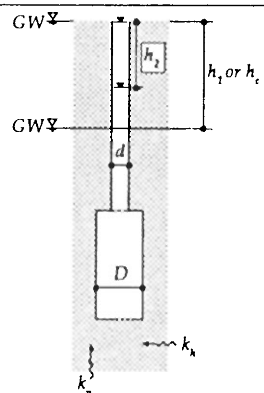
$$\frac{dy}{z-y} = \frac{dt}{T}$$

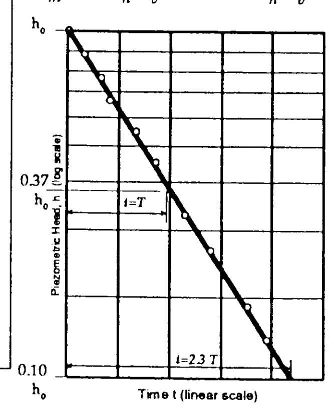
T = Basic time lag

General Conditions for Time Lag Determination

(After Hvorslev, 1951)

JAMMAL & ASSOCIATES, INC.

Condition	Diagram	Constant Head	Variable Head	Basic Time Lag	Notation
a) Laboratory Permeameter (consolidometer)		$k_v = \dfrac{4\,q\,L}{\pi\,D^2\,h_c}$	$k_v = \dfrac{d^2\,L}{D_2\,(t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ $k_v = \dfrac{L}{t_2 - t_1} \ln \dfrac{h_1}{h_2}$ *for d=D*	$k_v = \dfrac{d^2\,L}{D^2\,T}$ $k_v = \dfrac{L}{T}$ *for d=D*	D= Diameter intake sample, cm d= Diameter standpipe, cm L= Length intake sample, cm h_c= Constant piez. head, cm h_1= Piez. head for $t=t_1$, cm h_2= Piez. head for $t=t_2$, cm q= Flow of water, cm^3/sec t= Time, sec T= Basic time lag, sec k_v= Vert. perm. ground, cm/sec k_v'= Vert. perm. casing, cm/sec k_h= Horiz. perm. ground, cm/sec k_m= Mean coeff. perm., cm/sec m= Transformation ratio $k_m = \sqrt{k_h\,k_v}$ $m = \sqrt{k_h/k_v}$
b) Flush bottom at impervious boundary		$k_v = \dfrac{q}{2\,D\,h_c}$	$k_m = \dfrac{\pi\,d^2}{8\,D\,(t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ $k_m = \dfrac{\pi\,D}{8\,(t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ *for d=D*	$k_m = \dfrac{\pi\,d^2}{8\,D\,T}$ $k_m = \dfrac{\pi\,D}{8\,T}$ *for d=D*	
c) Flush bottom in uniform soil		$k_v = \dfrac{q}{2.75\,D\,h_c}$	$k_m = \dfrac{\pi\,d^2}{11\,D\,(t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ $k_m = \dfrac{\pi\,D}{11\,(t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ *for d=D*	$k_m = \dfrac{\pi\,d^2}{11\,D\,T}$ $k_m = \dfrac{\pi\,D}{11\,T}$ *for d=D*	



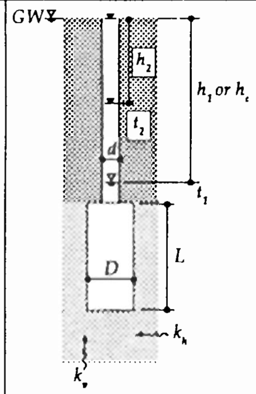
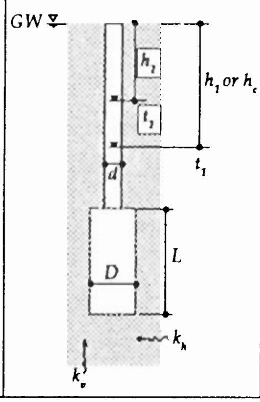
Determination Basic Time Lag T

Assumptions: Soil at intake, infinite depth and directional isotropy (k_v and k_h constant); no disturbance, segregation, swelling or consolidation of soil; no sedimentation or leakage; no air or gas in soil, well point, or pipe; hydraulic losses in pipes, well point or filter negligible. (After Hvorslev, U. S. Corps of Engineers, W.E.S., 1951)

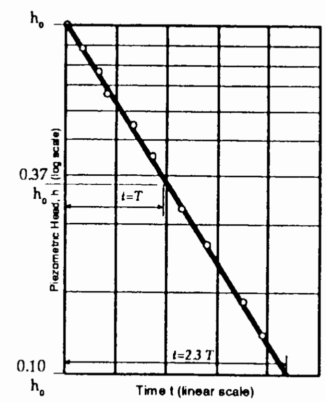
JAMMAL & ASSOCIATES, INC.

Formulas for Determination of Permeability

Figure 3-9

Condition	Diagram	Constant Head	Variable Head	Basic Time Lag	Notation
d) Well point filter at impervious boundary		$k_h=\dfrac{q \ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{2\pi L h_c}$	$k_h=\dfrac{d^2 \ln\left[\frac{2mL}{D}+\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8L(t_2-t_1)} \ln\dfrac{h_1}{h_2}$ $k_h=\dfrac{d^2 \ln\left(\frac{4mL}{D}\right)}{8L(t_2-t_1)} \ln\dfrac{h_1}{h_2}$ for $\dfrac{2mL}{D}>4$	$k_h=\dfrac{d_2 \ln\left[\frac{2mL}{D}\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8LT}$ $k_h=\dfrac{d^2 \ln\left(\frac{4mL}{D}\right)}{8LT}$ for $\dfrac{2mL}{D}>4$	D= Diameter intake sample, cm d= Diameter standpipe, cm L= Length intake sample, cm h_c= Constant piez. head, cm h_1= Piez. head for $t=t_1$, cm h_2= Piez. head for $t=t_2$, cm q= Flow of water, cm³/sec t= Time, sec T= Basic time lag, sec k_v= Vert. perm. ground, cm/sec k_v'= Vert. perm. casing, cm/sec k_h= Horiz. perm. ground, cm/sec k_m= Mean coeff. perm., cm/sec m= Transformation ratio $k_m=\sqrt{k_h k_v}$ $m=\sqrt{k_h/k_v}$
e) Well point filter in uniform soil		$k_h=\dfrac{q \ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{2\pi L h_c}$	$k_h=\dfrac{d^2 \ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{8L(t_2-t_1)} \ln\dfrac{h_1}{h_2}$ $k_h=\dfrac{d^2 \ln\left(\frac{2mL}{D}\right)}{8L(t_2-t_1)} \ln\dfrac{h_1}{h_2}$ for $\dfrac{mL}{D}>4$	$k_h=\dfrac{d_2 \ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{8LT}$ $k_h=\dfrac{d^2 \ln\left(\frac{mL}{D}\right)}{8LT}$ for $\dfrac{mL}{D}>4$	

Assumptions: Soil at intake, infinite depth and directional isotropy (k_v and k_h constant); no disturbance, segregation, swelling or consolidation of soil; no sedimentation or leakage; no air or gas in soil, well point, or pipe; hydraulic losses in pipes, well point or filter negligible. (After Hvorslev, U. S. Corps of Engineers, W.E.S., 1951)



Determination Basic Time Lag T

JAMMAL & ASSOCIATES, INC.

Formulas for Determination of Permeability

Figure 3-9 Continued

satisfied.

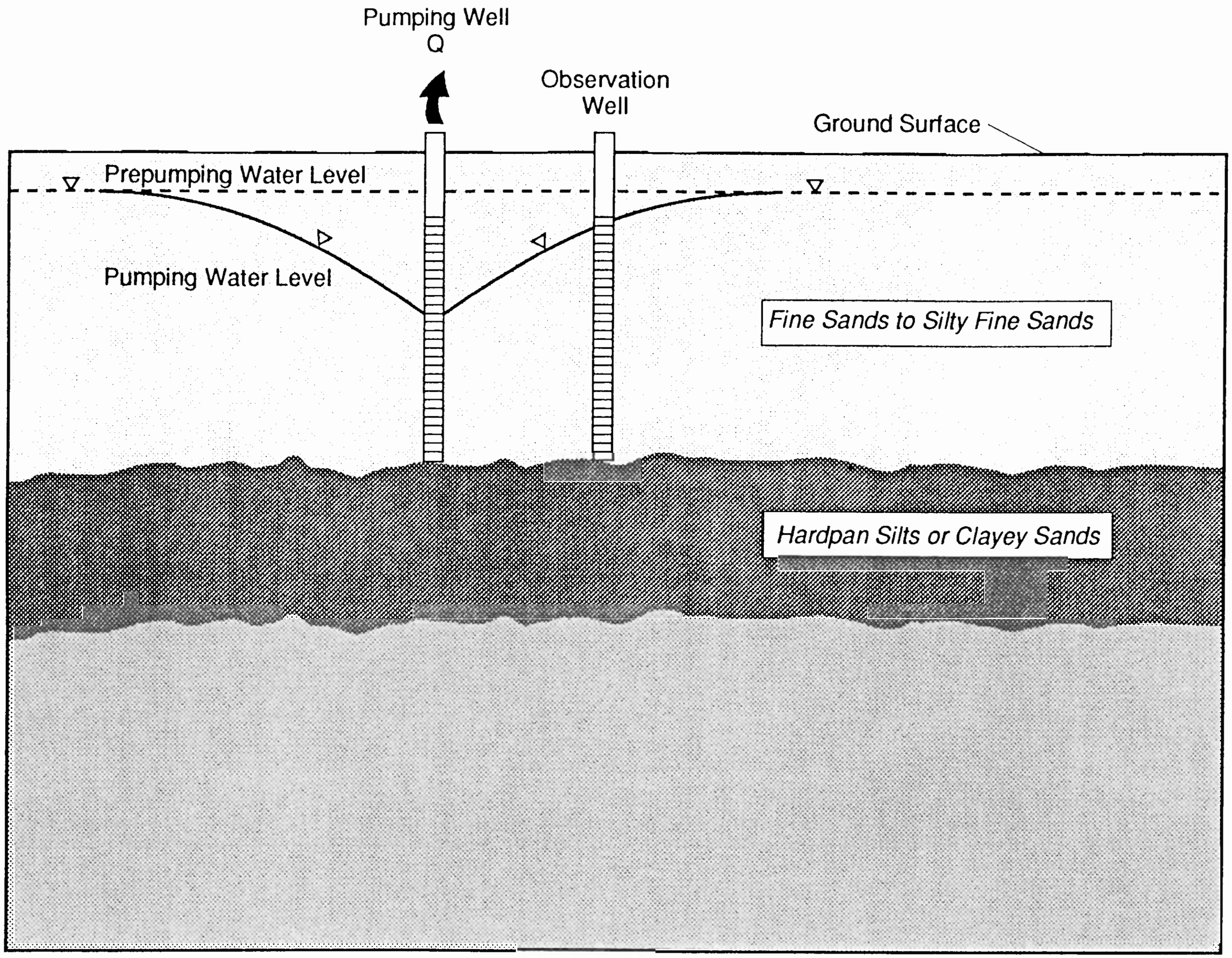
On the basis of our field hydraulic conductivity testing experience, we feel that open borehole permeability methods in well drained sandy soils are not appropriate and should not be used. Cased borehole methods generally provide reasonable results when used in measuring hydraulic conductivity of sandy soils above the groundwater table and at depths of 15 feet or less. Below the groundwater table and in deeper strata, only piezometer methods (properly installed and sufficiently developed) or pump tests provide reasonable results. In general, we find that piezometer tests yield lower permeability values as compared to pumping tests and laboratory methods. A summary of published laboratory and field methods for hydraulic conductivity measurement is presented in **Table 3-3**.

Pumping Tests

The third method of measuring permeability in the field is to conduct a pumping test. Both short term and long term pumping times can be used depending upon the aquifer type being tested, the pumping rate and distance between wells. In general, a pumping test consists of installing one pumping well and at least one observation well at some reasonable distance away from the pumping well **(Figure 3-10)**. The pumping well and observation well should be installed with the same characteristics (same depth and screen interval). Prior to initiating pumping, the static water level below the top of the casing should be measured in both wells. Then one well should be pumped at a high rate and the drawdown below the static water level measured in the observation well.

For shallow, unconfined aquifer pumping tests in sand aquifers, the yield is generally low and the groundwater typically mixes with air. Therefore, the volume of water pumped should be measured using calibrated containers (i.e., 55 gallon calibrated drums) and the time to fill each container or a fraction should be recorded. This will yield a better estimate of average pumping rate for the hydraulic conductivity calculation since the drawdown in the observation well is more a function of the total volume of water removed than the instantaneous flow rate. In highly transmissive aquifers where sufficiently high withdrawal rates are necessary to produce measureable drawdowns, other methods such as installation of an in-line flow meter or an orifice is more appropriate. Minimum pumping times to produce reliable results are 8 to 10 hours for short-term unconfined aquifer pumping tests. However, longer pumping periods are preferable. For shallow unconfined aquifers (sand aquifers), the wells are typically placed no further than 5 feet apart for short duration pumping tests and no greater than 10 feet for long duration pumping tests.

For unconfined aquifer short-term pump tests, very few methods are available to evaluate the data. One reliable method is the matchpoint method presented in



Pumping Test Conditions and Drawdown Profile in an Unconfined Aquifer

JAMMAL & ASSOCIATES, INC.

Figure 3-10

Lohman (1972). This method consists of plotting the drawdown versus time in the observation well on a log-log scale paper and superimposing a family of type curves developed by Boulton (1963). This family of type curves, developed by Boulton, is presented on **Figure 3-11**.

Methods to evaluate pumping test data in semi-confined leaky aquifers and confined aquifers are numerous and well documented (Theis 1935; Cooper and Jacob 1946; Hantush 1962 and Lohman 1972). Therefore, these methods of testing and analyses will not be presented herein.

Double-ring Infiltromenter Test

A popular method to estimate *in-situ* infiltration rate from stormwater retention ponds is the double-ring infiltrometer test (ASTM D-3385). This test involves the use of cylindrical devices in which an inner ring is placed within a larger outer ring. Typical diameters are 14 inches for the inner ring and 36 inches for the outer ring. Both rings are pushed or driven into the soil to a depth of 2 to 4 inches below grade. A constant water level is maintained inside both rings and the amount of water added to maintain this constant head within the inner ring is measured versus time. The infiltration rate is then plotted on a log scale versus time on an arithmetic scale. Infiltration at various times can be predicted using Horton's equation as follows:

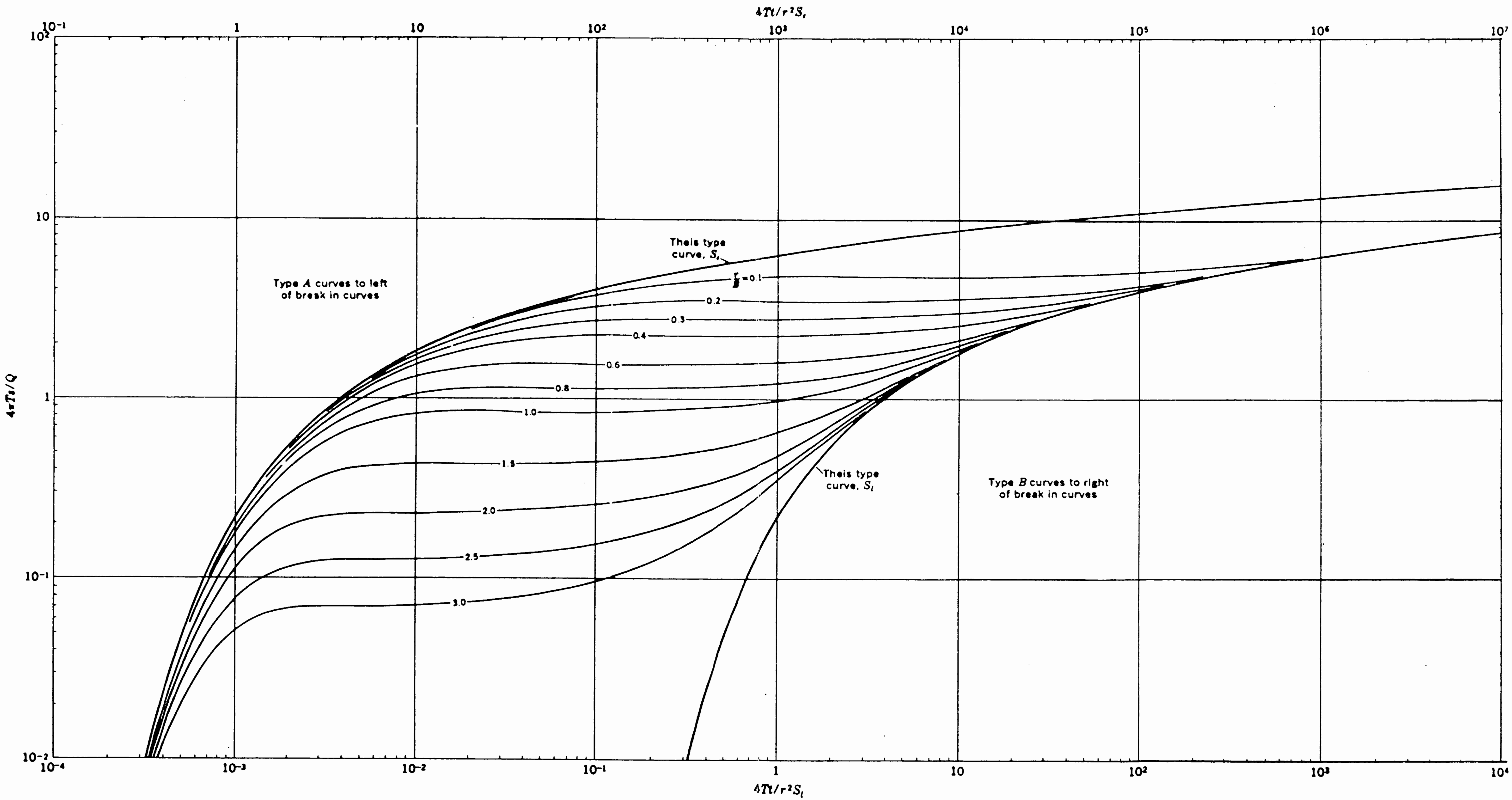
$$I_t = I_c + (I_o - I_c)e^{-kt} \qquad \textbf{(3-12)}$$

Where:

I_t = infiltration rate as a function of time
I_c = final or ultimate infiltration rate
I_o = initial infiltration rate
k = recession constant
t = time

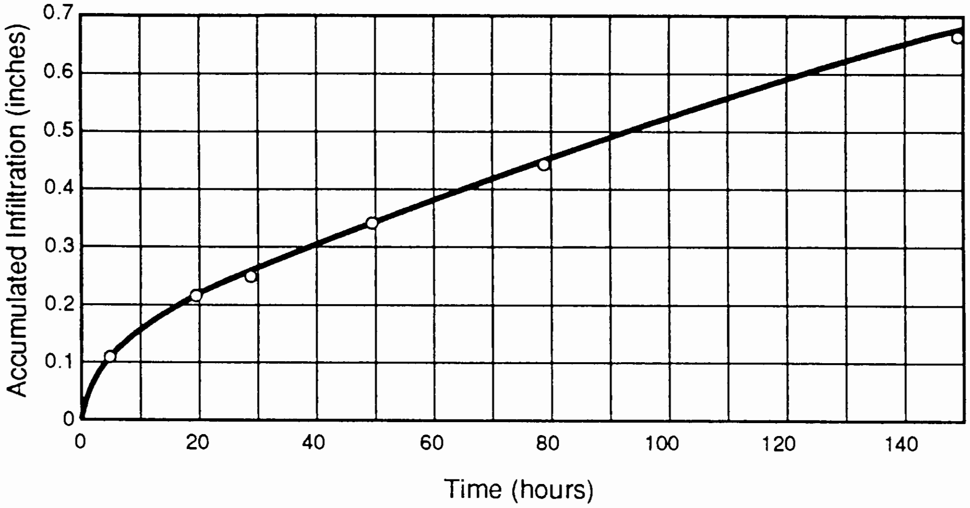
The terms in this equation are presented in **Figure 3-12**.

For most soils, k is not constant and it is difficult to obtain an average value. Horton's equation seems to be most suitable for describing infiltration when the water is applied by rain or sprinkling systems, and then only for relatively short time periods (Bouwer 1978). It should be realized that field data (I_t and t) for evaluation of the parameters in the empirical infiltration equation must be obtained for the same conditions as will occur for the infiltration systems to be predicted with equations. These conditions include duration of infiltration event, quality of water applied, depth of flooding, velocity of water above ground (ponded or flowing), soil conditions, and size and geometry of field tests (Bouwer 1978).

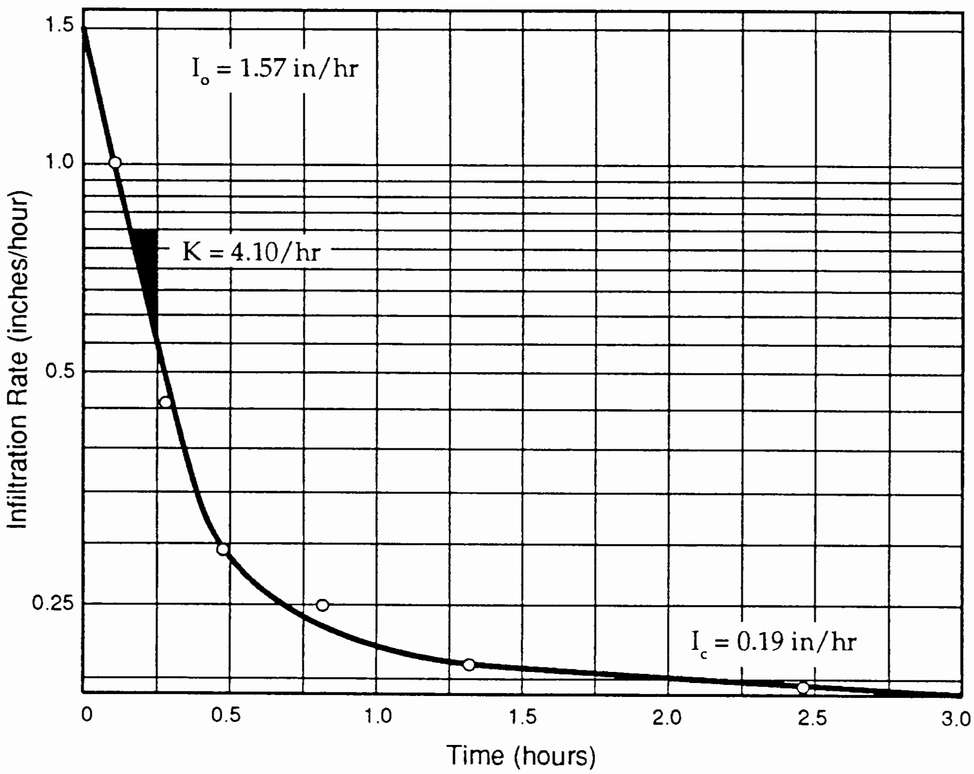


DELAYED-YIELD TYPE CURVES

After Boulton(1963, fig. 1)



Example Accumulated Infiltration Curve



Example Horton Infiltration Curve

$$I(t) = I_c + (I_o - I_c)e^{-Kt}$$

Where:

I(t)= Infiltration rate as a function of time cm/hr (in/hr) or other consistent ones
I_c= Final or ultimate infiltration rate-for a hydraulic gradient of unity, this is analogous to the soil permeability
I_o= Initial infiltration rate
K= Recession constant (/hr) or other consistent units
t= Time-units compatible with K

The total volume of infiltrate using Horton's equation is determined by integrating the area under the curve, or

$$I = \int_o^t I(t) = I_c + \frac{(I_o - I_c)}{K}(1 - e^{-Kt})$$

Where:

I= Total infiltration volume, cm (in) or other consistent units

JAMMAL & ASSOCIATES, INC.

(After Wanielista, 1979)

Figure 3-12

The main source of error with this technique is lateral diversion of the flow below the cylinder, which may be due to unsaturated flow (Bouwer 1961; Swartzendruber and Olsen 1961; Talsma 1970) or to restricting layers in the soil (Evans et al., 1950). Since the amount of infiltration contributing to the diversions will be minimal when infiltration takes place over a large area like a field or retention pond, the test results will lead to an over-estimation of infiltration rates. Diversions of flow below the cylinder due to unsaturated flow can be minimized by increasing the diameter of the cylinder. However, flow diversions due to lateral flow above restrictive layers deeper in the profile can be avoided only by using full scale ponds for the infiltration measurements (Bouwer 1978).

Based on our experience with using double-ring infiltrometer test data to estimate stormwater infiltration, we provide the following comments and conclusions:

1. If the test is conducted at the depth of the proposed pond bottom and the surface is representative of post-construction conditions, the test results are useful to estimate initial infiltration rates, prior to groundwater mounding conditions.

2. Once the groundwater mound rises to the pond bottom or higher, the results of a double-ring infiltrometer test are not valid. Since most stormwater retention ponds do operate with groundwater mounding into the pond, the use of double-ring infiltrometer tests should be restricted to the initial "unsaturated infiltration" analyses only.

3. The small area of recharge from the double-ring infiltrometer cannot produce a significant groundwater mound during the test period. Therefore, the scale factor between the test area and the area of a retention pond should be realized when using the results of a double-ring infiltrometer test.

4. We conclude that the double-ring infiltrometer test data is useful only to estimate the initial unsaturated infiltration from stormwater retention ponds, except in deep groundwater conditions where groundwater mound does not intersect the pond bottom throughout the entire duration of the stormwater runoff period.

Conclusions and Recommendations

There are many different field and laboratory test methods which can be used to explore and estimate hydrogeologic conditions and hydraulic parameters of an aquifer. In most instances, the limitations of the various methods are not clearly understood. To measure the horizontal hydraulic conductivity of the entire effective aquifer thickness, we recommend using short or long term pumping tests. This method, if used properly, provides the most reliable results. Slug tests are the next best means of measuring the hydraulic conductivity of the entire aquifer thickness, but the accuracy of this method is usually hindered by the need to install the piezometer in an undisturbed condition. For instance, if a clayey fine sand or clay is encountered in the profile in which the well is to be installed, unreliable results are usually obtained due to smearing of the soil surface during drilling and piezometer installation.

Laboratory permeability measurements on undisturbed samples generally yield accurate results, but the value of hydraulic conductivity is usually representative of a point of a soil stratum within the aquifer. Therefore, to characterize the entire aquifer system, permeability tubes would need to be collected in each soil strata comprising the aquifer system. This method is generally limited by the number of tests required and the fact that undisturbed samples must be collected.

Therefore, it is our opinion that the most effective method of hydraulic conductivity testing is a combination of laboratory and field tests that produce the most reliable results. These would include laboratory tests on undisturbed samples obtained from shallow depths, field auger/tube tests in sandy soils and above groundwater table, piezometer slug tests with properly installed and developed wells in deeper sandy deposits and short term or long term pump tests for multi-layer aquifer systems. A summary of recommended methods for the various exploration and testing techniques is presented in **Table 3-4**.

It should be realized that the information contained in this chapter is intended for planning purposes. Good, sound engineering judgement is still needed to determine when and where a particular method is applicable, to assess the limitations of each method and the validity of its results.

TABLE 3-4. ***Recommended Field and Laboratory Testing Methods for Stormwater Retention Pond Infiltration Analysis***

CONDITIONS	TEST METHOD
Soil Exploration	
Type and condition of soil	
<10 feet	hand or power auger borings
>10 Feet [60 Feet	power auger borings
In-situ density needed (any depth)	Standard Penetration Test Boring
Accurate groundwater level reading is critical	Hand or power auger boring and allow water level to stabilize for a minimum of 24 hours
Hydraulic Conductivity Measurement	
Shallow hydraulic conductivity measurement above groundwater table (sandy soil)	
<4 Feet	Excavate test pit with post-hole digger or shovel, hand drive shelby tube and perform laboratory permeameter tests
>4 feet < 10 Feet	Excavate test pit with backhoe or other equipment, collect shelby tubes by hand and perform laboratory permeameter tests.
>10 feet <50 Feet	Drill power auger boring to depth of proposed test. Install casing to bottom of borehole and screen the desired test interval. Conduct field hydraulic test using well permeameter method (U.S.B.R. Designation E-19).
Hydraulic Conductivity Measurement Below Groundwater Table (sandy soil) < 30 Feet Rice, 1971)	Install piezometer to desired depth, develop piezometers, stabilize for 24 hour minimum and conduct slug test or constant head test (Hvorslev, 1951, U.S. Navy, 1974 and Bouwer &
Accurate Determination of Hydraulic Conductivity is critical. Measurement below groundwater table. Any depth.	Install two wells and conduct short-term pumping test (lohman, 1972)
Estimate K_v (unsaturated initial infiltration)	Conduct Double Ring Infiltrometer tests. Alternatives, obtain undisturbed tube sample in the vertical direction. Conduct laboratory permeameter test and then estimate K_v (unsatu rated) by emperical methods.

TABLE 3-4. *(Continued)*

CONDITIONS	TEST METHOD
Deep hydraulic conductivity measurement below restrictive soils or confining unit (sandy soil). Groundwater table below bottom of restrictive soil	Install piezometer(s) to desired depth and screen below confining unit. Grout from bottom of confining unit to land surface. Conduct slug test in piezometer(s) (Hvorslev, 1951; U.S. Navy, 1974)
Deep hydraulic conductivity measurement below restrictive soil or confining unit (sandy soil). Groundwater table above confining unit. Leakance suspected to be high through confining unit.	Install two (2) piezometers to desired depth and screen below confining unit. Grout from bottom of confining unit to land surface. Conduct iong-term pumping test (Lohman, 1972)
Shallow or deep hydraulic conductivity measurement of restrictive soils (clayey sand, clays and hardpan)	Collect shelby tube soil sample by hand or with drill rig and conduct laboratory permeameter test in triaxial machine.
Approximate estimate of hydraulic conductivity after drilling is completed	Remold sample collected during drilling program to the approximate *in-situ* unit weight and conduct laboratory test in triaxial machine.
Unsaturated Vertical Infiltration Estimate, Direct Method	Conduct double ring infiltrometer test at pond bottom level. Compact test surface to the approximate post-construction density. Use final (I_c) infiltration rate determined during test.

CHAPTER 4

Presentation of Recommended Infiltration Analysis

Chapter 4

Presentation of Recommended Infiltration Analysis

General Considerations

In this Chapter, the hydraulics of stormwater retention pond operation will be discussed. The goal is to develop techniques for analyzing the infiltration capacity of retention ponds when the subsurface soil conditions and the input due to runoff are known. The presentation of the analytical models begins with the description of the problem to be analyzed. Next, the relevant analytical methods are presented, which are both mathematically understandable and easily applied. These techniques incorporate the saturated and unsaturated infiltration as normally occurs at stormwater retention ponds in shallow groundwater conditions typical to Florida.

Unsaturated Groundwater Flow

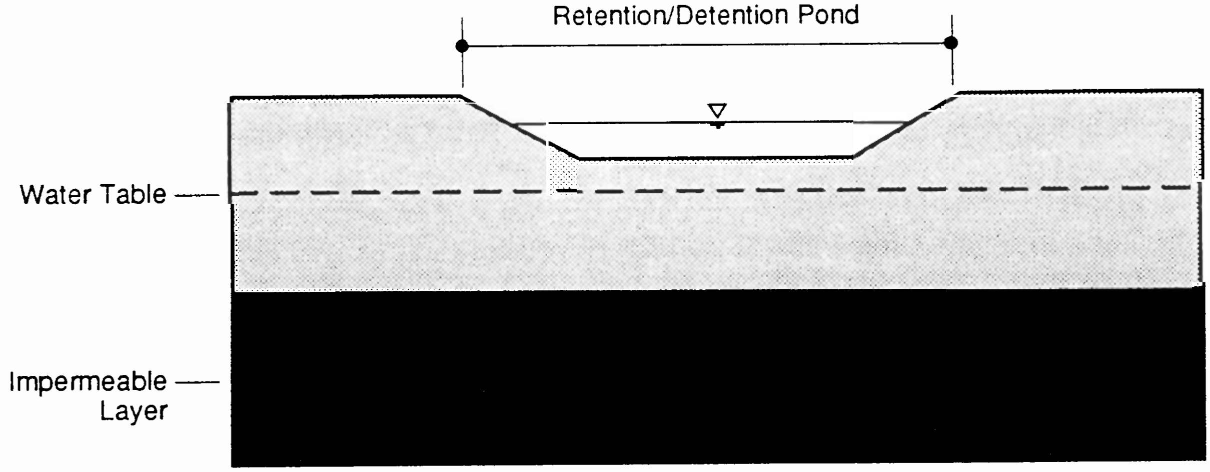
For presentation purposes, we assume that the subsurface conditions as presented in **Figure 4-1** exist where the soil is homogeneous and isotropic. Initially the soil above the water table is assumed to be at field capacity. Field capacity is defined as the moisture content of the soil when the gravitational water has drained away.

When water enters the pond, the standing water in the pond begins to infiltrate. The soil and moisture profile can be classified into zones as shown in **Figure 4-2**. (Bear, Zaslavsky and Irmaq 1968).

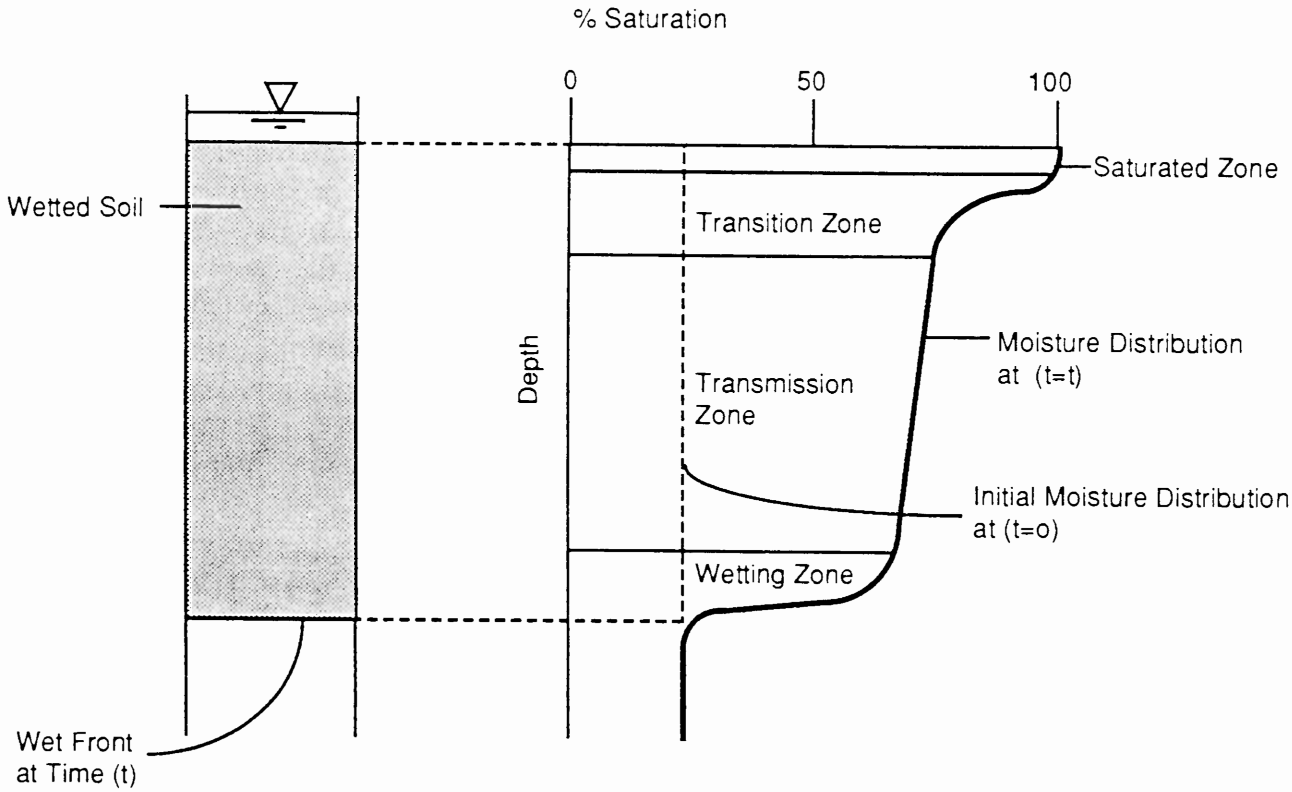
The thin surface layer that forms at the soil surface is apparently due to reduced permeability caused by the entrance of water into the soil. Below this layer, the moisture content decreases until it reaches a nearly constant value in the transmission zone. The transmission zone penetrates deeper and deeper into the ground behind the advancing wetting front. The wetting front is driven downward by the combined forces of gravity and capillary action.

Unsaturated flow follows Darcy's law just as saturated flow does, but both the coefficient of permeability and the gradients are variable (Isrealson and Hansen, 1962). Permeability depends on the degree of saturation, because the decrease in moisture content will produce a decrease in the cross sectional area of flow between the soil particles. The gradient changes because the head at the soil surface is constant and the head at the wetting front is constant (in homogeneous soil) but the distance between the two increases. The rate of water intake by the soil varies with several factors, including the depth of the water on the surface, temperature, soil structure, texture, moisture content of the soil, and time (Isrealson & Hansen, 1962). Laboratory and field experiments indicate that the rate of flow varies inversely with the square root of the time during the advance of the wetting front (Weaver 1971).

The second stage of seepage starts when the wetting front reaches the groundwater table. The effective capillary suction potential at the wetting front then disappears and vertical infiltration takes place under a constant gradient of 1.0 where the pressure head is approximately equal to the seepage path length. Simultaneously,



Idealized Subsurface Conditions



Moisture Zones During Infiltration
(After Bear, Zaslavasky and Irmaq, 1968)

JAMMAL & ASSOCIATES, INC.

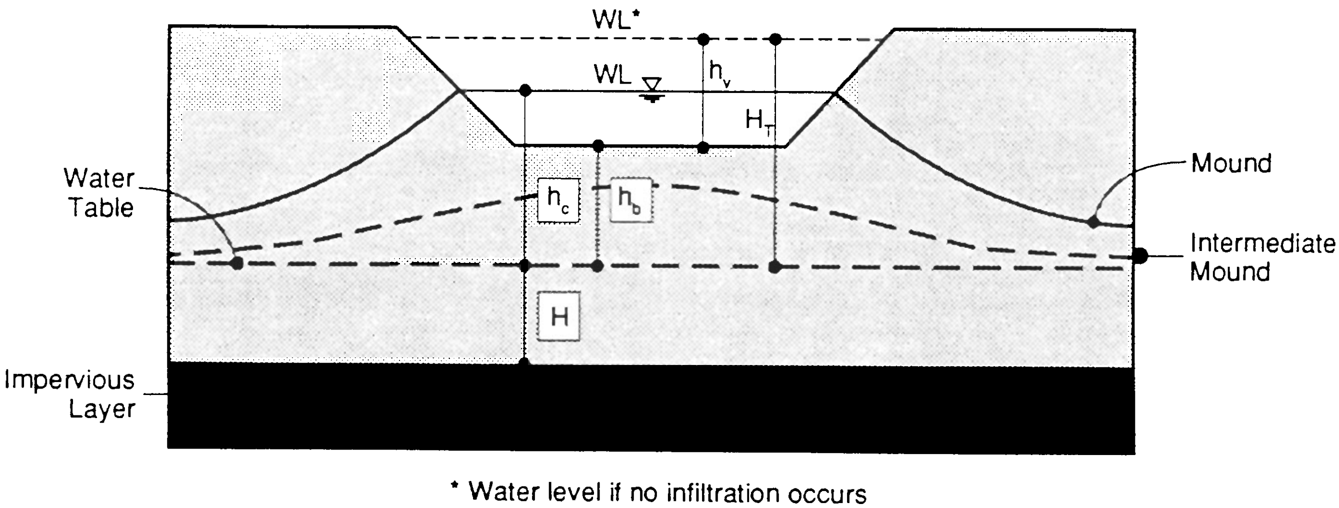
the vertical infiltration begins to add water to the water table aquifer. From this time, horizontal groundwater seepage in the saturated aquifer occurs simultaneously with storage in the unsaturated portion above the groundwater table. At this time, saturated flow begins and the water table mounds as shown on **Figure 4-3**.

When water invades the surface of an unsaturated soil, it flows into the ground and responds to the forces of capillary suction, gravity and the pressure of the ponded water at the surface. Capillary forces arise because of surface tension at the interfaces between soil, water and air. At any interface, the force due to surface tension is determined by a degree of curvature of the interface. The raised curvature of such an air/water interface depends on the dimensions of the pore space and the soil moisture content. In most natural soils, the wetting angle produces reduced pressure on the water side of the interface, indicating that the capillary forces do pull water into the dry soil. Since the greatest interfacial curvature occurs in the smallest pores, they will fill first due to greater suction assuming the displaced air can escape. As the moisture content of the soil increases, larger interstices will fill and the average interfacial curvature and corresponding pressure reduction will be less (Bouwer 1969). Therefore, in unsaturated soils, the water pressure is less than the atmospheric pressure and is a function of the degree of saturation. The functional relationship is different for different soils.

Soil particle diameter and grain size distribution are the primary factors in this relationship for granular soils. **Figure 4-4** shows typical curves of the capillary suction potential, h_{σ} versus degree of saturation (Brooks and Corey 1966). The units of h_{σ} are units of length because the suction potential is formed by dividing the negative pressure by the unit weight of water. These curves indicate that fine grained soils have a higher suction potential than the coarse grained soils. Also for poorly graded soils (i.e. uniform grain sizes), such as the one constructed from glass beads, the suction potential varies smoothly as the moisture decreases over a wide range.

Recommended Method for Unsaturated Infiltration Analyses

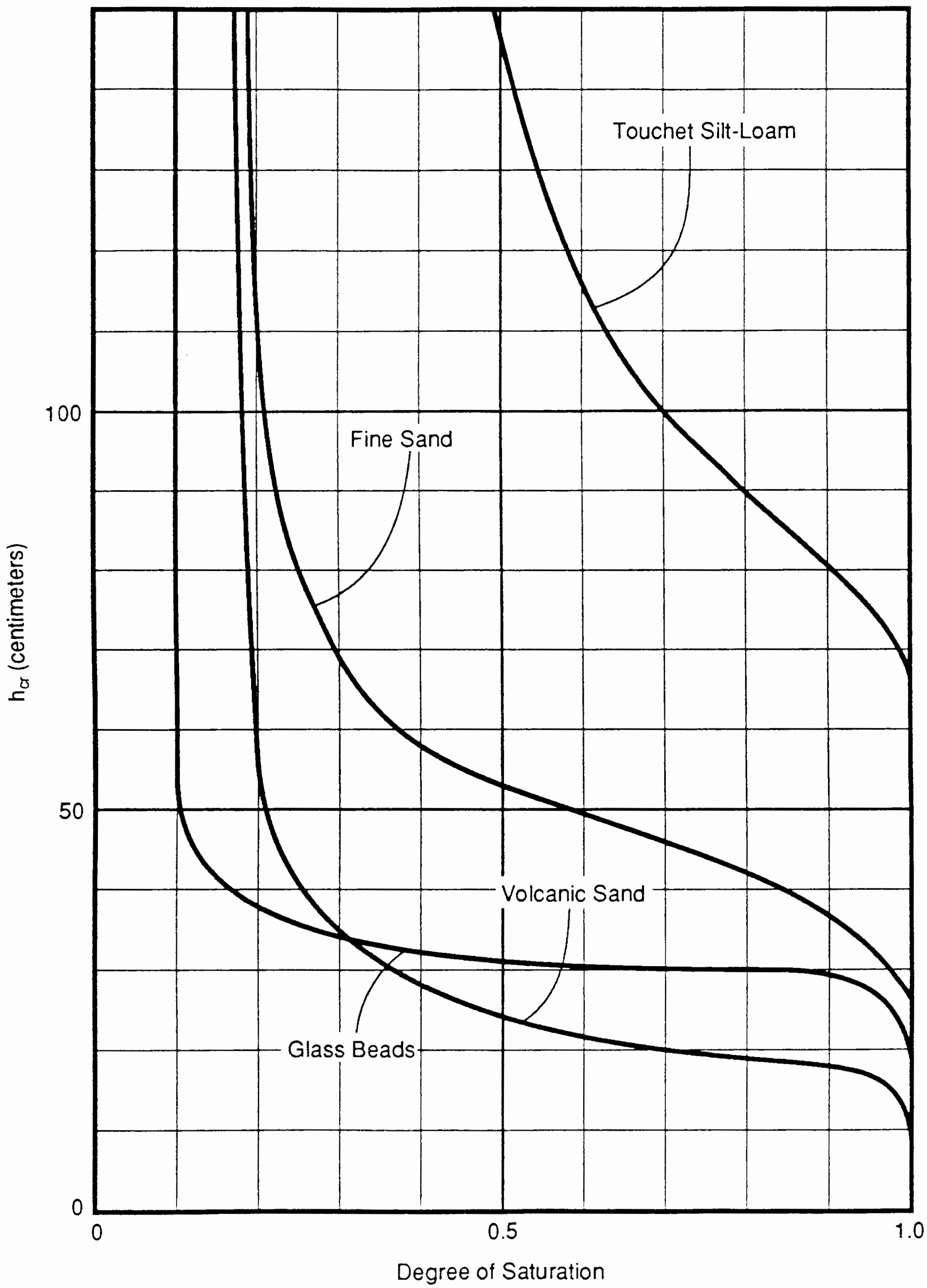
The equation of motion for infiltration is Darcy's law which has been found to apply to unsaturated flow even though the hydraulic conductivity is not constant with changing moisture content. The unsaturated hydraulic conductivity for any given soil, like capillary suction potential, is a function of the degree of saturation and grain size distribution. Rigorous theoretical treatment of the infiltration (exfiltration) phenomenon requires the determination of the variation in the hydraulic conductivity as the wetting front advances and other functional relationships by extensive laboratory testing of the particular soil in question. Infiltration into a soil for which these relationships have been found can then be described by numerical solutions of the diffusivity form of the unsaturated flow equation (Philips 1957).



Groundwater Mounding System

JAMMAL & ASSOCIATES, INC.

Figure 4-3



Typical Capillary Suction Potential Curves

(After Brooks and Corey, 1966)

JAMMAL & ASSOCIATES, INC.

Figure 4-4

For practical design of stormwater retention ponds a simpler analysis is needed without expensive instrumentation and testing procedures. For such purposes, the infiltration equation developed by Green and Ampt (1911), is sufficient. This equation was presented in 1911 as an empirical description of infiltration. Later, investigators modified the original Green and Ampt equation by theoretical determination of the original empirical constants (Bouwer 1969). In its modified form, the equation has been compared to more rigorous solutions and found to give almost identical results (Whisler and Bouwer 1970). Green and Ampt based their derivation on a simplified model of infiltration which treated the soil as a bundle of capillary tubes. Hydraulic conductivity and water content in the transmissive zone are considered constant as is the capillary suction potential acting on the advancing wetting front. Applying Darcy's law to the transmissive zone, the infiltration (I) is described as:

$$I = K_{vu} \frac{H_w + L_s - h_{cr}}{L_s} \qquad \textbf{(4-1)}$$

Where:

I = infiltration rate
K_{vu} = unsaturated vertical hydraulic conductivity
H_w = depth of ponded water
h_{cr} = capillary suction potential at the wetting front
L_s = depth of penetration of the wetting front

The rate of advance for the wetting front is:

$$\frac{dL_s}{dt} = \frac{I}{f} \qquad \textbf{(4-2)}$$

Where:

I = Infiltration rate
t = time
f = the effective storage coefficient (fillable porosity)

Combining **Equations 4-1** and **4-2** and integrating gives the relationship of depth of wetting to time.

$$t = \frac{f}{K_{vu}} \left(L_s - (H_w - h_{cr}) \ln \left(\frac{H_w + L_s - h_{cr}}{H_w - h_{cr}}\right)\right) \qquad \textbf{(4-3)}$$

Where:

t = time since start of infiltration

Equations 4-1 and **4-3** can be used to calculate the approximate infiltration rate and movement of the wetting front for soil profiles where f varies with depth. Normally, f decreases with depth because the moisture content of the soil tends to increase with depth. The values of f, K_{vu} and h_{cr} to be used in these equations are important enough to warrant some discussion. The void space in which water is stored after the passing of the wetting front (effective storage coefficient, f) is the difference between the initial moisture content and the moisture content in the transmissive zone. Experiments by Bodman and Coleman (1943) indicate that the transmissive zone for sand is about 90% saturated. Therefore, a reasonable engineering approach for shallow aquifer infiltration analysis is to let the moisture content of the transmissive soil be 90% of the soil porosity and the initial moisture content be the actual measured field capacity. The effective storage coefficient is the fractional difference between these two values. In terms of soil mechanics, the effective storage coefficient can be approximated as:

$$f = 0.9\ n - \left(\frac{w\gamma_d}{\gamma_w}\right) \qquad \textbf{(4-4)}$$

Where:

f = effective storage coefficient
n = total porosity
w = moisture content (fraction on a dry weight basis)
γ_d = dry unit weight of soil
γ_w = unit weight of water

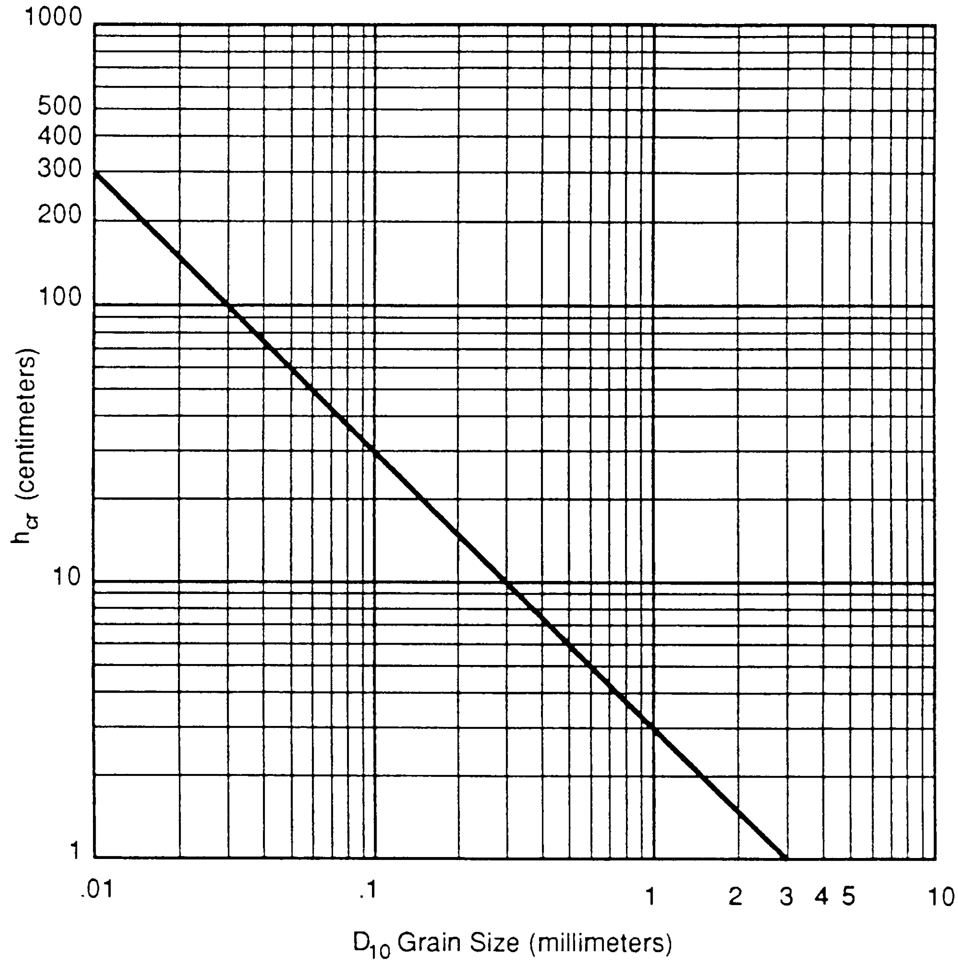
The relationship between the grain size distribution of the soil and h_{cr} is presented on **Figure 4-5** for estimation of the value of h_{cr} at field capacity. This chart has been constructed by the New York State Department of Transportation (Weaver 1971) to represent well-graded cohesionless soils in a natural state of moisture packing. Therefore, this chart should not be used if the soils analyzed consist of fine grained silty or clayey material. Laboratory determination of h_{cr} should follow the procedures in ASTM Standard D-2325.

The unsaturated hydraulic conductivity of the transmissive zone, K_{vu}, can be determined by field testing with an air entry permeameter (Bouwer 1966) or approximated by the results of a double-ring infiltrometer test. Alternatively, K_{vu} can be estimated from an empirical relationship of hydraulic conductivity versus saturation. Studies by Brooks and Corey (1966) express such a relationship of hydraulic conductivity versus saturation as:

$$K_{vu} = K_{vs} \left(\frac{0.8-S_r}{1.0-S_r}\right)^z \qquad \textbf{(4-5)}$$

Where:

K_{vs} = saturated vertical hydraulic conductivity
S_r = degree of saturation at field capacity (percent)
z = (2+3b)/b



Suction Potential vs. Grain Size for Well-Graded Cohesionless Soils

(After Weaver, 1971)

The b mentioned here is an empirical parameter representing primarily the grain size distribution of the soil. Generally, well-graded granular soils have lower values of b than poorly graded soils. Values of b given by Brooks and Corey (1966) range from b = 1.8 for Touchet silt to b = 7.3 for soil constructed of glass beads and for well graded clean sands b is between 3.4 and 3.7. Values of b for natural sands in Florida are not known.

The K_{vu} value is normally less than K_{vs} at saturation. Based on Bouwer's research of K_{vs} versus K_{vu}, the K_{vu} for sand varies from 1/2 to 2/3 of K_{vs} (Bouwer 1978). Based on Jammal & Associates, Inc. experience, the correlation factor of 2/3 appears to be applicable for most sands in Florida.

Once the appropriate values of f, h_{cr} and K_{vu} have been established, Green and Ampt's equation can be solved.

For most stormwater retention pond analyses, the depth of ponded water H_w is typically equal to or greater than h_{cr} in sands. Assuming that $H_w = h_{cr}$, **Equation 4-3** can be rewritten as:

$$t = \frac{fL_s}{K_{vu}} \qquad \textbf{(4-6)}$$

And the time necessary for the wetting front to reach groundwater is:

$$\Delta t_{sat} = \frac{fh_b}{K_{vu}} \qquad \textbf{(4-7)}$$

Where:

h_b = height of pond bottom above groundwater table

Using the same assumption of $H_w = h_{cr}$, **Equation 4-1** becomes:

$$I = K_{vu} \qquad \textbf{(4-8)}$$

Equations 4-7 and **4-8** can be used to calculate the approximate infiltration rate and the time for the wetting front to reach the groundwater table.

On the basis of our literature review, selection of an analytical approach that is practical and for which hydraulic parameters are readily measurable, we recommend that the modified Green and Ampt's infiltration equations be used to estimate initial unsaturated infiltration rates from stormwater retention/seepage ponds constructed in unconfined shallow aquifers. The following modified Green and Ampt equations should provide conservative results, provided that the recommended factor of safety is utilized.

The b mentioned here is an empirical parameter representing primarily the grain size distribution of the soil. Generally, well-graded granular soils have lower values of b than poorly graded soils. Values of b given by Brooks and Corey (1966) range from b = 1.8 for Touchet silt to b = 7.3 for soil constructed of glass beads and for well graded clean sands b is between 3.4 and 3.7. Values of b for natural sands in Florida are not known.

The K_{vu} value is normally less than K_{vs} at saturation. Based on Bouwer's research of K_{vs} versus K_{vu}, the K_{vu} for sand varies from 1/2 to 2/3 of K_{vs} (Bouwer 1978). Based on Jammal & Associates, Inc. experience, the correlation factor of 2/3 appears to be applicable for most sands in Florida.

Once the appropriate values of f, h_{cr} and K_{vu} have been established, Green and Ampt's equation can be solved.

For most stormwater retention pond analyses, the depth of ponded water H_w is typically equal to or greater than h_{cr} in sands. Assuming that $H_w = h_{cr}$, **Equation 4-3** can be rewritten as:

$$t = \frac{fL_s}{K_{vu}} \qquad \textbf{(4-6)}$$

And the time necessary for the wetting front to reach groundwater is:

$$\Delta t_{sat} = \frac{fh_b}{K_{vu}} \qquad \textbf{(4-7)}$$

Where:

h_b = height of pond bottom above groundwater table

Using the same assumption of $H_w = h_{cr}$, **Equation 4-1** becomes:

$$I = K_{vu} \qquad \textbf{(4-8)}$$

Equations 4-7 and **4-8** can be used to calculate the approximate infiltration rate and the time for the wetting front to reach the groundwater table.

On the basis of our literature review, selection of an analytical approach that is practical and for which hydraulic parameters are readily measurable, we recommend that the modified Green and Ampt's infiltration equations be used to estimate initial unsaturated infiltration rates from stormwater retention/seepage ponds constructed in unconfined shallow aquifers. The following modified Green and Ampt equations should provide conservative results, provided that the recommended factor of safety is utilized.

$$I_d = \frac{K_{vu}}{FS} \qquad \textbf{(4-9)}$$

$$\Delta t_{sat} = \frac{f h_b}{I_d} \qquad \textbf{(4-10)}$$

Where:

I_d = design infiltration rate
Δt_{sat} = time to saturate soil below pond bottom
K_{vu} = unsaturated vertical hydraulic conductivity
FS = factor of safety (recommend FS = 2.0)

In these equations, it is assumed that there is **no** lateral infiltration during unsaturated flow. In reality, a certain amount of lateral infiltration will occur due to capillary forces in the soil matrix.

For design infiltration rate estimates during unsaturated flow below a pond, the following procedure is recommended:

1) Define the shallow aquifer characteristics below the pond.
2) Establish the elevation of the pond bottom, design high groundwater level and the area of the pond bottom.
3) Measure the unsaturated hydraulic conductivity, K_{vu}, using double ring infiltrometer field testing procedures or laboratory methods. The field test should be conducted at the same elevation as the proposed pond bottom or lower, if possible. The surface at the test site should be compacted to simulate pond bottom conditions after construction. Measurements of field K_{vu} at depths of more than 1 to 2 feet may not be possible, however, correlation of shallow strata test results with deeper strata may be possible. If field measurement of K_{vu} is not possible, measure K_{vs} using laboratory testing of undisturbed or remolded tube samples, and calculate K_{vu} using an empirical correlation of K_{vs} versus K_{vu}. The following correlation is recommended:

$$K_{vu} = \frac{2}{3} K_{vs} \qquad \textbf{(4-11)}$$

4) Calculate the design infiltration rate, I_d, and the saturation time, Δt_{sat}, using **Equations 4-9** and **4-10**. These equations incorporate a factor of safety to allow for pond bottom siltation/clogging.
5) The total volume of water required to saturate the soil below the pond bottom can be calculated as follows:

$$V_u = A_b h_b f \qquad \textbf{(4-12)}$$

Where:

A_b = area of pond bottom.

h_b = height of pond bottom above groundwater table

f = average effective storage coefficient

Saturated Groundwater Flow

Saturated flow beneath a typical stormwater retention pond is governed by the transmissive characteristics of the shallow aquifer, available lateral seepage gradients, pond geometry and other factors affecting the general form of Darcy's law for saturated seepage. Recharge into the groundwater aquifer creates a groundwater mound beneath the pond and its vicinity as presented on **Figure 4-3**. Once the groundwater mound intersects the pond bottom or if groundwater is at pond bottom, the infiltration to the soil will be governed by the saturated seepage in the groundwater aquifer with lateral gradients equal to the slope of the free surface, instead of the downward seepage with a gradient of 1.0. If the groundwater mound rises above the pond bottom at the end of the storm event, then the rate of water level decline in the pond after the storm is directly proportional to the rate of mound recession in the saturated aquifer. Therefore, for stormwater retention ponds constructed in areas of high groundwater conditions, it is important to predict the rate of growth and decay of the groundwater mound. Numerous analytical methods are available to evaluate the growth and decay of the groundwater mounds. For successful design of stormwater retention ponds, both the unsaturated and saturated seepage must be accounted for and incorporated into the analysis.

The majority of numerical solutions for groundwater mound growth and decay are a function of a uniform rate of recharge and spatially uniform storage coefficients (Chapter 2). Based on our literature review, we found two sets of analytical methods (Ortiz et al. 1978) and the "Three-Dimensional Finite Difference Groundwater Flow Computer Model" developed by the USGS (MODFLOW), where the storage coefficient was spatially variable. This was significant for our review and implementation of a practical method for stormwater retention pond analysis since the storage coefficient in the pond area is 1.0 and in the surrounding aquifer is less than 1.0. In the derivation of differential equations developed by Ortiz et al. (1979), it was assumed that an instantaneous slug of water was added to a pond area and decay of the groundwater mound was calculated. Ortiz et al. (1979) did not account for mound intersection of the pond bottom and the effects of level mound (pond stage) in the pond area were not incorporated. As a result the method developed by Ortiz et al. (1979) was not directly applicable for our purposes therefore, to account for unsaturated recharge from a retention pond followed by continual recharge from a typical 24-hour storm event (allowing the mound to intersect pond bottom), a combination of the modified Green and Ampt equations and MODFLOW was determined to be the most applicable.

Recommended Method for Saturated Infiltration Analyses

In the analysis of saturated flow processes, some idealized configuration must be assumed. First, a definite geometry must be chosen for the stormwater infiltration basin. In this report, a rectangular basin is used. Irregular shaped basins and circular shaped basins can be converted to an equivalent rectangle for the analysis presented herein.

A schematic of an unconfined aquifer is presented by **Figure 4-3**. The aquifer is assumed to be homogeneous and isotropic, and it is underlain by an impervious horizontal layer. The water table is also horizontal.

Due to the complex flow characteristics during stormwater infiltration from retention ponds, most of the simplistic methods of calculation were found inapplicable. Based on our review of gathered literature (Chapter 2) and considering the complexities of retention pond infiltration conditions, we have selected the Three-Dimensional Finite Difference Groundwater Flow Model (aka MODFLOW) developed by the U.S. Geological Survey (McDonald and Harbaugh, 1984) for saturated groundwater flow modeling. This model simulates groundwater flow in three dimensions for variable recharge rates and spatially variable storage coefficients required to satisfy the conditions of a stormwater retention pond system. In addition, it incorporates variable aquifer bottom elevations, variable hydraulic conductivity and can accommodate a layered system with any combination of unconfined, confined or semi-confined leaky aquifers. Input parameters such as initial head elevation, permeability, storage coefficient and bottom elevation of the aquifer can be varied from cell to cell within the grid system.

For the purpose of generating data points for a series of dimensionless curves, an idealized grid system was set-up for the MODFLOW computer model. A grid system with 24 rows and 24 columns was selected with the retention pond located in the center of the grid system. The retention pond was represented by 24 grid cells (4 columns and 6 rows. The storage coefficient within the retention pond cells was set to a value of 1.0 (indicating "no-soil" conditions). Another condition considered in our modeling was the horizontal hydraulic conductivity within the retention pond and the transitional grid cells. The horizontal hydraulic conductivity of the stormwater retention pond cells was set to a value of 1,000 feet per day. A high permeability condition in the stormwater retention pond in combination with an effective storage of 1.0 tells the model that those grid cells are comprised completely of water at a constant head elevation. The horizontal hydraulic conductivity for the transitional grid cells was calculated using the following equation:

$$K_{Htrans} = \frac{G}{\frac{1.5G}{K_H} - \frac{G/2}{1000}} \qquad \textbf{(4-13)}$$

Where:

G = distance of grid interval

K_H = horizontal hydraulic conductivity of the aquifer.

An explanation of how **Equation 4-13** was developed is presented on **Figure 4-6**.

To illustrate the relationship between transient water level in the retention pond and pond design parameters, a series of dimensionless curves have been developed. These curves were developed for rectangular retention ponds constructed in an unconfined aquifer. No vertical leakance to underlying aquifer systems was taken into account. The dimensionless curves were generated by making numerous computer runs using the MODFLOW computer model.

Based on our literature review outlined in Chapter 2, the dimensionless parameters developed by Ortiz, Zachmann, McWhorter and Sunada (1979) were modified and used to generate the dimensionless curves for estimating infiltration rates from stormwater retention ponds. The modified dimensionless parameters can be expressed as follows:

$$F_x = \left(\frac{W^2}{4K_H Dt} \right)^{1/2} \quad \text{(for x-axis)} \qquad \textbf{(4-14)}$$

$$F_y = \frac{h_c}{H_T} \quad \text{(for y-axis)} \qquad \textbf{(4-15)}$$

Where:

h_c = height of water level in the retention pond above the initial groundwater table

H_T = h_b+h_v

h_b = height of the pond bottom above the groundwater table

h_v = height of water level above pond bottom if **no** infilltration occured

W = the average width of the pond

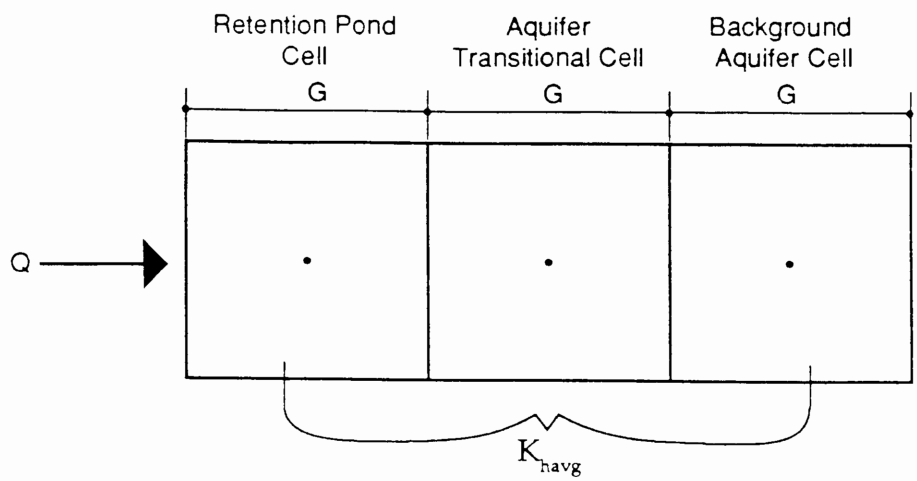
K_H = the average horizontal hydraulic conductivity of the aquifer

D = the average saturated thickness of the aquifer $(H+h_c/2)$

H = initial saturated thickness of aquifer

t = cumulative time since recharge started

Typical input parameters for a rectangular basin of 800 feet in length and 200 feet width is presented on **Figure 4-7**. The transitional horizontal hydraulic conductivity of the model grid cells between the retention pond and background unconfined aquifer was calculated based on the hydraulic conductivity of the aquifer and the default value of 1,000 feet per day in the retention pond area. Additionally, to eliminate boundary water accumulation affect at the edge of the grid system, if any, the head levels around the entire perimeter were set to a constant value. The recharge



Where:

G= Grid interval, feet

K_{h1000}= Hydraulic conductivity value in retention pond=1000 ft./day

K_{hback}= *In-situ* hydraulic conductivity of unconfined aquifer

K_{htrans}= Hydraulic conductivity of unconfined aquifer in transitional grid cell between retention pond and background aquifer (*see figure 4-7*)

$$K_{havg} = K_{hback} = \frac{1/2G + G + 1/2G}{\frac{1/2G}{K_{h1000}} + \frac{G}{K_{htrans}} + \frac{1/2G}{K_{hback}}}$$

Combining terms and rearranging the equation...

$$2G = K_{hback}\left(\frac{1/2G}{K_{h1000}} + \frac{G}{K_{htrans}} + \frac{1/2G}{K_{hback}}\right)$$

$$2G = \frac{GK_{hback}}{2K_{h1000}} + \frac{GK_{hback}}{K_{htrans}} + 1/2G$$

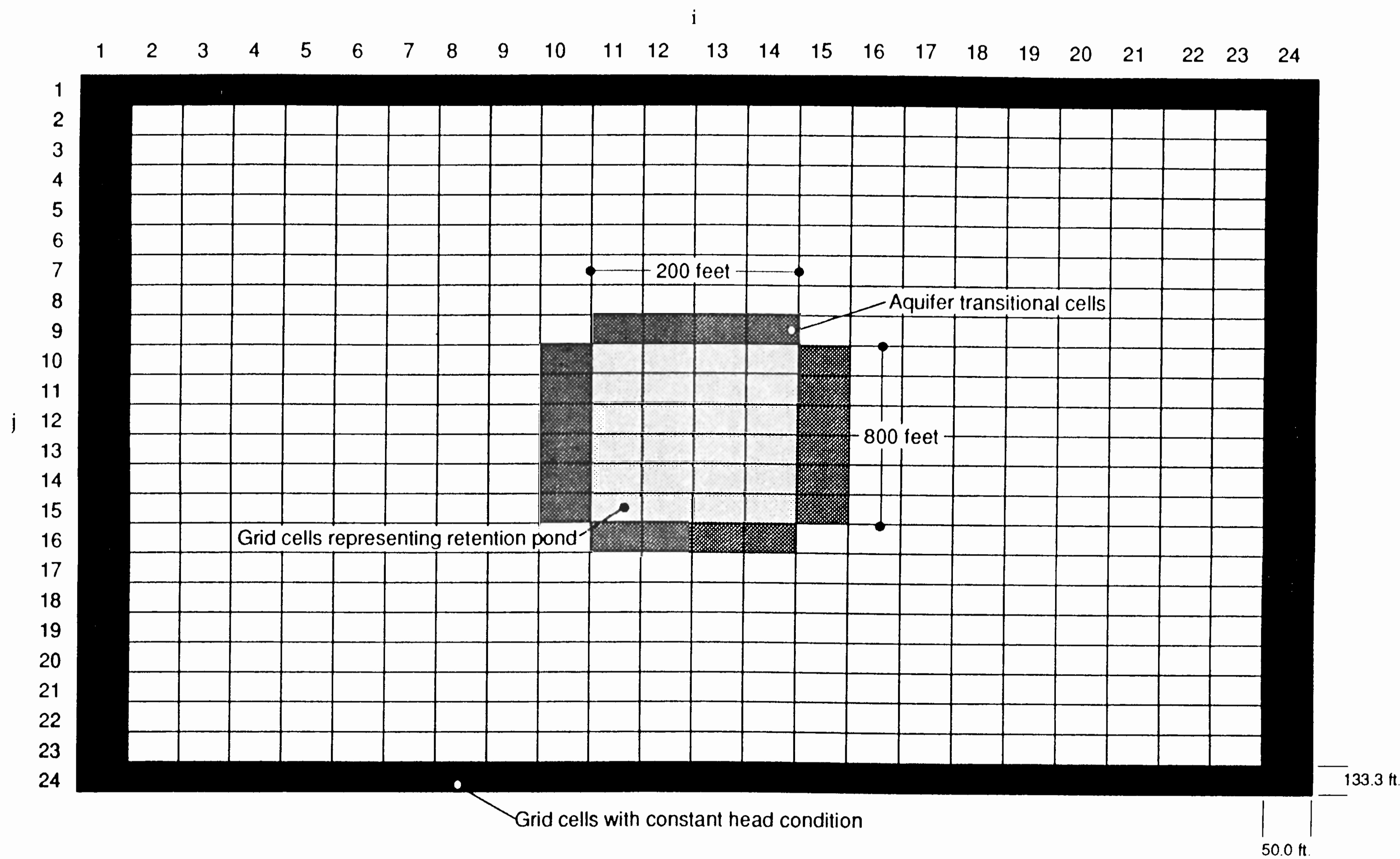
$$\frac{GK_{hback}}{K_{htrans}} = 1.5G - \frac{\frac{G}{2}K_{hback}}{1000} \quad ; \quad K_{htrans} = \frac{GK_{hback}}{1.5G - \frac{\frac{G}{2}K_{hback}}{1000}}$$

In a different form...

$$K_{htrans} = \frac{G}{\frac{1.5G}{K_{hback}} - \frac{G/2}{1000}}$$

Derivation of Equation 4-13

JAMMAL & ASSOCIATES, INC.



Plan View of 3-D Model Grid for Example Problem

JAMMAL & ASSOCIATES, INC.

Figure 4-7

rate, effective runoff time, and several arbitrary recovery times were entered into the program for analyses and generation of data points.

The results of the MODFLOW analyses are presented as head elevation (groundwater level) in each grid cell for each simulation time period. The results of the analyses for this example problem are presented on **Figure 4-8**. The head output data was then used in conjunction with the previously presented dimensionless parameters.

For the example problem herein, dimensionless parameter, F_x, was calculated based on the retention pond dimensions and aquifer data, as follows:

$W =$ Pond width = 200 feet
$K_H =$ Horizontal hydraulic conductivity = 5 feet per day
$t =$ Various time periods for which the water level in the pond is to be calculated, $t_5 = 22$ days (cummulative)
$D =$ Average saturated thickness of the aquifer.

$$D = H + \frac{h_c}{2} = 10\text{ ft} + \frac{11.3\text{ ft}}{2} = 15.65\text{ ft}$$

Now, using **equation 4-14**,

$$F_x = \left(\frac{200^2}{4(5)(15.65)(22)}\right)^{1/2} = 2.41$$

The dimensionless parameter, F_y, was calculated based on the input data and the results of the MODFLOW analysis, as follows:

$h_b =$ Height of the pond bottom above the groundwater table, which is equal to 5 feet.
$h_v =$ Height of water level above pond bottom if **no** infiltration occured. (15 ft/day)(0.6 days) = 9 ft.
$H_T =$ $h_b + h_v = 5\text{ ft} + 9\text{ ft} = 14\text{ ft}$
$h_c =$ Height of the water level in the retention pond above the initial ground water table at time t_5. Knowing the groundwater table elevation used as input, **Figure 4-8**, determine the resulting head elevation in the retention pond area from the MODFLOW analyses. For stress period No. 5 the resulting difference in elevation is 121.3 - 110.0 = 11.3 feet.

$$F_y = h_c/H_T = \frac{11.3}{14.0} = 0.807$$

Now that F_x and F_y are known, this point is plotted on the curve of **Figure 4-11**. This analytical approach was used for a combination of stormwater retention pond configurations, dimensions and aquifer parameters.

Basin Length in Y direction	800 ft.
Basin width in X direction	200 ft.
Water table elevation	110 ft., m.s.l.
Aquifer bottom elevation	100 ft., m.s.l.
Average horizontal permeability	5 ft./day
Fillable porosity of effective aquifer	0.3
Average recharge rate into pond	15 ft./day
Average background recharge	0 ft./day
Effective runoff time	0.6 days (stress period 1)
Four (4) time steps after storm	1.4, 2.0, 4.0, 14.0 days
Pond bottom elevation	115 ft., m.s.l.

Stress Period Time = 0.6 days
Total Time = 0.6 days

Head in layer 1 at end of time step 1 in **stress period 1**

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
2	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
4	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
5	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
6	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
8	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
9	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.1	110.1	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
10	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
11	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
12	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
13	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
14	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
15	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.7	**123.9**	**123.9**	**123.9**	**123.9**	110.7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
16	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.1	110.1	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
17	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
18	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
19	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
20	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
21	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
22	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
23	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
24	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0

Computer Input and Output for Modflow Retention Pond Saturated Infiltration Analysis

JAMMAL & ASSOCIATES, INC.

Figure 4-8(2)

Stress Period Time = 1.4 days
Total Time = 2.0 days

Head in layer 1 at end of time step 1 in **stress period 2**

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
2	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
4	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
5	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
6	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	[illegible]	110.0	110.0	110.0	110.0
7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	[illegible]	[illegible]	110.0	110.0	110.0	110.0
8	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	[illegible]	[illegible]	110.0	110.0	110.0	110.0	110.0
9	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.3	110.3	110.3	110.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
10	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
11	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
12	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
13	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
14	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
15	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	112.3	**123.6**	**123.6**	**123.6**	**123.6**	112.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
16	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.3	110.3	110.3	110.3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
17	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
18	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
19	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
20	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
21	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
22	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
23	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
24	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0

Stress Period Time = 2.0 days
Total Time = 4.0 days

Head in layer 1 at end of time step 1 in **stress period 3**

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
2	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
4	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
5	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
6	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
8	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
9	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	110.7	110.7	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
10	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.0	**123.2**	**123.2**	**123.2**	**123.2**	114.0	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
11	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.1	**123.2**	**123.2**	**123.2**	**123.2**	114.1	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
12	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.1	**123.2**	**123.2**	**123.2**	**123.2**	114.1	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
13	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.1	**123.2**	**123.2**	**123.2**	**123.2**	114.1	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
14	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.1	**123.2**	**123.2**	**123.2**	**123.2**	114.1	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
15	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	114.0	**123.2**	**123.2**	**123.2**	**123.2**	114.0	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
16	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.6	110.7	110.7	110.6	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
17	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
18	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
19	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
20	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
21	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
22	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
23	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
24	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0

Computer Input and Output for Modflow Retention Pond Saturated Infiltration Analysis

JAMMAL & ASSOCIATES, INC.

Figure 4-8 Continued (2)

Stress Period Time = 4.0 days
Total Time = 8.0 days

Head in layer 1 at end of time step 1 in **stress period 4**

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
2	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
4	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
5	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
6	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
8	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
9	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.4	111.1	111.2	111.2	111.1	110.4	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
10	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.1	**122.6**	**122.6**	**122.6**	**122.6**	116.1	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
11	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.3	**122.6**	**122.6**	**122.6**	**122.6**	116.3	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
12	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.3	**122.6**	**122.6**	**122.6**	**122.6**	116.3	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
13	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.3	**122.6**	**122.6**	**122.6**	**122.6**	116.3	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
14	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.3	**122.6**	**122.6**	**122.6**	**122.6**	116.3	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
15	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.3	111.5	116.1	**122.6**	**122.6**	**122.6**	**122.6**	116.1	111.5	110.3	110.1	110.0	110.0	110.0	110.0	110.0	110.0
16	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.4	111.1	111.2	111.2	111.1	110.4	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
17	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
18	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
19	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
20	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
21	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
22	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
23	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
24	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0

Stress Period Time = 14.0 days
Total Time = 22.0 days

Head in layer 1 at end of time step 1 in **stress period 5**

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
2	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
3	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
4	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
5	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
6	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
7	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
8	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.1	110.2	110.2	110.3	110.3	110.2	110.2	110.1	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
9	110.0	110.0	110.0	110.0	110.0	110.1	110.2	110.4	110.8	111.4	112.3	112.6	112.6	112.3	111.4	110.8	110.4	110.2	110.1	110.0	110.0	110.0	110.0	110.0
10	110.0	110.0	110.0	110.0	110.1	110.2	110.6	111.6	113.7	117.9	**121.3**	**121.3**	**121.3**	**121.3**	117.9	113.7	111.6	110.6	110.2	110.1	110.0	110.0	110.0	110.0
11	110.0	110.0	110.0	110.0	110.1	110.3	110.7	111.8	114.0	118.2	**121.3**	**121.3**	**121.3**	**121.3**	118.2	114.0	111.8	110.7	110.3	110.1	110.0	110.0	110.0	110.0
12	110.0	110.0	110.0	110.0	110.1	110.3	110.8	111.9	114.1	118.2	**121.3**	**121.3**	**121.3**	**121.3**	118.2	114.1	111.9	110.8	110.3	110.1	110.0	110.0	110.0	110.0
13	110.0	110.0	110.0	110.0	110.1	110.3	110.8	111.9	114.1	118.2	**121.3**	**121.3**	**121.3**	**121.3**	118.2	114.1	111.9	110.8	110.3	110.1	110.0	110.0	110.0	110.0
14	110.0	110.0	110.0	110.0	110.1	110.3	110.7	111.8	114.0	118.2	**121.3**	**121.3**	**121.3**	**121.3**	118.2	114.0	111.8	110.7	110.3	110.1	110.0	110.0	110.0	110.0
15	110.0	110.0	110.0	110.0	110.1	110.2	110.6	111.6	113.7	117.9	**121.3**	**121.3**	**121.3**	**121.3**	117.9	113.7	111.6	110.6	110.2	110.1	110.0	110.0	110.0	110.0
16	110.0	110.0	110.0	110.0	110.0	110.1	110.2	110.4	110.8	111.4	112.3	112.6	112.6	112.3	111.4	110.8	110.4	110.2	110.1	110.0	110.0	110.0	110.0	110.0
17	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.1	110.1	110.2	110.2	110.3	110.3	110.2	110.2	110.1	110.1	110.0	110.0	110.0	110.0	110.0	110.0	110.0
18	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
19	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
20	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
21	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
22	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
23	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0
24	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0	110.0

Computer Input and Output for Modflow Retention Pond Saturated Infiltration Analysis

JAMMAL & ASSOCIATES, INC.

Figure 4-8 Continued (3)

Using the calculated dimensionless parameters, F_x and F_y, four families of dimensionless curves were developed. A family of dimensionless curves are defined herein as having the same effective storage coefficient in the surrounding unconfined aquifer but each curve corresponds to a specific length to width ratio of the stormwater retention pond. Therefore, four families of curves for storage factor, f = 0.1, 0.2, 0.3 and 0.4 have been developed. Five individual curves, for length to width ratios of 1, 2, 4, 10 and 100 have been developed for each family. The resulting dimensionless curves are presented on **Figures 4-9** through **4-12**. These dimensionless curves can easily be used to determine the water level in a stormwater retention pond given the hydraulic parameters of the aquifer, the recharge rate and duration, the physical configuration of the pond and the desired time period.

Summary and Conclusions

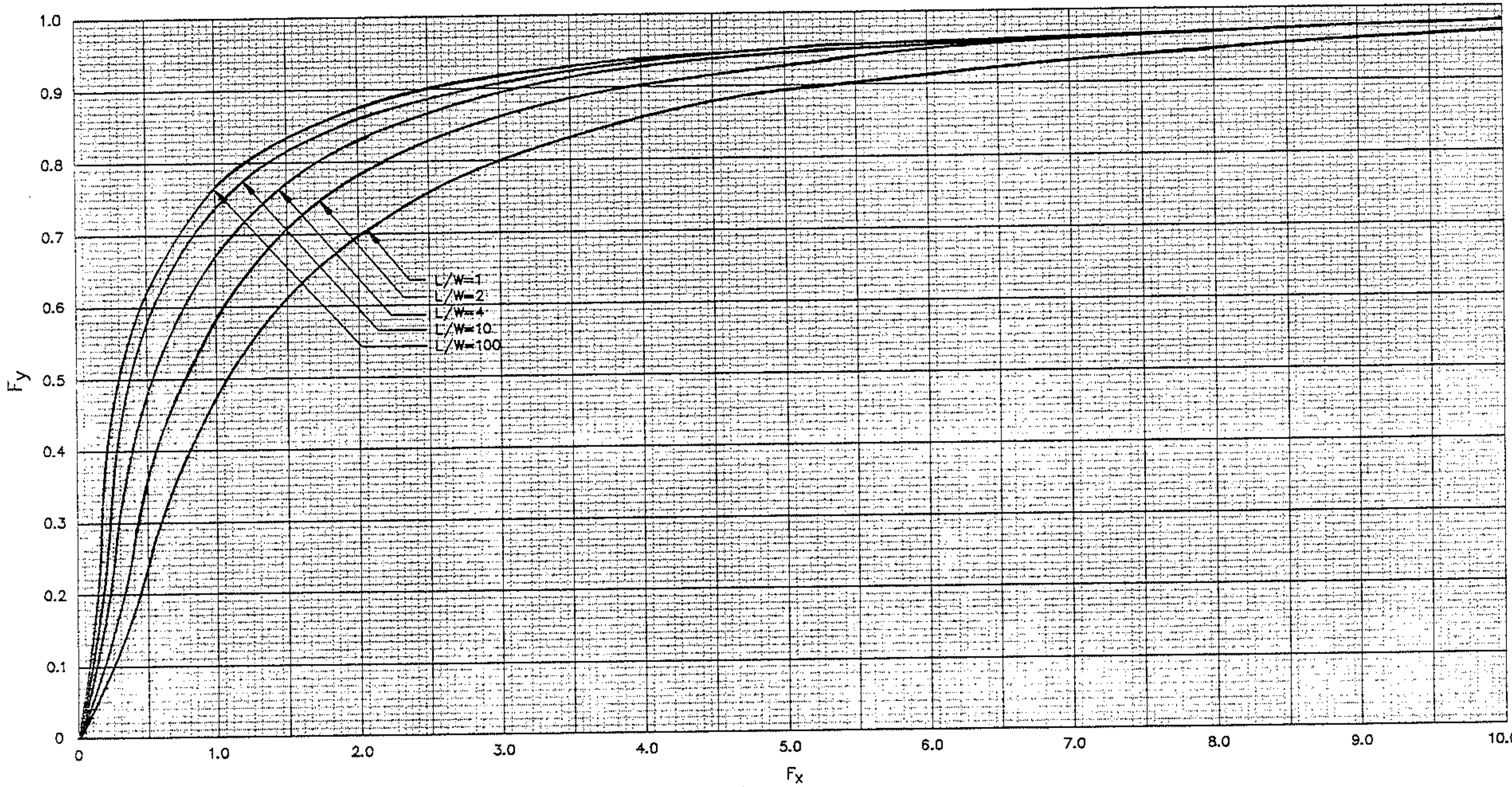
An analysis of the hydraulic operation of a stormwater retention pond has been presented in two phases. The first phase is concerned with the rate at which water flows out of a pond by vertical infiltration through its bottom. The equation developed by Green and Ampt (1911) was selected to describe the vertical infiltration for the retention pond.

The second phase of the analysis deals with the response of the water table to recharge from the retention pond. From a pond designer's point of view, this is important because when the groundwater mound intersects the pond bottom, the infiltration rate analysis becomes a function of lateral dissipation through the saturated aquifer and storage in the unsaturated zone instead of vertical infiltration.

It should be noted that the hydraulic conductivity referred to in the two phases of the analysis are not the same. Unsaturated infiltration is concerned with vertical flow while the groundwater mound is mainly influenced by horizontal flow. The difference between the hydraulic conductivities in the two directions is usually associated with horizontal layering of the soil. A representative hydraulic conductivity in the horizontal direction should be found by appropriate field or laboratory test methods. Analysis of saturated infiltration was presented in the form of dimensionless curves which were generated using the MODFLOW computer program.

The analytical, unsaturated and saturated infiltration models presented in this chapter are mathematically correct and have been theoretically proven to provide reasonable results. However, fully instrumented full scale field tests will be needed to verify the accuracy of the selected models and their application for stormwater retention pond design. The saturated flow model (MODFLOW) has been used to simulate the results of several full scale load tests conducted in Florida and has been proven to provide reasonable simulation capacity.

RECHARGE



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

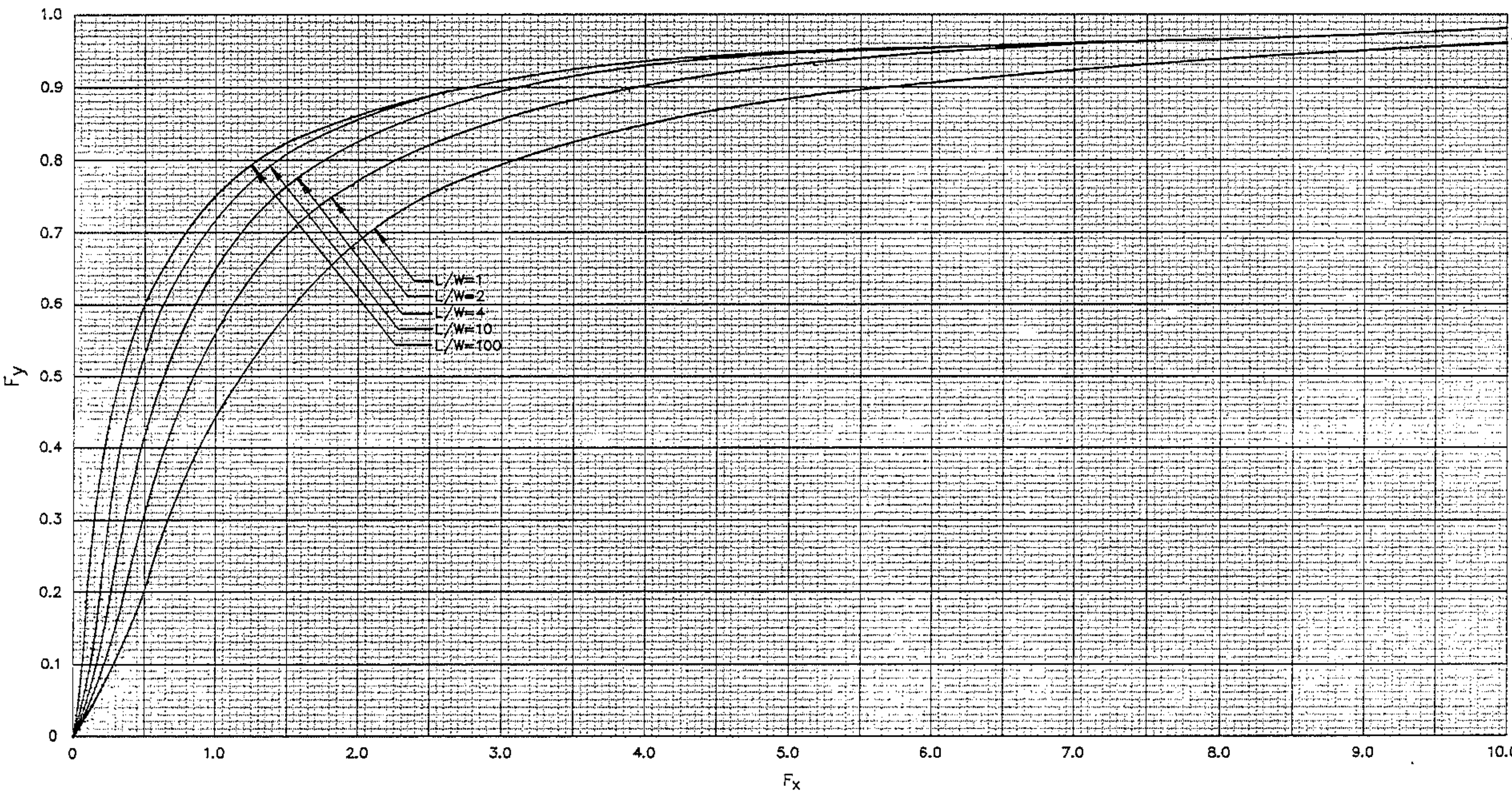
Dimensionless Curves Relating Geometric and Hydraulic Parameters
to Infiltration from Retention Ponds.

$f = 0.1$

JAMMAL & ASSOCIATES, INC.

Figure 4–9a

RECOVERY



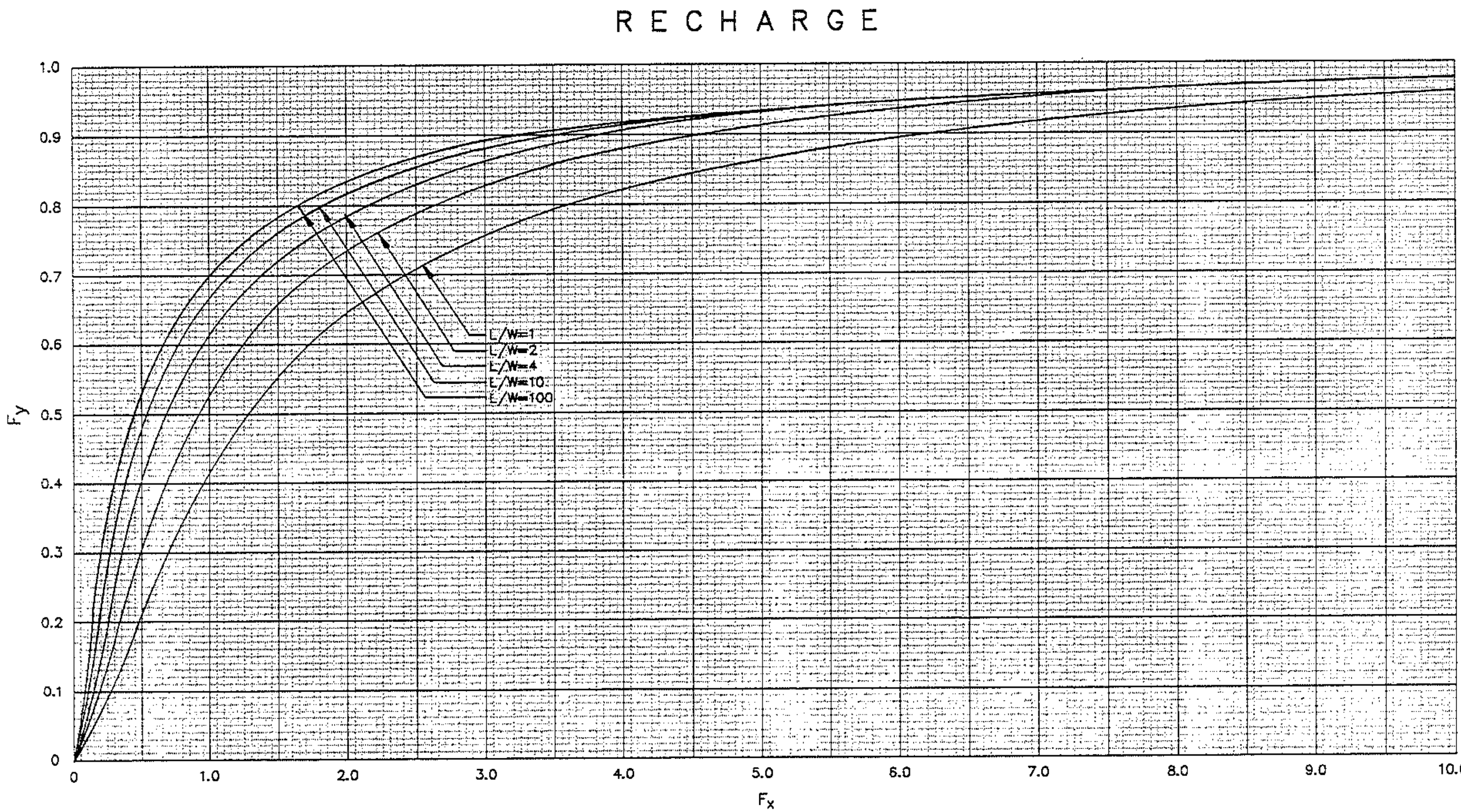
$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

f = 0.1

JAMMAL & ASSOCIATES, INC.

Figure 4–9b



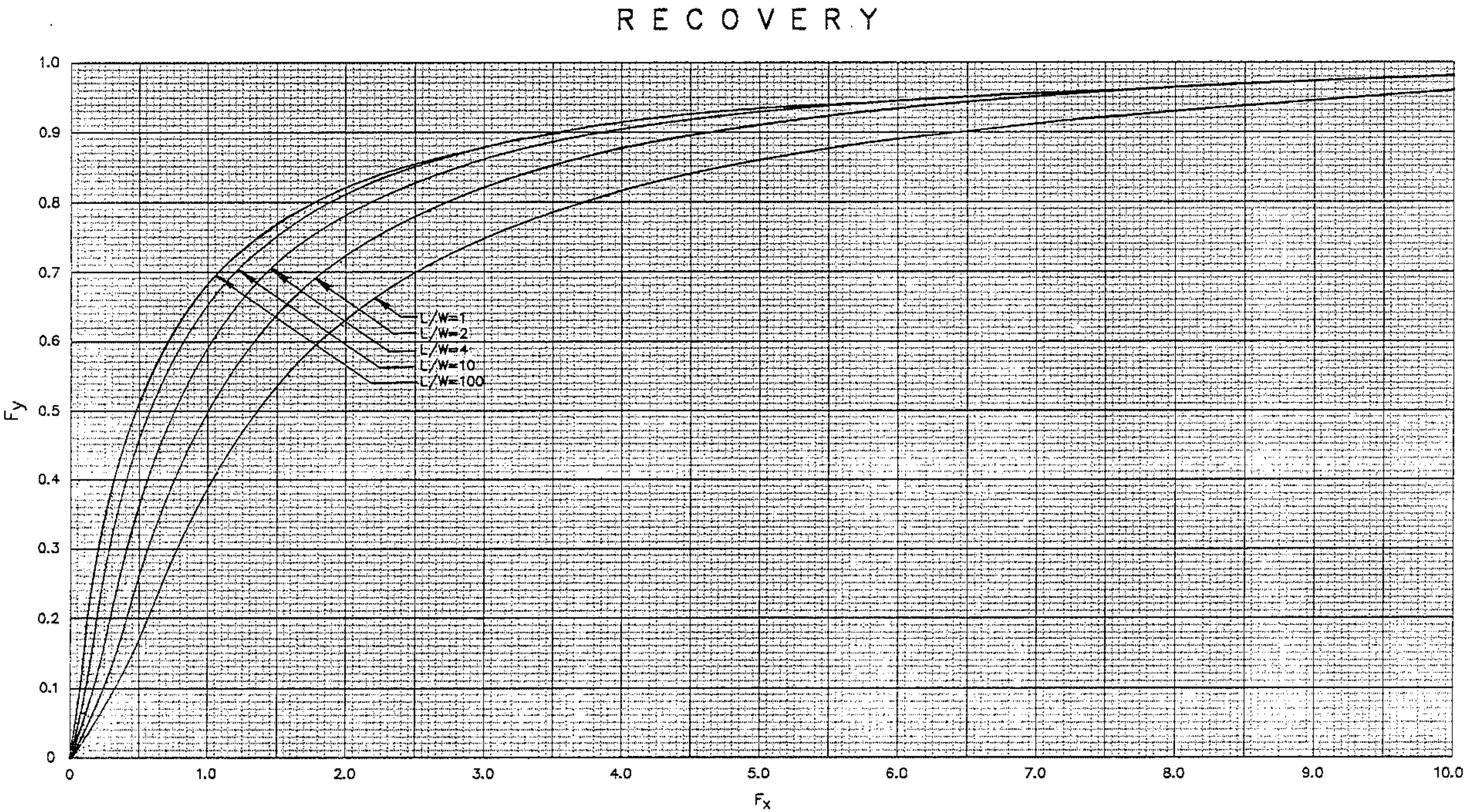
$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

f = 0.2

JAMMAL & ASSOCIATES, INC.

Figure 4–10a



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

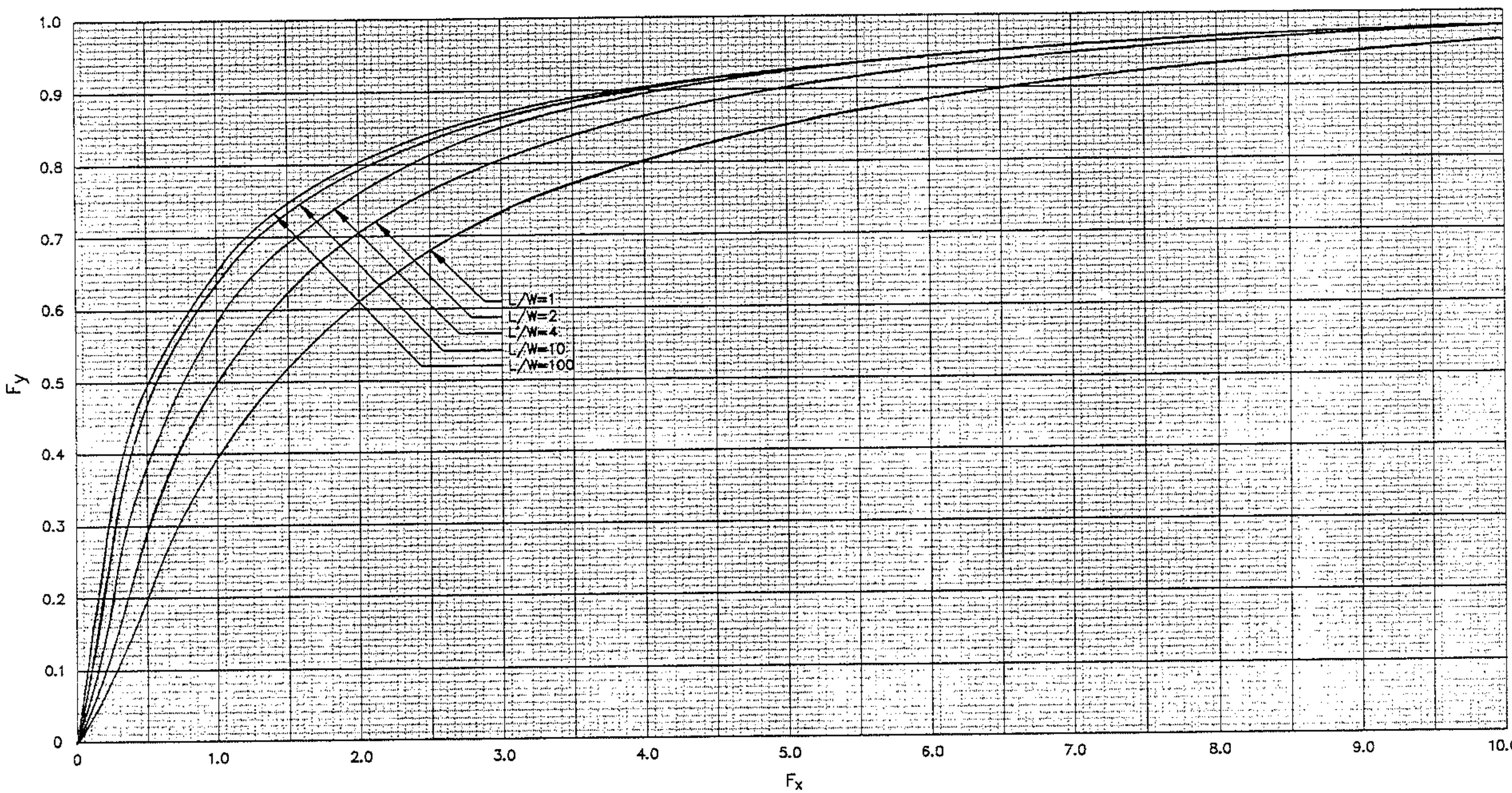
Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

f = 0.2

JAMMAL & ASSOCIATES, INC.

Figure 4–10b

RECHARGE



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

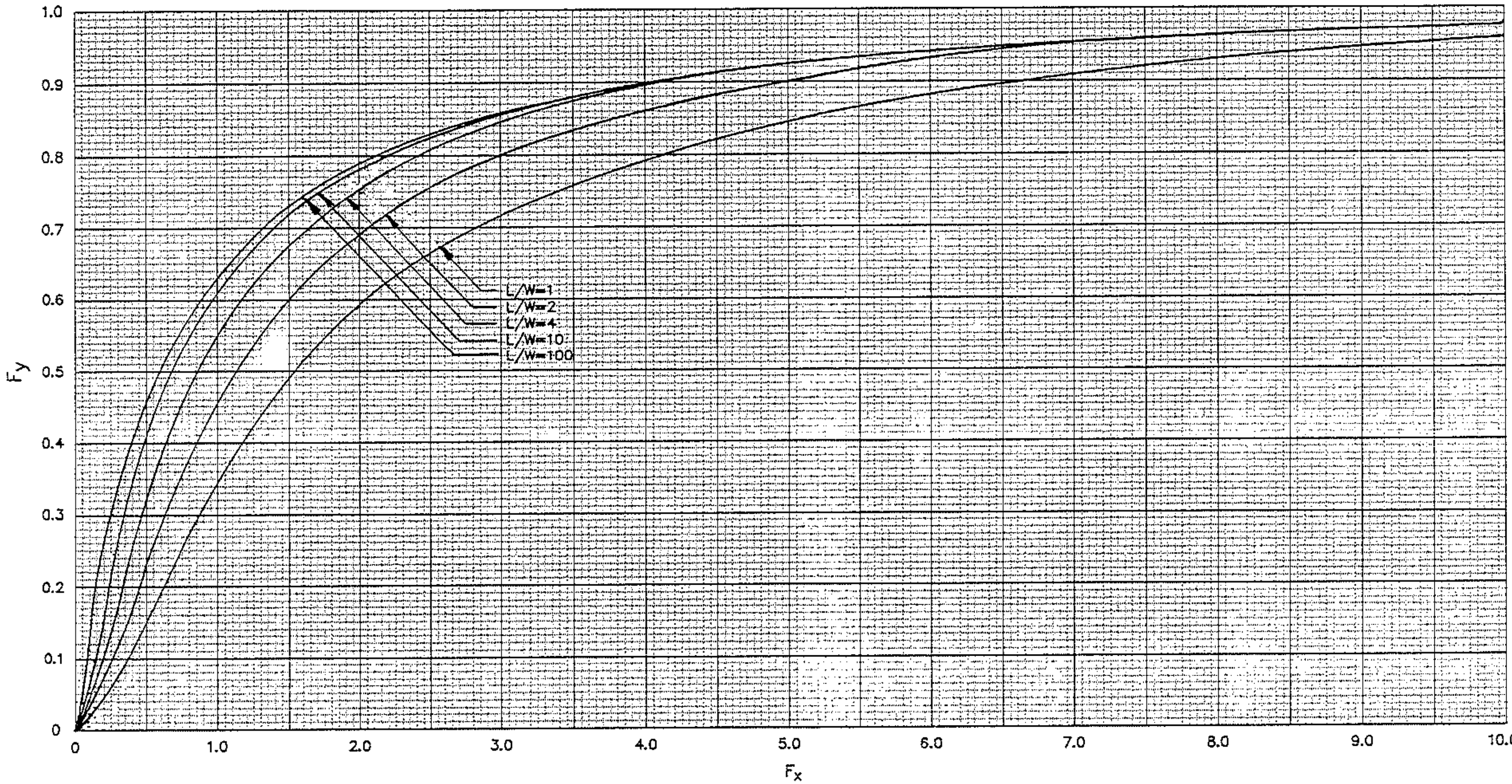
Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

$f = 0.3$

JAMMAL & ASSOCIATES, INC.

Figure 4–11a

RECOVERY



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

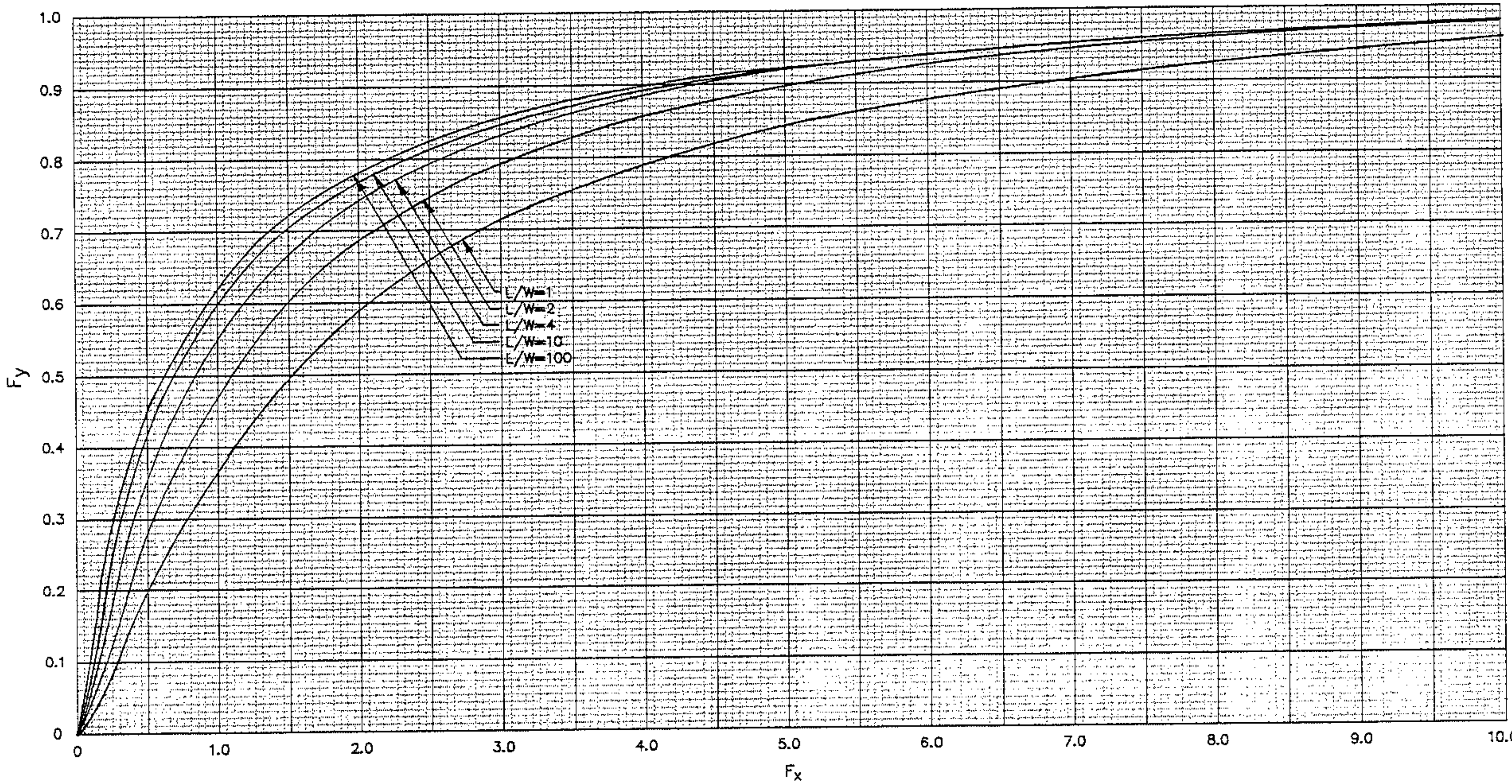
Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

$f = 0.3$

JAMMAL & ASSOCIATES, INC.

Figure 4–11b

RECHARGE



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

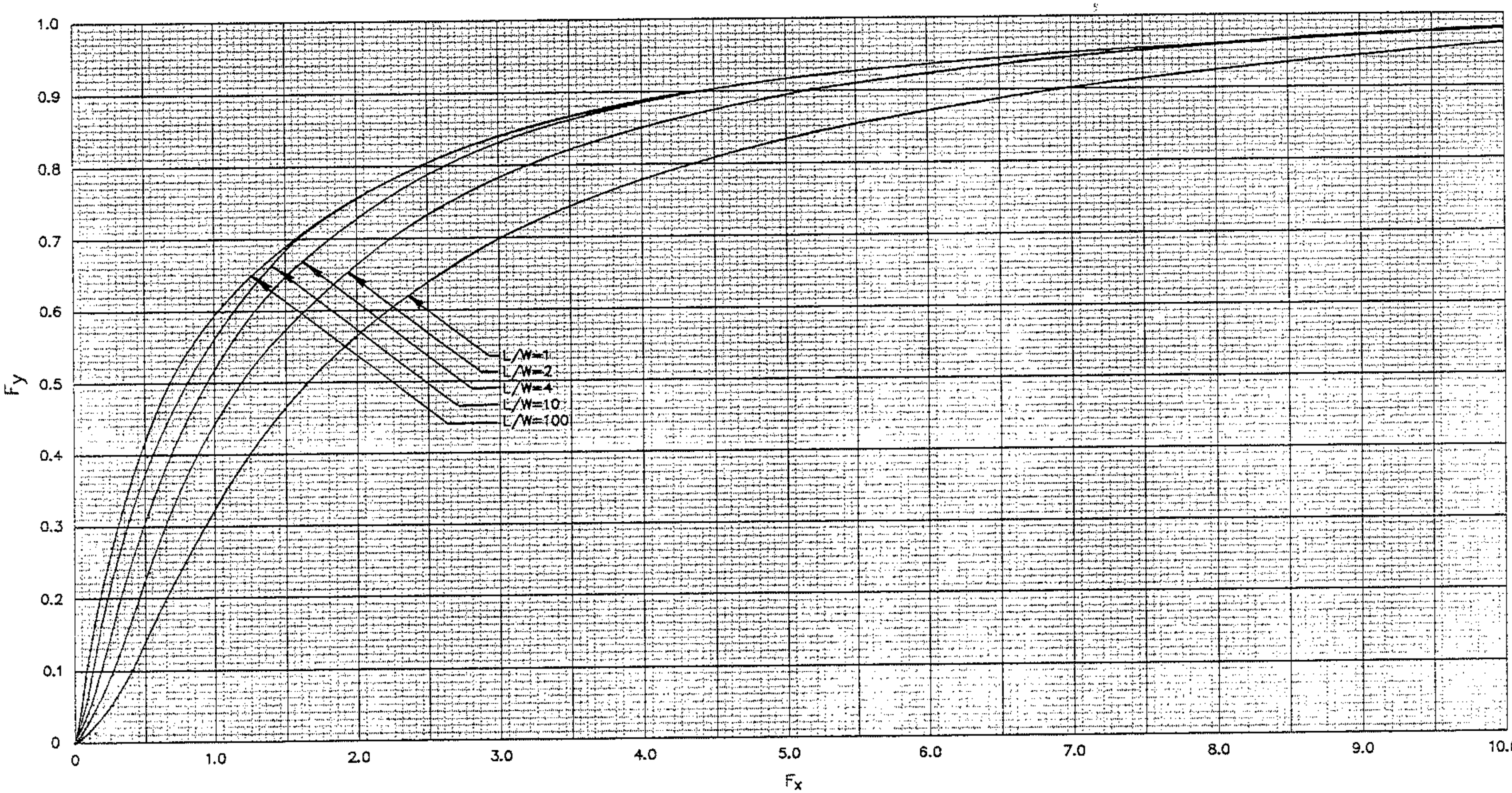
Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

$f = 0.4$

JAMMAL & ASSOCIATES, INC.

Figure 4–12a

RECOVERY



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \qquad F_y = \frac{h_c}{H_T}$$

Dimensionless Curves Relating Geometric and Hydraulic Parameters to Infiltration from Retention Ponds.

$f = 0.4$

JAMMAL & ASSOCIATES, INC.

Figure 4–12b

One of the limitations of this MODFLOW model is that it can not account for the unsaturated lateral flow under high hydraulic gradients (typical condition during storm event), and as a result the model underestimates the total infiltration capacity of a retention pond. Given the conservatism incorporated in the overall analytical approach presented in this report, the factor of safety of 2.0 for retention volume recovery, which is required by the regulatory agencies, should not be applied.

The factor of safety of 2.0 is already incorporated into the vertical infiltration rate equation. The factor of safety is important for the vertical infiltration to account for potential long term clogging due to the effects of sediment accumulation.

The horizontal component of infiltration is not affected by the clogging of the pond bottom. Therefore, when using this analytical approach to estimate retention volume recovery, the total recovery time (i.e. 72 hours) should be used instead of the typical half time (i.e. 36 hours).

CHAPTER 5

Example Problems

Chapter 5
Example Problems

To demonstrate the effectiveness of the analytical methodology developed in this report and the versatility of utilizing the generated design curves of **Figures 4-9 through 4-12,** we present herein five example problems. The example problems utilize variable pond configurations, aquifer characterisitcs, depth to groundwater table, varied design criteria and design storm events. The first example problem starts from pond design planning phases and is carried through field investigation, laboratory testing, data reduction, aquifer characterization, equivalent pond configuration selection, summarizing required data of surface stormwater runoff and step-by-step analyses to size the retention pond and select the pond bottom elevation. The second example problem presents an analysis of a circular retention pond, where the pond is already designed and requires a final check for its adequacy to store and infiltrate the runoff volume for a design storm event. The third example problem deals with a long and narrow "swale type" retention system. The fourth example problem incorporates a retention system analysis to estimate maximum design water level. And the last example problem demonstrates the use of the recommended analytical approach to design an underground exfiltration trench system.

Example 1

Proposed Retention Pond (Given)
1. Allocated area as presented on **Figure 5-1,** approximately 0.6 acres
2. Estimated runoff volume, V_{ro} = 1.40 acre-feet, see hydrograph, **Figure 5-2**
3. Approximate pond depth = 3 feet
4. Design high water level of pond = 80 feet NGVD (maximum)
5. Storm event is 100 year-24 hour
6. There is no outfall from retention pond (100% retention)

Required Field Testing

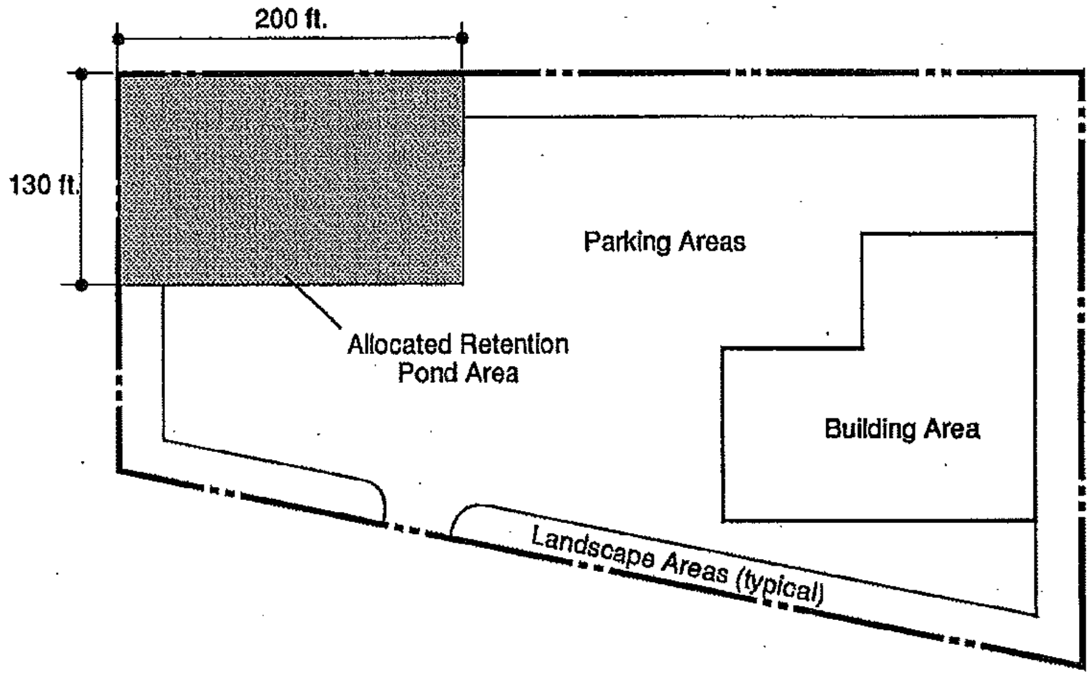
1. Estimate number of borings to be drilled. From **Chapter 3, Equation 3-1:**

$$B = 1 + \sqrt{2A} + \frac{L}{2\pi W}$$

Where:

B = Number of borings required
A = Area to be investigated (0.6 acres)
L = Length of area "A"(200 feet)
W = Width of area "A"(130 feet)

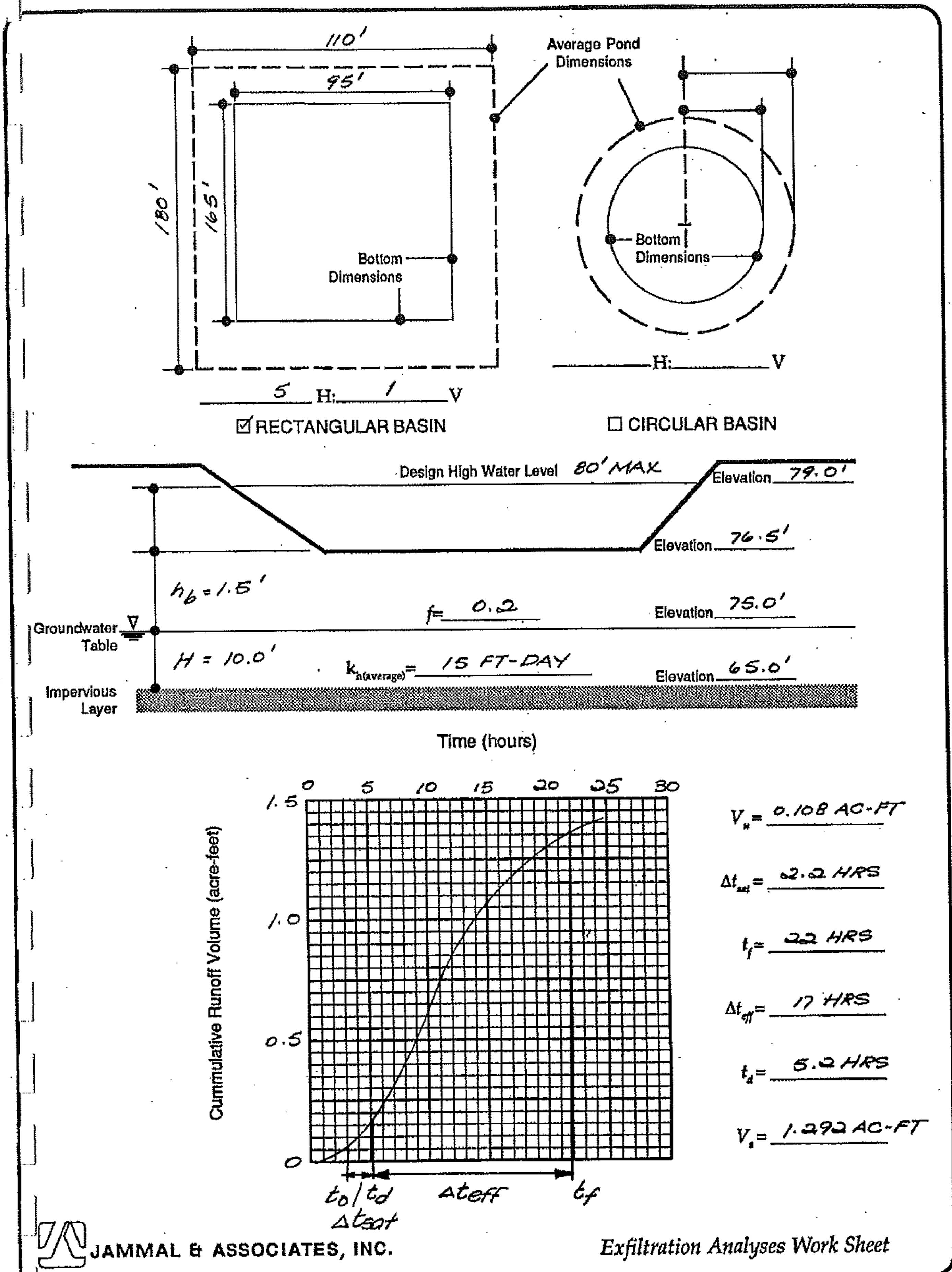
$$B = 1 + \sqrt{2(0.6)} + \frac{200}{2\pi(130)} = 2.44 \text{ borings}$$



Site Plan-Example Problem No. 1

JAMMAL & ASSOCIATES, INC.

Figure 5-1



110'
95'
180'
165'
Average Pond Dimensions
Bottom Dimensions
5 H: 1 V
☑ RECTANGULAR BASIN
Bottom Dimensions
H: V
☐ CIRCULAR BASIN
Design High Water Level 80' MAX
Elevation 79.0'
Elevation 76.5'
h_b = 1.5'
f = 0.2
Elevation 75.0'
Groundwater Table
H = 10.0'
$k_{h(average)}$ = 15 FT-DAY
Elevation 65.0'
Impervious Layer
Time (hours)
0 5 10 15 20 25 30
Cummulative Runoff Volume (acre-feet)
1.5
1.0
0.5
0
t_0 | t_d
Δt_{sat}
Δt_{eff}
t_f
V_u = 0.108 AC-FT
Δt_{sat} = 2.2 HRS
t_f = 22 HRS
Δt_{eff} = 17 HRS
t_d = 5.2 HRS
V_s = 1.292 AC-FT
JAMMAL & ASSOCIATES, INC.
Exfiltration Analyses Work Sheet

Figure 5-2

Suggest drilling 2 borings
Drill 1 standard penetration test boring to 15 feet
Drill 1 auger boring to 15 feet

2. Estimate number of hydraulic conductivity tests (P)
   Also from **Chapter 3, equation 3-2:**

$$P = 1 + \frac{B}{4} = 1 + \frac{2}{4} = 1.5 \text{ tests}$$

Suggest conducting 2 hydraulic conductivity tests

3. Measure depth and elevation of stabilized (24-hour minimum) groundwater table.
4. Estimate normal wet season or post-development wet season groundwater table (depth and elevation).

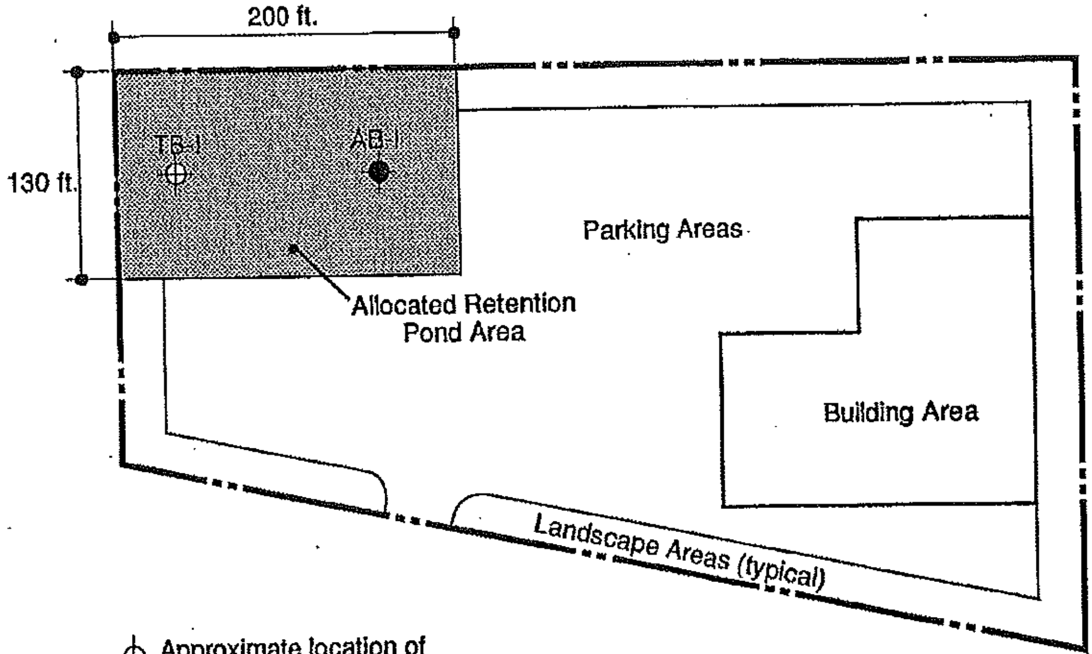
Results of Field Exploration

1. **Figure 5-3** presents the soil profiles and pertinent aquifer parameters.
2. The effective aquifer system was characterized by a geotechnical engineer as follows:

Estimated elevation of top of confining layer = 65 feet NGVD
Estimated normal high groundwater table =75 feet NGVD
Average horizontal hydraulic conductivity of the effective aquifer =15 feet/day
Estimated effective storage coefficient = 0.2
Vertical hydraulic conductivity =10 feet/day

Design Criteria
1. Design optimum size of pond and elevation of pond bottom.
2. Calculate retention volume recovery time (100% recovery) after storm event.
3. Design retention pond as a "dry pond bottom" system.
4. Pond side slopes = 5H : 1V.

Analytical Approach
1. Use modified Green & Ampt Equation for unsaturated infiltration calculations.
2. Use dimensionless curves of **Figure 4-10** (effective storage coefficient of 0.2) for saturated infiltration rate estimates during and after the storm event.

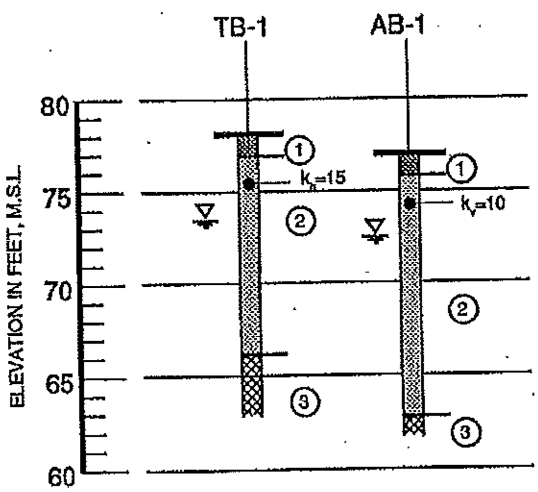


Approximate location of standard penetration test boring

Approximate location of auger boring

LOCATION PLAN

NTS



SOIL PROFILES

NTS

LEGEND

1 Topsoil

2 Light brown to brown fine sand

3 Brown clayey sand

Depth to groundwater level: Dec. 20, 1988

k_h Horizontal coefficient of permeability in feet/day

k_v Vertical coefficient of permeability in feet/day

Location Plan and Soil Profiles for Example Problem No. 1

JAMMAL & ASSOCIATES, INC.

Computations

Step 1

Select a trial average pond size of 180 feet by 110 feet.
Select a trial pond bottom at elevation 76.5 feet NGVD.
DHWL (try) = 79 ft NGVD
For side slopes of 5H:1V, the pond bottom is 165 feet by 95 feet.
See **Figure 5-2** for selected dimensions.

Step 2

Calculate infiltration volume and time for unsatruated flow, using equation **4-9** on page **4-6.**

$$I_d = \frac{K_{vu}}{FS}$$

$$K_{vu} \approx \frac{2}{3} K_{vs} = \frac{2}{3} \text{ (10 ft/day)} = 6.7 \text{ ft/day}$$

For design, use factor of safety of 2.0 to allow for siltation/sediment clogging.

$$I_d = \frac{6.7}{2.0} = 3.3 \text{ ft/day}$$

Time to saturate below pond bottom can be calculated using equation **4-10** on page **4-6.**

$$\Delta t_{sat} = \frac{h_b f}{I_d} = \frac{1.5\text{ft}(0.2)}{3.3 \text{ ft/day}}$$

$$\Delta t_{sat} = 0.09 \text{ days} = 2.2 \text{ hours}$$

Total infiltration capacity during unsaturated flow, V_u, is :

$V_u = A_b h_b f$
$A_b =$ Area of pond bottom
$h_b =$ Separation distance between high groundwater table and pond bottom
$f =$ Average effective storage of soil above groundwater table.
$V_u =$ (165 ft)(95 ft)(1.5 ft)(0.2)
$V_u =$ 4,702.5 ft^3 = 0.108 acre-feet

Step 3

Calculate infiltration volume, V_I, and direct storage volume, V_D in the pond during the saturated flow and mounding (during storm event).

Use dimensionless curves of **Figure 4-10** for f=0.2.

First calculate the length over width ratio of pond.

$$\frac{L}{W} = \frac{180}{110} = 1.64$$

Interpolate between

$$\frac{L}{W} = 1.0 \text{ and } \frac{L}{W} = 2.0$$

Calculate dimensionless parameter, F_x.

$$F_x = \left(\frac{W^2}{4K_H Dt}\right)^{1/2}$$

W= average pond width 110 feet

K_H= average horizontal hydraulic conductivity 15 feet/day

D= average saturated thickness of aquifer.

If the assumed DHWL of 79 feet NGVD is correct, D = initial saturated thickness of aquifer plus 1/2 of distance between groundwater table and DHWL (10 feet + 4 feet / 2) = 12 feet. **See Figure 5-2.**

t= effective time for saturated flow infiltration. Subtracting time for unsaturated infiltration, Δt_{sat}, and ineffective time at the beginning and end of storm, the effective time is the increment between t_d and t_f, Δt_{eff}= 0.71 days on **Figure 5-2.**

Therefore,

$$F_x = \left(\frac{110^2}{4(15)(12)(0.71)}\right)^{1/2} = 4.86$$

Using **Figure 4-10** and interpolating between curves of L/W=1.0 and L/W=2.0, estimate the value of F_y.

$$F_y \approx \frac{h_c}{H_T} = 0.89$$

Where,

h_c= height of water level in pond above initial groundwater table as a result of infiltration.

H_T= Height of water level in pond above initial groundwater table if no infil tration occured.

Also,

H_T= $h_b + h_v$

h_b= separation distance between pond bottom and initial groundwater table (1.5 feet)

h_v= height of water level in pond above pond bottom, in no infiltration occured.

The value of h_v can be calculated from the runoff volume as follows:
Total runoff volume V_{ro}=1.4 acre feet.
Volume loss due to unsaturated infiltration,

V_u= 0.108 acre feet.

Therefore the remaining runoff volume during saturated infiltration is:

$V_s = V_{ro} - V_u = 1.4 - 0.108$

V_s= 1.292 acre feet

Dividing V_s by average pond area, A_a will produce the value of h_v.

$$h_v = \frac{V_s}{A_a}$$

$$A_a = \frac{(180\text{ ft})(110\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}} = 0.454\text{ acres}$$

$$h_v = \frac{1.292\text{ acre-feet}}{0.454\text{ cares}} = 2.84\text{ feet}$$

Therefore,

H_T= 1.5 feet+2.84 feet=4.34 feet

And,

$h_c = F_y H_T$=0.89(4.34)=3.86 feet

Step 4

1. Compare calculated DHWL of pond with assumed DHWL.
2. Calculated DHWL = initial groundwater table.(estimated high groundwater table) + h_c = 75 feet + 3.86 feet = 78.86 feet
3. If calculated DHWL is greater than assumed DHWL or if calculated DHWL is significantly less than assumed DHWL go to *Step 1*, adjust pond size and/or pond bottom and repeat *steps 1 through 4*.
4. If calculated DHWL ($DHWL_c$) ≈ assumed DHWL ($DHWL_a$) then go to *Step 5*.

For this example, $DHWL_c$=78.86 feet ≈ 79.0 feet (*Go to Step 5*).

Step 5

1. Calculate time of retention volume recovery.

At the time when pond has just recovered.

$h_c = h_b$=1.5 feet

H_T= 4.34 feet (same, since no additional water was added to the pond)

$$F_y = \frac{h_c}{H_T} = \frac{1.5\text{ ft}}{4.34\text{ ft}} = 0.345$$

Using **Figure 4-10**, for F_y=0.345 and L/W=1.64 determine the value of F_x.

$$F_x=0.83=\left(\frac{W^2}{4K_H Dt}\right)^{1/2}$$

Rearrange the equation to solve for t.

$$t=\frac{W^2}{4K_H DF_x^2}$$

Where,

W= 110 feet (same average pond width)
K_H= 15 ft/day
D= 10 Ft+1.5/2=10.75 ft
F_x^2= 0.83^2=0.686

$$t=\frac{110^2}{4(15)(10.75)(0.689)}=27.2 \text{ days}$$

Time of retention volume recovery, t_r, can be calculated by subtracting the final time of the stormwater runoff, t_f, (**Figure 5-2**).

t_r= $t-t_f$=27.4 days - 0.9 days
t_r= 26.3 days.

Step 6
1. Compare calculated volume recovery time, t_r, with the required design volume recovery time, t_{rd}.
2. If $t_r > t_{rd}$ then go to *Step 1*. Modify pond (raise pond bottom, decrease pond stage and/or increase L/W ratio of pond) and redo *Steps 1 through 6*.
3. If $t_r < t_{rd}$, then the selected pond configuration and design is adequate.

End of Example 1.

Example 2

Given:
1. Circular Pond
150 feet diameter at pond bottom
200 feet diameter at DHWL
Side slopes 5H:1V
Design stage of pond is 5 feet.

2. Geotechnical report indicates:
Impervious layers at elevation 22 feet NGVD
Estimated high groundwater table at elevation 60 feet NGVD
Average horizontal hydraulic conductivity 20 ft/day
Vertical hydraulic conductivity 18 ft/day
Average ground surface elevation in the pond area 75 ft NGVD
Estimated effective storage coefficient = 0.3

3. Civil Engineer calculates total runoff volume for a 25 year - 24 hour storm at 3.0 acre-feet. Drainage design for the development indicates that the design high water level of the pond should be at elevation 74 feet NGVD.
Runoff hydrograph is presented on **Figure 5-4.**

Required:
1. The pond is to retain 100% of the runoff volume and recover in 3 days after the storm event.
2. Calculate storage and infiltration capacity of the pond during and after the storm event. Check if the pond size is adequate and check if the pond recovers within the specified time period.

Select Design Parameters:
Use **Figure 5-4** as a working sheet for parameter dimensioning.

Calculate pond area at pond bottom.

$$A_b = \frac{\pi}{4}(150)^2 = 17{,}671 \text{ ft}^2 = 0.405 \text{ acres}$$

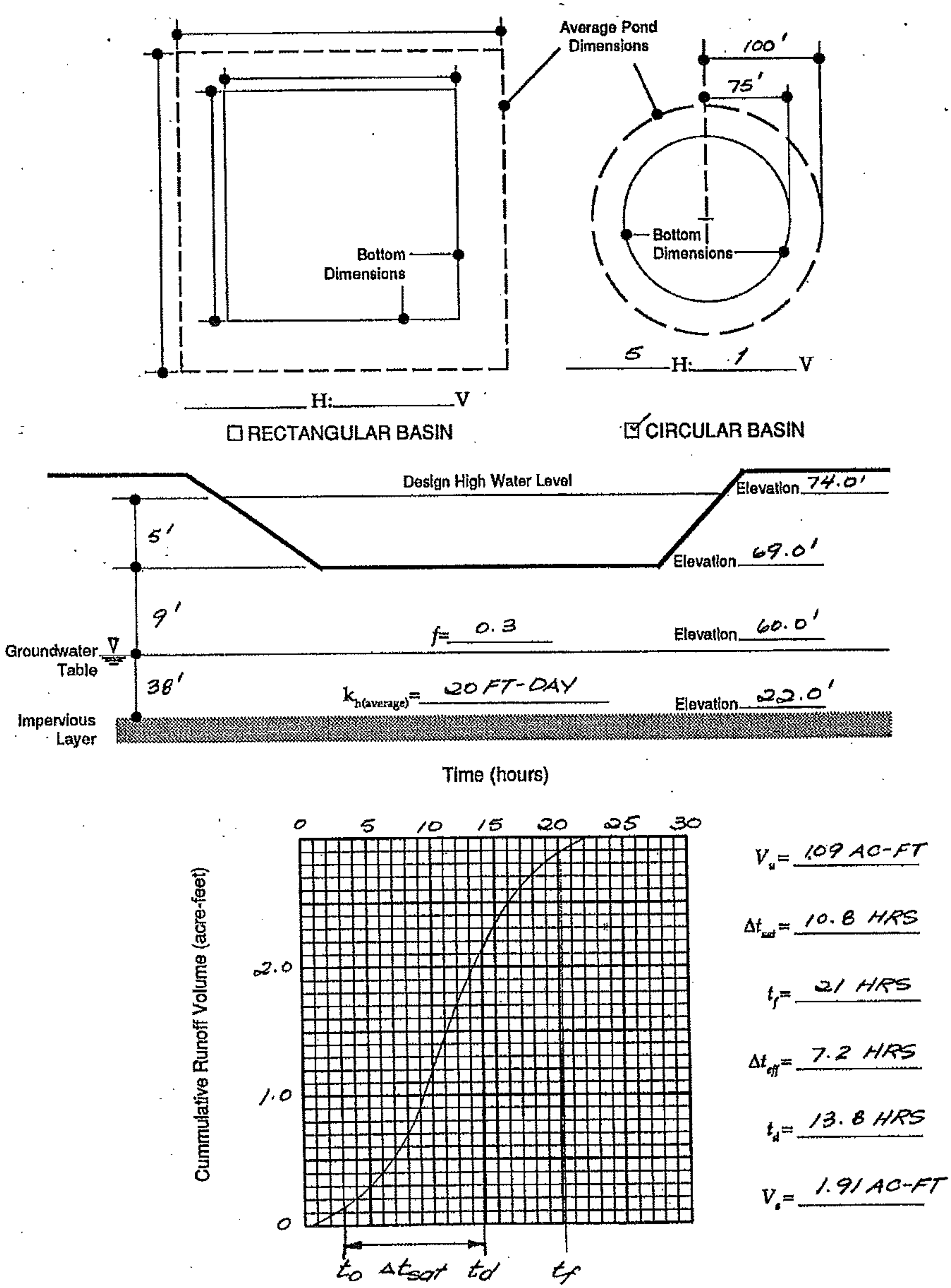
Calculate pond area at DHWL:

$$A_h = \frac{\pi}{4}(200)^2 = 31{,}416 \text{ ft}^2 = 0.721 \text{ acres}$$

The average pond area is:

$$A_a = \frac{A_b + A_h}{2} = \frac{17{,}671 + 31{,}416}{2} = 24{,}542 \text{ ft}^2$$

$A_a =$ 0.563 acres
DHWL = 74 feet NGVD
Pond bottom = 69 feet NGVD



JAMMAL & ASSOCIATES, INC.

Exfiltration Analyses Work Sheet

Figure 5-4

Computations:

Step 1
Calculate unsaturated infiltration rate, I_d.

$$K_{vs} = 18 \text{ ft/day}$$

$$K_{vu} = \frac{2}{3}K_{vs} = \frac{2}{3}(18) = 12 \text{ ft/day}$$

Apply a factor safety of 2.0.

$$I_d = \frac{K_{vu}}{FS} = \frac{12 \text{ ft/day}}{2.0}$$

$$I_d = 6 \text{ ft/day}$$

Calculate time to saturate soil below pond bottom, Δt_{sat}:

$$\Delta t_{sat} = \frac{h_b f}{I_d}$$

h_b = separation between pond bottom and high groundwater table.
h_b = 69 ft-60 ft = 9 ft
f = effective storage above groundwater table
f = 0.3
I_d = design unsaturated infiltration rate
I_d = 6 ft/day

$$\Delta t_{sat} = \frac{9\text{ft}(0.3)}{6\text{ft/day}} = 0.45 \text{ days} = 10.8 \text{ hours}$$

Calculate volume of water infiltrated during unsaturated flow, V_u:

$V_u = h_b f A_b$=9 ft(0.3)(0.405 acres)
V_u = 1.09 acre-ft

Step 2
In this example we will calculate the potential infiltration and storage volume given the design high water level, instead of calculating the height of water level rise due to runoff volume. The results of this calculated infiltration and storage volume will then be compared to the actual runoff volume.

For f= 0.3, use Figure 4-11 for saturated infiltration analysis.

$$F_x = \left(\frac{W^2}{4K_H D\, t}\right)^{1/2}$$

W = equivalent pond width
$W = \sqrt{A_a} = \sqrt{24{,}543} = 157$ ft

K_H= average horizontal hydraulic conductivity of the effective aquifer.
K_H= 20 ft/day

D= average saturated thickness during groundwater mounding

$$D = H + \frac{h_c}{2}$$

H= initial saturated thickness = 60 ft - 22 ft = 38 ft

h_c= height of water level in pond above initial groundwater table = 74 ft - 60 ft = 14 ft

$$D = 38 + \frac{14}{2} = 45 \text{ ft}$$

Δt_{eff}= effective time increment for saturated infiltration from the time infiltrating water reaches the groundwater table to the time identified by t_f.

Δt_{eff}= 7.2 hrs = 0.3 days

Therefore,

$$F_x = \left(\frac{157^2}{4(20)(45)(0.3)}\right)^{1/2} = 4.78$$

Using Figure 4-11 and the dimesionless curve for L/W=1.0, the value of F_y was estimated.

F_y= 0.853

Also,

$$F_y = \frac{h_c}{H_T} = 0.853$$

$$H_T = \frac{h_c}{F_y} = \frac{14 \text{ ft}}{0.853} = 16.41 \text{ ft}$$

h_c= 14 feet (previously calculated)
H_T= $h_b + h_v$
h_b= 9 ft (previously calculated)
h_v= $h_s + h_I$
h_s= height of DHWL above pond bottom
h_s= $h_c - h_b$ = 14 ft - 9 ft = 5 ft
h_I= equivalent height of infiltrated water during saturated infiltration time, Δt_{eff}.
h_v = $H_T - h_b$= 16.41 ft - 9 ft = 7.41 ft

h_I= $h_v - h_s$= 7.41 ft - 5 ft = 2.41 ft

Calculate the potential volume of infiltration from pond during saturated iniftration time, Δt_{eff}.

$$V_I = h_I A_a = 2.41 \text{ ft } (0.563 \text{ acres}) = 1.36 \text{ acre-ft}$$

The average potential infiltration rate can be approximated by :

$$I_s = \frac{h_I}{\Delta t_{eff}} = \frac{2.41 \text{ ft}}{7.2 \text{ hrs}} = 0.335 \text{ ft/hr} = 4.02 \text{ inches/hr}$$

Calculate the direct storage volume of the pond.

$$V_D = h_s A_a = 5 \text{ ft } (0.563 \text{ acres}) = 2.82 \text{ acre-ft}$$

The total storage and infiltration capacity of pond during saturated infiltration is:

$$V_s = V_I + V_D = 1.36 \text{ acre-ft} + 2.82 \text{ acre-ft}$$

$$V_s = 4.18 \text{ acre-ft}$$

The total storage and infiltration capacity of pond during unsaturated and saturated infiltration is:

$$V_T = V_u + V_s = 1.09 \text{ acre-ft} + 4.18 \text{ acre-ft} = 5.27 \text{ acre-ft}$$

Check if total storage and infiltration capacity of pond is sufficient to handle the design stormwater runoff volume.

The stormwater runoff volume, V_{ro}=1.40 acre-ft

$$V_T = 5.27 \text{ acre-ft}$$
$$V_T > V_{ro}$$

Therefore, the pond is more than adequatly sized to handle the design runoff volume.

Step 3

Calculate time of pond volume recovery. Assume pond was full at end of storm event or at elevation 74 feet NGVD.

$$F_x = \left(\frac{W^2}{4K_H D\, t}\right)^{1/2}$$

All parameters are the same except for D and t

$$D = H + \frac{h_b}{2} = 38\text{ ft} + \frac{9\text{ ft}}{2} = 42.5\text{ ft}$$

t= To be calculated. Total time increment during saturated infiltration for water level to receed to the pond bottom level.

Therefore, first calculate, F_y, then obtain a value for , F_x. "t" can then be calculated from F_x.

$$F_y = \frac{h_c}{H_T}$$

For recovery, $h_c = h_b = 9$ ft

H_T = 16.41 ft(same)

$$F_y = \frac{9\text{ ft}}{16.41\text{ ft}} = 0.548$$

From **Figure 4-11** and for dimensionless curve of L/W=1.0:

F_x = 1.75

Solving for time, t,

$$t = \frac{W^2}{4K_H D F_x^2}$$

$$t = \frac{157^2}{4(20)(42.5)(1.75)^2} = 2.36\text{ days} = 56.8\text{ hrs}$$

The calculated time, t, represents the time increment from the time when the infiltrating water reached groundwater to the time when water level in the pond receeded to the pond bottom level, after the storm event.

Therefore, the total time of recovery, t_r, after the storm event is:

$t_r = t - (t_{ST} - t_d)$

t_{ST} = Total time of storm event (for this example t_{ST} = 24 hrs)

t_r = 56.8 hrs - (24 hrs - 13.8 hrs) = 46.6 hrs

Compare the calculated recovery time, t_r, to the specified maximum recovery time, t_{rd}, of 3 days.

$t_r < t_{rd}$

Therefore, the pond will recover the retention volume in less than the specified maximum recovery time.

End of Example 2

Example 3

Given:
A swale system is designed for stormwater retention at a site.
The swale system is approximately 200 feet long and has an average width of 5 feet.
The swale is designed to be an average 2 feet deep with a design high water level a 1.5 feet above swale invert **(Figure 5-5)**.
The geotechincal investigation report indicates the following:
Clean fine sand extending to the investigated depth of 10 feet below ground surface.
Estimated normal high groundwater table is at 3 feet below ground surface (1.0 feet below swale bottom).
Horizontal hydraulic conductivity is 12 feet per day.
Vertical hydraulic conductivity is 10 feet per day.
Effective storage coefficient is 0.2.
Design storm event is 25 year-96 hour with runoff hydrograph as presented on **Figure 5-5.**

Required:
Calculate the direct storage volume and the infiltration volume of the swale during the storm event and calculate the time of volume recovery for the swale after the storm event.

Step 1
Calculate unsaturated infiltration volume for the swale.

$$K_{vu} = \frac{2}{3} K_{vs} = \frac{2}{3}(10) = 6.7 \text{ feet/day}$$

Calculate the design infiltration rate during unsaturated infiltration. Use FS = 2.0

$$I_d = \frac{K_{vu}}{FS} = \frac{6.7}{2.0} = 3.3 \text{ feet/day}$$

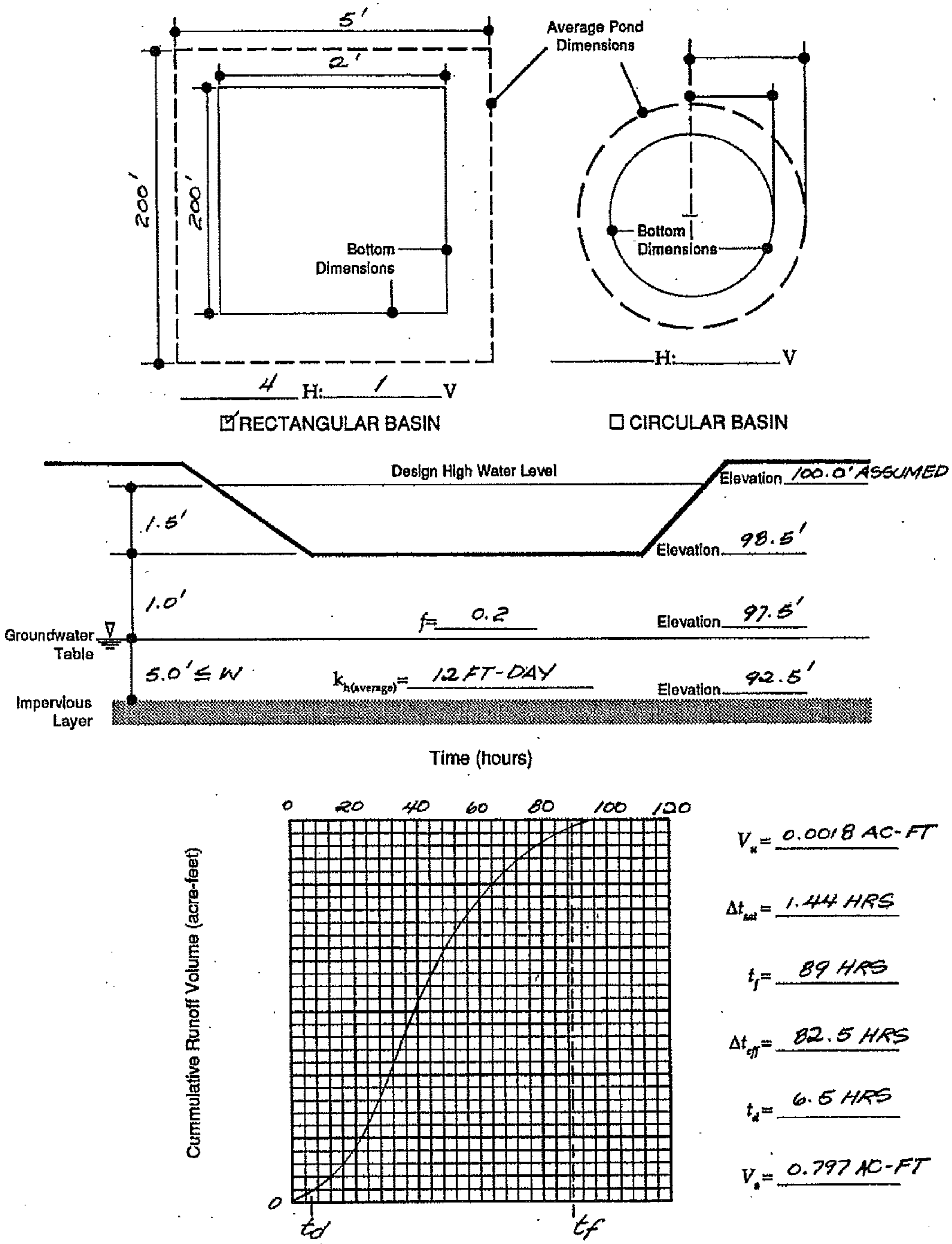
Calculate the time to saturate soil between pond bottom and groundwater table.

$$\Delta t_{sat} = \frac{h_b f}{I_d}$$

h_b = separation distance between pond bottom and groundwater table
h_b = 1.0 ft (given)

f = 0.2 (given)

Therefore,



JAMMAL & ASSOCIATES, INC.

Exfiltration Analyses Work Sheet

Figure 5-5

$$\Delta t_{sat} = \frac{1.0(0.2)}{3.3} = 0.06 \text{ days} = 1.44 \text{ hours}$$

Calculate the total infiltration volume during unsaturated infiltration.

$V_u = h_b A_b f$

$A_b =$ pond bottom area

$A_b =$ 200 ft (2.0 ft)=400 ft^2 (assuming swale bottom width of 2.0 feet)

$V_u =$ 1.0(400)(0.2) = 80 ft^3 =0.0018 acre-ft

Step 2

Calculate saturated infiltration volume and direct storage volume.

$$F_x = \left(\frac{W^2}{4K_H Dt}\right)^{1/2}$$

W= average swale width = 5 ft (given)

$K_H =$ 12 ft/day (given)

D= average saturated thickness = $H + \frac{h_c}{2}$

(Note that H ≤ W – Bouwer, 1978)

Therefore,

H = W = 5 feet

$h_c =$ height of DHWL above initial groundwater table **(Figure 5-5)**

$h_c =$ 2.5 feet (given)

$$D = 5 + \frac{2.5}{2} = 6.25 \text{ feet}$$

$t = t_f - t_d = \Delta t_{eff} = 89 \text{ hrs} - 6.5 \text{ hrs} = 82.5 \text{ hrs}$

$\Delta t_{eff} =$ 3.44 days

(See Figure 5-5)

$$F_x = \left(\frac{5^2}{4(12)(6.25)(3.44)}\right)^{1/2} = 0.156$$

Swale length to width ratio,

$$\frac{L}{W} = \frac{200 \text{ ft}}{5 \text{ ft}} = 40$$

Using **Figure 4-10**, estimate a value of F_y.

$F_y =$ 0.07

$$F_y = \frac{h_c}{H_T}$$

$$H_T = \frac{h_c}{F_y} = \frac{2.5}{0.07} = 35.71 \text{ ft}$$

$$H_T = h_b + h_v = h_b + h_I + h_s$$

$$h_I + h_s = H_T - h_b = 35.71 - 1.0 \text{ ft} = 34.71 \text{ ft}$$

The direct storage volume of swale is:

$V_D = h_s A_a$
$h_s =$ height of DHWL above swale bottom
$h_s =$ 1.5 feet (given)
$A_a =$ average area of pond
$A_a =$ 5 ft (200 ft) = 1,000 ft^2

And,

$$V_D = 1.5 \text{ ft}(1{,}000 \text{ ft}^2) = 1{,}500 \text{ ft}^3 = 0.0344 \text{ acre-ft}$$

The infiltration volume during saturated infiltration is:

$V_I = h_I A_a$
$h_I =$ equivalent height of water in pond during saturated infiltration
$h_I = H_T - h_b - h_s = 35.71 \text{ ft} - 1.0\text{ft} - 1.5 \text{ ft} = 33.21 \text{ ft}$
$V_I = 33.21 \text{ ft}(1{,}000 \text{ ft}^2) = 33210 \text{ ft}^3 = 0.7624 \text{ acre-ft}$

The average infiltration rate during the saturated infiltration time increment is:

$$I_s = \frac{h_I}{\Delta t_{eff}} = \frac{33.21 \text{ ft}}{82.5 \text{ hrs}} = 0.403 \text{ ft/hr} = 4.83 \text{ inches/hr}$$

The total storage and infiltration capacity of pond during unsaturated and saturated infiltration is:

$$V_T = V_u + V_D + V_I = 0.0018 + 0.0344 + 0.7624$$

$$V_T = 0.7986 \text{ acre-ft}$$

Step 3
Calculate time of swale volume recovery

$$F_y = \frac{h_c}{H_T}$$

For recovery $h_c = h_b$, H_T was calculated to be 35.71 ft, and

Therefore,

$$F_y = \frac{1.0}{35.71} = 0.028$$

From **Figure 4-10** and L/W = 40

$$F_x = 0.05$$

$$F_x = \left(\frac{W^2}{4K_H D\, t}\right)^{1/2}$$

$$t = \frac{W^2}{4K_H D\, F_x^2}$$

Where,

$W =$ 5 feet

$K_H =$ 12 feet/day

$$D = H + \frac{h_c}{2} = 5 + \frac{1.0}{2} = 5.5 \text{ ft}$$

$$t = \frac{5^2}{4(12)(5.5)(.05)^2} = 37.9 \text{ days} = 909 \text{ hours}$$

$t_r = t - (t_{ST} - t_d)$

$t_{ST} =$ 96 hrs for this example

$t_d = t_o + \Delta t_{sat}$ **(Figure 5-5)**

$t_r =$ 909 hours - (96 hours - 6.44 hours) = 819.44 hours

Summary

$V_I =$ 1.08 ac-ft (total infiltration volume during storm event)

$V_D =$ 0.0344 ac-ft (total direct storage volume of swale)

$t_r =$ 819 hours (time of retention volume recovery of the swale, after the storm event)

End of Example 3

Example 4

Given:

Land Locked retention pond system
(There is no positive outfall from pond)
Design stormwater runoff hydrograph is presented on **Figure 5-6**
(100-year, 24-hour storm)
Retention pond dimensions are also presented on **Figures 5-6**

Site soil and groundwater conditions are as follows:

1. Elevation of impervious soil layers 20 feet NGVD
2. Estimated normal high groundwater table 40 feet NGVD
3. Average horizontal hydraulic conductivity 15 feet/day
4. Vertical hydraulic conductivity 9 feet/day
5. Proposed pond bottom elevation 43 feet NGVD
6. Average effective storage 0.2

Required:

1. Estimate design high water level (DHWL) for the proposed retention pond.
2. Estimate total direct storage volume in the pond.
3. Estimate total infiltration volume during the storm event (unsaturated and saturated infiltration).

Calculations:

Step 1

Calculate the infiltration volume and infiltration rate during unsaturated infiltration.

Volume of unsaturated infiltration, V_u, is:

$V_u = A_b H_b f$

$A_b =$ area of pond bottom

$A_b = 100 \text{ ft} \times 400 \text{ ft} = 40{,}000 \text{ ft}^2$

$h_b =$ vertical separation between pond bottom and groundwater table

$h_b = 43 \text{ ft} - 40 \text{ ft} = 3 \text{ ft}$

$f =$ effective storage coefficient

$f = 0.2$ (given)

$V_u = 40{,}000 \text{ ft}^2 (3 \text{ ft}) (0.2) = 24{,}000 \text{ ft}^3$

$V_u = 0.55$ acre-feet

The average design infiltration rate, I_d is:

$$I_d = \frac{K_{vu}}{FS}$$

K_{vu} = unsaturated vertical hydraulic conductivity

$$K_{vu} = \frac{2}{3} K_{vs} = \frac{2}{3} (9 \text{ ft/day}) = 6 \text{ ft/day}$$

FS = Factor of safety

FS = 2.0 (recommended)

$$I_d = \frac{6.0}{2.0} = 3.0 \text{ ft/day}$$

The total time to saturate soil below pond bottom, Δt_{sat}, is:

$$\Delta t_{sat} = \frac{h_d f}{I_d} = \frac{3 \text{ ft}(0.2)}{3 \text{ ft/day}} = 0.2 \text{ days}$$

$$\Delta t_{sat} = 0.2 \text{ days} \left(\frac{24 \text{ hrs}}{1 \text{ day}} \right) = 4.8 \text{ hrs}$$

Step 2

Calculate design high water level (DHWL) of the pond.

To conservatively characterize the runoff hydrograph, assume zero (0) runoff during initial phases and final phases, as presented by t_o and t_f on **Figure 5-6**.

Subtract the unsaturated infiltration volume, V_u, from the initial part of the hydrograph as shown on **Figure 5-6.**

Subdivide the remaining hydrograph into 4 selected segments **(Figure 5-6)**. These segments can be selected based on the characteristics of the runoff hydrograph.

Calculate cummulative runoff volume for each segment with corresponding effective time, Δt_{eff} **(Figure 5-6)**.

Using the dimensionless infiltration curves of **Figure 4-10,**

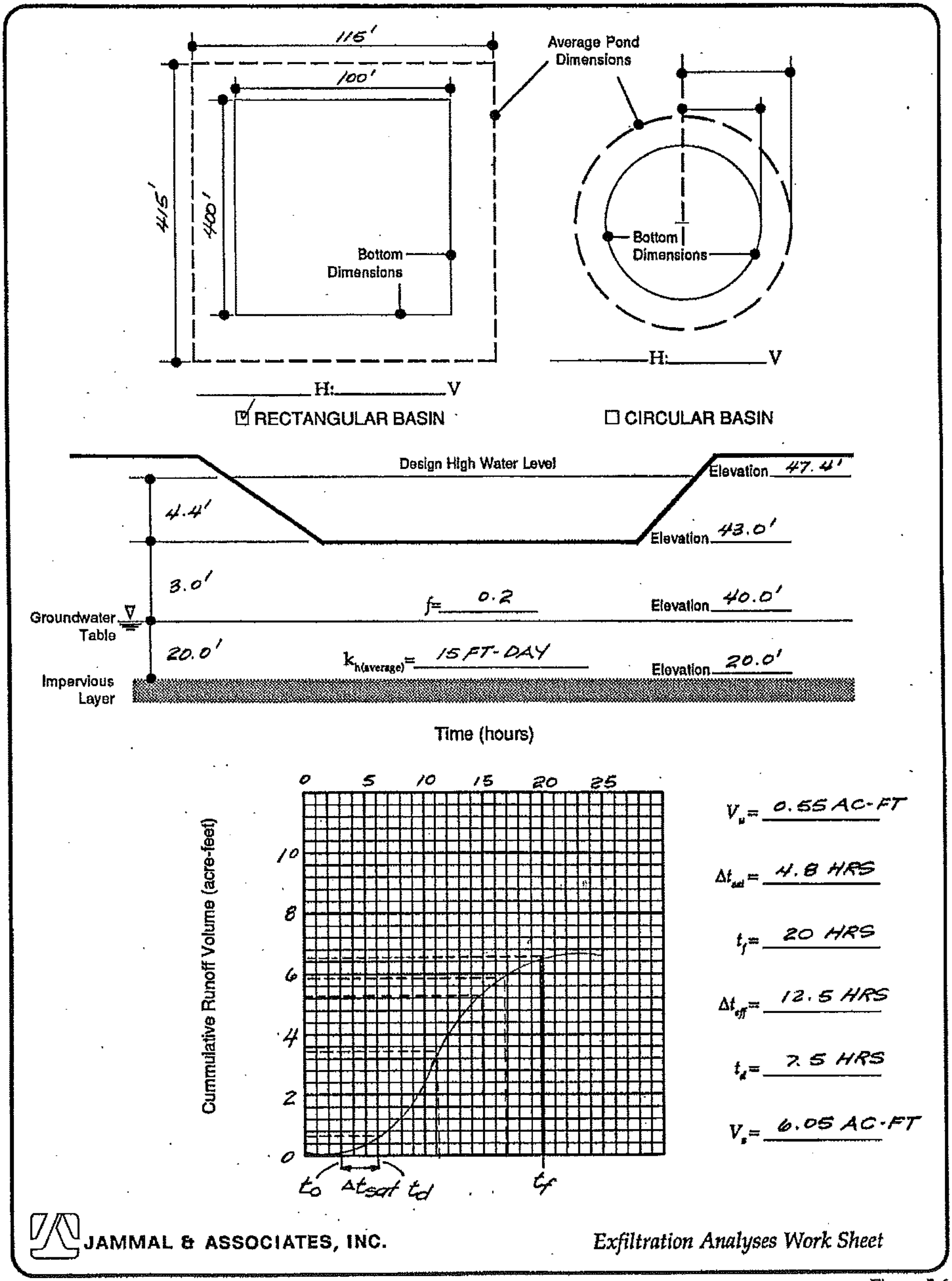
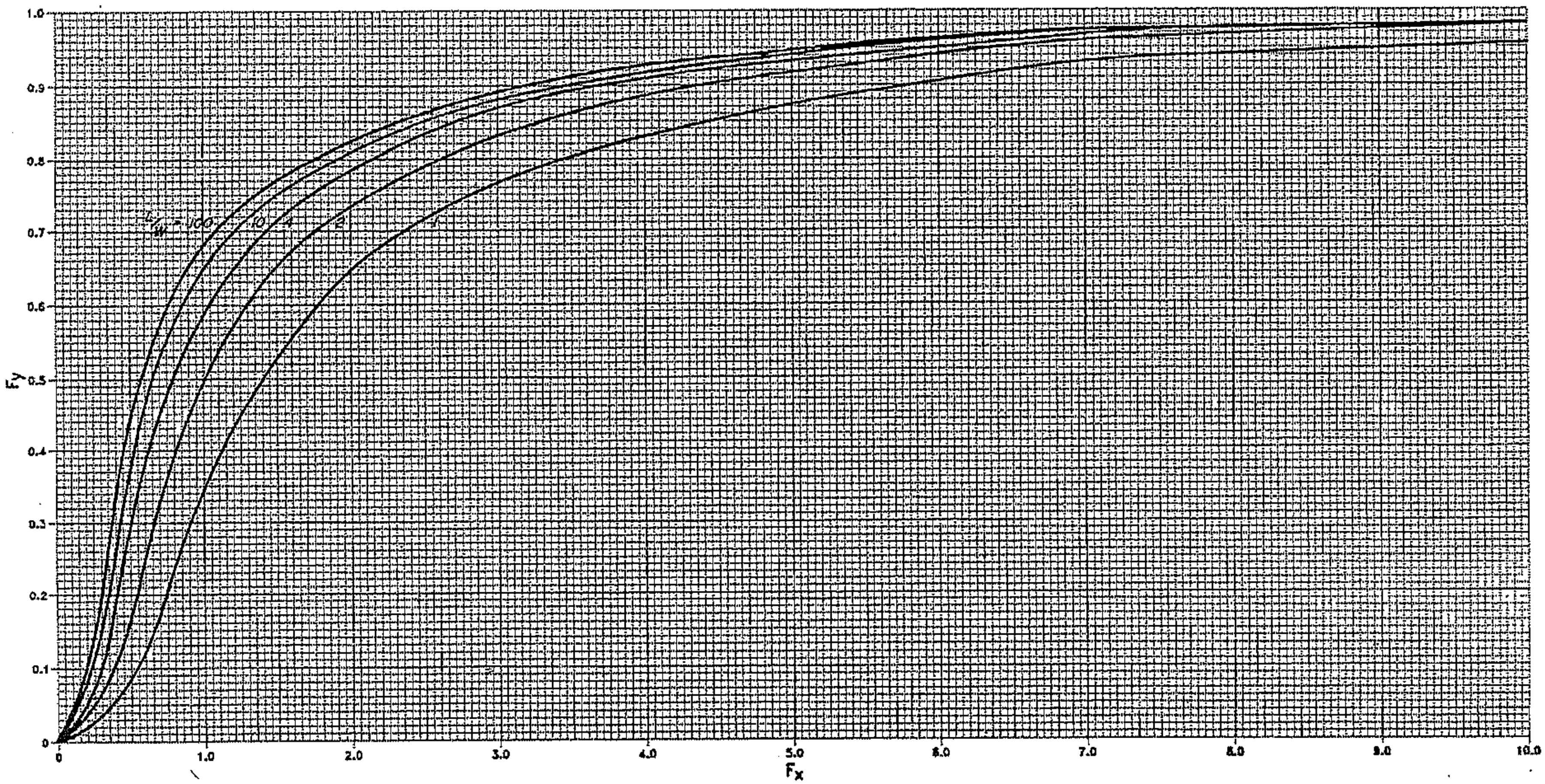

115'
100'
415'
400'
Average Pond Dimensions
Bottom Dimensions
Bottom Dimensions
H: V
H: V
RECTANGULAR BASIN
CIRCULAR BASIN
Design High Water Level
Elevation 47.4'
4.4'
Elevation 43.0'
3.0'
f= 0.2
Elevation 40.0'
Groundwater Table
20.0'
k h(average) = 15 FT-DAY
Elevation 20.0'
Impervious Layer
Time (hours)
0 5 10 15 20 25
Cummulative Runoff Volume (acre-feet)
0 2 4 6 8 10
t0 Δtsat td tf
Vu = 0.55 AC-FT
Δtsat = 4.8 HRS
tf = 20 HRS
Δteff = 12.5 HRS
td = 7.5 HRS
Vs = 6.05 AC-FT
JAMMAL & ASSOCIATES, INC.
Exfiltration Analyses Work Sheet

Figure 5-6



$$F_x = \left(\frac{W^2}{4KDt}\right)^{1/2} \quad F_y = \frac{h_c}{H_T}$$

Dimensionless Curves Relating Pond Design Parameters to Pond Water Level in a Rectangular Recharge Basin over an Unconfined Aquifer ($f = 0.2$)

JAMMAL & ASSOCIATES, INC.

Figure 4-10

calculate the height of water level in the pond for each effective time increment Δt_{eff}

$$F_x = \left(\frac{W^2}{4K_H Dt}\right)^{1/2}$$

W = average width of pond

$$W = \left(\frac{100\text{ ft} + 130\text{ ft}}{2}\right) = 115\text{ ft}$$

K_H = 15 ft/day

D = average saturated thickness

$$D = H + \frac{h_c}{2}$$

H = 20 feet **(Figure 5-6)**

h_c = to be calculated by interative process. It will be necessary to assume a value of h_c, calculate, compare, adjust the value and repeat the process.

The following initial assumptions will be made:

Δt_{eff} (hrs)	Δt_{eff} (days)	Assumed h_c (ft)
4.0	0.167	4.25
6.5	0.270	5.50
9.0	0.370	6.60
12.5	0.520	7.50

t = effective infiltration time increment for incremental analysis, cummulative time needs to be used as presented in the summary table above for Δt_{eff}.

For the first time increment Δt_{eff1}

$$F_{x1} = \left(\frac{W^2}{4K_H D \Delta t_{eff1}}\right)^{1/2}$$

$$F_{x1} = \left(\frac{115^2}{4(15)(22.12)(0.167)}\right)^{1/2} = 7.72$$

Similarly, the remaining three (3) values of F_x can be calculated and the values for

F_y can be obtained from **Figure4-10** for f=0.2.

Increment No.	Δt_{teff} (days)	D (Ft.)	F_x	F_y
1	0.167	22.12	7.72	0.97
2	0.270	22.75	5.99	0.94
3	0.370	23.30	5.05	0.93
4	0.520	23.75	4.22	0.91

The height of water level above groundwater table, h_c, can be calculated as follows:

$$F_y = \frac{h_c}{H_T}$$

h_c = $F_y H_T$
H_T = $h_b + h_v$
h_b = height of pond bottom above groundwater table
h_b = 3 feet
h_v = height of water level above pond bottom if no infiltration occured

$$h_v = \frac{V_s}{A_a}$$

V_s = Total volume of runoff directed to the pond during saturated infiltration period (varies with Δt_{eff})
A_a = Average pond area modelled
A_a = 47,725 ft2 (415 ft x 115 ft)
A_a = 1.095 acres

Therefore, h_v, H_T and h_c for each Δt_{eff} can be calculated as follows:

Δt_{eff}(days)	h_v (ft)	H_T (ft)	h_c (ft)	V_S (ac-ft)
0.167	2.60	5.60	5.43	2.85
0.27	4.24	7.24	6.81	4.65
0.37	4.79	7.79	7.24	5.25
0.52	5.52	8.55	7.78	6.05

Now, compare the calculated values of h_c to the assumed values of h_c, adjust as necessary and repeat the process:

Δt_{eff}	Assumed h_c	Calculated h_c	Adjusted h_c
0.167	4.25	5.43	5.2
0.270	5.50	6.81	6.6
0.370	6.60	7.24	7.0
0.52	7.50	7.78	7.5

The adjusted values of h_c should be very close to the calculated values of h_c with a small reduction or increase towards the assumed value of h_c.

Calculate new values for D and F_x, obtain values of F_y from **Figure 4-10** and re-calculate values of h_c.

Δt_{eff}	D	F_x	F_y	New h_c
0.167	22.60	7.64	0.95	5.3
0.270	23.30	5.92	0.92	6.6
0.370	23.50	5.03	0.90	7.0
0.520	23.75	4.22	0.87	7.4

The value of H_T did not change from last iteration and was reused to calculate new h_c.

Since the values of new h_c and adjusted h_c are very close, the iteration analysis can be terminated.

From this iterative analysis, the maximum value of h_c occured at the end of time increment Δt_{eff} = 0.52 days = 12.5 hours.

Therefore, the DHWL for this pond should be:

DHWL = initial groundwater table + h_{cmax}.

DHWL = 40 feet + 7.4 feet

DHWL = 47.4 feet NGVD

Step 3

Calculate total direct storage volume of pond, V_D

$V_D = A_a (h_{cmax} - h_b)$

$V_D = 1.095 \text{ acres } (7.4 \text{ feet} - 3 \text{ feet})$

$V_D = 4.82 \text{ acre-feet}$

Step 4
Calculate infiltration volume during saturated infiltration, V_I.

$V_I = A_a (H_{tlast} - h_{clast})$
$H_{Tlast} =$ runoff volume height for the last time increment
$H_{Tlast} = 8.55$ feet
$h_{clast} =$ Water level height for the last time increment
$h_c = 7.4$ feet
$V_I = 1.095 \text{ acres } (8.55 \text{ ft} - 7.4 \text{ ft})$
$V_I = 1.26 \text{ acre-feet}$

The total infiltration volume during the storm event, V_{TI}:

$V_{TI} = V_u + V_I$
$V_{TI} = 0.55 \text{ acre-feet} + 1.26 \text{ acre-feet}$
$V_{TI} = 1.81 \text{ acre-feet}$

Summary

1. Estimated DHWL = 47.4 feet NGVD
2. Estimated total direct storate volume = 4.82 acre-feet
3. Estimated total infiltration volume = 1.81 acre-feet

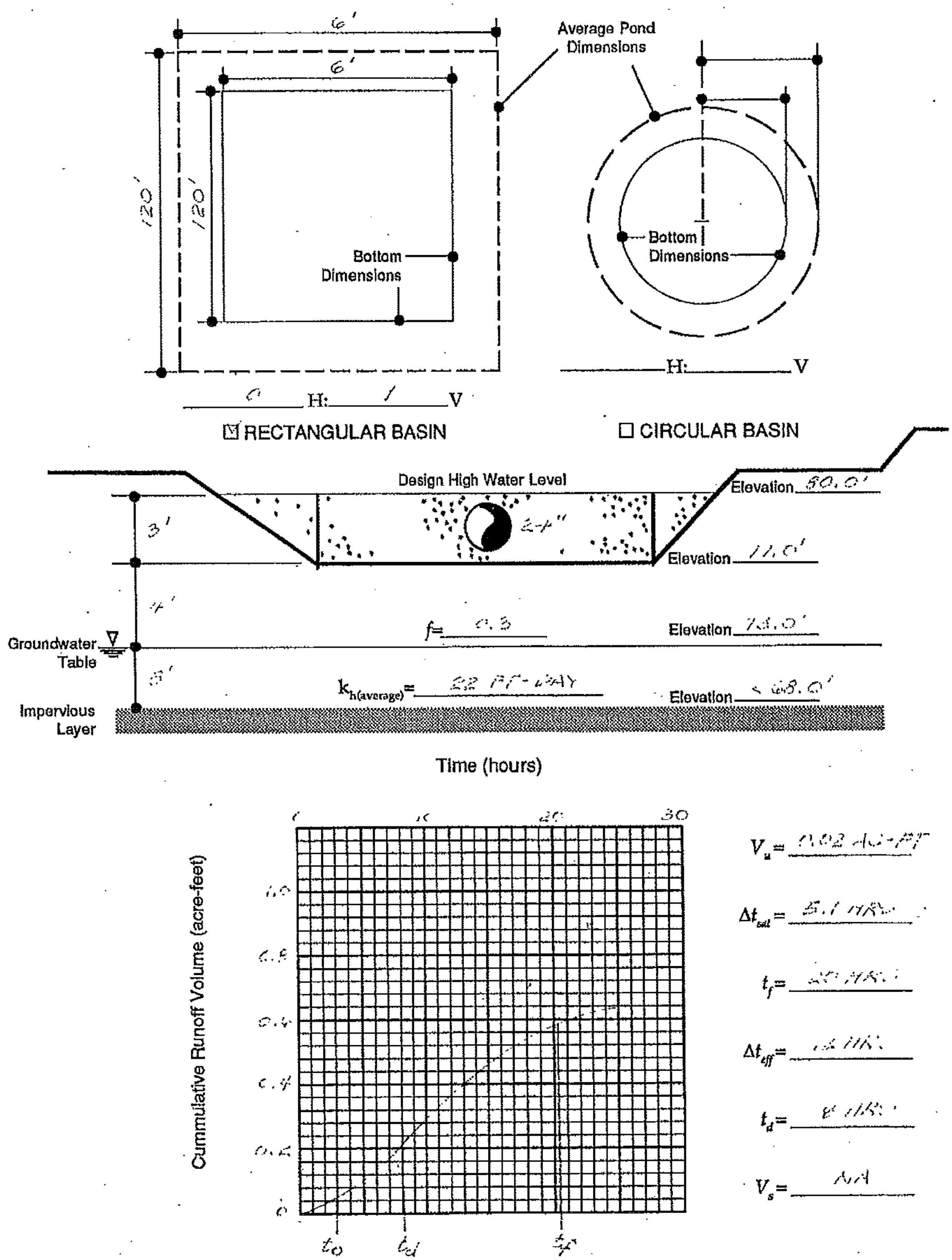
End of Example 4

Example 5

Given:

Proposed stormwater exfiltration trench retention system.

Design stormwater runoff hydrograph is presented on **Figure 5-7** for 25-year, 24-hour storm.



JAMMAL & ASSOCIATES, INC.

Exfiltration Analyses Work Sheet

Figure 5-7

Exfiltration trench system details are presented on **Figure 5-7.**

Site soil and groundwater conditions are as follows:

1. Elevation of impervious soil layer >14 feet in depth
2. Average ground surface elevation 82 feet NGVD
3. Average horizontal hydraulic conductivity 22 feet/day
4. Vertical hydraulic conductivity 17 feet/day
5. Proposed trench bottom elevation 77 feet NGVD
6. Estimated high groundwater table 73 feet NGVD
7. Average effective storage 0.3
8. Effective porosity of trench gravel 0.45

Required:

1. Estimate total infiltration volume during the storm event (unsaturated and saturated). Assume overflow will occur at the top of trench bed (elevation 80 feet NGVD). Typically, the exfiltration trenches are permitted for the purpose of retaining the first 1/2 inch or 1 inch of runoff. However, infiltration from trenches will continue throughout the storm duration.
2. Estimate total direct storage volume in the exfiltration trench.

Calculations:

Step 1:

Calculate unsaturated infiltration volume, V_u

$V_u = A_b H_b f$

A_b = average area of exfiltration trench bottom

A_b = 720 ft^2 (120 ft x 6 ft)

A_b = 0.0165 acres

h_b = vertical separation between the trench bottom and groundwater table

h_b = 77 ft - 73 ft = 4 ft

f = effective storage of soil above groundwater table

f = 0.3

V_u = 0.0165 (4 feet) (0.3) = 0.02 acre-feet

Calculate time to saturate soil below trench bottom, Δt_{sat}

$$\Delta t_{sat} = \frac{h_b f}{I_d}$$

h_b and f as previously defined

$$I_d = \frac{K_{vu}}{FS}$$

K_{vu} = unsaturated vertical hydraulic conductivity
K_{vu} = 2/3 K_{vs}; where 2/3 is an empirical correction factor (*Bouwer, 1978*)
K_{vs} = vertical hydraulic conductivity (saturated soil)
K_{vs} = 17 feet/day (given)
K_{vu} = 2/3 (17) = 11.33 ft/day
FS = Factor of safety
FS = 2.0 (recommended)
I_d = 11.33/2.0 = 5.67 feet/day
Δt_{sat} = 4 ft(0.3)/5.67 = 0.212 days
Δt_{sat} = 5.1 hours

Step 2:
Calculate saturated infiltration volume, V_I

V_I = $A_a(H_T - h_c)$
A_a = average area of exfiltration trench during saturated infiltration
A_a = A_b = 0.0165 acres
h_c = height of water level in trench at end of storm event. top of exfiltration trench elevation minus groundwater table elevation (assuming trench is overflowing at the end of the storm)
h_c = (80 - 73) ft NGVD = 7 feet
H_T = To be caculated

To solve for H_T, calculate the value of F_x

$$F_x = \left(\frac{W^2}{4K_H D\, t}\right)^{1/2}$$

W = average width of trench
W = 6 feet (given)
K_H = average horizontal hydraulic conductivity
K_H = 22 ft/day (given)
D = average saturated thickness of aquifer

$$D = H + \frac{h_c}{2}$$

H = initial saturatred thickness ≤W

Investigated depth of 14 feet corresponds to elevation 68 feet NGVD. Assume impermeable layer is at elevation 68 feet NGVD.

Since groundwater table is at 73 feet NGVD ·

$H = (73 - 68)$ ft NGVD $= 5$ feet $< W$

$D = 5 + 7/2 = 8.5$ feet

$t = \Delta t_{eff}$ = effective time increment of stormwater runoff during saturated infiltration

$\Delta t_{eff} = t_f - t_d$ **(see Figure 5-7)**

$\Delta t_{eff} =$ 12 hours $= 0.5$ days

And,

$$F_x = \left(\frac{6^2}{4(22)(8.5)(0.5)}\right)^{1/2}$$

$F_x = 0.31$

Calculate length over width ratio.

$$\frac{L}{W} = \frac{120 \text{ ft}}{6 \text{ ft}} = 20$$

Use **Figure 4-11** for L/W = 20 to estimate a value of F_y

$F_y = 0.165$

Also,

$$F_y = \frac{h_c}{H_T}$$

$$H_T = \frac{h_c}{F_y} = \frac{7 \text{ feet}}{0.165} = 42.42 \text{ feet}$$

Therefore,

$V_I =$ 0.0165 acres (42.42 ft - 7 feet)

$V_I =$ 0.585 acre-feet

Calculate total infiltration volume during storm event, V_{TI}

$V_{TI} = V_u + V_I =$ 0.02 acre-feet + 0.585 acre-feet

$V_{TI} =$ 0.605 acre-feet

Step 4:

Calculate total direct storage volume in the exfiltration trench at the overflow elevation of 80 feet NGVD.

$V_D = V_p + (V_{TR} - V_P)(P)$

V_P = total volume of distribution pipes in the trench system

V_{TR} = total volume of trench = width x length

P = effective porosity of trench gravel

For this example,

$V_p = A_p L_p$
A_p = area of distribution pipe
L_p = total length of pipe

$A_p = \frac{\pi}{4}(2)^2 = 3.14 \text{ ft}^2$

$L_p = 120 \text{ ft}$
$V_p = 3.14 \text{ ft}^2 (120 \text{ ft}) = 377 \text{ ft}^3$
V_{TR} = width x height x length
V_{TR} = 6 feet x 3 feet x 120 feet
V_{TR} = 2,160 ft3
$V_D = 377 \text{ ft} + (2{,}160 \text{ ft}^3 - 377 \text{ ft}^3)(0.45)$
$V_D = 1{,}179 \text{ ft}^3 = 0.027$ acre-feet

Summary:

1. Estimated total infiltration volume (unsaturated and saturated infiltration)
   $V_{TI} = 0.605$ acre-feet

2. Estimated total direct storage volume in the exfiltration trench
   $V_D = 0.027$ acre-feet

End of Example 5

Nomenclature for Chapter 5

A_a Average area of the retention pond. Average area between pond bottom and water level height in the pond.

A_b Area of pond bottom.

A_p Cross-sectional area of pipe(s). Typically, the distribution pipe(s) in an underground exfiltration trench.

D Average saturated thickness of effective aquifer during saturated infiltration ($D = H + h_c/2$).

F_x Dimensionless parameter representing physical and hydraulic characteristics of the retention pond and effective aquifer system

$$F_x = \left(\frac{W^2}{4K_H Dt} \right)^{1/2}$$

F_y Dimensionless parameter representing percent of water level decline below a maximum level. The maximum level would occur if NO soil infiltration occured.

$$F_y = \frac{h_c}{H_T}$$

FS Factor of Safety.

f Average effective storage (fillable porosity) coefficient of soil above groundwater table and within the groundwater mound.

H Initial saturated thickness of the effective aquifer. Height of initial groundwater table above an impermeable layer in the effective unconfined aquifer.

H_T Height of the water level in the retention pond above initial groundwater table, if NO infiltration occured. (For case of $K_H = 0$)

h_b Height of pond bottom above initial groundwater table.

h_c Height of water level in the retention pond above the initial groundwater table at time, t, after the start of saturated infiltration.

h_I Equivalent height of water in the retention pond which has infiltrated (exfiltration from the pond) during the saturated infiltration period.

h_s Height of water level in the retention pond above pond bottom at time, t, after start of saturated infiltration.

h_v $= h_s + h_I$

I_d Design average infiltration rate at the retention pond bottom during unsaturated infiltration. A factor of safety is incorporated into I_d to allow for long-term operation and account for pond bottom siltation.

K_H Average horizontal hydraulic conductivity of the effective unconfined aquifer.

K_{vs} Vertical hydraulic conductivity (saturated soil specimen test).

K_{vu} Unsaturated infiltraton vertical hydraulic conductivity. Direct measurement or emperically estimated from K_{vs}

$(K_{vu} = \frac{2}{3} K_{vs})$

L Average length of retention pond. Midway between pond bottom and water level at time t, after start of saturated infiltration.

P Effective porosity of gravel in underground exfiltration trench retention systems.

t Time

t_f Time increment from start of stormwater runoff ($t = 0$) to the time when runoff stops flowing into the pond.

Δt_{eff} Incremental time of saturated infiltration inflow to the retention pond. Can be any increment of time selected for analysis.

Δt_{sat} Incremental time of unsaturated infiltration.

t_{ST} Time of storm event

t_d Time increment from start of stormwater runoff ($t = 0$) to the time when the soil between pond bottom and the groundwater table has been saturated.

V_D Direct storage volume of retention pond at a given water level.

V_I Total volume of water resulting from saturated infiltration from retention pond during selected saturated infiltration period Δt_{eff}.

V_p Total volume of distribution pipe(s) in an underground exfiltration trench system.

V_{ro} Runoff volume associated with a design storm event

V_s $= V_D + V_I$

V_T $= V_u + V_s$

V_{TR} Total volume of underground exfiltration trench system

V_u Total volume of unsaturated infiltration occuring during time, Δt_{sat}

W Average width of retention pond, midway between pond bottom and water level at time t, after start of saturated infiltration.

CHAPTER 6

Pond Construction Considerations

Chapter 6

Construction Considerations

Retention pond construction procedures and the overall sequence of site construction are two key factors that will control the effectiveness of system operation. Even the most accurate infiltration analysis and the best engineering design of a retention pond can not compensate for sub-standard construction methods and/or construction sequence.

Since stormwater retention ponds are typically required to be constructed during the initial phases of site development, the pond is often exposed to poor quality surface runoff. Stormwater runoff during construction contains considerable amounts of suspended solids, organics, clays, silts, trash and other undesirable materials. One such source of undesirable material is generated during construction of roadways and pavement areas. The subgrade stabilization material within these areas typically consist of clayey sand or soil cement. If a storm is to occur prior to placement of the roadway wearing surface (asphalt and associated tacking), considerable amounts of these materials would end up in the retention pond. Another obvious source of fine material generated during construction is disturbed surface soil which typically releases relatively large quantities of organics and other fine particles. Fine particles of clay, silt and organics accumulate at the bottom of the retention pond creating a poor infiltrating surface.

A review of SWFWMD Management and Storage of Surface Water (MSSW) Permit Information Manual was conducted to identify construction considerations and requirements already published. Based on this review, it was determined that all the construction considerations and requirements set forth in the SWFWMD MSSW Permit Information Manual are appropriate and very important for successful construction and operation of a retention pond. The recommendations presented in this chapter are mostly supplemental to the information presented in the SWFWMD MSSW Permit Information Manual. It is important that the recommended construction sequence and the construction phasing presented herein be incorporated into the requirements and permit regulations for stormwater retention ponds.

To construct an effective retention pond and to avoid degradation of the pond exfiltration capacity due to the sequence and common practices of construction, the following construction procedures are recommended:

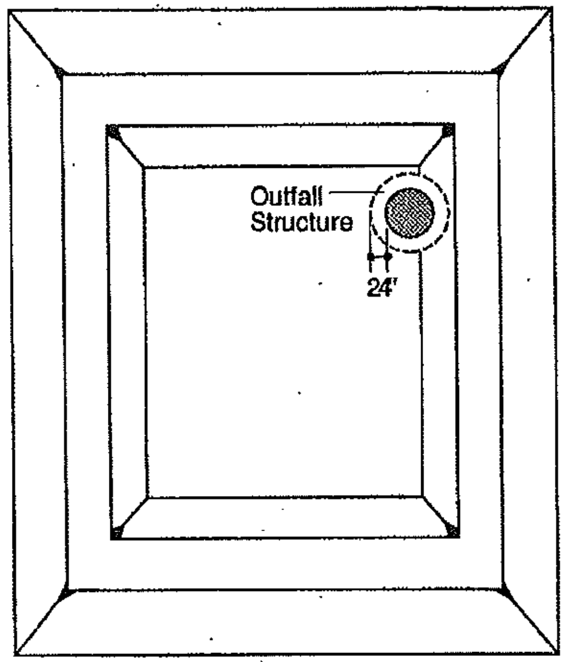
1. In the initial phase, Phase I, construct the stormwater retention pond by under-excavating the pond bottom and sides by a minimum of twelve inches. If possible, the outfall structure should be installed as designed during this initial phase of retention pond construction. The retention pond should remain undercut until all construction has been completed and, if possible, the pond should be allowed to stabilize for a period of one month after completion of construction prior to completing final pond excavation.

2. The second phase of construction, Phase II, of the retention pond should be initiated after all construction work has been completed and at least one month has been allowed for runoff stabilization within the effective stormwater runoff area of the pond. The Phase II construction should consist of excavating the retention pond interior slopes and pond bottom to the design specifications. The excess soil and undesirable material should be carefully excavated and removed from the pond so that all accumulated silts, clays, organics and other fine sediment material has been removed from the pond area. **Figure 6-1** presents a schematic of stormwater retention pond construction phases. The excavated material should be disposed of beyond the limits of the direct surface runoff area of the retention pond as presented on Figure 6-1.

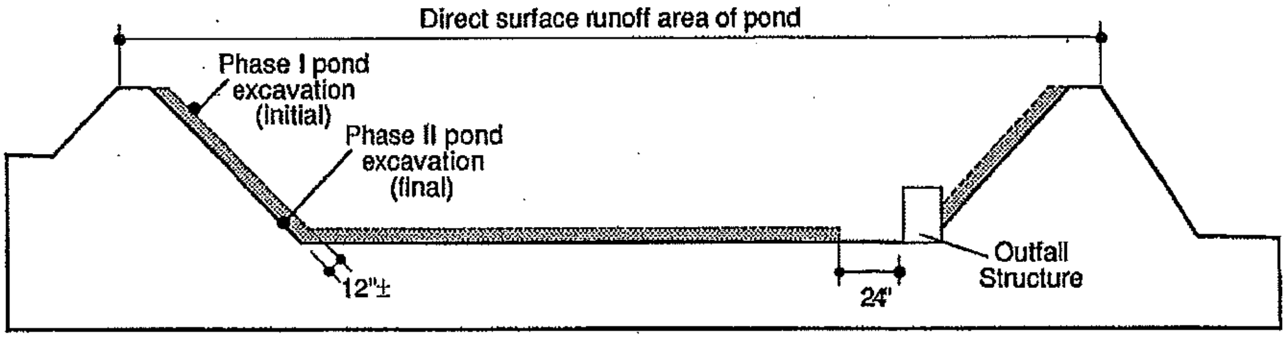
3. Once the retention pond has been excavated to the desired elevation, the entire pond bottom should be deep raked and loosened to create a permeable pond bottom for optimum percolation.

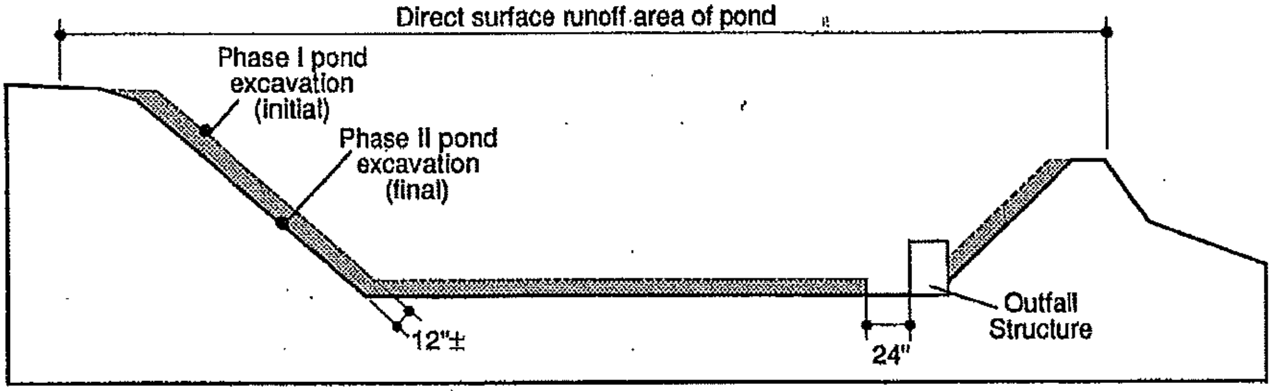
4. The entire pond area including outside slopes, berm crests and inside slopes should be sodded or mulched and seeded immediately after completion of Phase II construction of the pond. The pond bottom should not be seeded or sodded. Construction traffic should not be allowed on the pond bottom after deep raking to avoid compaction.

5. In general, sodding of the pond bottom area is not recommended due to the considerable amount of organic material present in the root zone of the sod. Wherever possible, sod grown in sand should be used for sodding of the effective pond area.



PLAN VIEW OF RETENTION POND





Schematic of Stormwater Retention Pond Construction Sequence

JAMMAL & ASSOCIATES, INC.

Figure 6-1

CHAPTER 7

Pond Maintenance Requirements

Chapter 7
Maintenance Requirements

To maintain an operational stormwater retention system the following minimum inspection and maintenance program should be implemented:

1. The stormwater retention pond should be inspected on a monthly basis for the following critical items:

a. Blockage of the outfall structure.
b. Operational integrity of the skimmer system.
c. Pond slope erosion problems.
d. Accumulation of pond bottom sediments.
e. Effectiveness of percolation.

A sample field inspection form has been generated and is included as **Figure 7-1.** The inspection form was generated in such a manner so that a problem can be identified and the appropriate recommended corrective action which is listed directly on the form can be implemented. Accordingly, the inspection form should be automatically submitted to the appropriate section of the operation and maintenance entity to initiate corrective action.

2. Due to the variability of site specific soil and ground water conditions and surface runoff conditions, it is not possible to determine one optimal routine inspection interval for all retention pond systems. Therefore, we recommend herein, that all ponds initially be inspected on a monthly basis for at least the first three months of operation. At the end of three months of operation, the inspection schedule should be adjusted on the basis of the results of the first three inspections. If all items during all three inspection periods were associated with problems, the inspection period should be adjusted to every three months. However if some of the items on the inspection form were not associated with any problem during the initial period of inspection then it should be individually evaluated on a site-by-site basis to determine the required inspection schedule. Formal and impartial inspections should be conducted every 2 years by a registered engineer to evaluate continued proper function of the system and adequacy of O & M procedures. These findings should be submitted in a written report to the appropriate authorities.

In addition to these maintenance requirements, all existing requirements presented in the current SWFWMD MSSW Permit Information Manual should be followed. The specifics of all other requirements, as presented in SWFWMD MSSW Permit Information Manual, are not included herein to avoid redundancy.

RETENTION POND INSPECTION FORM

Retention pond identification ______________________________

Date of inspection ______________________________

Name of inspector ______________________________

Description of weather *(day of inspection)* ______________________________

Description of weather *(one day prior to inspection)* ______________________________

A.) Is the outfall structure (including discharge pipe) blocked or partially blocked?

yes☐ no☐ If yes, explain the problem. ______________________________

____

B.) Is the skimmer system of the outfall structure broken, blocked or otherwise inoperable?

yes☐ no☐ If yes, explain the problem. ______________________________

____

C.) Are there any significant pond berm erosion problems? yes☐ no☐ If yes, explain the problem. ______________________________

____

D.) Are there significant amounts of pond bottom sediment accumulation that requires removal to restore pond operation? yes☐ no☐ If yes, explain the problem.

____

____

E.) Is there significant evidence that the pond is not percolating as designed and requires maintenance to improve percolation? yes☐ no☐ If yes, explain the problem.

____

____

F.) Other comments ______________________________

____

JAMMAL & ASSOCIATES, INC.

Figure 7-1

BIBLIOGRAPHY

Bibliography

Amar, A. C., 1975, "Ground-Water Recharge Simulation", Journal of the Hydraulics Division, ASCE, No. HY9, p. 1235-1247

Andreyev, N. E., 1985 "Exfiltration from Stormwater Ponds in Central Florida, MS Thesis, University of Central Florida, Orlando, Florida

Annual Book of ASTM Standards, 1986, Soil and Rock; Building Stones, Volume 4, Philadelphia, PA

Bamgboye, O.A., 1979, "Groundwater Basin Modeling Using the Finite Element Method - Yolo County Case Study", MS Thesis, California University, Davis, 126 p.

Bear, J., D. Zaslavsky and S. Irmay, 1968, Physical Principles of Water Percolation and Seepage. Paris: UNESCO, p. 249.

Beaver, R. D., J. P. Hartman, and M. P. Wanielista, "Infiltration and Stormwater Retention/Detention Ponds", Research Report, College of Engineering, Florida Technological University, Orlando, Florida, 14 p.

Besbes, M. and G. DeMarsily, 1984, "From Infiltration to Recharge: Use of a Parametric Transfer Function", Journal of Hydrology, Vol. 74, p. 271-293.

Boast, C. W. and D. Kirkham, 1971, Auger Hole Seepage Theory, Soil Science Society American Proceedings, Vol. 35, p. 365-374.

Bodman, G. B. and E. A. Coleman, 1943, Moisture and Energy Conditions During Downward Entry of Water Into Soils. Soil Science Society of America Proceedings, Vol. 8

Bouwer, H., 1966, "Rapid Field Measurement of Air Entry Value and Hydraulic Conductivity as Significant Parameters in Flow System Analysis", Water Resources Research, Vol. 2, p. 729-738.

Bouwer, H., 1969, "Infiltration of Water into Nonuniform Soil", Journal of the Irrigation and Drainage Division, ASCE, No. IR4, p. 451-462. Bouwer, H., 1978, Groundwater Hydrology, McGraw-Hill Book Company, New York, 480 p.

Branscome, J. and R. S. Tomasello, 1987, "Field Testing of Exfiltration Systems", South Florida Water Management District, Water Resources Division. Technical Publication 87-5, 49 p.

Brock, R. R., 1976, "Dupuit-Forchheimer and Potential Theories for Recharge From Basins"; Water Resources Research, Vol. 12, No. 5, p. 909-911.

Brooks, R. H. and A. T. Corey, 1966, Properties of Porous Media Affecting Fluid Flow. Proceedings of the American Society of Civil Engineers, Irrigation and Drainage Division: Vol. 92, No. IR2. Cabrera, G., and G. Mathey, 1984, "Space-Time Finite-Element Model for Two-Stream - Unconfinued-Aquifer Systems", Journal of Hydrology, Vol. 73, p. 299-314.

Cedergren, H. R., 1977, Seepage, Drainage and Flow Nets, John Wiley & Sons, New York, 534 p.

Driscoll, F. G., 1986, Groundwater and Wells, H. M. Smyth Company, St. Paul, Minnesota.

Das, Braja M., 1982, Soil Mechanics Laboratory Manual, Engineering Press, Inc., San Jose, California, 116 p.

Dunn, I. S., L. R. Anderson and F. W. Kiefer, 1980, Geotechnical Analysis, John Wiley & Sons, New York, 414 p.

Evans, D. D., D. Kirkham and R. K. Frevert, 1950, "Infiltration and Permeability in Soil Overlying an Impermeable Layer.", Proceedings of Soil Science Society of America.

Glover, R. E., 1964, Groundwater Movement, U. S. Bureau of Reclamation, Engineering Monograph 31, p. 67

Green, W. H. and G. A. Ampt, 1911, "Studies on Soil Physics I. The Flow of Air and Water Through Soils.", Journal Agricultural Science, Vol. 4, p. 1-24.

Guvanasen, V., and R. E. Volker, 1982, "Solutions for Solute Transport in an Unconfined Aquifer", Journal of Hydrology, Vol. 58, No. 1-2, p. 89-102.

Hannon, J. B., 1980, Underground Disposal of Stormwater Runoff, Design Guidelines Manual, California Department of Transporation.

Hantush, M. S., 1967, "Growth and Decay of Groundwater-Mounds in Response to Uniform Percolation", Water Resources Research, Vol. 3, No. 1, p. 227-234.

Hvorslev, M. J., 1951, "Time Lag and Soil Permeability Ground-Water Observations" Waterways Experiment Station, Corps of Engrs, Vicksburg, Miss., Bulletin No. 36.

Hunt, B. W., 1973, "Seepage from a Shallow Reservoir", Journal of the Hydraulics Division, ASCE, No. HY1, p. 23-30.

Johnson, A. I., 1963, "A Field Method for Measurement of Infiltration", U.S. Geological Survey Water-Supply Paper 1544-F, 27 p.

Karamouzis, D., and G. Terzidis, 1984, "Computation of the Unsteady Two-Dimensional Free Surface of Groundwater by the Finite-Element Method", Journal of Hydrology, Vol. 74, p. 53-65.

Kruseman, G. P. and N. A. De Ridder, 1983, Analysis and Evaluation of Pumping Test Data, International Institute for Land Reclamation and Improvement, 200 p.

Lambe, William T., 1951, Soil Testing for Engineers, John Wiley & Sons, Inc., New York, 165 p.

Latinopoulos, P., 1981, "The Response of Groundwater to Artificial Recharge Schemes", Water Resources Research, Vol. 17, No. 6, p. 1712-1714.

Lohman, S. W., 1972, Groundwater Hydraulics, Geological Survey Professional Paper 708, U.S. Government Printing Office, Washington, D.C., 70 p.

Marino, M. A., 1974a, "Water-Table Fluctuation in Response to Recharge", Journal of the Irrigation and Drainage Division, ASCE, No. IR2, p. 117-125.

Marino, M.A., 1974b, "Growth and Decay of Groundwater Mounds Induced by Percolation", Journal of Hydrology, Vol. 22 p. 295-301.

Marino, M.A., 1974c, "Rise and Decline of the Water Table Induced by Vertical Recharge", Journal of Hydrology, Vol. 23, p. 289-298.

Marino, M. A., 1975a, "Artificial Groundwater Recharge, I. Circular Recharging Area", Journal of Hydrology, Vol. 25, p. 201-208.

Marino, M. A., 1975b, "Artificial Groundwater Recharge, II. Rectangular Recharging Area", Journal of Hydrology, Vol. 26, p. 29-37.

Marino, M. A., 1975c, "Response of Unconfined Aquifer Systems to Deep Percolation", California University, Davis, Dept. of Water Science and Engineering, Water Science and Engineering Paper, No. 2004, 32 p.

Martin, R. E., 1980, "Processes Occurring During Natural Recharge of an Unconfined Aquifer, North of Perth, Western Australia", Proceedings of the Groundwater Recharge Conference Townsville, Australia, July 14-18, Australian Water Resources Council Conference Series 3, p. 233-238.

McDonald, M. G. and A. W. Harbaugh, 1984, A Modular Three-Dimensional Finite Difference Ground-Water Flow Model, U.S. Geological Survey, p. 528.

Morel-Seytoux, H. J. and J. Khanji, 1974, "Derivation of an Equation of Infiltration", Water Resources Research, Vol. 10, No. 4, p. 795-800.

Ortiz, N. V., D. B. McWhorter, D. K. Sunada and H. R. Duke, 1978, "Growth of Groundwater Mounds Affected by In-Transit Water", Water Resources Research, Vol. 14, No. 6, p. 1048-1088.

Ortiz, N.V., D. W. Zachmann, D. B. McWhorter and D. K. Sunada, 1979, "Effects of In-Transit Water on Groundwater Mounds Beneath Circular and Rectangular Recharge Areas", Water Resources Research, Vol. 15, No. 3, p. 577-582.

Page, R. W., 1977, "Guide for Data Collection to Calibrate a Predictive Digital Ground-Water Model of the Unconfined Aquifer in and Near the City of Modesto, California", U.S. Geological Survey, Menlo Park, California, Water-Resources Investigations 76-41, 46 p.

Patterson, R. J., 1974, "Waste Disposal Systems From a Groundwater Hydrology and Pollution Pond of View", Water and Sewage Works, April 30, 1974, p. 92-99.

Philip, J. R., 1957, "The Theory of Infiltration, I. The Infiltration Equation and It's Solution", Soil Science, Vol. 83, p. 345-357.

Rai, S. N., and R. N. Singh, 1980, "Dynamic Response of an Unconfined Aquifer System Subjected to Transient Recharge", Geophysics Research Bulletin (Hyderabad), Vol. 18, No. 2, p. 49-56.

Rai, S. N., and R. N. Singh, 1981, "A Mathematical Model of Water Table Fluctuation in a Semi-Infinite Aquifer Induced by Localized Transient Recharge", Water Resources Research, Vol. 17, No. 4, p. 1028-1032.

Rao, N. H., and P. B. S. Sarma, 1981, "Ground-Water Recharge from Rectangular Areas", Ground Water, Vol. 19, No. 3, p. 271-274.

Rao, N. H., and P. B. S. Sarma, 1984, "Recharge to Aquifers with Mixed Boundaries", Journal of Hydrology, Vol. 74, p. 43-51.

Seaburn, G. E., 1970, "Preliminary Results of Hydrologic Studies at Two Recharge

Basins on Long Island, New York", U.S. Geologic Survey Professional Paper 627-C, 17 p.

Southwest Florida Water Management District, 1988, Management and Storage of Surface Waters, Permit Information Manual, Volume I.

Southwest Florida Water Management District, 1987, Management of Surface Waters, Permit Information Manual, Volume IV.

Swartzendruber, D. and T. C. Olson, 1961, "Model Study of the Double-Ring Infiltrometer as Affected by Depth of Wetting and Particle Size", Soil Science, pp. 219-225.

Texas Water Development Board, 1974, "Groundwater Simulation Program, GSWIM Program Documentation and User's Manual", Texas Department of Water Resources, Austin, May 1974, 68 p.

Tinsley, P. S. and R. M. Ragan, 1974, "Investigations of theResponse of an Unconfined Aquifer to Localized Recharge", Maryland Office of Water Resources Research, Technical Report No. 20, 80 p.

Todd, 1980, Groundwater Hydrology, 2nd Edition, John Wiley & Sons, Inc., New York, 535 p. U.S. Dept. of the Interior, Bureau of Reclamation, 1974, Earth Manual, U.S. Gov't Printing Office, Washington D.C., Second Printing

Viswanathan, M. N., 1983, "Rainfall/Water-Table Level Relationship of an Unconfined Aquifer", Ground Water, Vol. 21, No. 1, p. 49-56. Walton, W.C., 1970, Groundwater Resource Evaluation, McGraw Hill Book Company, New York, 664 p.

Wanielista, M. P., 1979, Stormwater Management, Ann Arbor Science Publishers, Inc., Michigan, 383 p.

Watson, K. K., and F. D. Whisler, 1977, "Profile Desaturation During Sediment Deposition in a Groundwater Recharge Trench", Journal of Hydrology, Vol. 33, No. 3/4, p. 397-401.

Weaver, R. J., 1971, "Recharge Basins for Disposal of Storm Drainage", New York State Department of Transportation Engineering Research and Development Bureau. Research Report 69-2.

Whisler, F. D. and H. Bouwer, 1970, "Comparison of Methods for Calculating Vertical Drainage and Infiltration for Soils", Journal of Hydrology, Vol. 10, p. 1-19.

Youngs, E. G., 1980, "The Analysis of Groundwater Seepage in Heterogeneous Aquifers", Hydrological Sciences Bulletin, Vol. 25, No. 2, p. 155-165.

APPENDIX A

ASTM Standard Procedures

D1587-83
D2325-68
D2434-68
D3385-75

Standard Practice for
THIN-WALLED TUBE SAMPLING OF SOILS[1]

This standard is issued under the fixed designation D 1587; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This practice has been approved for use by agencies of the Department of Defense and for listing in the DOD Index of Specifications and Standards.

1. Scope

1.1 This practice covers a procedure for using a thin-walled metal tube to recover relatively undisturbed soil samples suitable for laboratory tests of structural properties. Thin-walled tubes used in piston, plug, or rotary-type samplers, such as the Denison or Pitcher, must comply with the portions of this practice which describe the thin-walled tubes (5.3).

NOTE 1—This practice does not apply to liners used within the above samplers.

2. Applicable Documents

2.1 *ASTM Standards:*

D 2488 Practice for Description and Identification of Soils (Visual-Manual Procedure)[2]

D 3550 Practice for Ring-Lined Barrel Sampling of Soils[2]

D 4220 Practices for Preserving and Transporting Soil Samples[2]

3. Summary of Practice

3.1 A relatively undisturbed sample is obtained by pressing a thin-walled metal tube into the in-situ soil, removing the soil-filled tube, and sealing the ends to prevent the soil from being disturbed or losing moisture.

4. Significance and Use

4.1 This practice, or Practice D 3550, is used when it is necessary to obtain a relatively undisturbed specimen suitable for laboratory tests of structural properties or other tests that might be influenced by soil disturbance.

5. Apparatus

5.1 *Drilling Equipment*—Any drilling equipment may be used that provides a reasonably clean hole; that does not disturb the soil to be sampled; and that does not hinder the penetration of the thin-walled sampler. Open borehole diameter and the inside diameter of driven casing or hollow stem auger shall not exceed 3.5 times the outside diameter of the thin-walled tube.

5.2 *Sampler Insertion Equipment*, shall be adequate to provide a relatively rapid continuous penetration force. For hard formations it may be necessary, although not recommended, to drive the thin-walled tube sampler.

5.3 *Thin-Walled Tubes*, should be manufactured as shown in Fig. 1. They should have an outside diameter of 2 to 5 in. and be made of metal having adequate strength for use in the soil and formation intended. Tubes shall be clean and free of all surface irregularities including projecting weld seams.

5.3.1 *Length of Tubes*—See Table 1 and 6.4.

5.3.2 *Tolerances*, shall be within the limits shown in Table 2.

5.3.3 *Inside Clearance Ratio*, should be 1 % or as specified by the engineer or geologist for the soil and formation to be sampled. Generally, the inside clearance ratio used should increase with the increase in plasticity of the soil being sampled. See Fig. 1 for definition of inside clearance ratio.

5.3.4 *Corrosion Protection*—Corrosion, whether from galvanic or chemical reaction, can damage or destroy both the thin-walled tube and the sample. Severity of damage is a function of

[1] This practice is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.02 on Sampling and Related Field Testing for Soil Investigations.

Current edition approved Aug. 17, 1983. Published October 1983. Originally published as D 1587 – 58 T. Last previous edition D 1587 – 74.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

time as well as interaction between the sample and the tube. Thin-walled tubes should have some form of protective coating. Tubes which will contain samples for more than 72 h shall be coated. The type of coating to be used may vary depending upon the material to be sampled. Coatings may include a light coat of lubricating oil, lacquer, epoxy, Teflon, and others. Type of coating must be specified by the engineer or geologist if storage will exceed 72 h. Plating of the tubes or alternate base metals may be specified by the engineer or geologist.

5.4 *Sampler Head*, serves to couple the thin-walled tube to the insertion equipment and, together with the thin-walled tube, comprises the thin-walled tube sampler. The sampler head shall contain a suitable check valve and a venting area to the outside equal to or greater than the area through the check valve. Attachment of the head to the tube shall be concentric and coaxial to assure uniform application of force to the tube by the sampler insertion equipment.

6. Procedure

6.1 Clean out the borehole to sampling elevation using whatever method is preferred that will ensure the material to be sampled is not disturbed. If groundwater is encountered, maintain the liquid level in the borehole at or above ground water level during the sampling operation.

6.2 Bottom discharge bits are not permitted. Side discharge bits may be used, with caution. Jetting through an open-tube sampler to clean out the borehole to sampling elevation is not permitted. Remove loose material from the center of a casing or hollow stem auger as carefully as possible to avoid disturbance of the material to be sampled.

NOTE 2—Roller bits are available in downward-jetting and diffused-jet configurations. Downward-jetting configuration rock bits are not acceptable. Diffuse-jet configurations are generally acceptable.

6.3 Place the sample tube so that its bottom rests on the bottom of the hole. Advance the sampler without rotation by a continuous relatively rapid motion.

6.4 Determine the length of advance by the resistance and condition of the formation, but the length shall never exceed 5 to 10 diameters of the tube in sands and 10 to 15 diameters of the tube in clays.

NOTE 3—Weight of sample, laboratory handling capabilities, transportation problems, and commercial availability of tubes will generally limit maximum practical lengths to those shown in Table 1.

6.5 When the formation is too hard for push-type insertion, the tube may be driven or Practice D 3550 may be used. Other methods, as directed by the engineer or geologist, may be used. If driving methods are used, the data regarding weight and fall of the hammer and penetration achieved must be shown in the report. Additionally, that tube must be prominently labeled a "driven sample."

6.6 In no case shall a length of advance be greater than the sample-tube length minus an allowance for the sampler head and a minimum of 3 in. for sludge-end cuttings.

NOTE 4—The tube may be rotated to shear bottom of the sample after pressing is complete.

6.7 Withdraw the sampler from the formation as carefully as possible in order to minimize disturbance of the sample.

7. Preparation for Shipment

7.1 Upon removal of the tube, measure the length of sample in the tube. Remove the disturbed material in the upper end of the tube and measure the length again. Seal the upper end of the tube. Remove at least 1 in. of material from the lower end of the tube. Use this material for soil description in accordance with Practice D 2488. Measure the overall sample length. Seal the lower end of the tube. Alternatively, after measurement, the tube may be sealed without removal of soil from the ends of the tube if so directed by the engineer or geologist.

NOTE 5—Field extrusion and packaging of extruded samples under the specific direction of a geotechnical engineer or geologist is permitted.

NOTE 6—Tubes sealed over the ends as opposed to those sealed with expanding packers should contain end padding in end voids in order to prevent drainage or movement of the sample within the tube.

7.2 Prepare and immediately affix labels or apply markings as necessary to identify the sample. Assure that the markings or labels are adequate to survive transportation and storage.

8. Report

8.1 The appropriate information is required as follows:

8.1.1 Name and location of the project,

8.1.2 Boring number and precise location on project, 8.1.3 Surface elevation or reference to a datum,

8.1.4 Date and time of boring—start and finish,

8.1.5 Depth to top of sample and number of sample,

8.1.6 Description of sampler: size, type of metal, type of coating,

8.1.7 Method of sampler insertion: push or drive,

8.1.8 Method of drilling, size of hole, casing, and drilling fluid used,

8.1.9 Depth to groundwater level: date and time measured,

8.1.10 Any possible current or tidal effect on water level,

8.1.11 Soil description in accordance with Practice D 2488,

8.1.12 Length of sampler advance, and

8.1.13 Recovery: length of sample obtained.

9. Precision and Bias

9.1 This practice does not produce numerical data; therefore, a precision and bias statement is not applicable.

TABLE 1 Suitable Thin-Walled Steel Sample Tubes[A]

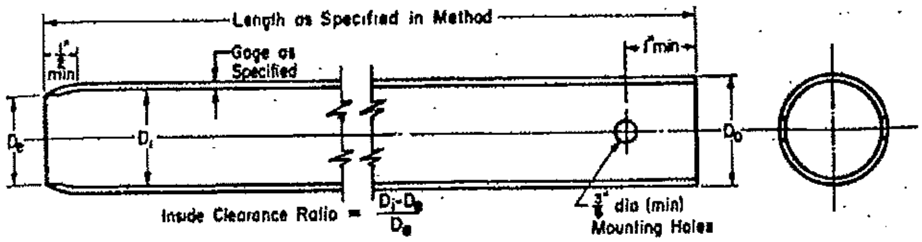
Outside diameter:			
in.	2	3	5
mm	50.8	76.2	127
Wall thickness:			
Bwg	18	16	11
in.	0.049	0.065	0.120
mm	1.24	1.65	3.05
Tube length:			
in.	36	36	54
m	0.91	0.91	1.45
Clearance ratio, %	1	1	1

[A] The three diameters recommended in Table 1 are indicated for purposes of standardization, and are not intended to indicate that sampling tubes of intermediate or larger diameters are not acceptable. Lengths of tubes shown are illustrative. Proper lengths to be determined as suited to field conditions.

TABLE 2 Dimensional Tolerances for Thin-Walled Tubes

Nominal Tube Diameters from Table 1[A] Tolerances, in.			
Size Outside Diameter	2	3	5
Outside diameter	+0.007 −0.000	+0.010 −0.000	+0.015 −0.000
Inside diameter	+0.000 −0.007	+0.000 −0.010	+0.000 −0.015
Wall thickness	±0.007	±0.010	±0.015
Ovality	0.015	0.020	0.030
Straightness	0.030/ft	0.030/ft	0.030/ft

[A] Intermediate or larger diameters should be proportional. Tolerances shown are essentially standard commercial manufacturing tolerances for seamless steel mechanical tubing. Specify only two of the first three tolerances; that is, O.D. and I.D., or O.D. and Wall, or I.D. and Wall.



NOTE 1—Minimum of two mounting holes on opposite sides for 2 to 3½ in. sampler.

NOTE 2—Minimum of four mounting holes spaced at 90° for samplers 4 in. and larger.

NOTE 3—Tube held with hardened screws.

NOTE 4—Two-inch outside-diameter tubes are specified with an 18-gage wall thickness to comply with area ratio criteria accepted for "undisturbed samples." Users are advised that such tubing is difficult to locate and can be extremely expensive in small quantities. Sixteen-gage tubes are generally readily available.

Metric Equivalents

in.	mm
⅛	6.77
½	12.7
1	25.4
2	50.8
3½	88.9
4	101.6

FIG. 1 Thin-Walled Tube for Sampling

Standard Test Method for
CAPILLARY-MOISTURE RELATIONSHIPS FOR COARSE- AND MEDIUM-TEXTURED SOILS BY POROUS-PLATE APPARATUS[1]

This standard is issued under the fixed designation D 2325; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 Note—Section 2 was added editorially and editorial changes were made throughout in October 1984.

1. Scope

1.1 This test method covers the determination of capillary-moisture relationships for coarse- and medium-textured soils as indicated by the soil-moisture tension relations for tensions between 10 and 101 kPa (0.1 and 1 atm). Under equilibrium conditions, moisture tension is defined as the equivalent negative gage pressure, or suction, corresponding to a soil moisture content. This test method determines the equilibrium moisture content retained in a soil subjected to a given soil-water tension. This test method is not suitable for very fine-textured soils.

Note 1—For determination of capillary-moisture relationships for fine-textured soils, refer to Test Method D 3152.

2. Applicable Documents

2.1 *ASTM Standards:*

D421 Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants[2]

D698 Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop[2]

D3152 Test Method for Capillary Moisture Relationships for Fine-Textured Soils by Pressure-Membrane Apparatus[2]

3. Summary of Method

3.1 Saturated soil samples are placed in contact with a saturated porous plate installed within a pressure chamber. The bottom of each plate is covered by a rubber membrane, or otherwise sealed to be airtight. The bottom of each plate is maintained at atmospheric pressure by means of a small drain tube or opening through the side of the pressure chamber. A desired air pressure admitted to the pressure chamber, and consequently to the top of the porous plate, creates a pressure drop across the porous plate. The saturated soil samples on the plates establish equilibrium with the water in the plate. The water, held at a tension less than the pressure drop across the porous plate, will then move out of the soil, through the plate, and out through the drain tube. When water has ceased to flow from the sample and porous plate (indicating equilibrium for that particular tension), the moisture content of each sample is determined. A series of these tests at various tensions is required to prepare a complete curve of the capillary-moisture relationship for any particular soil.

4. Apparatus

4.1 An assembly of the apparatus is shown in Fig. 1.

4.1.1 *Porous Plate Apparatus*, consisting of the following:

4.1.1.1 *Pressure Container*, (such as a pressure cooker), of approximately 15-L (16-qt) capacity.

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.04 on Hydrologic Properties of Soils and Rocks.

Current edition approved Sept. 13, 1968. Originally issued as D 2325 – 64 T. Last previous edition D 2325 – 64 T.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

4.1.1.2 *Porous Ceramic Plates*, 1 to 4 (see Fig. 2), approximately 280 mm (11¼ in.) in diameter and 6 mm (¼ in.) in thickness, with an air entry value of 203 kPa (2 atm).

4.1.1.3 *Brass Spout*—The brass spout (one per porous plate) shall consist of a brass tube and associated washers, gaskets, and brass nuts. It shall provide an airtight joint when inserted through the porous plate 38 mm (1.5 in.) from the edge of the plate. The length of the unthreaded portion of the brass tube shall be 9.5 mm (⅜ in.); the length of the threaded portion shall be 15.8 mm (⅝ in.); the inside diameter of the tube shall be 1.7 mm (1/16 in.); the outside diameter of the upper unthreaded portion shall be 4 mm (5/32 in.); the outside diameter of the lower threaded portion shall be 4.8 mm (3/16 in.). The tap size for the hole through the porous plate shall be 5.5 mm (7/32 in.).

4.1.1.4 *Disks of 10-mesh Brass Screen*, from 1 to 4, of slightly smaller diameter than bottom of porous ceramic plates.

4.1.1.5 *Rubber Membrane*—The membrane shall consist of sheet neoprene, 0.79 mm (1/32 in.) in thickness, with a hardness of 35 by the Shore Durometer. Place a disk of brass screen over the bottom of each porous ceramic plate to provide space for the flow of water between the membrane and the ceramic plate (see Fig. 2). Then place the rubber membrane snugly over the brass screen, glue it securely to the outer edge of the ceramic plate, and wrap the edge tightly with wire (see Fig. 2).

4.1.1.6 *Tubing*—A flexible tubing tube, 3 mm (⅛ in.) in diameter, to carry the outflow water from the brass spout on each porous plate to a short length of rigid tubing passing through a rubber stopper installed in the wall of the pressure container.

4.1.1.7 *Assembly*—Support and separate plates by means of plastic plugs approximately 15 mm (0.6 in.) in diameter by 25 mm (1 in.) in length.

4.1.2 *Sample Retainer Rings*—Rigid plastic rings, 10 mm (0.4 in.) in height by 50 mm (2 in.) in inside diameter with a wall thickness of approximately 3 mm (⅛ in.), capable of holding approximately 25 g of disturbed sample. The same rings shall be used to contain the undisturbed samples. The rings shall be numbered in pairs (A1 and A'1, A2 and A'2, etc.).

4.1.3 *Manometer*, mercury type for measuring pressures of 34 to 101 kPa (⅓ to 1 atm); water type for measuring pressures below 34 kPa.

4.1.4 *Pressure Regulator*—A sensitive control valve or regulator for fine pressure control.

4.1.5 *Water Trap and Humidity Control*—A transparent plastic cylinder approximately 100 mm (4 in.) in outside diameter by 150 mm (6 in.) high with a wall thickness approximately 6 mm (¼ in.). The cylinder shall be sealed on both ends with an air inlet and outlet near the top of the cylinder and a drain outlet approximately 25 mm (1 in.) from the bottom. (This cylinder traps water if back pressure draws water out of the pressure container, and the 25 mm (1 in.) of water in the bottom maintains a humid atmosphere for the air to pass through.)

4.1.6 *Test Specimen Cutter*—A cylindrical ring with a sharp cutting edge on one end. The inside diameter shall be 50 mm (2 in.) and the height shall be 20 mm (0.8 in.). A metal blank 50 mm (2 in.) in diameter by 10 mm (0.4 in.) thick with a detachable handle, shall be available.

4.1.7 *Spatula*—A short, wide-blade spatula (or small pancake turner) for removing samples from pressure plates.

4.1.8 *Test Specimen Packer Disk*—A flat steel disk, 50 mm (2 in.) in diameter and 3 mm (⅛ in.) thick, that can be loaded to 9000 g.

NOTE 2—A pocket-type penetrometer has been found convenient for loading the disk.

4.1.9 *Plate Hook*—A three-pronged hook assembly for lifting porous plate.

4.1.10 *Moisture Sample Containers*—Suitable containers made of material resistant to corrosion and not subject to change in weight or disintegration on repeated heating and cooling. Containers shall have close-fitting lids to prevent loss of moisture from samples before initial weighing and to prevent absorption of moisture from the atmosphere following drying and before final weighing. One container is needed for each moisture content determination. Containers should be 60 or 90-cm^3 (2 or 3-oz) capacity. The containers are numbered in pairs to coincide with the retainer rings.

4.1.11 *Saturation Tray*—A waterproof tray about 30 mm (1.2 in.) in depth, large enough to hold at least 4 porous plates while samples are being saturated thereon.

4.1.12 *Balance*—A balance with a capacity of at least 200 g and sensitive to 0.01 g.

4.1.13 *Desiccator*—A desiccator of suitable size to hold samples for cooling after removal from the oven.

4.1.14 *Oven*—A thermostatically controlled drying oven capable of maintaining temperatures at 110 ± 5°C (230 ± 9°F).

4.1.15 *Trimmers*—Wire saw, bevelled straightedge, spatula, and other small tools for trimming the test specimen.

5. Samples

5.1 Make tests in duplicate on specimens cut from undisturbed or remolded samples or on specimens packed from loose disturbed samples. Place duplicates in paired retainer rings diametrically opposite each other on the pressure plate.

6. Preparation of Test Specimens from Disturbed Samples

6.1 Take a sample weighing approximately 25 g from the thoroughly mixed portion of the air-dried soil passing the 2.00-mm (No. 10) sieve, which has been obtained in accordance with Practice D 421.

6.2 Pour each sample into a retainer ring, pack, and level to fill the ring by pressing the top surface with the test specimen packer disk, using an applied force of 9000 g. Record on the report form the sample type, disturbed, and the numbers of the paired sample retainer rings containing the duplicate samples.

7. Preparation of Test Specimens from Undisturbed Samples

7.1 Cut a block of the material, from which the test specimen is to be prepared, with two plane faces. Determine and record the natural moisture content and dry unit weight of the sample block. Also record the direction (perpendicular or transverse) of the sampling in relation to the structural or depositional layers.

7.2 Place the test specimen cutter, with the cutting edge downward, on top of one of the plane faces and force the cutter down lightly and gradually as excess material is trimmed from the outside, using the minimum pressure required on the cutter. The trimming motions shall be from the cutter outward and downward, leaving a column of soil slightly larger than the outside diameter of the cutting edge. When the cutter is more than half full of soil, remove the excess at the bottom with the wire saw, invert the cutter and use a straightedge to make the soil flush. Invert the cutter again, place it on the smooth face of the metal blank, and carefully force it downward until the blank is flush. Remove the excess soil at the top with the wire saw, true the end with the straightedge, and remove the blank by means of the detachable handle.

7.3 Place the specimen cutter with the specimen downward, over a retainer ring, and use the metal blank to gently insert the test specimen into the ring. Record on the report the sample type, undisturbed, and the numbers of the paired sample retainer rings containing the duplicate samples.

7.4 Maintain the samples in closed containers until time for testing.

8. Preparation of Test Specimens from Compacted Samples

8.1 Compact the sample to a density and moisture content desired for anticipated service conditions in accordance with Test Methods D 698.

8.2 After the compacted sample has been ejected from the compaction mold, cut the test specimen by the process used for undisturbed samples as described in 7.1 through 7.4. Record on the report form, the sample type, compacted, and the numbers of the paired sample retainer rings containing the duplicate samples.

9. Saturating and Testing of Porous Plates

9.1 Install, by stacking with plastic plugs for spacers, as many porous plates in the pressure container as are to be used in the test. Fill the container with water, place the lid on the container and lock it in the closed position.

9.2 Close valves *C* and *E*, open valves *A* and *B*, and set the pressure at 101 kPa or 776 mmHg (1 atm or 15 psi) by adjusting the control valve on the pressure regulator and noting the pressure on the mercury manometer. Open valve *C* on the water trap cylinder. The measured rate of water outflow from the porous plates should be at least 10 mL/min for satisfactory operation of the plates.

9.3 Check the plates for air-entry value, as follows: release the air pressure by closing valve *C*, and by opening valve *D* on the lid, and empty the excess water from the pressure container and plates. Close and lock the lid of the container and apply the desired pressure. After approximately 10 min at this pressure, the outflow of water from the plate outlets should cease and there should be no bubbling of air from these outlets. This will indicate that the entry values for the plates are above the value of the applied pressure.

9.4 If trouble is encountered in air-pressure control, submerge the pressure container in water, with the pressure still on, to check for leaks in the lid gasket or container connections.

9.5 Exercise care that the pressure is released by means of valve *D* before the lid is opened or injury may occur to the operator or damage to the container.

10. Procedure

10.1 Place the required number of saturated porous plates in the saturation tray, one porous plate for approximately 12 sample retainer rings. Place the retainer rings containing duplicate samples prepared as described under Sections 6, 7 and 8, on a porous plate, locating duplicate samples diametrically opposite each other.

10.2 Place a control sample retainer ring in the center of the porous plate. In this retainer ring insert a disk of porous stone with standard sample dimensions, or pour and pack into this retainer ring a control sample consisting of a medium-textured soil with approximately equal parts of sand-, silt-, and clay-size particles. Record on the report form the number of the sample retainer ring containing the control sample.

10.3 Thoroughly saturate the samples by pouring 3 mm (⅛ in.) of water on each plate and gradually increasing the depth of water over a minimum period of several hours until the water is at the top edge of the sample. Hold the water at this depth for at least 24 h. Place surcharge weights equivalent to field overburden weight on top of the samples during the soaking period.

10.4 Remove the excess water from the plates with a suction hose or syringe. Place the plates in the pressure container with each plate supported by the 25-mm (1-in.) high plastic blocks. Insert the outflow tubes in perforated rubber stoppers in outlet holes where the plates are used; insert solid rubber stoppers in the holes where plates are not used. Place the container lid in position and lock it in the closed position.

10.5 Close valve *D* on the container lid and valve *E* on the water-trap outlet. Open air-control valves *A* and *B*, and adjust the pressure regulator until the desired pressure (Table 1) is observed on the mercury or water manometer.

10.6 Open valve *C* on the water trap outlet and admit the pressure to the pressure container. Allow the water from the outflow tubes to flow into 10-mL graduates so it can be noted when moisture equilibrium is obtained, at which time the test is discontinued. It may take 18 to 48 h for some soils to reach this equilibrium. Consider equilibrium to be reached when no water flows out of the outlet tubes during a ½-h period. As each plate of samples reaches equilibrium, place a pinch clamp on each outflow tube to prevent backflow of water to the samples when the pressure is released.

10.7 Close valve *C* and release the air pressure by opening valve *D* on the lid of the pressure chamber and remove the lid.

10.8 Lift each plate out by means of the plate hook. By means of the wide-blade spatula, quickly transfer the samples to the sample containers, and immediately weigh them on a balance. Record the weight of moist sample plus container (W'_{cw}) on the report form.

10.9 Dry the samples to constant weight in an oven at 110 ± 5°C (230 ± 9°F). Weigh the samples and record weights of oven-dry samples plus containers (W_{cl}) on the report form.

10.10 Follow the above procedure until moisture contents have been obtained for at least 5 different tensions between 10 and 101 kPa (0.1 and 1 atm).

11. Calculations

11.1 Calculate the moisture content of the soil, *w*, in percent as follows:

$$w = [(W'_{cw} - W_{cl})/(W_{cl} - W'_c)] \times 100$$
$$= (W_w/W_s) \times 100$$

where:

W'_{cw} = weight of container, ring and wet sample, g,
W_{cl} = weight of container, ring and dry sample, g,
W'_c = weight of container and ring, g,
W_w = weight of water, g, and
W_s = weight of dry soil, g.

11.2 Obtain the moisture content in volume percent by multiplying the moisture content by the dry unit weight (g/cm³).

12. Report

12.1 Report the moisture content, tension data, and calculations on the form "Capillary-Moisture Relations for Soils" (Fig. 3).

12.2 Plot the moisture content and tension data on a graph similar to that shown in Fig. 4. Extrapolate the curve to the total porosity (converted to percent of dry weight) on the zero tension line. If desired, the moisture data can also be converted to moisture content in volume percent or to degree of saturation, but this should be clearly identified on the graph (Fig. 4).

TABLE 1 Pressure Conversion Factors

Tension (atmosphere)[A]	Equivalent Pressure: Pounds per Square Inch (psi)	Equivalent Pressure: Millimetres of mercury	Equivalent Pressure: Millimetres of water	Equivalent Pressure: Kilopascals	Capillary Head: Feet of water	Capillary Head: Centimetres of water
0.1	1.5	76	1 033	10	3.4	103.3
0.2	2.9	152	2 066	20	6.8	206.6
0.3	4.4	228	3 099	30	10.2	309.9
0.33	5.3	253	3 440	33	11.3	340.9
0.4	5.9	304	4 132	40	13.6	413.2
0.5	7.4	380	5 165	51	16.9	516.5
0.6	8.8	456	6 198	61	20.3	619.8
0.7	10.3	532	7 231	71	23.7	723.1
0.8	11.8	608	8 264	81	27.1	826.4
0.9	13.2	684	9 297	91	30.5	929.7
1.0	14.7	760	10 330	101	33.9	1 033.0

[A] 1 atmosphere = 760 mm mercury (0°C) = 14.7 psi = 406.8 in. water (39.2°F) = 1033 cm water (4°C) = 33.899 ft water (39.2°F) = 101 kPa.

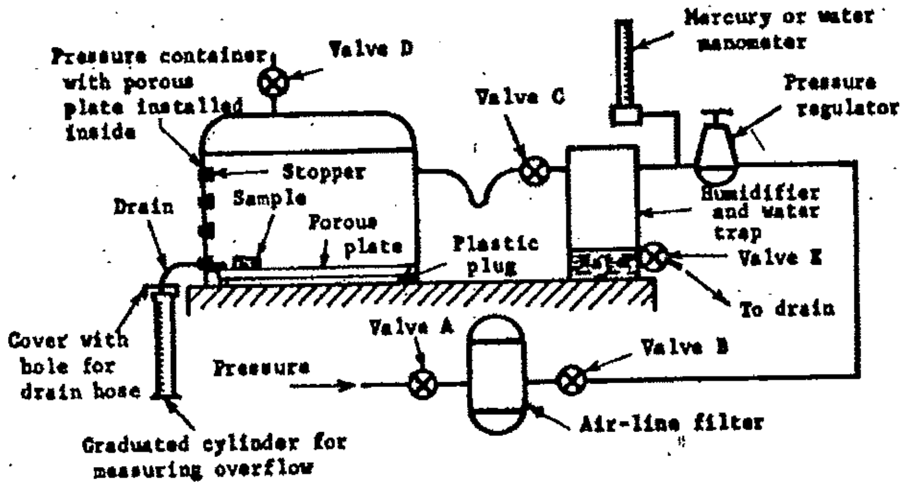


FIG. 3 Suggested Porous Plate Tension Apparatus

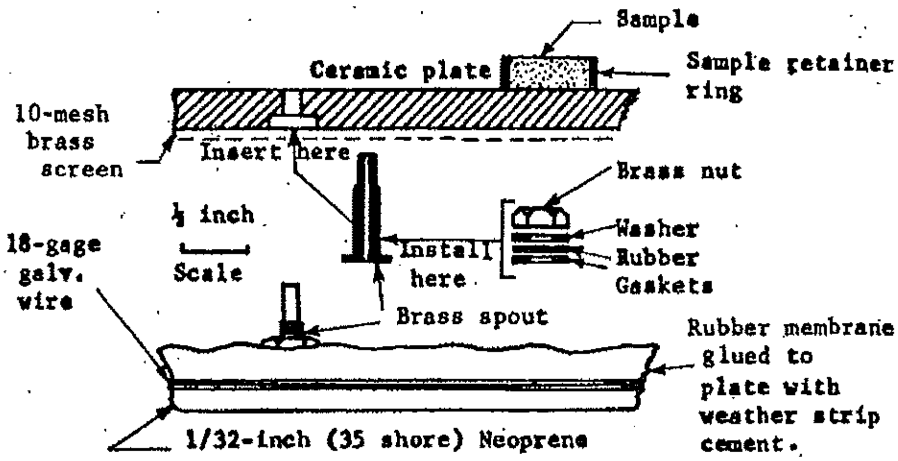


FIG. 2 Porous Plate Construction

CAPILLARY-MOISTURE RELATIONS FOR SOILS

Location: ______________________ Sample No: ______________
Depth: ______________________ Date: ______________
Identification: ______________________ Soil type: ______________
Initial moisture content: ______________ %; Sample type: ______________
Initial dry unit weight: ______________ g/cm³; ______________ lb/ft³; Porosity: ______________ %
Specific gravity: ______________; Remarks: ______________

(1) Tension, ______						
(2) Container number						
(3) Wt of container and ring + wet sample, g (W_{rws})						
(4) Wt of container and ring + dry sample, g (W_{cs})						
(5) Wt of moisture, g (W_w), (3 − 4)						
(6) Wt of container and ring g (W_c)						
(7) Wt of dry sample, g (W_s) (4 − 6)						
(8) Moisture content, % (ω) (5 ÷ 7) × 100						
(9) Unit wt of dry sample (γ_d)						
(10) Moisture content, volume percent (ω_v) (8 × 9)						

FIG. 3 Laboratory Form for Capillary Moisture Relations for Soils

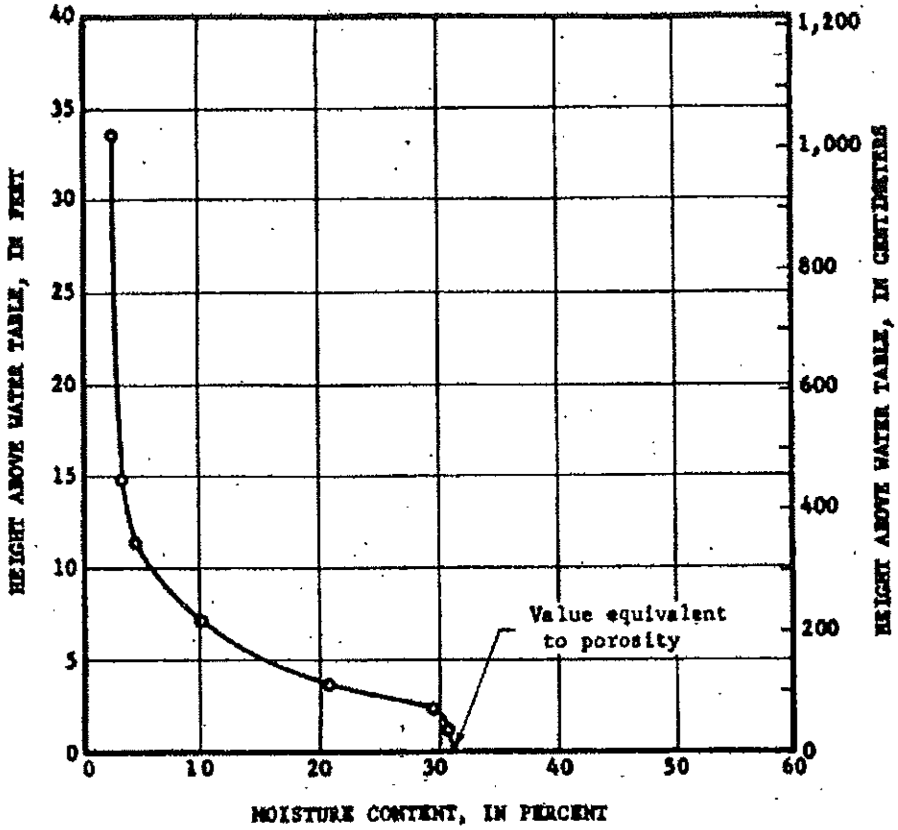


FIG. 4 Example of Data on Capillary-Moisture Relations of Soils

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, Pa. 19103.

Standard Test Method for
PERMEABILITY OF GRANULAR SOILS (CONSTANT HEAD)[1]

This standard is issued under the fixed designation D 2434; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] Note—Section 2 was added editorially and subsequent sections renumbered in July 1984.

1. Scope

1.1 This test method covers the determination of the coefficient of permeability by a constant-head method for the laminar flow of water through granular soils. The procedure is to establish representative values of the coefficient of permeability of granular soils that may occur in natural deposits as placed in embankments, or when used as base courses under pavements. In order to limit consolidation influences during testing, this procedure is limited to disturbed granular soils containing not more than 10 % soil passing the 75-μm (No. 200) sieve.

2. Applicable Documents

2.1 *ASTM Standards:*

D422 Method for Particle-Size Analysis of Soils[2]

D2049 Test Method for Relative Density of Cohesionless Soils[3]

3. Fundamental Test Conditions

3.1 The following ideal test conditions are prerequisites for the laminar flow of water through granular soils under constant-head conditions:

3.1.1 Continuity of flow with no soil volume change during a test,

3.1.2 Flow with the soil voids saturated with water and no air bubbles in the soil voids,

3.1.3 Flow in the steady state with no changes in hydraulic gradient, and

3.1.4 Direct proportionality of velocity of flow with hydraulic gradients below certain values, at which turbulent flow starts.

3.2. All other types of flow involving partial saturation of soil voids, turbulent flow, and unsteady state of flow are transient in character and yield variable and time-dependent coefficients of permeability; therefore, they require special test conditions and procedures.

4. Apparatus

4.1 *Permeameters,* as shown in Fig. 1, shall have specimen cylinders with minimum diameters approximately 8 or 12 times the maximum particle size in accordance with Table 1. The permeameter should be fitted with: (*1*) a porous disk or suitable reinforced screen at the bottom with a permeability greater than that of the soil specimen, but with openings sufficiently small (not larger than 10 % finer size) to prevent movement of particles; (*2*) manometer outlets for measuring the loss of head, *h*, over a length, *l*, equivalent to at least the diameter of the cylinder; (*3*) a porous disk or suitable reinforced screen with a spring attached to the top, or any other device, for applying a light spring pressure of 22 to 45-N (5 to 10-lbf) total load, when the top plate is attached in place. This will hold the placement density and volume of soil without significant change during the saturation of the specimen and the permeability testing to satisfy the requirement prescribed in 3.1.1.

4.2 *Constant-Head Filter Tank,* as shown in Fig. 1, to supply water and to remove most of

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.04 on Hydrologic Properties of Soil and Rocks.

Current edition approved Sept. 13, 1968. Originally issued 1965. Replaces D 2434 – 65 T.

[2] *Annual Book of ASTM Standards,* Vol 04.08.

[3] Discontinued—See *1983 Annual Book of ASTM Standards,* Vol 04.08.

the air from tap water, fitted with suitable control valves to maintain conditions described in 3.1.2.

NOTE 1—De-aired water may be used if preferred.

4.3 *Large Funnels,* fitted with special cylindrical spouts 25 mm (1 in.) in diameter for 9.5-mm (⅜-in.) maximum size particles and 13 mm (½ in.) in diameter for 2.00-mm (No. 10) maximum size particles. The length of the spout should be greater than the full length of the permeability chamber—at least 150 mm (6 in.).

4.4 *Specimen Compaction Equipment*[2]—Compaction equipment as deemed desirable may be used. The following are suggested: a vibrating tamper fitted with a tamping foot 51 mm (2 in.) in diameter; a sliding tamper with a tamping foot 51 mm (2 in.) in diameter, and a rod for sliding weights of 100 g (0.25 lb) (for sands) to 1 kg (2.25 lb) (for soils with a large gravel content), having an adjustable height of drop to 102 mm (4 in.) for sands and 203 mm (8 in.) for soils with large gravel contents.

4.5 *Vacuum Pump or Water-Faucet Aspirator,* for evacuating and for saturating soil specimens under full vacuum (see Fig. 2).

4.6 *Manometer Tubes,* with metric scales for measuring head of water.

4.7 *Balance,* of 2-kg (4.4-lb) capacity, sensitive to 1 g (0.002 lb).

4.8. *Scoop,* with a capacity of about 100 g (0.25 lb) of soil.

4.9 *Miscellaneous Apparatus*—Thermometers, clock with sweep second hand, 250-mL graduate, quart jar, mixing pan, etc.

5. Sample

5.1 A representative sample of air-dried granular soil, containing less than 10 % of the material passing the 75-μm (No. 200) sieve and equal to an amount sufficient to satisfy the requirements prescribed in 5.2 and 5.3, shall be selected by the method of quartering.

5.2 A sieve analysis (See Method D 422) shall be made on a representative sample of the complete soil prior to the permeability test. Any particles larger than 19 mm (¾ in.) shall be separated out by sieving (Method D 422). This oversize material shall not be used for the permeability test, but the percentage of the oversize material shall be recorded.

NOTE 2—In order to establish representative values of coefficients of permeabilities for the range that may exist in the situation being investigated, samples of the finer, average, and coarser soils should be obtained for testing.

5.3 From the material from which the oversize has been removed (see 5.2), select by the method of quartering, a sample for testing equal to an amount approximately twice that required for filling the permeameter chamber.

6. Preparation of Specimens

6.1 The size of permeameter to be used shall be as prescribed in Table 1.

6.2 Make the following initial measurements in centimetres or square centimetres and record on the data sheet (Fig. 3); the inside diameter, D, of the permeameter; the length, L, between manometer outlets; the depth, H_1, measured at four symmetrically spaced points from the upper surface of the top plate of the permeability cylinder to the top of the upper porous stone or screen temporarily placed on the lower porous plate or screen. This automatically deducts the thickness of the upper porous plate or screen from the height measurements used to determine the volume of soil placed in the permeability cylinder. Use a duplicate top plate containing four large symmetrically spaced openings through which the necessary measurements can be made to determine the average value for H_1. Calculate the cross-sectional area, A, of the specimen.

6.3 Take a small portion of the sample selected as prescribed in 5.3 for water content determinations. Record the weight of the remaining air-dried sample (see 5.3), W_1, for unit weight determinations.

6.4 Place the prepared soil by one of the following procedures in uniform thin layers approximately equal in thickness after compaction to the maximum size of particle, but not less than approximately 15 mm (0.60 in.).

6.4.1 For soils having a maximum size of 9.5 mm (⅜ in.) or less, place the appropriate size of funnel, as prescribed in 4.3, in the permeability device with the spout in contact with the lower porous plate or screen, or previously formed layer, and fill the funnel with sufficient soil to form a layer, taking soil from different areas of the sample in the pan. Lift the funnel by 15 mm (0.60 in.), or approximately the unconsolidated layer thickness to be formed, and spread the soil with a slow spiral motion, working from the perimeter of the device toward the center, so that a uniform layer is formed. Remix the soil in the pan for each successive layer to reduce segregation caused by taking soil from the pan.

6.4.2 For soils with a maximum size greater than 9.5 mm (⅜ in.), spread the soil from a scoop. Uniform spreading can be obtained by sliding a scoopful of soil in a nearly horizontal position down along the inside surface of the device to the bottom or to the formed layer, then tilting the scoop and drawing it toward the center with a single slow motion; this allows the soil to run smoothly from the scoop in a windrow without segregation. Turn the permeability cylinder sufficiently for the next scoopful, thus progressing around the inside perimeter to form a uniform compacted layer of a thickness equal to the maximum particle size.

6.5 Compact successive layers of soil to the desired relative density by appropriate procedures, as follows, to a height of about 2 cm (0.8 in.) above the upper manometer outlet.

6.5.1 *Minimum Density (0 % Relative Density)*—Continue placing layers of soil in succession by one of the procedures described in 6.4.1 or 6.4.2 until the device is filled to the proper level.

6.5.2 *Maximum Density (100 % Relative Density):*

6.5.2.1 *Compaction by Vibrating Tamper*—Compact each layer of soil thoroughly with the vibrating tamper, distributing the light tamping action uniformly over the surface of the layer in a regular pattern. The pressure of contact and the length of time of the vibrating action at each spot should not cause soil to escape from beneath the edges of the tamping foot, thus tending to loosen the layer. Make a sufficient number of coverages to produce maximum density, as evidenced by practically no visible motion of surface particles adjacent to the edges of the tamping foot.

6.5.2.2 *Compaction by Sliding Weight Tamper*—Compact each layer of soil thoroughly by tamping blows uniformly distributed over the surface of the layer. Adjust the height of drop and give sufficient coverages to produce maximum density, depending on the coarseness and gravel content of the soil.

6.5.2.3 *Compaction by Other Methods*—Compaction may be accomplished by other approved methods, such as by vibratory packer equipment, where care is taken to obtain a uniform specimen without segregation of particle sizes (See Test Method D 2049).

6.5.3 *Relative Density Intermediate Between 0 and 100 %*—By trial in a separate container of the same diameter as the permeability cylinder, adjust the compaction to obtain reproducible values of relative density. Compact the soil in the permeability cylinder by these procedures in thin layers to a height about 2.0 cm (0.80 in.) above the upper manometer outlet.

NOTE 3—In order to bracket, systematically and representatively, the relative density conditions that may govern in natural deposits or in compacted embankments, a series of permeability tests should be made to bracket the range of field relative densities.

6.6 *Preparation of Specimen for Permeability Test:*

6.6.1 Level the upper surface of the soil by placing the upper porous plate or screen in position and by rotating it gently back and forth.

6.6.2 Measure and record: the final height of specimen, $H_1 - H_2$, by measuring the depth, H_2, from the upper surface of the perforated top plate employed to measure H_1 to the top of the upper porous plate or screen at four symmetrically spaced points after compressing the spring lightly to seat the porous plate or screen during the measurements; the final weight of air-dried soil used in the test ($W_1 - W_2'$) by weighing the remainder of soil, W_2, left in the pan. Compute and record the unit weights, void ratio, and relative density of the test specimen.

6.6.3 With its gasket in place, press down the top plate against the spring and attach it securely to the top of the permeameter cylinder, making an air-tight seal. This satisfies the condition described in 3.1.1 of holding the initial density without significant volume change during the test.

6.6.4 Using a vacuum pump or suitable aspirator, evacuate the specimen under 50 cm (20 in.) Hg minimum for 15 min to remove air adhering to soil particles and from the voids. Follow the evacuation by a slow saturation of the specimen from the bottom upward (Fig. 2) under full vacuum in order to free any remaining air in the specimen. Continued saturation of the specimen can be maintained more adequately by the use of (*1*) deaired water, or (*2*) water maintained in an in-flow temperature sufficiently high to cause a decreasing temperature gradient in the specimen during the test. Native water or water of low mineral content (Note 4) should be used for the test, but in any case the fluid should be described on the report form (Fig. 3). This satisfies the condition described in 3.1.2 for saturation of soil voids.

NOTE 4—Native water is the water occurring in the rock or soil *in situ*. It should be used if possible, but it (as well as de-aired water) may be a refinement not ordinarily feasible for large-scale production testing.

6.6.5 After the specimen has been saturated and the permeameter is full of water, close the bottom valve on the outlet tube (Fig. 2) and disconnect the vacuum. Care should be taken to ensure that the permeability flow system and the manometer system are free of air and are working satisfactorily. Fill the inlet tube with water from the constant-head tank by slightly opening the filter tank valve. Then connect the inlet tube to the top of the permeameter, open the inlet valve slightly and open the manometer outlet cocks slightly, to allow water to flow, thus freeing them of air. Connect the water manometer tubes to the manometer outlets and fill with water to remove the air. Close the inlet valve and open the outlet valve to allow the water in the manometer tubes to reach their stable water level under zero head.

7. Procedure

7.1 Open the inlet valve from the filter tank slightly for the first run to conditions described in 3.1.3, delay measurements of quantity of flow and heat until a stable head condition without appreciable drift in water manometer levels is attained. Measure and record the time, t, head, h (the difference in level in the manometers), quantity of flow, Q, and water temperature, T.

7.2 Repeat test runs at heads increasing by 0.5 cm in order to establish accurately the region of laminar flow with velocity, v, (where $v = Q/At$), directly proportional to hydraulic gradient, i (where $i = h/L$). When departures from the linear relation become apparent, indicating the initiation of turbulent flow conditions, 1-cm intervals of head may be used to carry the test run sufficiently along in the region of turbulent flow to define this region if it is significant for field conditions.

NOTE 5—Much lower values of hydraulic gradient, h/L, are required than generally recognized, in order to ensure laminar flow conditions. The following values are suggested: loose compactness ratings, h/L from 0.2 to 0.3, and dense compactness ratings, h/L from 0.3 to 0.5, the lower values of h/L applying to coarser soils and the higher values to finer soils.

7.3 At the completion of the permeability test, drain the specimen and inspect it to establish whether it was essentially homogeneous and isotropic in character. Any light and dark alternating horizontal streaks or layers are evidence of segregation of fines.

8. Calculations

8.1 Calculate the coefficient of permeability, k, as follows:

$$k = QL/Ath$$

where:

k = coefficient of permeability,
Q = quantity of water discharged,
L = distance between manometers,
A = cross-sectional area of specimen,
t = total time of discharge,
h = difference in head on manometers.

8.2 Correct the permeability to that for 20°C (68°F) by multiplying k (see 8.1) by the ratio of the viscosity of water at test temperature to the viscosity of water at 20°C (68°F).

9. Report

9.1 The report of permeability test shall include the following information:

9.1.1 Project, dates, sample number, location, depth, and any other pertinent information,

9.1.2 Grain size analysis, classification, maximum particle size, and percentage of any oversize material not used,

9.1.3 Dry unit weight, void ratio, relative density as placed, and maximum and minimum densities,

9.1.4 A statement of any departures from these test conditions, so the results can be evaluated and used,

9.1.5 Complete test data, as indicated in the laboratory form for test data (see Fig. 3), and

9.1.6 Test curves plotting velocity, Q/At, versus hydraulic gradient, h/L, covering the ranges of soil identifications and of relative densities.

TABLE 1 Cylinder Diameter

Maximum Particle Size Lies Between Sieve Openings	Minimum Cylinder Diameter			
	Less than 35 % of Total Soil Retained on Sieve Opening		More than 35 % of Total Soil Retained on Sieve Opening	
	2.00-mm (No. 10)	9.5-mm (⅜-in.)	2.00-mm (No. 10)	9.5-mm (⅜-in.)
2.00-mm (No. 10) and 9.5-mm (⅜-in.)	76 mm (3 in.)	...	114 mm (4.5 in.)	...
9.5-mm (⅜-in.) and 19.0-mm (¾-in.)	...	152 mm (6 in.)	...	229 mm (9 in.)

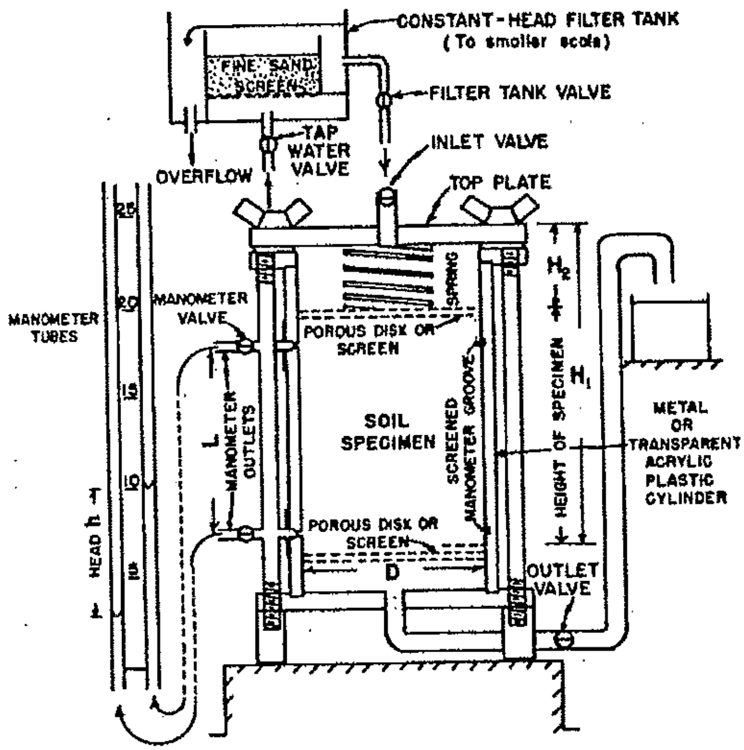


FIG. 1 Constant-Head Permeameter

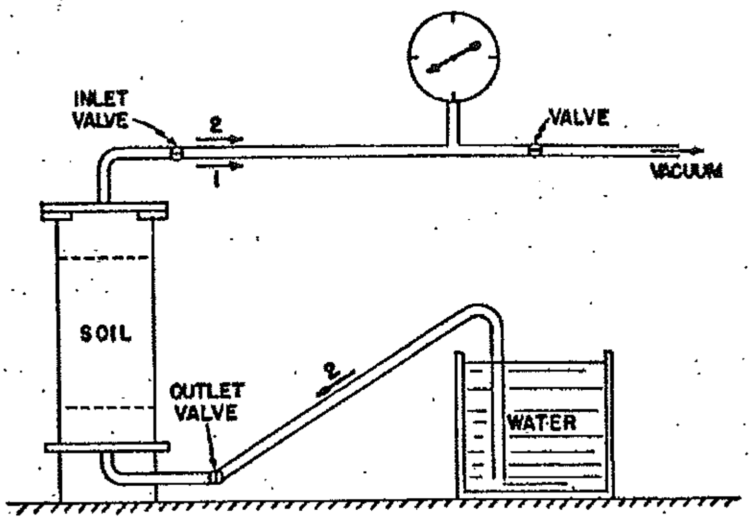


FIG. 2 Device for Evacuating and Saturating Specimen

PERMEABILITY TEST ON GRANULAR SOIL

Test No. Date of Test
Location of Sample Date Sampled Report
Boring— Sample— Depth—

(*a*) Description of Soil ______

Materials Used: ______

(*b*) Unit Weight Determination:

Diameter, *D*, cm
Area, *A*, cm^2
Length, *L*, cm

Height Before, H_1
Height After, H_2
Height Net, cm

Weight Before, W_1
Weight After, W_2
Weight Net, g

Moisture Content (air-dried) ______

W (max) Dry Unit Weight, lb/ft^3 *W* ______

W (min) Void Ratio, *e*, ______

Relative Density, *RD* ______

(*c*) Permeability Test (Degree of Compactness)

Test No.	Manometers		Head, *h* cm	*Q* cm^3	*t* s	*Q/At*	*h/L*	Temperature, °C	*k* cm/s
	H_1	H_2							
1									
2									
3									
4									
5									
6									

FIG. 3 Permeability Test Data Sheet

APPENDIX B

Evaluation of SFWMD Hydraulic Conductivity Equation

Review of Technical Publication 87-5, *"Field Testing of Exfiltration Systems"*, SFWMD (December, 1987), indicates that the method of hydraulic conductivity measurement and subsequent calculation of exfiltration rate *"substantially overestimates exfiltration trench capacity"*. Field load test comparison data indicate overestimation of exfiltration rates by 32 to 158%. The results of this study generally confirm our evaluation of K_{SF} as being inappropriate and suggests using alternate methods of analyses.

APPENDIX C

Glossary

Appendix C
Glossary

Aquifer
A porous, water-bearing geologic unit. Generally restricted to materials capable of yielding an appreciable quantity of water.
Borehole Permeability Test
In-situ permeability test conducted in uncased auger hole.
Capillary Rise
The height above a free water surface that water will rise by capillary action.
Capillary Suction
Capillary force that draws water against the force of gravity in dry soils.
Closed Watershed
A surface drainage basin with no positive surface outfall.
Coefficient of Permeability
A property of a soil which when multiplied by hydraulic gradient (i) yields velocity of flow. Permeability is generally a constant which does not vary for a particular in-situ soil. The coefficient of permeability typically is applied to the movement of all fluids through porous media. In this text, coefficient of permeability applies only to the movement of water through soil. Hydraulic conductivity is the coefficient of permeability when water is the fluid passing through the soil.
Detention
The delay of stormwater runoff prior to discharge into receiving waters.
Detention Pond
An open excavation or depression in the ground surface used for temporary storage of stormwater prior to release downstream.
Discharge Structure
A structural device usually of concrete, metal, etc., through which water is discharged from a project to the receiving water.
Effective Particle Size
The diameter of particles, spherical in shape, equal in size and arranged in a given manner of a sample of granular material that would have the same transmission constant as the actual material under consideration.
Effective Storage Coefficient
The fraction of the total soil porosity above the groundwater that is not occupied by moisture and residual air. Generally inversely proportional to the soil moisture content and soil density. Used interchangeably with "fillable porosity".
Elevation
The height in feet above mean sea level according to the National Geodetic Vertical Datum (NGVD) of1929 or other relative reference datum.
Exfiltration
Process by which water in a surface impoundment discharges via soil infiltration.
Exfiltration Trench
A trench excavated in permeable soil, utilizing either a perforated slotted pipe back-filled with coarse aggregate; or with vertical walls and porous rock with a slab cover

to form a channel void of pipe conduit and backfill.

Groundwater

Subsurface water occupying the saturated zone. In a strict sense, the term applies only to the water below the water table.

Groundwater Recharge

Water descending to the zone of saturation from the atmosphere which gravitates to the zone of saturation under natural conditions or which is added to the zone of saturation by infiltration of stormwater from retention ponds.

Groundwater Table

The upper surface of the zone of saturation, except where these surfaces formed by an impermeable body (perched groundwater table). Same as "groundwater".

Hydraulic Head

The height of a free surface of a body of water above a given point.

Hydrograph

A graph of direct runoff resulting from rainfall generated over the watershed area during a specified storm event.

Impermeable Strata

A strata in which texture is such that water cannot move through it under pressures ordinarily found in subsurface water.

Impervious

Not allowing or allowing only with great difficulty the movement of water.

Infiltration

Process by which surface water passes through surface soil, synonymous with percolation and unsaturated flow.

Infiltration Basin

Natural or excavated depression in the ground surface for storing infiltrating stormwater.

Infiltration Pond

A natural or man-made surface reservoir for collection and infiltration of stormwater.

Infiltration Rate

The rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour, feet per day or cubic feet per second.

Monitoring Well

A piezometer, which is used for water quality sampling. For a monitoring well, the annulus is always grouted to land surface and the well head is encased in a concrete pad. Since the monitoring well is used for water quality sampling, only threaded casing and screen are used.

Mounding

A condition that exists when the water table rises to the elevation of the bottom of an infiltration system. When this occurs, percolation rates are controlled by the groundwater gradient laterally away from the system rather than vertical infiltration rates.

Normal Water Level

The design starting water elevation used when determining stage/storage design, computations in retention or detention areas. For dry retention ponds, the normal water level is the pond bottom level.

Observation Well

A piezometer which is typically used to measure the drawdown produced by an pumping well or well point system.

Off-line Treatment System

A system only for water quality treatment that collects project runoff and has no direct discharge capability other than percolation and evaporation. A system utilizing detention with effluent filtration is not an off-line treatment system.

On-line Treatment System

A dual purpose system that collects project runoff for both water quality and water quantity requirements. Water quality volumes are recovered through percolation and evaporation while water quantity volumes are recovered through a combination of percolation, evaporation and surface drainage.

Open Hole Section

The portion of a soil boring which is not lined with casing or a screen. For deep water supply wells, the open hole section is typically the portion of the well that extends into the limestone and is below the casing.

Overflow Elevation

A design elevation of the discharge structure at or below which water is contained behind the structure.

Perched Groundwater Table

Groundwater that is separated from the main body of groundwater by an aquitard or aquiclude.

Percolation

The movement or flow of water through the pores of a soil or other porous medium. Generally used interchangebly with "infiltration".

Pervious Soil

Soil containing voids through which water will move under ordinary hydrostatic pressure.

Piezometer

A groundwater level measuring device which usually consist of a solid casing which extends to a predetermined depth below ground surface with a perforated section (screen) in the bottom of the casing. A sand filter pack is usually installed around the screen and the annulus above the screen is either filled with sand or grout to land surface.

Piezometer Permeability Test

In-situ permeability test conducted in a piezometer. Both steady state and unsteady state methods can be used.

Pollutants

Harmful or objectionable contaminants in water.

Positive System

A storm drain system which pipes discharge directly into a stream, river, canal, pond or lake.

Pumping Well

A piezometer from which groundwater is withdrawn during a pumping test.

Recharge

Addition of water to the zone of saturation from precipitation or infiltration.

Retention

The prevention of direct discharge of storm runoff into a receiving water; included as examples are such systems which discharge via percolation, exfiltration and evaporation processes and which generally have residence times less than three days.

Retention Pond

See infiltration basin.

Retention System

A facility designed for the purpose of storing stormwater.

Runoff

That part of precipitation which runs off the surface of a drainage area and recharges a retention system, stream or other body of water or drain or sewer.

Saturated Flow

Movement of water through soil under hydrostatic pressure with all void spaces of the soil filled with water.

Saturated Soil

Soil that has its void spaces filled with water to the point at which runoff occurs.

Seasonal High Water Level

Elevation to which the groundwater or surface water can be expected to rise due to a normal wet season.

Soil Porosity

The percentage of the soil volume that is not occupied by solid particles, including all porous space filled with air and water.

Storm Duration

The period or length of the storm.

Storage Basin

A basin excavated in the earth for detention or retention of water for future flow.

Surface Water

Water appearing on the surface in a diffused state with no permanent source of supply or regular course for a considerable time; as distinguished from water appearing in water courses, lakes or ponds.

Swale
A slight depression in the ground surface where water collects.

Transmission Zone
A moisture zone draining infiltration of water which is characterized by an essentially constant moisture content. Nearly the entire depth of the profile of the wetted soil will be in this zone.

Unsaturated Flow
Flow of water through unsaturated or dry soil. The same as "unsaturated infiltration".

Watershed
The catchment area for rainfall which is delineated as the drainage area producing a runoff.

Wells
Wells consist of shallow to deep vertical excavations, generally with perforated or slotted pipe backfilled with selected aggregate.

Well Screen
A special form of slotted or perforated well casing that admits water from an aquifer consisting of unconsolidated granular material while preventing the granular material from entering the well.