Special Publication SJ93-SP10

Full-Scale Hydrologic Monitoring of Stormwater Retention Ponds and Recommended Hydro-Geotechnical Design Methodologies

Indian River Lagoon Basin
St. Johns River Water Management District
Florida

Prepared by:

Professional Service Industries, Inc.
Jammal & Associates Division

Prepared for:

St. Johns River Water Management District
Palatka, Florida

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VOLUME I

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AUTHORIZATION AND ACKNOWLEDGEMENT

This report was prepared under the terms of an agreement dated July 3, 1990 between PSI/Jammal and Associates Division and the Governing Board of the St. Johns River Water Management District. The report covers all of the tasks contained in the mutually agreed upon scope of work of the said agreement. We appreciate the cooperation and guidance of the St. Johns River Management District professional staff, particularly Messrs. Harold A. Wilkening, III, P.E., and Mr. Perry Jennings throughout the duration of the study.

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EXECUTIVE SUMMARY

Background and Study Purpose

Water management districts require that new developments have a comprehensive stormwater management system which incorporates one or more Best Management Practices or BMP's to ensure that stormwater runoff is being effectively treated. Some of the most common BMP's recommended by the St. Johns River Water Management District (SJRWMD) are: *retention systems*, wet detention ponds, filtration systems, underdrain systems, swales, exfiltration trenches, and wetland stormwater management systems.

The focus of this study is on *retention systems*. Retention systems are storage areas designed to store a defined quantity of runoff, allowing the runoff to percolate through the permeable soils of the basin floor and side slopes into the shallow ground water aquifer. Unlike filtration or underdrain systems which rely on artificial methods like drainage pipes, stormwater in retention systems is drawn down by natural soil infiltration and dissipation into the ground water table. The most common type of retention system consists of man-made or natural depressional areas where the floor is graded as flat as possible and turf is established to promote infiltration and stabilize basin slopes. Soil permeability and water table conditions must be such that the retention system can percolate the desired runoff volume within a specified time following a storm event.

Computer-based ground water flow models are now routinely used by practicing hydrogeologists and engineers to predict the time for percolation of the retained runoff volume. The reliability of the output of these models cannot exceed the reliability of the input data. Input data assessment is probably the most neglected single task in the ground water modeling process. As our numerical modeling expertise to simulate increasingly complex systems increases, the accuracy of future simulations hinges on the quality and completeness of the input data.

St. Johns River Water Management District (SJRWMD) has recognized these difficulties during the past several years during its review of numerous permit applications pertaining to the design, construction and operation of hundreds of stormwater retention ponds within its permitting jurisdiction. With this motivation, SJRWMD contracted PSI/Jammal & Associates Division to conduct full-scale, short-term and long-term hydrologic monitoring of retention ponds. This field data was used, inter alia, to evaluate and to recommend hydrogeologic characterization techniques and design methodologies for computing the time of percolation of impounded stormwater runoff. Although the results of this study and the design recommendations have district-wide applicability, all of the ponds selected for instrumentation were located within the Indian River Lagoon Basin of SJRWMD where soil infiltration potential is, for the most part, limited.

Selection of Study Sites

A total of four ponds were selected for instrumentation after preliminary review of twenty potential ponds. Two of the four ponds are located on the western bank of the Indian River, between U.S. Highway 1 and the river. One of these ponds is at Tom Statham Park which is just east of the Titusville-Cocoa Airport and south of the NASA Causeway. The other pond is at the Fisherman's Landing Park, south of the community of Grant in Brevard County.

The third pond is located at the Tutor Time Child Care facility on Merritt Island, southeast of the intersection of the NASA Causeway and the Kennedy Parkway, while the fourth pond is within the Airport Warehouses industrial park at the Merritt Island Airport.

Hydrogeologic Testing

As a first step, subsurface explorations were conducted at each of the selected sites using field techniques routinely employed by geotechnical engineering consultants in Florida. These tests included Standard Penetration Test (SPT) borings and auger borings, and a wide variety of conventional hydraulic conductivity test methods, including:

<u>Laboratory Test</u>

- Permeameter Test on Undisturbed Sample

<u>Insitu (or Field) Tests</u>

- Cased Hole - Soil Flush With Bottom, Falling Head
- Cased Hole - Soil Flush With Bottom, Constant Head
- Cased Hole - With Uncased or Screened Extension, Falling Head
- Cased Hole - With Uncased or Screened Extension, Constant Head
- Uncased or Fully Screened Auger Hole - Constant Head Test
- Uncased or Fully Screened Auger Hole - Falling Head
- Pumping Test
- Double Ring Infiltrometer Test (note: this test gives an infiltration rate and not a hydraulic conductivity value)

Load Testing & Hydrologic Monitoring

Following the hydrogeologic exploration and aquifer testing, long-term hydrologic monitoring equipment was installed at the Airport Warehouses and Tom Statham Park sites. Observations wells, staff gauges, and rain gauges were equipped with pressure transducers and data loggers for continuous measurement of ground water and surface water levels and rainfall. The monitoring duration was approximately 1 year and included most of calendar year 1992, which was an above-average rainfall year.

Using water from nearby fire hydrants, the other 2 ponds (Fisherman's Landing and Tutor Time) were artificially recharged to their overflow elevations to simulate runoff from a design storm event. Ground water levels adjacent to and water levels within the retention ponds were continuously monitored--using observation wells and staff gauges, respectively--in the days preceding and following the rapid filling of these ponds.

Selected Models For Analysis of Retention Ponds

After the data collection phase of the study, the measured hydrologic responses of the ponds were compared with the predictions of 5 ground water flow models. The simulation models selected included:

Model	Description
#1	Simplified Analytical Method
#2	Glover's Line Source Theory
#3	MODRET (modified)
#4	PONDFLOW
#5	Hantush's Mounding Equation

This list of pond recovery models is representative of the current state of the geotechnical engineering practice in Florida, except for the Simplified Analytical Method (Model #1) which is a product of this research. In addition, during our evaluation, it was determined that the MODRET model, which is currently the most popular computer program used in the SJRWMD for stormwater retention pond analysis, is numerically unstable in some situations. A modified version of MODRET is used in this research to overcome this instability problem and the model is referred to hereinafter as "Modified MODRET".

These models are all similar in that the receiving aquifer system is idealized as a laterally infinite, single-layered, homogenous, isotropic water table aquifer of uniform thickness, with a horizontal water table prior to hydraulic loading. The three dimensional shape of the pond is assumed to be that of an equivalent rectangular trench.

All of the selected models require input values for the pond dimensions, retained stormwater runoff volume, and the following set of aquifer parameters:

- Thickness or elevation of base of mobilized (or effective) aquifer
- Weighted horizontal hydraulic conductivity of mobilized aquifer
- Fillable porosity of mobilized aquifer
- Ambient water table elevation which, for design purposes, is usually the normal seasonal high water table

Calculated recovery times are most sensitive to the input value for the aquifer hydraulic conductivity. However, with the exception of this parameter, estimating input values for the other aquifer parameters is fairly straightforward. The potential for error in estimating the weighted hydraulic conductivity of the receiving aquifer is manifested by the wide range of values obtained from the various test methods in this research. Estimates varied by as much as two orders of magnitude and this points to the need for careful scrutiny of test results.

Evaluation Procedure

A systematic procedure was used to evaluate and compare the hydraulic conductivity test results, the hydrologic monitoring data, and the theoretical models:

1. Events were selected for modeling which included the short-term load tests at Tutor Time and Fisherman's Landing as well as selected storm events from the two long-term monitoring sites (Airport Warehouses and Tom Statham Park).

2. The results of the hydrogeologic exploration were reviewed to estimate all aquifer parameters but the hydraulic conductivity parameter.

3. For each site and each selected event, a matrix was set up with the 5 ground water models representing the columns and the hydraulic conductivity test methods representing the rows. Recovery time simulations were conducted to fill each cell of these matrices using the corresponding hydraulic conductivity value and ground water flow model. In simpler terms, if there were thirty five geotechnical consultants analyzing one of the study ponds, each one using a unique combination of the 5 models and 7 hydraulic conductivity test methods, there could be 35 unique predictions of recovery time.

These matrices of predicted recovery times were compared to the observed recovery times. Based on i) the review of these matrices comparing real-world data with model predictions, ii) an assessment of the rationality and tractability of the mathematical models, and iii) the economic and practical considerations of the current state of the geotechnical engineering practice, design recommendations were made on field and laboratory methods for aquifer characterization and methodologies for computing recovery time. These recommendations are described below.

Recommendations

<u>Definition of Aquifer Thickness</u>

Standard Penetration Test (SPT) borings are recommended for definition of the aquifer thickness especially where the ground water table is high. This type of boring provides a continuous measure of the relative density/consistency of the soil (as manifested by the SPT "N" values) which is important in the Indian River Lagoon (IRL) Basin for detecting the top of cemented or very dense "hardpan" type layers. Such layers restrict the vertical movement of groundwater and are typical of the IRL Basin (excluding the relic sand dunes). If carefully utilized, manual "bucket" auger borings can also be used to define the thickness of the uppermost aquifer (i.e., depth to "hardpan" or restrictive layer), especially for small ponds and swales. Power flight auger borings may also be used with caution since this method may result in some mixing of soil from a given level with soils from strata above, thereby masking the true thickness of the aquifer.

Estimated Normal Seasonal High Groundwater Table

In estimating the normal seasonal high groundwater table (SHWT), the contemporaneous measurements of the water table are adjusted upward or downward taking into consideration numerous factors, including: antecedent rainfall, redoximorphic features (i.e., soil mottling), stratigraphy (including presence of hydraulically restrictive layers), vegetative indicators, effects of development, and hydrogeologic setting. The application of these adjustment requires considerable experience. The scope of the present study did not, however, include development or evaluation of the methodologies for estimating the SHWT.

In general, the measurement of the depth to the groundwater table is less accurate in SPT borings when drilling fluids are used to maintain an open borehole. Therefore, when SPT borings are drilled, it may be necessary to drill an auger boring adjacent to the SPT to obtain a more precise stabilized water table reading. In poorly drained soils, the auger boring should be left open long enough (at least 24 hours) for the water table to stabilize in the open hole.

Estimation of Horizontal Hydraulic Conductivity of Aquifer

Based on the findings of this study as well as practical and economic considerations, the following hydraulic conductivity tests are recommended for retention ponds:

i) Laboratory permeameter tests on undisturbed samples
ii) Uncased or fully screened auger hole (constant head)
iii) Cased hole with uncased or screened extension with the base of the extension at least 1 foot above the confining layer
iv) Pump test, when accuracy is important and hydrostratigraphy is conducive to such a test method.

Of the above methods, the most cost-effective is the laboratory permeameter test on an undisturbed horizontal sample. However, it becomes difficult and expensive to obtain undisturbed hydraulic conductivity tube samples under the water table or at depths greater than 5 feet below ground surface. In such cases--where the sample depth is over 5 feet below ground surface or below the water table--it is more appropriate to use the insitu screened auger hole method or the cased borehole horizontal hydraulic conductivity test.

Pump tests are recommended in cases where the mobilized aquifer is relatively thick (greater than 10 feet) and where the environmental, performance, or size implications of the system justifies the extra cost of such a test.

The main limitation of the laboratory permeameter test on a tube sample is that it represents the hydraulic conductivity at a point in the soil profile which may or may not be representative of the entire thickness of the mobilized aquifer. In most cases, the sample is retrieved at a depth of 2 to 3 feet below ground surface where the soil is most permeable, while the mobilized aquifer depth may be 5 to 6 feet. It is not practical or economical to obtain and test permeability tubes at each point in the soil profile where there is a change in density, degree of cementation, or texture. Some judgement and experience must therefore be used to estimate representative hydraulic conductivities of the less permeable zones of the mobilized aquifer from review of the soil profile. In such an evaluation, geotechnical engineers usually consider, among other factors, particle size distribution (particularly the percent by weight passing the U.S. No. 200 sieve), degree of cementation, density, color, presence of roots, sample orientation (i.e., horizontal or vertical), remolding, and compaction. For the less experienced, valuable insight into the variation of saturated hydraulic conductivity with depth in typical Florida soils can be gleaned from the comprehensive series of soil characterization reports published by the Soil Science Department at the University of Florida.

The screened auger hole or cased borehole with wellpoint horizontal hydraulic conductivity test methods are suitable for use where the mobilized aquifer is stratified and there is a high water table. Ideally, these tests should be screened over the entire thickness of the mobilized aquifer to obtain a representative value of the weighted horizontal hydraulic conductivity. Tests performed below the water table avoid the need to saturate the soil prior to testing.

If the mobilized aquifer is thick with substantial saturated and unsaturated zones, it is worthwhile to consider performing a laboratory permeameter test on an undisturbed sample from the upper unsaturated profile and also performing one of the insitu tests to characterize the saturated portion of the aquifer.

<u>Estimation of Fillable Porosity</u>

In Florida, the receiving aquifer system for retention ponds predominantly comprises poorly graded (i.e., relatively uniform particle size) fine sands. In these materials, the water content decreases rather abruptly with the distance above the water table and they therefore have a well-defined capillary fringe.

Unlike the hydraulic conductivity parameter, the fillable porosity value of the poorly graded fine sand aquifers in Florida are in a much narrower range (20 to 30 percent), and can therefore be estimated with much more reliability. For fine sand aquifers, it is therefore recommended that a fillable porosity in the range 20 to 30 percent be used in infiltration calculations. The higher values of fillable porosity will apply to the well- to excessively-drained, hydrologic group "A" fine sands, which are generally deep, contain less than 5 percent by weight passing the U.S. No. 200 (0.074 mm) sieve, and have a natural moisture content of less than 5 percent. No specific field or laboratory testing requirement is recommended to estimate this parameter.

Methodologies for Recovery Analysis

Based on the comparison of measured and predicted response of the ponds in this study, an assessment of the rationality of the formulations and the tractability of the mathematical models, the following three methodologies are recommended for retention pond recovery analysis:

1. Simplified Analytical Method
2. PONDFLOW
3. Modified MODRET

The following technical guidelines are associated with the utilization of these recommended methodologies:

- The required separation between the retention pond bottom and the seasonal high water table depends on the length/width ratio of the pond, the actual width of the pond, the average transmissivity of the mobilized aquifer, and the depth of the treatment volume within the pond. Establishing the pond bottom elevation 2 to 4 feet above the estimated SHWT covers a wide range of practical cases.

- If there is groundwater relief within the footprint of the pond, the average groundwater contour should be considered representative of the "flat" water table elevation for model input.

- Unless the vertical distance between the normal seasonal high water table and the pond bottom is greater than 2 feet, unsaturated flow prior to saturated lateral mounding should be conservatively ignored in recovery analyses. In other words, there should be no credit for soil storage immediately beneath the pond if the seasonal high water table is less than 2 feet below the pond bottom. This is not an unrealistic assumption since the models do not take into account capillary rise as well as the partially mounded water table conditions which may be remnant from a previous storm event.

- Recharge of the pond with the pollution abatement (or treatment) volume should be simulated as a "slug" or "instantaneous" loading (i.e., treatment volume fills the pond within an hour or less). There should be no credit for seepage during the storm. This recommendation does not, however, apply to recovery calculations for closed ponds (i.e., ponds with no positive outfall) since the design storm events for such systems are usually 24 to 96 hours long, and infiltration during such storm events can be signficant.

- The mobilized aquifer thickness used in the model should not be greater than the width of the pond.

In situations where the water table is deep and the ground water mound is not anticipated to rise above the pond bottom, the Hantush mounding equation may be applied. A more complicated fully three dimensional models with multiple layers, such as MODFLOW, may be warranted if the aquifer is markedly heterogeneous and non-uniform (such as cases with strongly sloping water tables or aquifer bases, hydraulic barriers adjacent to one side of the pond, sand filled trenches within pond, etc.).

Future Research Needs

Significant variations in hydraulic conductivity are inherent within and among the various soil horizons that comprise the receiving aquifer system for stormwater retention ponds. Further study of the applicability and limitations of the hydraulic conductivity test methods is warranted. Guidelines for assessing the reasonableness of the saturated hydraulic conductivity parameter used in models should be developed, since model predicted recovery times are virtually linearly related to this parameter. One approach would be to develop correlations between hydraulic conductivity and more economical soil tests such as particle size distribution analyses and other classification tests.

SECTION 1.0: INTRODUCTION

1.1 BACKGROUND AND PROBLEM STATEMENT

There is an increasing use of artificial groundwater recharge from shallow basins in water management schemes throughout Florida. Examples include recharging stormwater to augment the water supply and treat stormwater, recharging treated sewage effluent to obtain further treatment as it passes through the soil by taking advantage of the cleansing mechanism of the soil mantle, and recharging treated wastewater as a means of disposal. In the design and operation of such recharge/infiltration systems, it is necessary to predict, for a given geometry, soil properties and recharge rate, the motion of groundwater, and the velocity field as a result of recharging from a retention pond. This will enable the designer to:

1) more effectively size a stormwater retention pond which would recover within a stipulated time period following a particular design storm event;

2) estimate stormwater infiltration rates during and after a design storm event to be used with a surface water runoff routing model;

3) avoid a high water table condition beneath a dry bottom retention pond; and

4) minimize adverse groundwater mounding impacts adjacent to the pond.

As is well recognized, hydrogeology is basically an applied science. The geologic environment through which water moves, rarely, if ever, corresponds exactly to the postulates of mathematical theory or analog. Nevertheless, analytical and numerical modeling, when combined with appropriate hydrogeological and laboratory investigations, is a powerful tool for the assessment of soil infiltration characteristics and groundwater movement beneath and in the vicinity of retention basins. The mathematical treatment embodied in an analytical procedure or a numerical model may contain several approximations and simplifying assumptions for the practicing hydrogeologist or engineer to be sure that he understands the practical implications of the procedure. One must, therefore, learn from experience the limitations of various theoretical approaches and interpret the differences between the observed performance and the applied theories.

Computer models are now routinely used by practicing hydrogeologists and engineers for the synthesis and assessment of infiltration characteristics of stormwater retention ponds throughout Florida. This trend will probably accelerate in the future as the need for more realistic models increases. No model makes any sense if it is not based on a rational hydrogeological conceptualization of the underlying groundwater system. Unfortunately, reliable application of computer models requires considerable effort in quantifying pertinent hydrogeologic properties, particularly the saturated hydraulic conductivity of the mobilized aquifer. The required experimentation can be extremely laborious, time-consuming and expensive especially in view of the fact that field scale flow processes are generally quite variable in time and space. Nevertheless, the reliability of the output of a model cannot exceed the reliability of the input data. Input data assessment is probably the most neglected single task in groundwater modeling process. As our conceptual understanding and numerical modeling expertise to simulate increasingly complex systems increases, the accuracy of future simulations may well hinge on the quality and completeness of the input data.

St. Johns River Water Management District (SJRWMD) has recognized these difficulties during the past several years during its review of numerous permit applications pertaining to the design, construction and operation of hundreds of stormwater retention ponds within its permitting jurisdiction. Jammal & Associates, Inc. was retained in June 1990 to conduct the necessary field and laboratory investigations as well as modeling assessments for selected pond sites. The Indian River Lagoon (IRL) Basin of the SJRWMD was chosen as the study area because many soils in this basin have limited infiltration potential. Figure 1 shows the limits of the IRL Basin within SJRWMD.

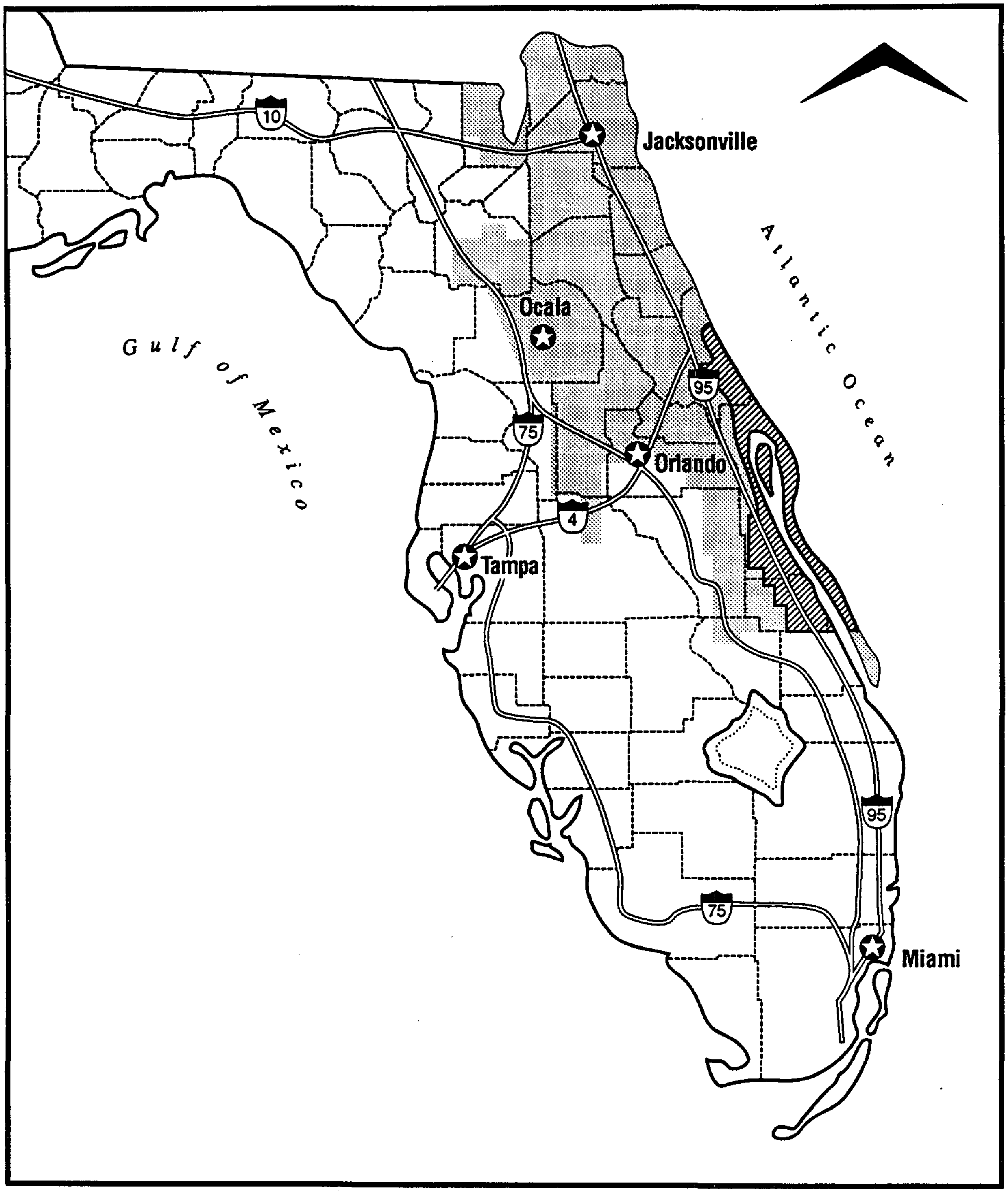
Using the recommended procedures documented in this report, it would be possible for a designer to select appropriate field and laboratory testing method(s) for determination of aquifer parameters, particularly hydraulic conductivity, as well as select an appropriate analytical or numerical model for a meaningful assessment of infiltration characteristics of soils and retention volume recovery analysis of proposed retention ponds.

Corroboratory material presented in this report supplements the basic theoretical and design information presented in an earlier publication entitled *"Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers"* prepared for the Southwest Florida Water Management District (Jammal & Associates, Inc. 1989). In summary, the latter documented the following:

- A comprehensive review of the field and laboratory test methods utilized by geotechnical engineers in the practice of analyzing retention ponds

- A recommended methodology for analyzing the recovery of retention ponds (i.e., the computer program MODRET)

- Stormwater retention pond construction and maintenance considerations



LEGEND

St. Johns River Water Management District

Indian River Lagoon SWIM Planning Basin

Project Location Within Florida

Figure: 1

1.2 OVERVIEW OF CURRENT REGULATORY FRAMEWORK GOVERNING RETENTION POND DESIGN

Historically, stormwater management systems have been regulated by both the Florida Department of Environmental Protection (FDEP) and the water management districts (WMDs), with FDEP rules governing water quality treatment and WMD Management and Storage of Surface Water (MSSW) rules governing water quantity considerations such as drainage and flood control. In 1989, the Florida legislature gave the water management districts clear authority and directive to regulate all aspects of stormwater management systems, including both stormwater quantity and quality, under the WMD MSSW rules. As a result, most of the WMDs, including St. Johns River Water Management District, now regulate all aspects of stormwater management systems under their rules with statewide oversight provided by the FDEP. Currently, both FDEP and the water management districts require that any new development must have a comprehensive stormwater management system which incorporates a number of different Best Management Practices or BMPs to ensure that stormwater is being effectively treated.

Some of the most common BMPs recommended by the St. Johns River Water Management District are outlined below:

1. Retention systems are storage areas designed to store a defined quantity of runoff, allowing the runoff to percolate through the permeable soils of the basin floor and side slopes into the shallow groundwater aquifer. The most common type of retention system consists of man-made or natural depressional areas where the floor is graded as flat as possible and turf is established to promote infiltration and stabilize basin slopes. Soil permeability and water table conditions must be such that the retention system can percolate the desired runoff volume within a specified time following a storm event.

2. Wet detention ponds are permanently wet ponds which are designed to slowly release collected stormwater runoff through an outlet structure. Wet detention systems are the recommended BMP for sites with moderate to high water table conditions. Wet detention ponds have a vegetated littoral zone which treat stormwater by physical, chemical, and biological processes. These ponds are sized to contain a permanent pool of water which results in an average residence time of at least 14 days during the wet season. The pond is also designed such that the flow path through the pond has a length to width ratio of at least 2:1. SJRWMD strongly recommends wet detention ponds since, in addition to their treatment capabilities, they are not as maintenance-intensive as other systems appropriate for sites with high water table conditions (such as filtration systems described next).

3. Stormwater filtration systems consist of a perforated pipe which collects and conveys stormwater following infiltration through a sand filter. Filters are generally used where space, soil hydraulic conductivity, and/or high water table conditions dictate

that recovery of the stormwater treatment volume cannot be achieved by natural percolation (i.e., retention systems) or sedimentation (i.e., wet detention). The filter trench is normally backfilled to the surface with fine aggregate (such as washed sand) that is more permeable than the surrounding soil. SJRWMD requires that the fine aggregate filter be at least 2 feet thick. Filters are normally installed in the bottom or banks of detention basins and may be used in either dry or wet basins. Filters are a maintenance-intensive BMP because of the likelihood of clogging over time, and their use is usually restricted to projects with a drainage area of less than 10 acres.

4. Underdrain systems consist of a dry detention basin underlain with a network of subsurface drains to: i) control the water table below the basin bottom, and ii) enhance percolation of impounded stormwater from within the pond. Unlike filtration systems, the backfill around the drainage pipes consists of indigenous soils (typically poorly graded fine sand) which provide better pollutant removal capabilities than filter media (i.e., washed medium- to coarse-grained sand). SJRWMD requires at least 2 feet of soil cover between the bottom of the basin and the underdrain pipes.

5. Exfiltration trench is a subsurface system consisting of a conduit such as perforated pipe surrounded by natural or artificial aggregate which temporarily stores and infiltrates stormwater runoff. Stormwater enters the perforated pipes and infiltrates through the base and sides of the trench into the shallow ground water aquifer. The perforated pipe increases the storage available in the trench and promotes a more uniform distribution of recharge within the trench. Generally, exfiltration trenches are used where space is limited and/or land costs are relatively high. However, the operational life of an exfiltration trench is believed to be short (possibly 5 to 10 years) because of sediment accumulation and clogging by fines. These systems, if not properly designed, may therefore require extensive maintenance or complete replacement during the design life of the system.

6. Swales are man-made or natural systems, shaped or graded to specified dimensions and designed for the conveyance and rapid infiltration of stormwater runoff. Unlike retention ponds, swales are open conveyance systems in that there are no physical barriers such as berms or check dams to impound the runoff in the swale prior to discharge to the receiving water. Swales are designed to infiltrate a defined quantity of runoff through the permeable soils of the swale floor and the side slopes into the shallow ground water aquifer. The swale holds water only during and immediately after a storm event and thus the system is normally dry.

7. The wetlands stormwater management system design and performance criteria in the stormwater rule are an initial step by SJRWMD in a field where limited knowledge exists. Only wetlands which are connected to other waters by an artificial or intermittent water course or isolated wetlands may be used as stormwater treatment wetlands. Like wet detention ponds, these systems are designed to slowly release collected stormwater runoff through an outlet structure. However, the diversion of

> stormwater into the wetland for treatment should not adversely affect the hydroperiod of the wetland. In addition, inlet structures must be designed to preclude channelized flow and residence time within the wetland should be maximized.

The retention BMP is identified as the one that is primarily used in the IRL Basin. The regulations governing retention in this region are found in the St. Johns River Water Management District manual under Chapter 40C-42, Regulation of Stormwater Management Systems. The specific treatment criteria involved in the permitting of retention systems is summarized below:

- Provide off-line treatment, through soil infiltration for the "first flush" of runoff from the watershed. Off-line means the storage of a specified portion of the stormwater in such a manner so that subsequent runoff in excess of the specified volume of stormwater does not flow into the area storing the initial stormwater. The "first flush" is defined as the greater of:

  - the first ½ inch of runoff from the site, or,

  - the first 1¼ inches from contributing impervious areas.

- For on-line retention systems, an additional ½ inch of runoff must be provided over that volume specified for off-line treatment.

- Must make available the entire treatment volume within a period of 72 hours following the design storm event, assuming average antecedent conditions. The recovery of the storage volume must be provided by percolation through soil, evaporation or evapotranspiration.

1.3 MECHANICS OF THE INFILTRATION PROCESS

Infiltration from retention ponds involves the vertical downward movement of groundwater under the influence of vertical head differential. The percolative capacity of retention ponds is principally a function of the ability of the subsurface soils to store and percolate applied stormwater to the shallow aquifer, and the subsequent capacity for lateral flow. The latter is controlled by the thickness and hydraulic conductivity of the aquifer and the position of the groundwater table relative to the pond bottom. Stormwater within the pond percolates downward through the unconfined aquifer and upon reaching the water table or a restrictive layer begins to mound. As the recharge mound rises in elevation, lateral flow is induced under the increasing horizontal hydraulic gradient. The magnitude of rise of the recharge mound and the rate of rise depend on several factors including the thickness, fillable porosity, saturated hydraulic conductivity of the unconfined aquifer, leakance through the semi-confining layers, transmissivity of confined aquifers, the hydraulic application rate, the geometry of the loaded area, and the distance to and control levels of nearby surface water and subsurface drainage features.

Saturated flow through a porous medium is similar to laminar flow in smooth, narrow tubes, but is more complex because of the fortuitous path water must follow in soil voids. Downward and lateral movement of water is driven by a hydraulic gradient and retarded by friction and intermolecular attraction in both cases. However, in a porous medium, the pore spaces consist of passages that may be irregular, and interconnected or frequently discontinuous, which significantly complicates flow on a microscopic scale. For simplicity, flow through a saturated, porous medium is often represented on a larger scale as a velocity vector, or an overall average of the microscopic velocities within the total volume of porous medium. Development of equations to quantify groundwater flow requires the combination of two fundamental physical laws - Darcy's law and the law of mass conservation. Indeed, most of the models used for simulation of groundwater flow are based on a synthesis of these two laws.

In many situations, however, flow will take place through an unsaturated (vadose) zone between the ground surface and underlying water table. The fundamental principles that apply to the saturated flow can be extended to unsaturated flow. It should be noted that unsaturated flow processes are quite complicated due to complex relationships between air entrapment, water content, pressure head, capillary head, and unsaturated hydraulic conductivity. The driving forces for flow under saturated conditions are total head gradients that include positive pressure heads whereas for flow under unsaturated conditions, driving forces are total head gradients that include negative pressure heads (known as capillary suction). In the presence of an unsaturated zone, flow will take place by infiltration beneath the surface. If water application is discontinued, infiltration will cease and the previously infiltrated water in storage in the vadose zone will continue downward but usually at a slower rate until surface tension forces are equal to gravitational forces. The "slug" of water will remain suspended until another slug reaches it and pushes it further down. Once the wetting front reaches the water table, recharge to the surface of the saturated aquifer begins. As the wetting front begins to merge with the underlying saturated zone, the incoming discharge changes direction and flows mainly below the water table and generally parallel to the existing water table. As moisture content increases, water flows through pores of increasing size at increasing pressures until saturation, when atmospheric pressure is attained.

1.4 STUDY PURPOSE AND SCOPE

The objective of this investigation was to establish and to recommend the most applicable, readily available methods of geotechnical testing--principally, soil borings and insitu and laboratory hydraulic conductivity tests--to characterize the mobilized aquifer systems, and appropriate methods of calculating stormwater infiltration during and after a storm event. The selection of the testing methods was based on an evaluation of full-scale load tests on two existing ponds during long-term operating conditions and two more existing ponds under short-term hydraulic load test conditions. For this purpose, four ponds within the IRL Basin were selected, instrumented and monitored to collect sufficient full-scale field data for this evaluation.

Development of reliable methods of determination of hydraulic conductivity and selection and utilization of appropriate analytical or numerical model(s) for simulation of the infiltration behavior of subsoils beneath and in the vicinity of retention ponds were considered key ingredients in the evaluation process. Accordingly, it is envisioned that concentrated efforts must be directed toward retrieval, compilation and analysis of site-specific and regional hydrogeologic, geophysical, hydraulic and soil data as well as modeling and simulation for a meaningful assessment of infiltration characteristics of the underlying groundwater system so as to evaluate the infiltration capacity of retention ponds.

The specific objectives and scope of this study included:

- Review of pertinent published literature regarding available field and laboratory methods of determination of hydraulic conductivity that are frequently used by consultants in the state of Florida for assessment of infiltration.
- Review of pertinent literature regarding commonly used analytical and numerical models for the simulation of retention volume recovery.
- Preliminary hydrogeologic exploration of eight (8) short-listed retention pond sites within the IRL Basin.
- Site-specific hydrogeologic exploration of the four (4) selected pond sites.
- General watershed characterization of contributing drainage areas for the four (4) selected ponds.
- Installation and testing of site instrumentation for hydrologic monitoring, including measurement of stormwater inflow from contributing watersheds, measurement of rainfall (for long-term sites), and surface water/groundwater level monitoring at observation wells equipped with pressure transducers and data logger units for continuous monitoring at all four (4) selected sites.
- Collection of hydrologic data at the two long-term monitoring sites for a period of approximately 1 year.
- Performance of load tests at the two short-term monitoring sites.
- Hydrogeologic characterization of each site based on field and laboratory investigations.

- Comparison of various ground water flow models of pond recovery and their ability to predict the observed pond recovery.

- Presentation of results, including findings and recommendations on field and laboratory testing methods and models to simulate the recovery of ponds.

The field and laboratory data from all test sites were compiled and reduced in a consistent manner so that the results can be compared to various available methods of infiltration analysis currently being used for assessment of infiltration characteristics and design of stormwater retention ponds. Field results were compared with commonly used analytical equations and numerical models. Recommendations are presented for the use of the most appropriate method(s) of hydraulic conductivity determination, and analytical procedure/numerical models for assessment of efficiency of stormwater retention ponds.

1.5 REPORT ORGANIZATION

This report is divided into two volumes: Volumes I and II. The first volume contains the main body of the report while the second volume contains the digital data from the hydrologic monitoring programs. This data in Volume I is in graphical form.

Section 2 of the main report (Volume I) describes the regional hydrogeologic setting of the Indian River Lagoon Basin and describes typical shallow soil stratigraphies within this area which encompasses portions of Volusia, Brevard, and Indian River Counties. In Section 3, general characteristics of the sites selected for hydrologic monitoring and load testing are described. Section 4 discusses the extent and results of the hydrogeologic exploration and testing at each site. Hydrologic monitoring data collected from long-term monitoring and short-term load tests at the ponds are presented and discussed in Section 5, while Section 6 summarizes the results of a comparative study of the common numerical and analytical methodologies for retention pond recovery analysis using the hydrologic data of this study as test cases. Liberal use of tables and graphical displays of field and simulation results is made herein to facilitate meaningful comparisons between observed data and theoretically simulated results. Recommended design guidelines as a result of this research effort are contained in Section 7 of the report.

References cited in the report are listed in alphabetical order in Section 8. Figures and tables are included in the body of the report after their first mention.

Volume II contains the digital results obtained from the data logging equipment converted to a usable format.

SECTION 2: REGIONAL GEOLOGY AND HYDROGEOLOGY

2.1 PHYSIOGRAPHY

The study area (shown in Figure 1, page 11) was delineated by Brooks (1981) as the Eastern Flatwoods District. The Eastern Flatwoods District physiographic region is characterized as an area which originated as a sequence of barrier islands and lagoons during Plio-Pleistocene and Recent Times. The dominant physiographic features within the study area are terrace deposits of the Atlantic Coastal and Ten-Mile ridges west of the lagoon and the barrier islands to the east of the lagoon. The terrace deposits within the broad limits of the study area are the Silver Bluff terrace, which forms the western limits of the Indian River Lagoon, and the Talbot terrace, which extends further inland. Elevations are generally less than +25 feet NGVD, decreasing with proximity to the lagoon areas.

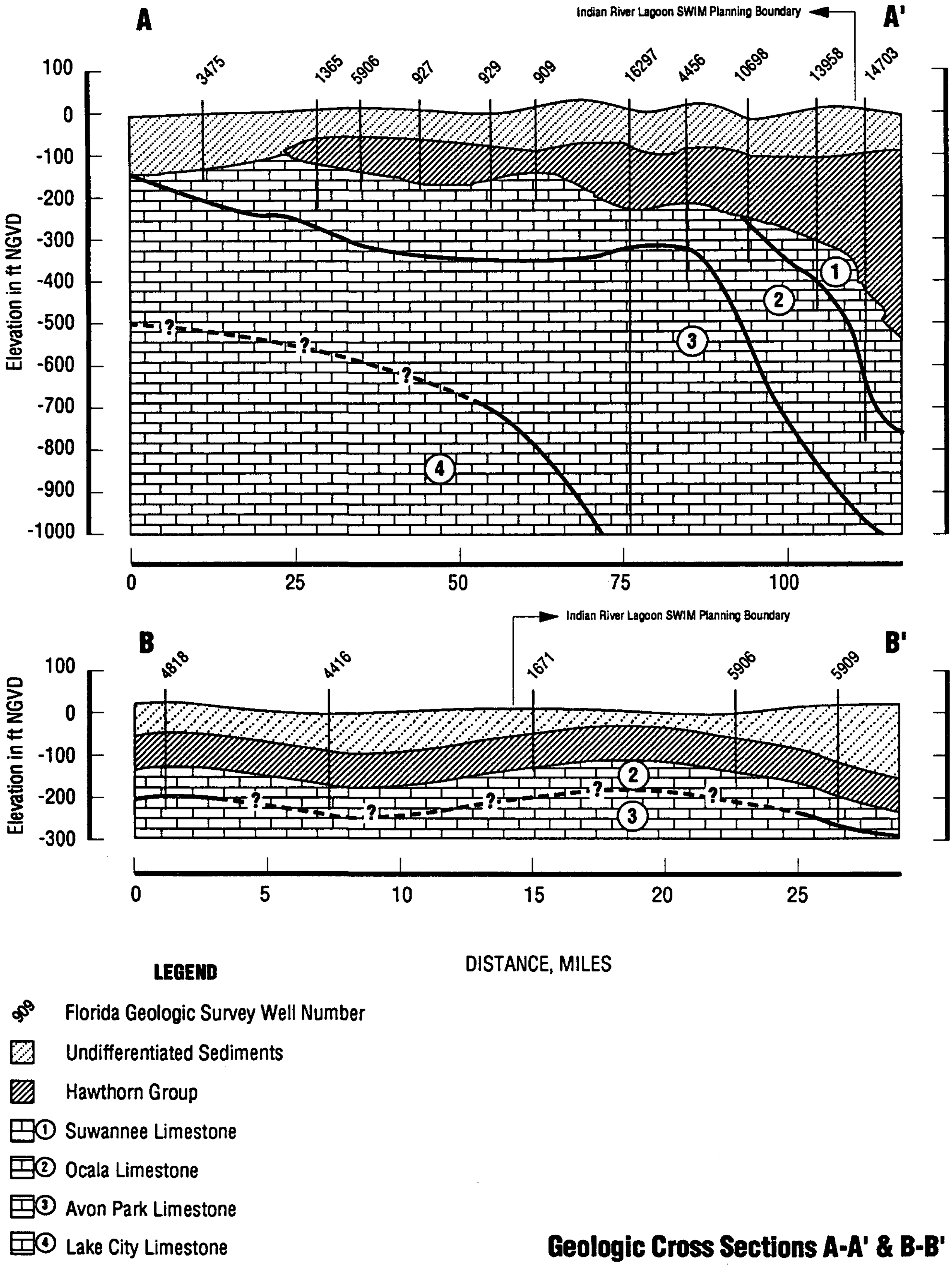
2.2 GEOLOGY

The general geologic characteristics of the study area have been described in reports published by the U.S. Geological Survey, the Florida Geological Survey, and the St. Johns River Water Management District. Among these reports are Toth (1988), Scott (1988), Crain et al. (1975), Brown et al. (1962), and Rutledge (1985). In general, the Indian River Lagoon area is underlain by a sequence of limestones and dolostones overlain by as much as 100 to 300 feet of siliceous and calcareous clastic deposits.

Generalized geologic cross-sections depicting the spatial relationships between the geologic formations present in the study area are presented on Figures 2 and 3, with accompanying cross-section location lines indicated on Figure 4. General descriptions of the geologic formations typically encountered within the study area follows.

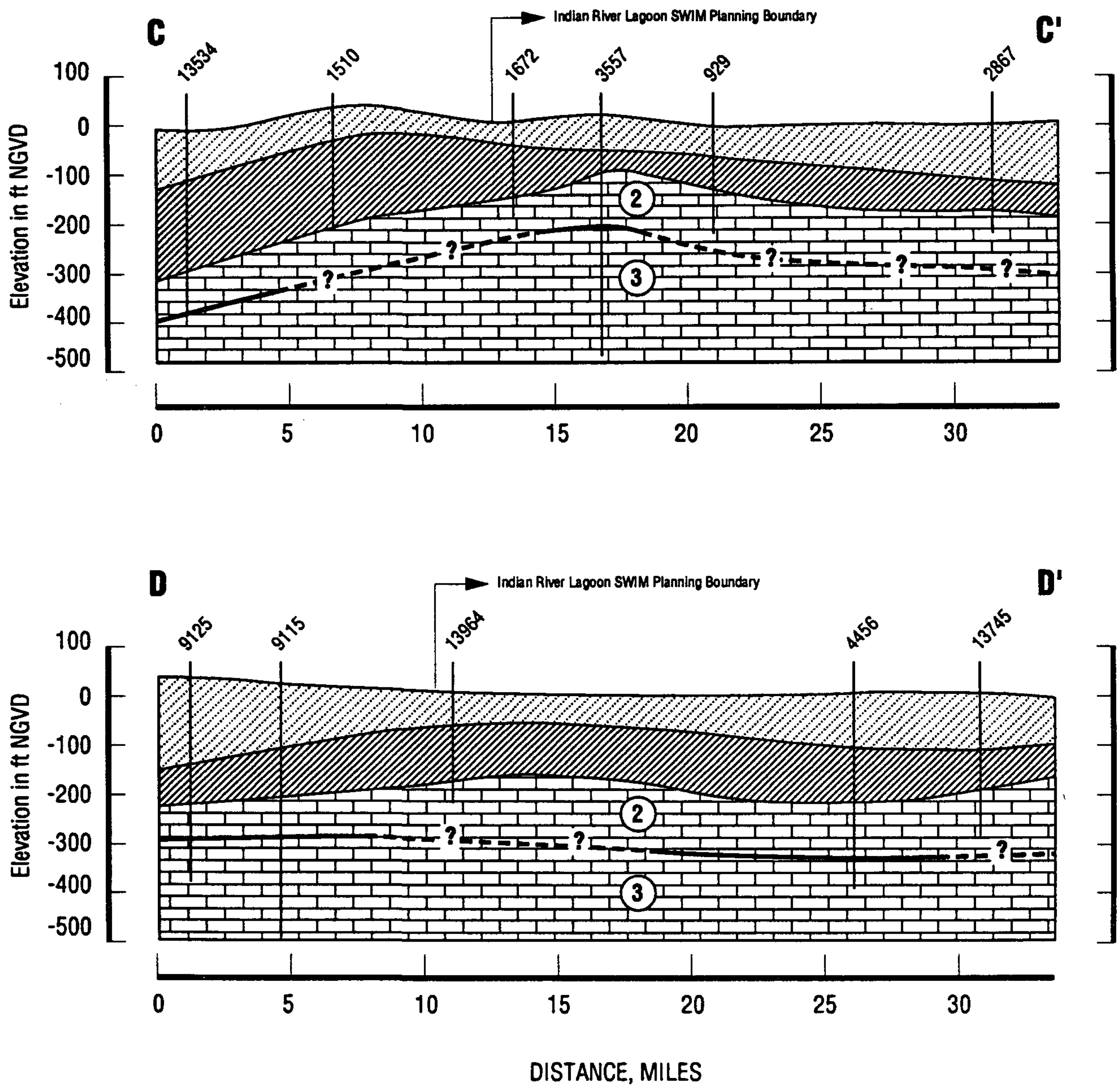
The deepest limestone formation generally penetrated within the study area is the Avon Park Limestone (Miller, 1986). This formation occurs at elevations varying from -100 feet NGVD in the northern portion of the study area (coastal Volusia County) to elevations below -900 feet NGVD in the southern portion of the study area (coastal Indian River County). The Avon Park Limestone is reported to range in thickness from 200 to 650 feet within the study area.

The geologic formation that generally marks the top of the limestone/dolostone sequence that comprises the upper Floridan aquifer in the study area is the Ocala Limestone. The Ocala Limestone is encountered at elevations of -75 feet NGVD in the northern portion of the study area, and below elevation -800 feet NGVD within the southern portion of the area. The average thickness of the Ocala Limestone within the study area is 100 feet.



Geologic Cross Sections A-A' & B-B'

Figure: 2



LEGEND

909 Florida Geologic Survey Well Number

Undifferentiated Sediments

Hawthorn Group

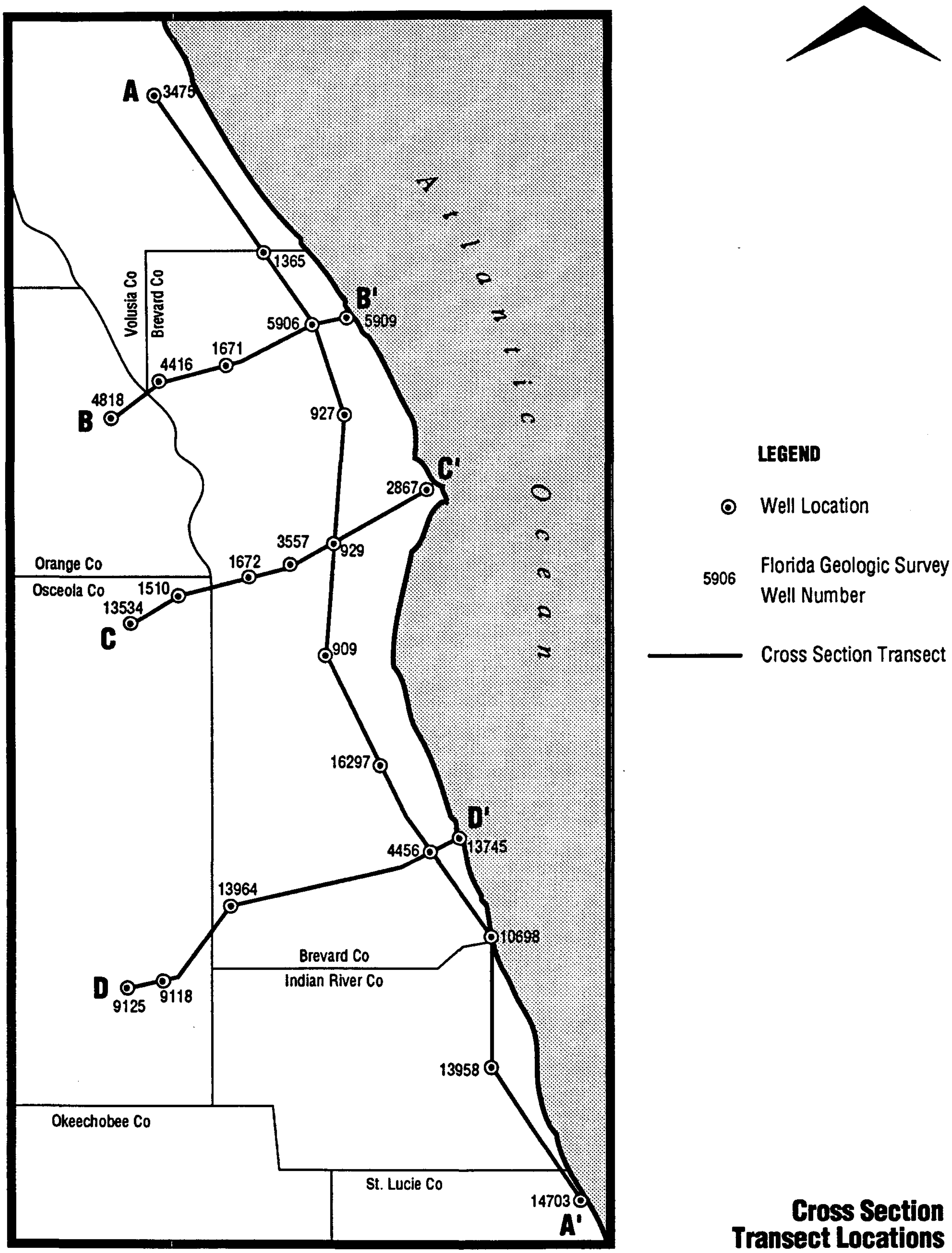
① Suwannee Limestone

② Ocala Limestone

③ Avon Park Limestone

Geologic Cross Sections C-C' & D-D'

Figure: 3



Cross Section Transect Locations

Figure: 4

In the extreme southern portion of the study area, in coastal Indian River County, the uppermost formation portion of the Floridan aquifer comprises the Suwannee Limestone, which overlies the Ocala Limestone. The Suwannee Limestone is encountered at elevations of -155 feet NGVD to below elevation -600 feet NGVD within the southern portion of the area. The thickness of the Suwannee Limestone ranges from 0 feet in south Brevard and western Indian River Counties to 194 feet south east of Vero Beach.

Overlying the Floridan aquifer is the semi-confining unit known as the Hawthorn Group. The Hawthorn is generally absent in the northern portion of the study area, and increases in thickness towards the south to more than 500 feet near Vero Beach. According to Scott (1988), the Hawthorn thins towards the north. The Peace River Formation of the Hawthorn Group consists of an interbedded mixture of sand, clay, limestone and dolomite with fine to coarse grained phosphatic sand, phosphorite gravel and pebbles.

Overlying the Hawthorn Group are undifferentiated sands, shell, clay, limestone and coquina, which comprises the surficial aquifer. These deposits may vary from 110 to 300 feet in thickness (Toth, 1988).

In general, all of the geologic units described above dip toward the south and east, and increase in thickness down-dip.

2.3 HYDROGEOLOGY

The deepest formation generally penetrated by water supply wells in Brevard County is the Avon Park Limestones. For the purpose of discussion, the sequence of lithologic materials present in this area can be divided into a carbonate group and a clastic group. The carbonate group is the Floridan aquifer comprised of the lower dolomitic portions of the Hawthorn Group (where hydraulically connected with the underlying limestones), the Ocala Group Limestones, and the Avon Park Limestone. The clastic group is the surficial aquifer which comprises undifferentiated sand and clay, and those portions of the Hawthorn Group that are not hydraulically connected with the Ocala Group Limestones (intermediate aquifers).

2.3.1 Surficial Aquifer

The sandy material in the upper part of the clastic unit forms the surficial aquifer. The mixture of clay and sandy clay in the lower part of the clastic unit forms a semi-confining bed that separates the surficial aquifer from the Floridan aquifer. The sandy and coquina material of the surficial aquifer and the clayey materials of the confining bed have an important function in the hydrology of this portion of central Florida. Those parts of the aquifer that in the aggregate are permeable, readily store water that infiltrates from the land surface. In the Indian River Lagoon Basin within Brevard County, the permeability of the confining bed is sufficiently less than that of the sandy or clayey material above it such that a water table is established whose distance below land surface depends on the quantity of water available for recharge, and the thickness and hydraulic conductivities of the aquifer and confining beds.

Within the Indian River Lagoon area, the configuration of the water table generally conforms to the configuration of the land surface. The water table of the surficial aquifer system fluctuates in response to recharge from rainfall; evapotranspiration of water from the water table where it is within about 5 feet of the land surface; lateral discharge of water to the coastal lagoons and the St. Johns River Marsh, lakes and streams; and downward leakage to the intermediate aquifer.

The surficial aquifer system is the principal source of fresh groundwater for most domestic wells and some municipal supply wells in Brevard County.

2.3.2 Intermediate Aquifer

Within the confining sediments of the Hawthorn Group, thin lenses of sand, shell, and limestone yields small to moderate amounts of water in some places. In other places, this aquifer system yields little or no water, has low permeabilities, and acts as a confining unit (Toth, 1988). In Volusia and north Brevard counties, where Hawthorn deposits are thin or absent, little is know about intermediate aquifers. In north-central Indian River County, the intermediate aquifer occurs as a thin lens less than 20 feet thick. This aquifer is artesian and is recharged by water that leaks from the surficial clastic aquifer.

2.3.3 Floridan Aquifer

Underlying the clastic unit is the Floridan aquifer which comprises a thick sequence of limestone/dolostone bedrock. With the exception of area around the City of Titusville, the water within the upper Floridan aquifer is highly mineralized. Therefore, most wells that tap the Floridan aquifer in coastal sections of Brevard County are seldom used as sources of water for municipal supplies. In most places in the Indian River Lagoon area, the Floridan aquifer is partly confined beneath less permeable confining beds. Wells tapping the Floridan aquifer in most of the study area are flowing artesian wells.

The potentiometric surface of the Floridan aquifer defines the level to which ground water will rise in tightly cased wells that penetrate the aquifer. Overall, there has been a general decline in the potentiometric surface throughout the region since the 1950's due to increased water withdrawal from the Floridan Aquifer and low rainfall (Toth, 1988).

The direction of the regional groundwater flow in the Floridan aquifer is essentially to the east and northeast. However, in the northern section of Brevard County along the Indian River, the local flow patterns are toward the river. Locations east of the Indian River exhibit a northwesterly trend in the groundwater flow pattern, while west of the river the flow is generally to the northeast. This deviation from the regional pattern is attributable to significant upward leakage into the surficial aquifer. A regional potentiometric surface map of the Floridan aquifer is shown on Figure 5.

2.4 SURFICIAL SOILS

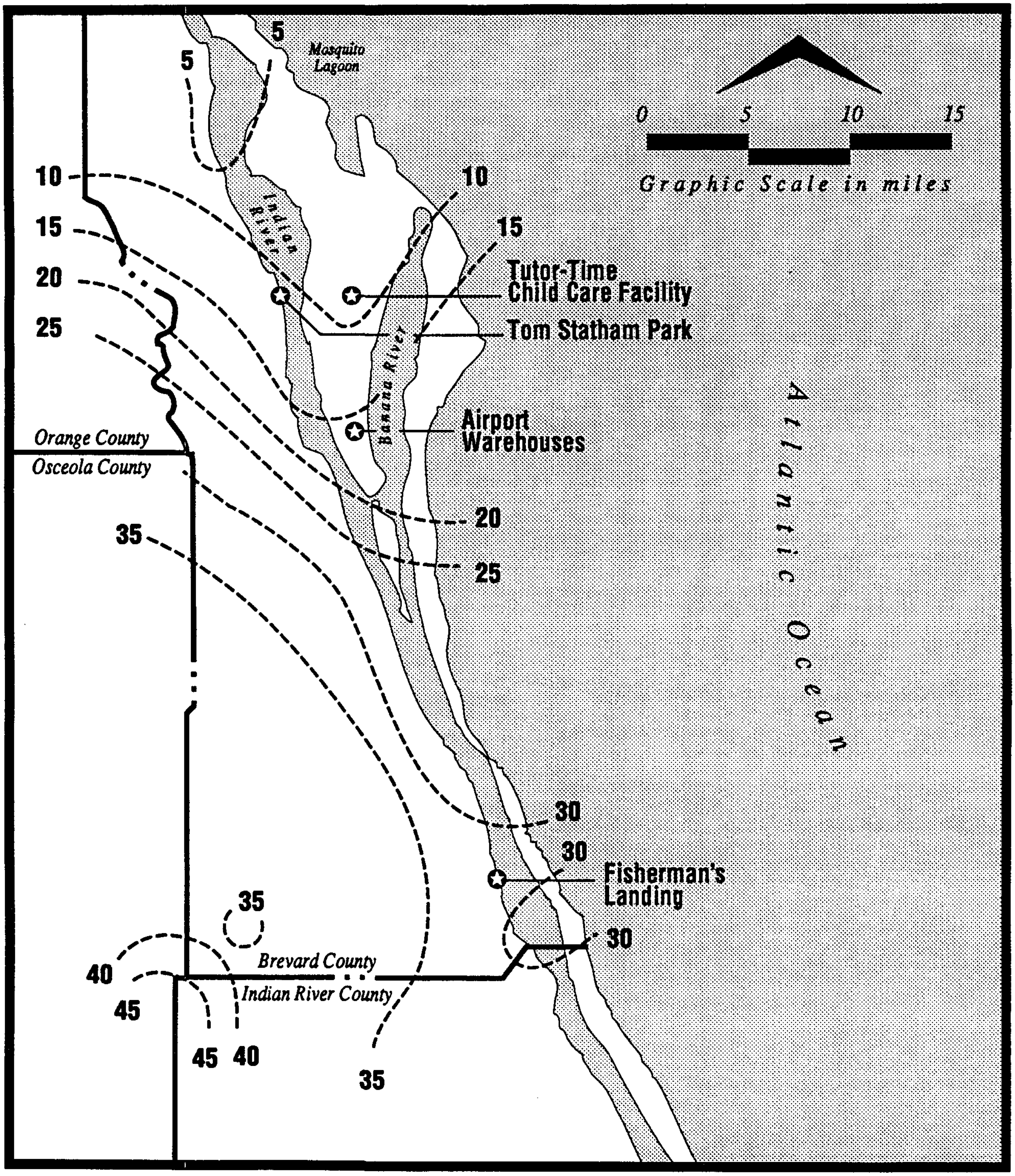
The United States Department of Agriculture (USDA), Soil Conservation Service (SCS) has mapped the shallow soils (i.e., less than 80 inches deep) within the Indian River Lagoon Basin and this information is published in the SCS Soil Surveys for Indian River, Brevard, and Volusia counties (Wettstein et al. 1987; Huckle et al. 1974; and Baldwin et al. 1980, respectively).

Figures 6 through 8 show the general soil map units within the IRL Basin in Indian River, Brevard, and Volusia counties, respectively. Within the study region, the soil map units of interest are grouped into four general landscapes:

- Soils of the Sand Ridges (SR): The soils in this general map unit consists of nearly level to gently sloping, excessively drained and moderately well drained soils that are sandy to a depth of 80 inches or more. They occur on high discontinuous dunes of the Atlantic Coastal Ridge which extends in a north-south direction parallel to the Indian River.

- Soils of the Flatwoods (FW): The soils of the flatwoods consist of poorly drained, nearly level sandy soils over dark-colored , weakly cemented sandy layers (hardpan) that are underlain by sands or silty sands. The soils of the Flatwoods have a spodic horizon into which organic matter has been translocated and has accumulated and is locally called hardpan. A hardpan is hardened or cemented soil horizon or layer. The soil material is sandy, loamy, or clayey and is cemented by iron oxide, silica, calcium carbonate or other substance. These associations are on the mainland between the Atlantic Coastal Ridge and the lowlands along the St. Johns River. The spodic horizon usually occurs at a depth of 30 to 40 inches below land surface. Figure 9 shows the typical pattern of the soils on the sand ridges and the adjacent flatwoods.

- Soils of the Coastal Islands (TM): This association is made up of nearly level and gently sloping ridges with narrow wet sloughs which generally parallel the ridges. It occurs along the coast near the Atlantic Ocean and extends the entire length of the study area. The soils are somewhat poorly drained to excessively drained sandy soils that contain shell fragments. Figure 10 shows a typical cross-section of the soils on the barrier islands.



LEGEND

✪ Study Sites

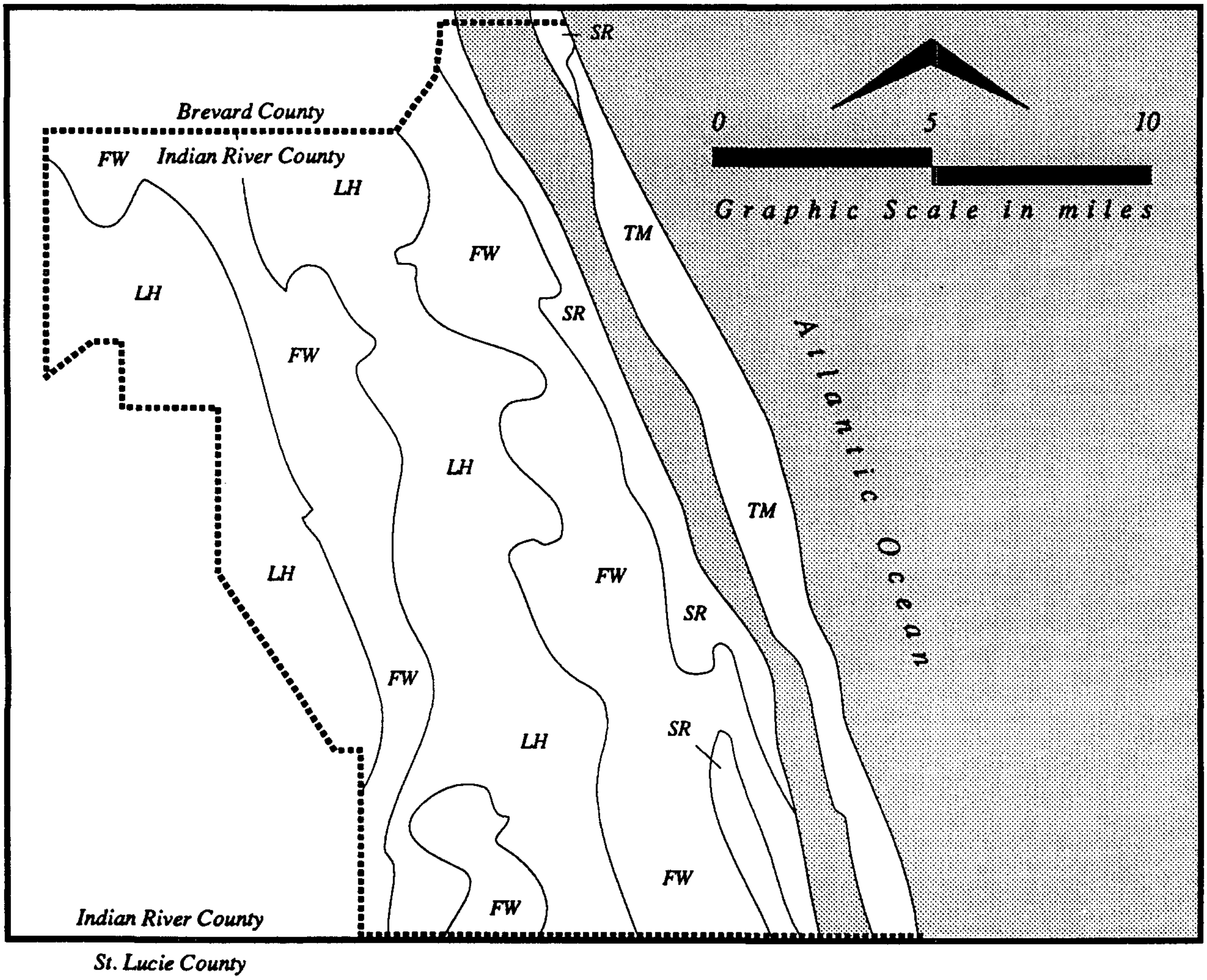
-- 45 -- Potentiometric Surface Contour of the Floridan Aquifer in feet NGVD for May, 1987 by G.R. Schiner

Potentiometric Surface Map of the Floridan Aquifer Within Indian River Lagoon Basin

Figure: 5

- Soils of the Hammocks, Sloughs, and Poorly Defined Drainageways (LH): The general map units in this group consist of nearly level, poorly drained and very poorly drained soils. These soils are on low broad flats and in sloughs, depressions, and poorly defined drainageways. For the most part, these soils have a silty/clayey subsoil or a dark-colored weakly cemented layers at a depth of less than 40 inches. Figure 11 shows a typical pattern of soils in this map unit.

An appreciation of representative values of aquifer parameters (primarily saturated hydraulic conductivity) for the uppermost aquifer system is particularly important when analyzing stormwater management ponds. Figure 12 shows the vertical variation in hydraulic conductivity for the soils of the sand ridges and the soils of the pine flatwoods within the IRL Basin (Wettstein et al. 1987; Huckle et al. 1974; and Baldwin et al. 1980). Note from this figure that the hydraulic conductivity of the flatwood soils decreases markedly at a depth of 30 to 50 inches below land surface (bls), while the soils of the sand ridges are characterized by relatively high hydraulic conductivity values to depths greater than 80 inches bls. Comprehensive data sets showing the variation of saturated hydraulic conductivity with depth for various soils throughout Florida have been published by the Soil Characterization Laboratory at the University of Florida (see, for example, Sodek et al. 1990). This data base provides valuable insight into the typical range of hydraulic conductivity values for the various soil horizons.



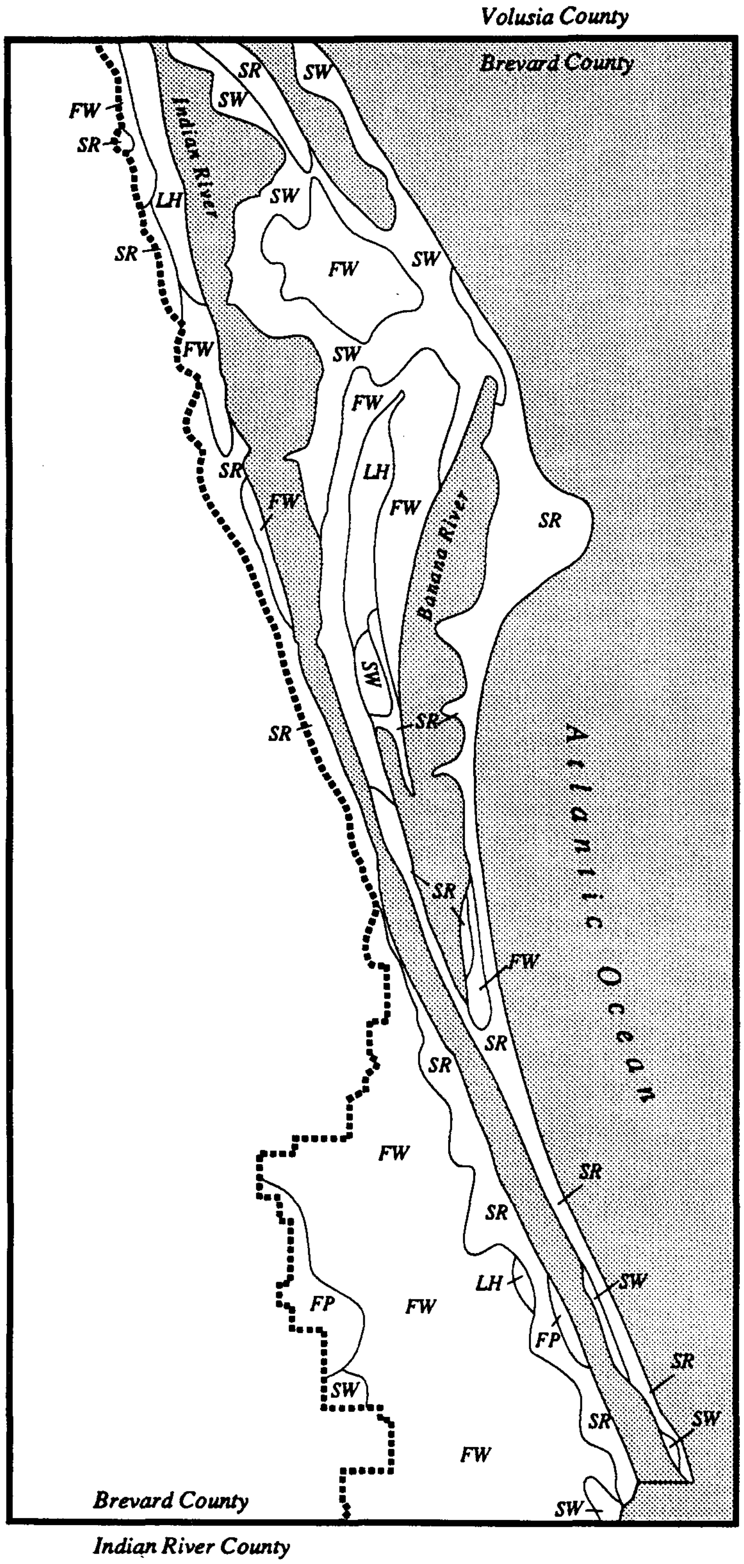
LEGEND

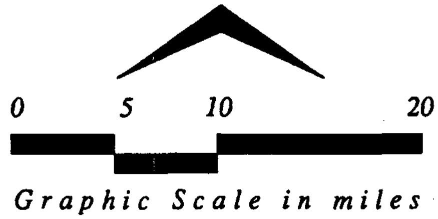
••••••••••	Indian River Lagoon SWIM Planning Boundary
SR	Soils of the Sand Ridges and Coastal Dunes
FW	Soils of the Flatwoods
TM	Soils of the Coastal Islands and Tidal Marshes
LH	Soils of the Low-Lying Hammocks

Adapted from U.S. Department of Agriculture Soil Conservation Service Soil Survey for Indian River County, Florida, issued January 1987

Generalized Soil Map of Study Area Within Indian River County

Figure: 6





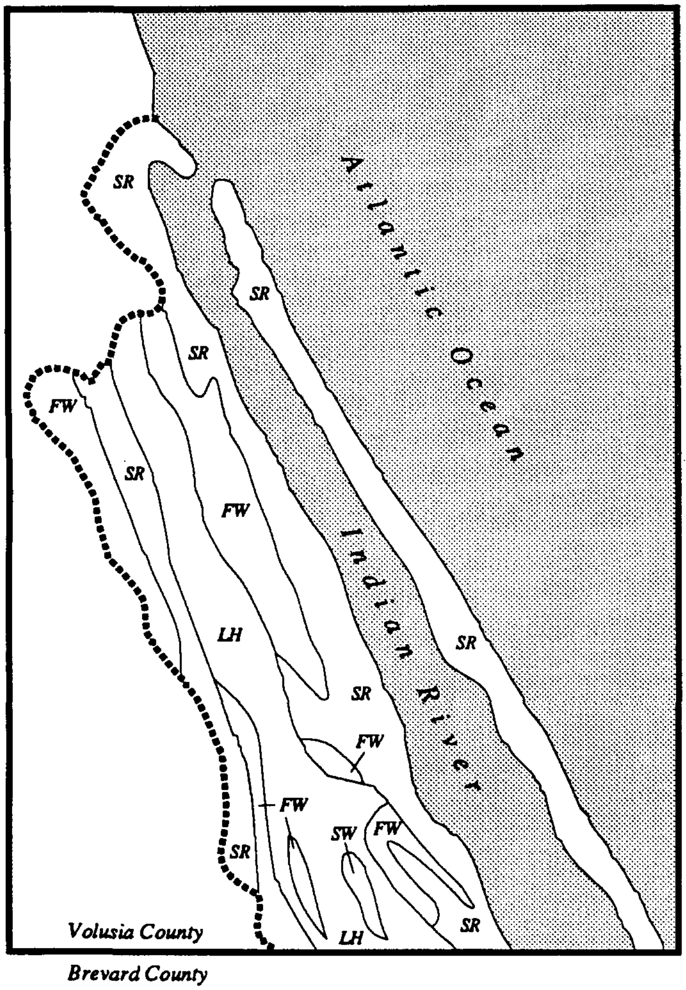
LEGEND

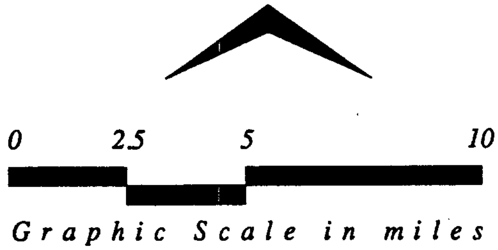
••••••••••	Indian River Lagoon SWIM Planning Boundary
SR	Soils of the Sand Ridges and Coastal Dunes
FW	Soils of the Flatwoods
FP	Soils of the St. Johns River Flood Plains
LH	Soils of the Low-Lying Hammocks
SW	Soils of the Inland and Coastal Wetlands

Adapted from U.S. Department of Agriculture Soil Conservation Service Soil Survey for Brevard County, Florida, issued November 1974

Generalized Soil Map of Study Area Within Brevard County

Figure: 7





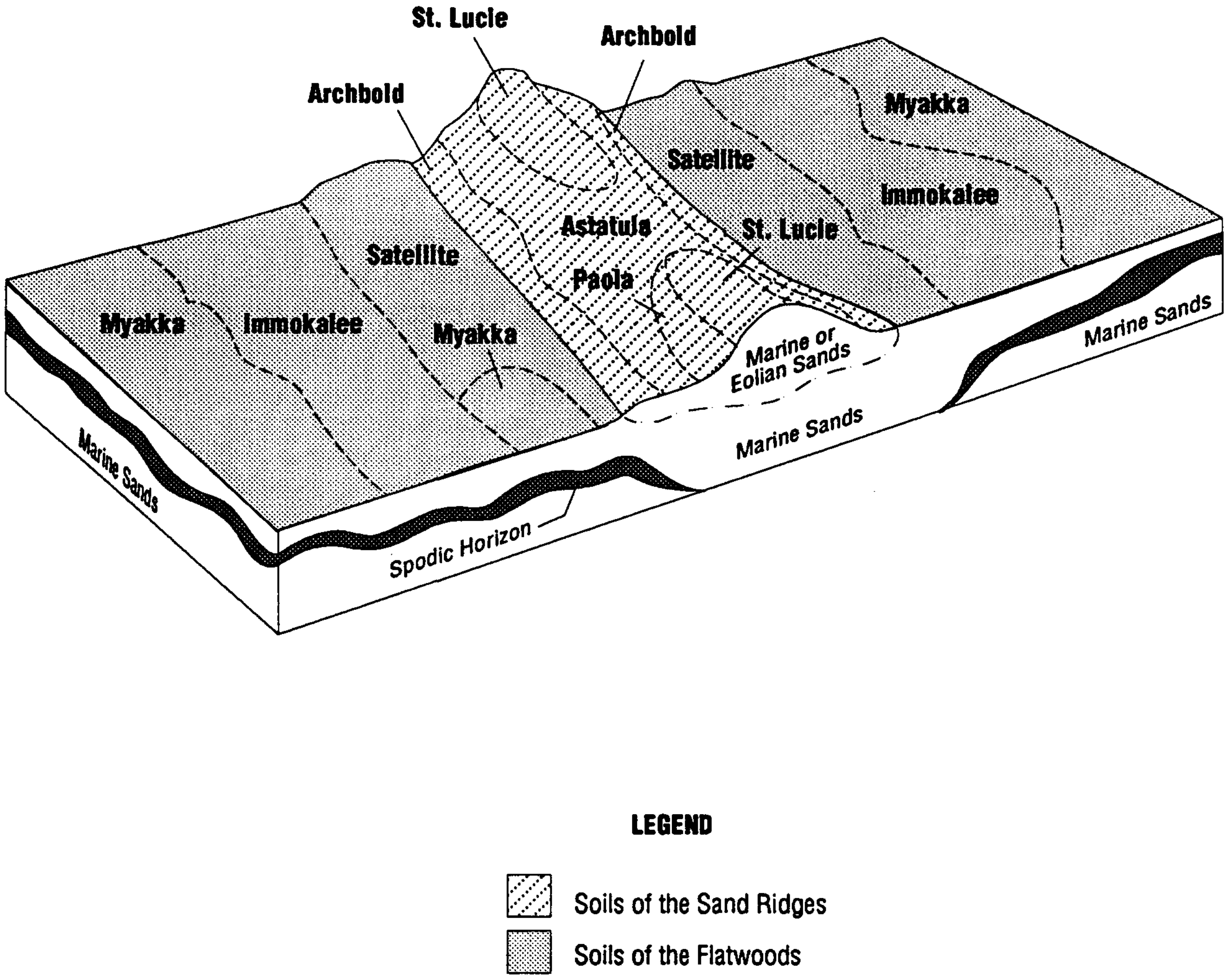
LEGEND

....	Indian River Lagoon SWIM Planning Boundary
SR	Soils of the Sand Ridges and Coastal Dunes
FW	Soils of the Flatwoods
LH	Soils of the Low-Lying Hammocks
SW	Soils of the Inland and Coastal Wetlands

Adapted from U.S. Department of Agriculture
Soil Conservation Service Soil Survey for Volusia
County, Florida, issued February 1980

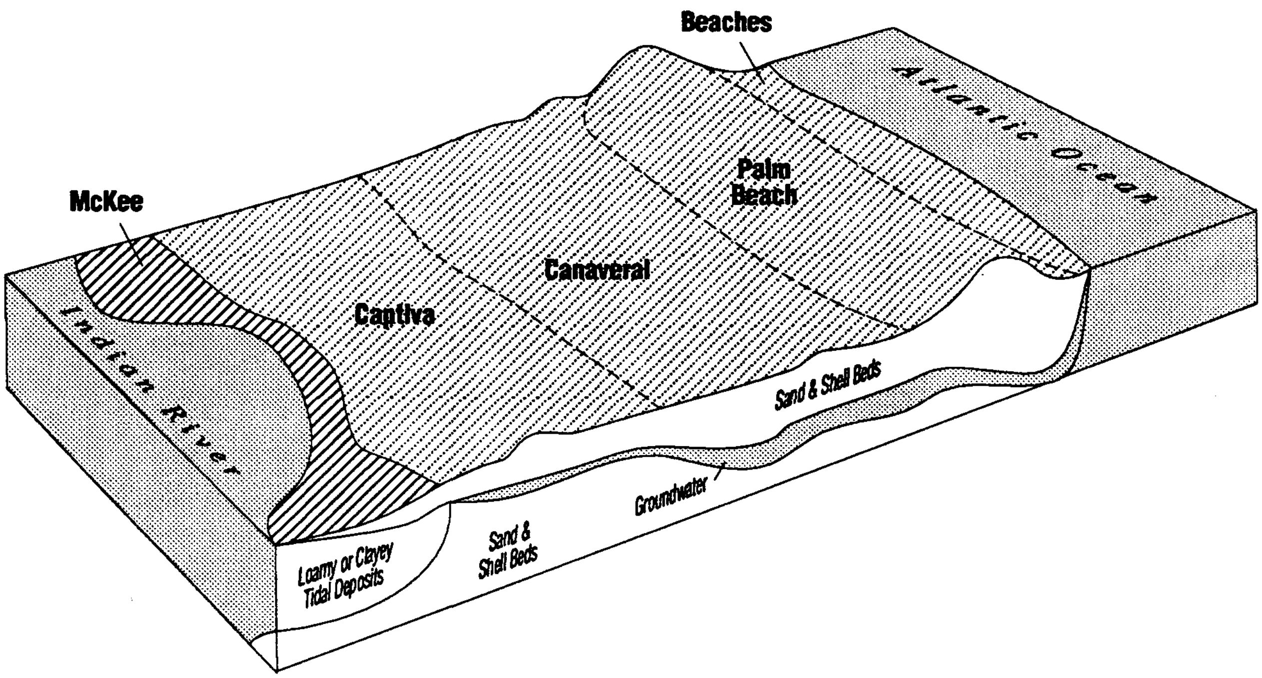
**Generalized Soil Map of
Study Area Within Volusia County**

Figure: 8



Adapted from U.S. Department of Agriculture
Soil Conservation Service Soil Survey for Indian
River County, Florida, issued January 1987

General Pattern of Soils of the Sand Ridges and Flatwoods



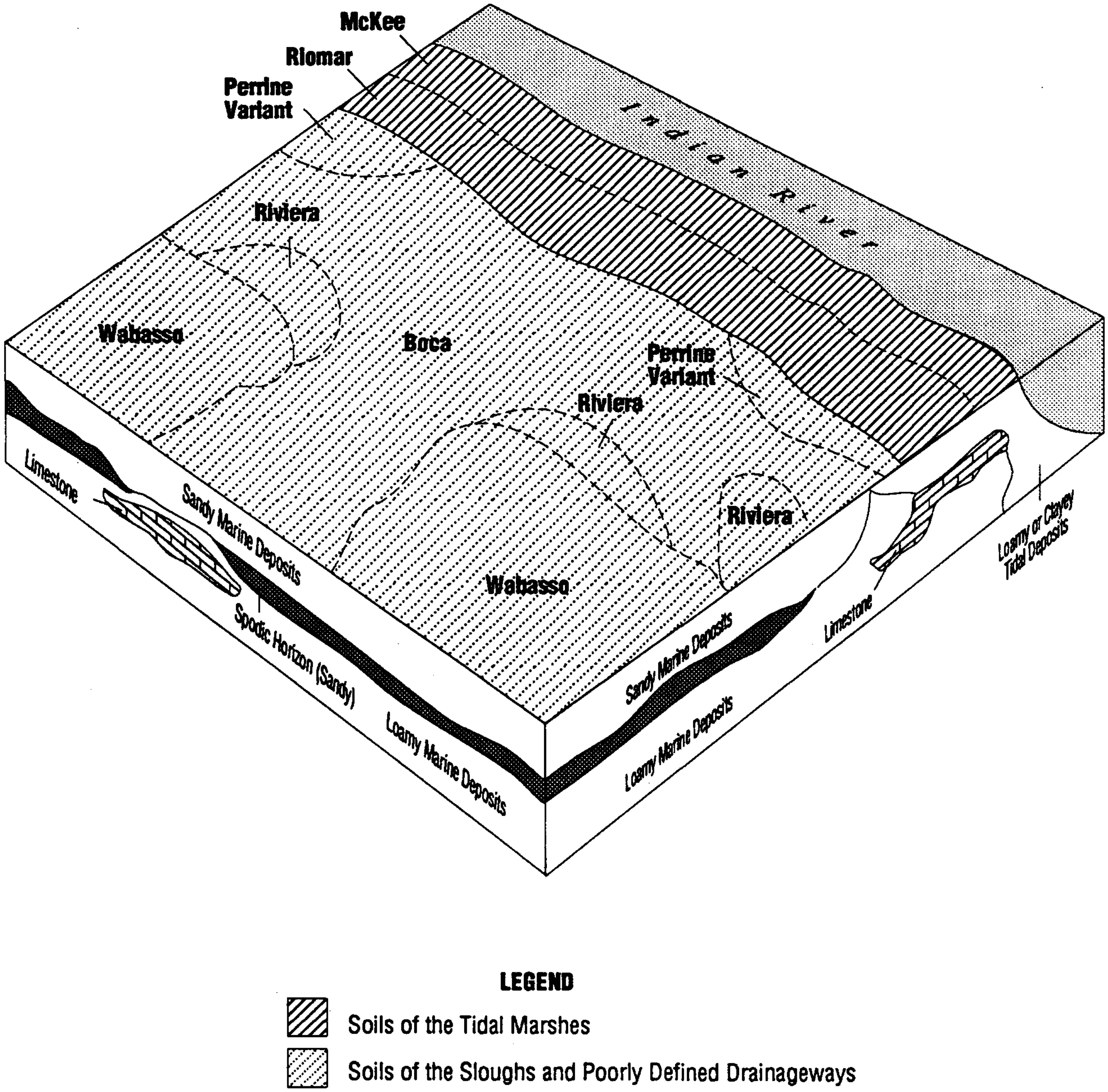
LEGEND

Soils of the Tidal Marshes

Soils of the Coastal Islands

Adapted from U.S. Department of Agriculture Soil Conservation Service Soil Survey for Indian River County, Florida, issued January 1987

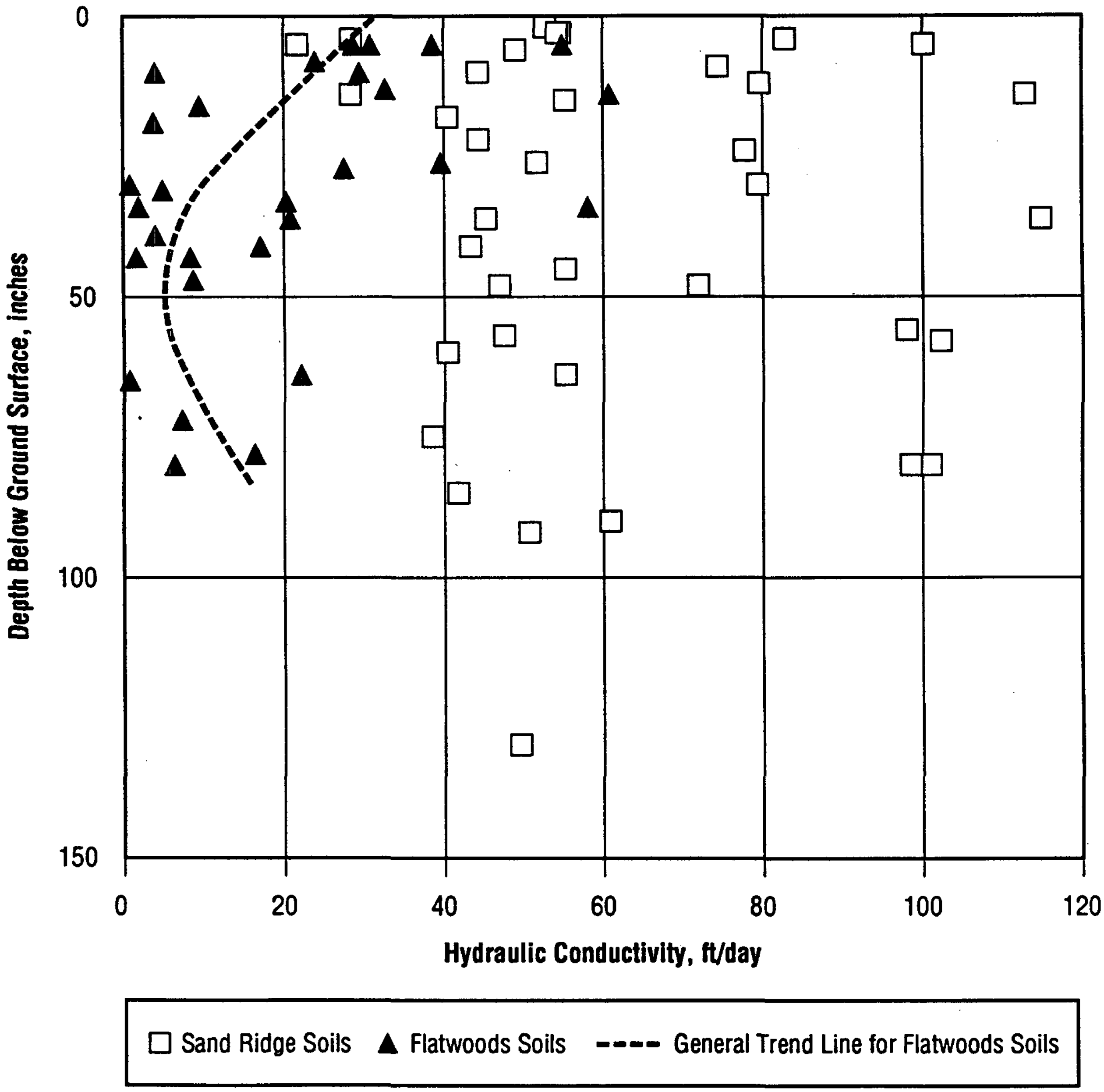
General Pattern of Soils of the Tidal Marshes and Coastal Islands



Adapted from U.S. Department of Agriculture Soil Conservation Service Soil Survey for Indian River County, Florida, issued January 1987

General Pattern of Soils of the Tidal Marshes and Poorly Defined Drainageways

Figure: 11



NOTE: Compiled from data presented in Wettstein et al. (1987), Huckle et al. (1974), and Baldwin et al. (1980)

Variation of Hydraulic Conductivity with Depth for Selected Flatwood Soils & Sand Ridge Soils

SECTION 3.0: SITE SELECTION & DESCRIPTION OF STUDY SITES

3.1 SELECTION OF STUDY SITES

As indicated earlier, the primary objective of this investigation was to evaluate stormwater retention ponds in areas which have rather limited infiltration potential. Accordingly, the purpose of the site selection process was to identify representative retention ponds located within the Indian River Lagoon (IRL) Basin of the SJRWMD which would be suitable candidates for load testing and hydrologic monitoring. Following review of SJRWMD files, twenty (20) potential sites were initially identified. Among the criteria used in assessing the desirability of the various sites for detailed hydrologic monitoring and evaluation were as follows:

- Shallow soil stratigraphy typical of the IRL basin
- Adequate access for truck-mounted drilling equipment to working areas around the pond
- Site security as it relates to protection against vandalism of hydrologic monitoring equipment
- Existence of well-defined inflow and outflow structures which can be conveniently and economically instrumented to measure inflow and outflow at appropriate (single) locations
- Availability of nearby water source, such as a fire hydrant or canal, for short duration load test
- Minimal interference from subsurface drains and canals or recharge sources such as septic tank drainfields within the zone of influence of the groundwater mound created by infiltration from the pond
- Well-defined stage versus storage relationship; localized depressions and heavy vegetation within the pond tend to mask the true configuration and geometry of the pond
- Measurable responses in the observation wells to facilitate meaningful analysis and evaluation of pond recovery data
- Size of the contributing drainage area and the percent impervious cover as it controls the magnitude of the runoff volumes and the resulting hydraulic stresses within the pond and adjacent aquifer

After inspection of the twenty (20) prospective sites and application of the aforementioned criteria, the list of candidate locations was reduced to the eight (8) sites shown on Figure 13, namely

1. Airport Warehouses, Manor and Oleander Drive, Merritt Island, Cocoa

2. Tom Statham Park, U.S. 1, Titusville

3. Tutor-Time Child Care Facility, S.R. 3 and 5th Avenue, Kennedy Space Center

4. Fisherman's Landing, U.S. 1, Grant

5. Port St. John Storage, U.S. 1, Port St. John

6. Merritt Island Store-All, Fortenberry and Plumosa Roads, Cocoa

7. New Georgiana Settlement, Courtenay Parkway, Merritt Island

8. United Parcel Service, 41st Street and 43rd Avenue, Vero Beach

SJRWMD staff contacted the property owners of the eight sites and obtained permission to conduct preliminary hydrogeologic explorations. The scope of these preliminary investigations included site reconnaissance, one to two 10-30 ft deep auger borings per site, ground water table measurements in the open boreholes, and visual classification of the soil samples. Based on review of the preliminary data and using the criteria set forth above, the following four (4) ponds were selected for detailed study:

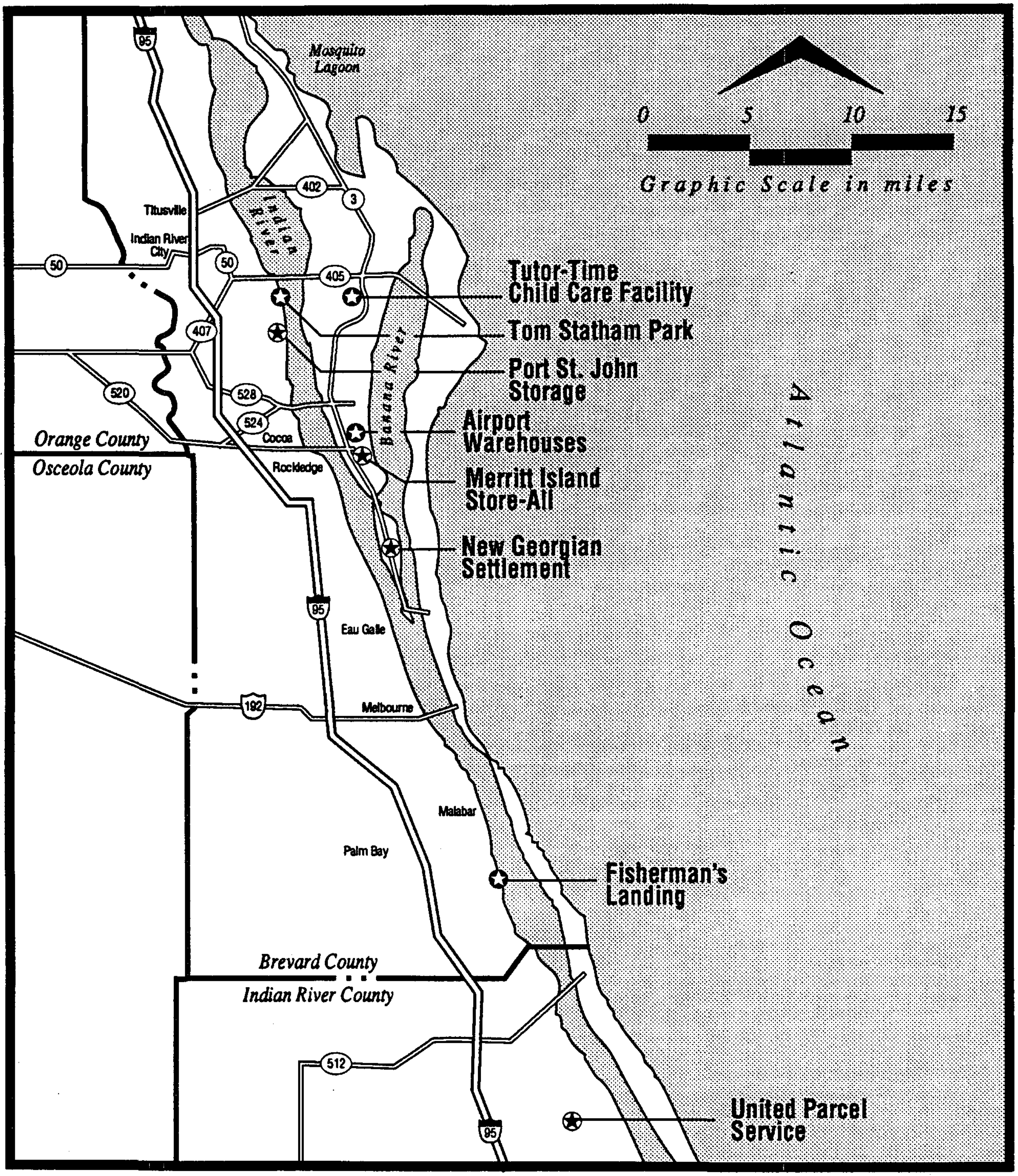
Long-Term Monitoring

- Airport Warehouses
- Tom Statham Park

Short-Term Load Test

- Tutor-Time Child Care Facility
- Fisherman's Landing

The drainage characteristics of these sites are briefly described in the following section.



LEGEND

Preliminary Study Site Selection

Selected Study Sites

Locations of Study Sites

Figure: 13

3.2 DESCRIPTION OF SELECTED STUDY SITES & RETENTION PONDS

3.2.1 Airport Warehouses

The Airport Warehouses site is located at the Merritt Island Airport in Brevard County, Florida. The airport is located in the town of Merritt Island, Florida near the southerly end of Merritt Island, between the Indian River to the west and Newfound Harbor of the Banana River to the east. A site vicinity map is presented on Figure 14, and a layout of the site itself is presented on Figure 15.

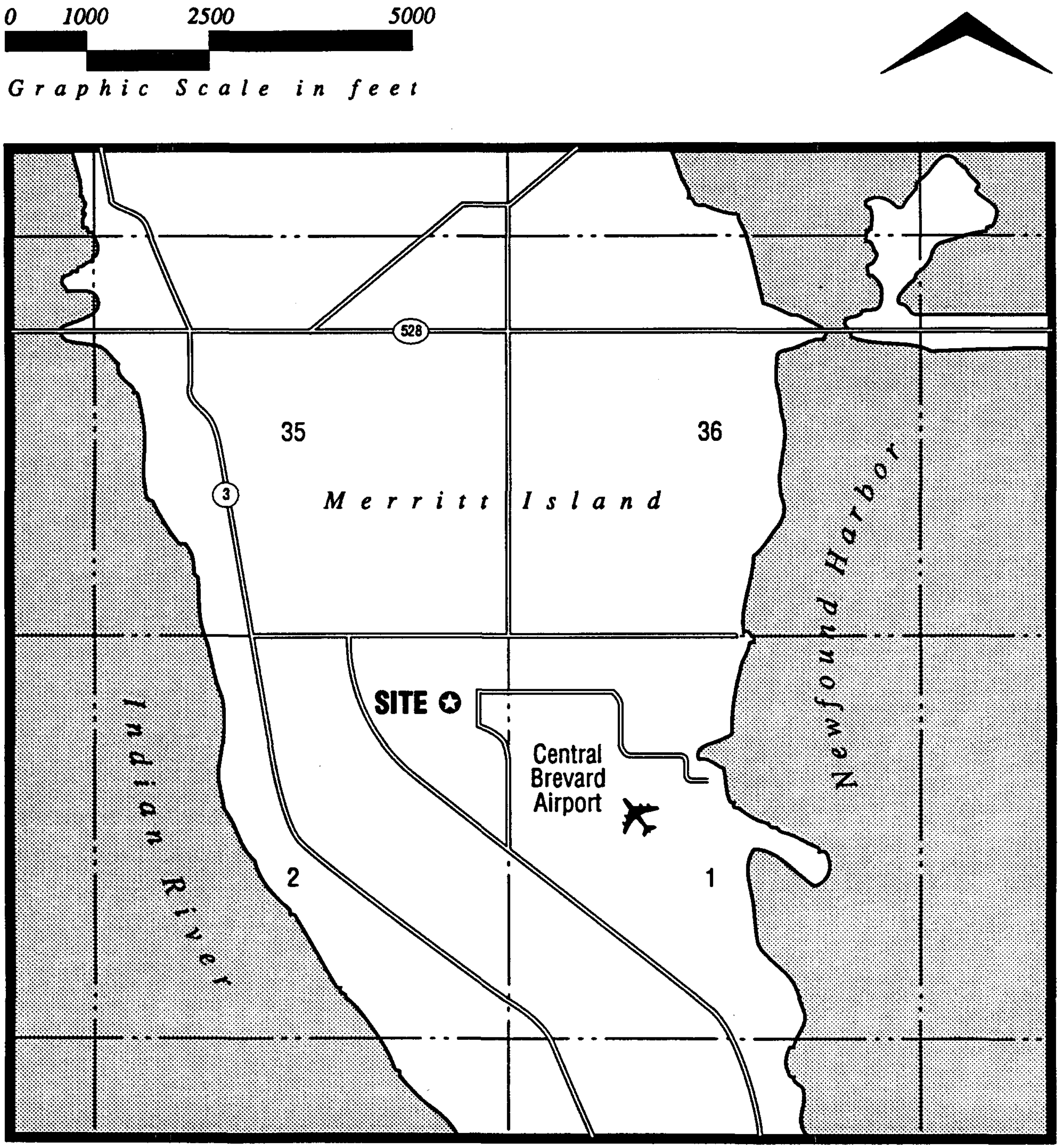
The stormwater retention pond at Airport Warehouses was designed as a closed, dry-bottom pond. Based on visual observations of stormwater recovery, it appears that the pond is operating as intended and the pond bottom is usually dry. There are no canals, ditches, or other drainage features adjacent to the pond.

The pond is approximately 57 ft × 64 ft at the top of slope, and has a bottom area of approximately 2,028 square feet. The side slopes are approximately 2 horizontal to 1 vertical. On average, the pond bottom is at an approximate elevation of +6 feet NGVD, and the top of slope elevation ranges from +10 to +11 ft NGVD. The stage versus storage relationship of the pond is tabulated below:

Stage (ft NGVD)	Cumulative Storage (ft^3)
+6	0
+7	2,141
+8	4,687
+9	7,579
+10	10,857

The pond receives runoff from a parking lot and from two (2) buildings through a 3-foot wide spillway and two (2) 6-inch diameter roof drain collectors. Overland flow into the pond is not a normal occurrence.

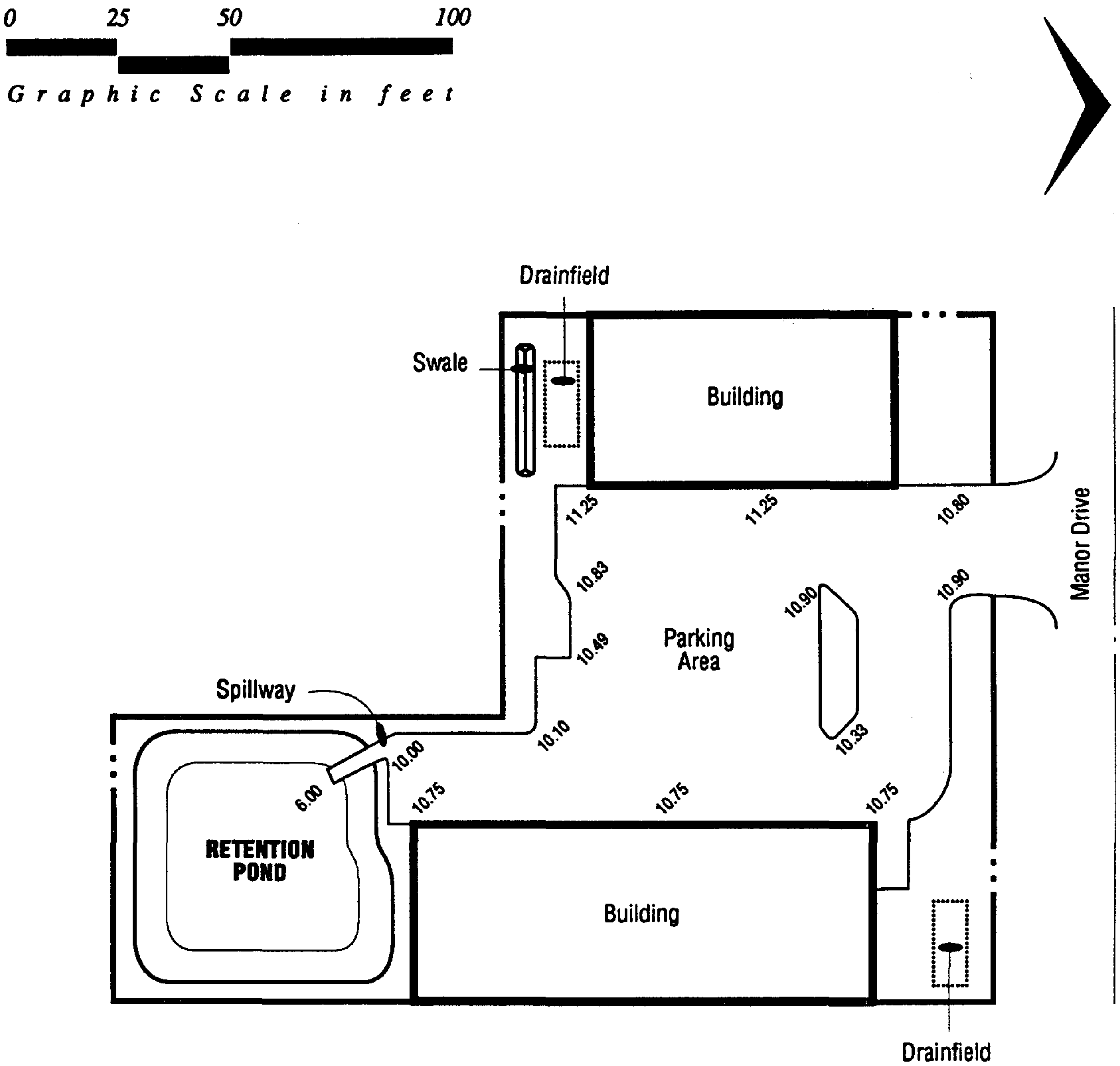
The Airport Warehouse site encompasses a total area of approximately 0.67 acres. The majority of the site drains directly into the retention pond with the exception of a small area of greenspace (870 square feet) located along the northern boundary of the property, where drainage flows off-site toward Manor Drive. Figure 16 delineates the contributing drainage basin for this pond and also shows the relatively impervious and pervious areas of the site with the general direction of surface water flow.



Adapted from USGS "Cocoa, Florida" quadrangle map
issued 1949, photorevised 1980
Site located within Section 2, Township 25 south, Range 36 east

Site Location
Airport Warehouses

Figure: 14

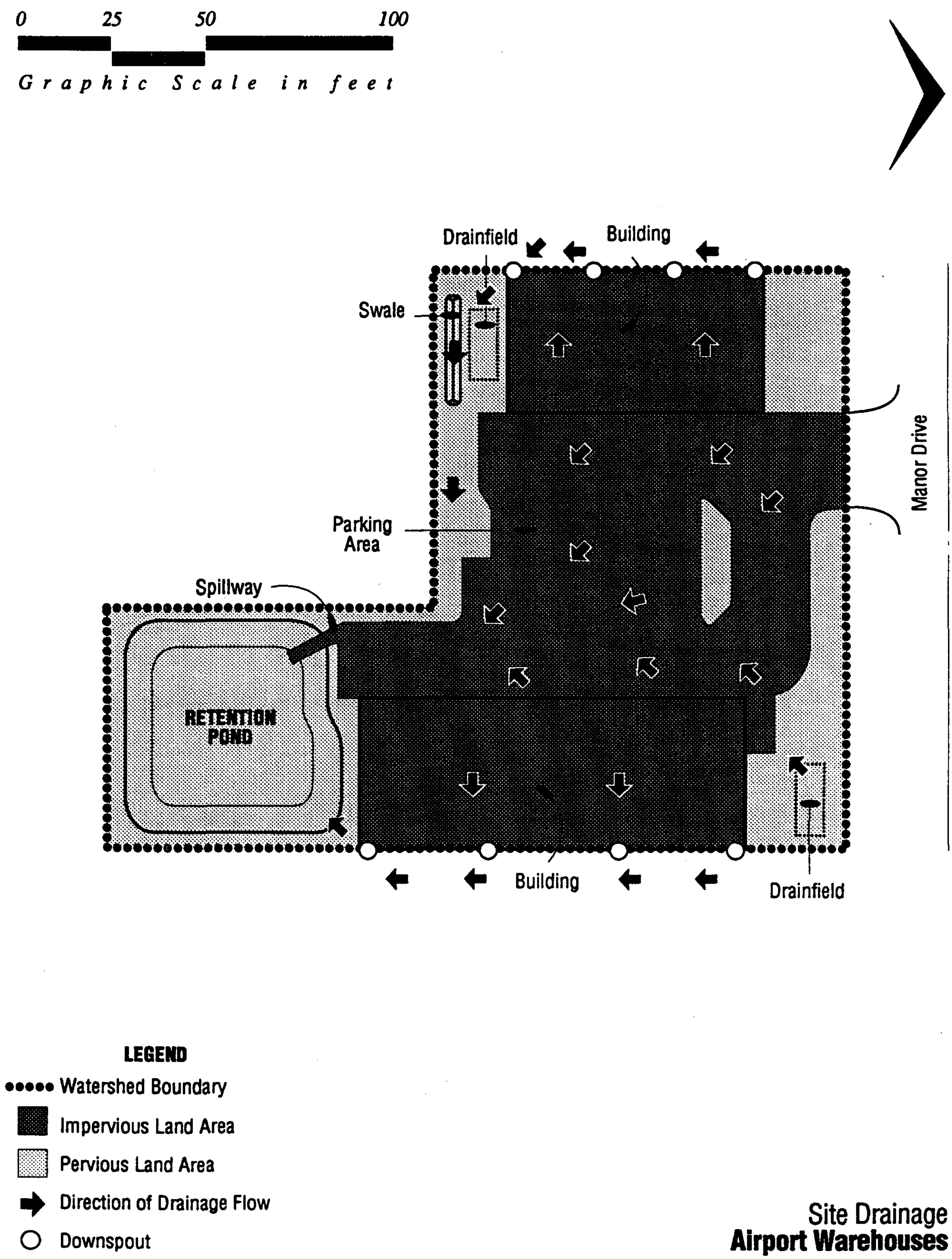


LEGEND

8.77 Finished Grade Elevation in ft NGVD

Site Features
Airport Warehouses

Figure: 15



Site Drainage
Airport Warehouses

Figure: 16

The total area within the contributing drainage area boundary, including the retention pond, is approximately 28,500 square feet (0.65 acres). Impervious ground cover, including parking areas and buildings, account for 68 percent of the drainage basin or 19,350 square feet (0.44 acres). Runoff from the parking area drains directly to a spillway located at the northwest corner of the pond while the roofs of the two warehouse buildings located along the east and west sides of the property have gutters and downspouts installed which also drain into the pond. Runoff from the east warehouse roof drains directly to the pond and the west warehouse drains into a swale before outfalling through a solid pipe connected to the pond. The pervious area within the contributing drainage area includes landscape areas and the pond itself. This accounts for 32 percent of the total contributing drainage area or 9,150 square feet (0.21 acres).

The principal advantages in the selection of the Airport Warehouses site were as follows:

- good access to pond and adjacent areas
- good security
- pond geometry and stage-storage relationship are well-defined
- control of stormwater inflow and no overflow structure
- extraneous flows entering the pond can be controlled
- soil conditions are typical and uniformly stratified
- substantial volume of runoff from impervious areas

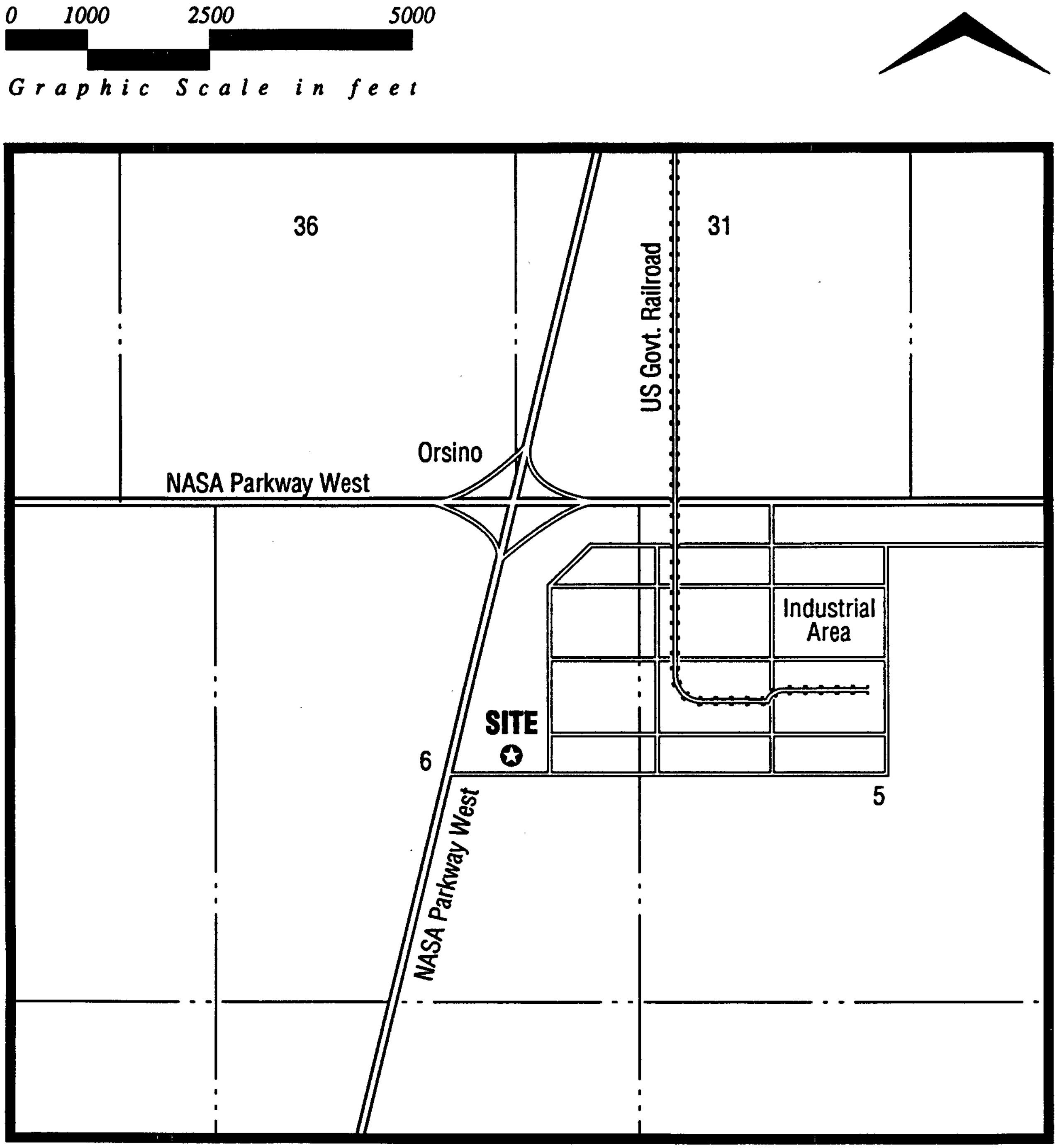
On the other hand, the disadvantages in selecting this pond included:

- inflow structure needed to be modified to accommodate instrumentation
- minor modifications to divert roof gutter flow and flow from parking lot on northern perimeter

3.3.2 Tutor Time

The Tutor-Time Child Care Facility study site is located on Merritt Island, south of Orsino, Brevard County, Florida. The site is situated southeast of the intersection of the NASA Causeway and the Kennedy Parkway. A site vicinity map is presented on Figure 17 and a layout of the site itself is presented on Figure 18.

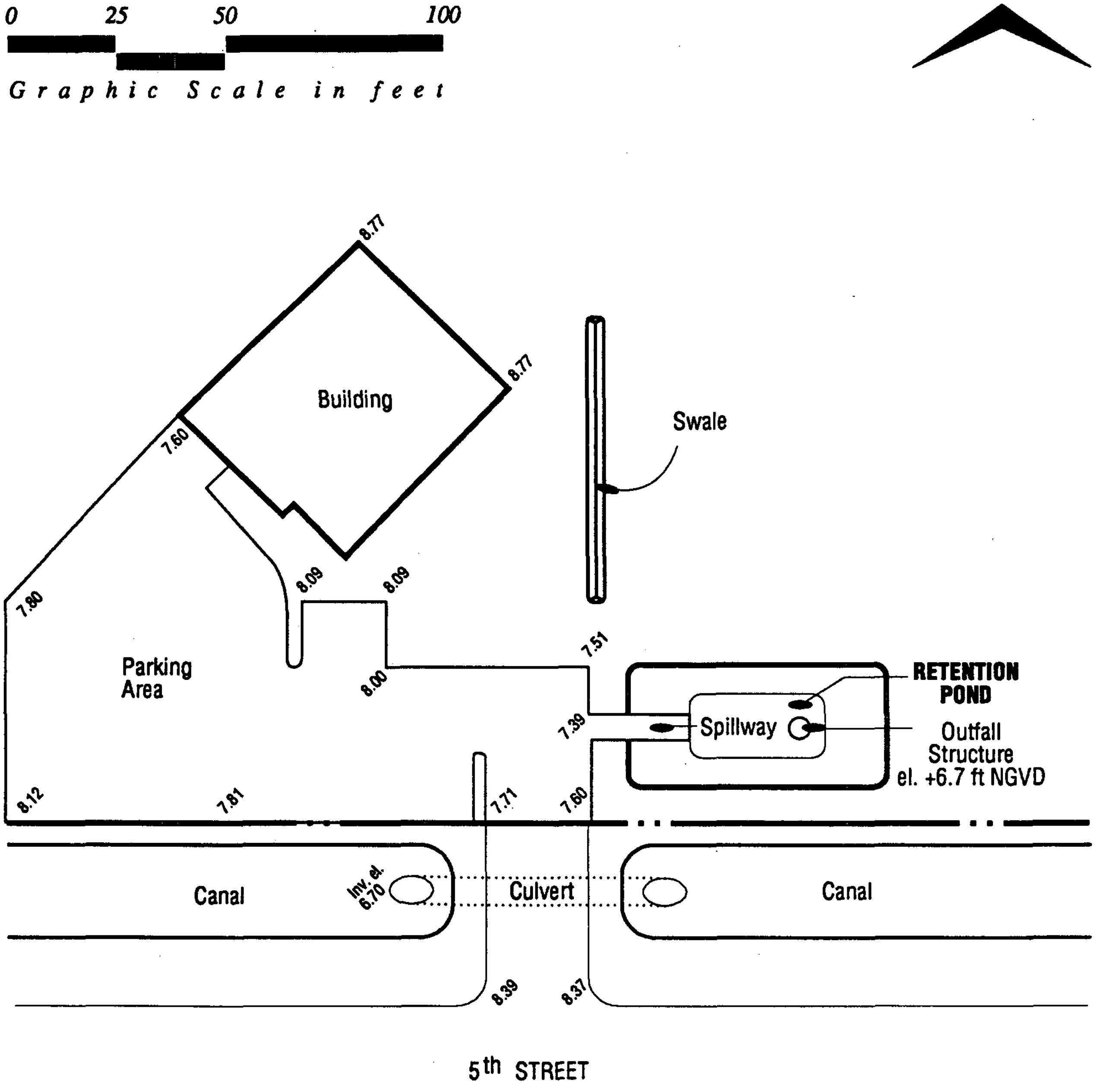
Two (2) retention areas are used to manage stormwater runoff at this facility and the east pond is the subject of our study. This pond is of approximate dimensions 30 × 55 feet at the top of slope with a bottom area of approximately 450 square feet. The side slopes are approximately 4 horizontal to 1 vertical. On average, the pond bottom elevation is +4.8 feet NGVD, and the top of berm ranges from elevation +7 to +8 feet NGVD. The stage versus storage relationship of the pond is tabulated below:



Adapted from USGS "Orsino, Florida" quadrangle map
issued 1976
Site located within Section 6, Township 23 south, Range 37 east

Site Location
Tutor-Time Child Care Facility

Figure: 17



LEGEND

8.77 Finished Grade Elevation in ft NGVD

Site Features
Tutor-Time Child Care Facility

Figure: 18

Stage (ft NGVD)	Cumulative Storage (ft^3)
+4.8	0
+5.0	93
+6.0	927
+6.65 (overflow)	1,840

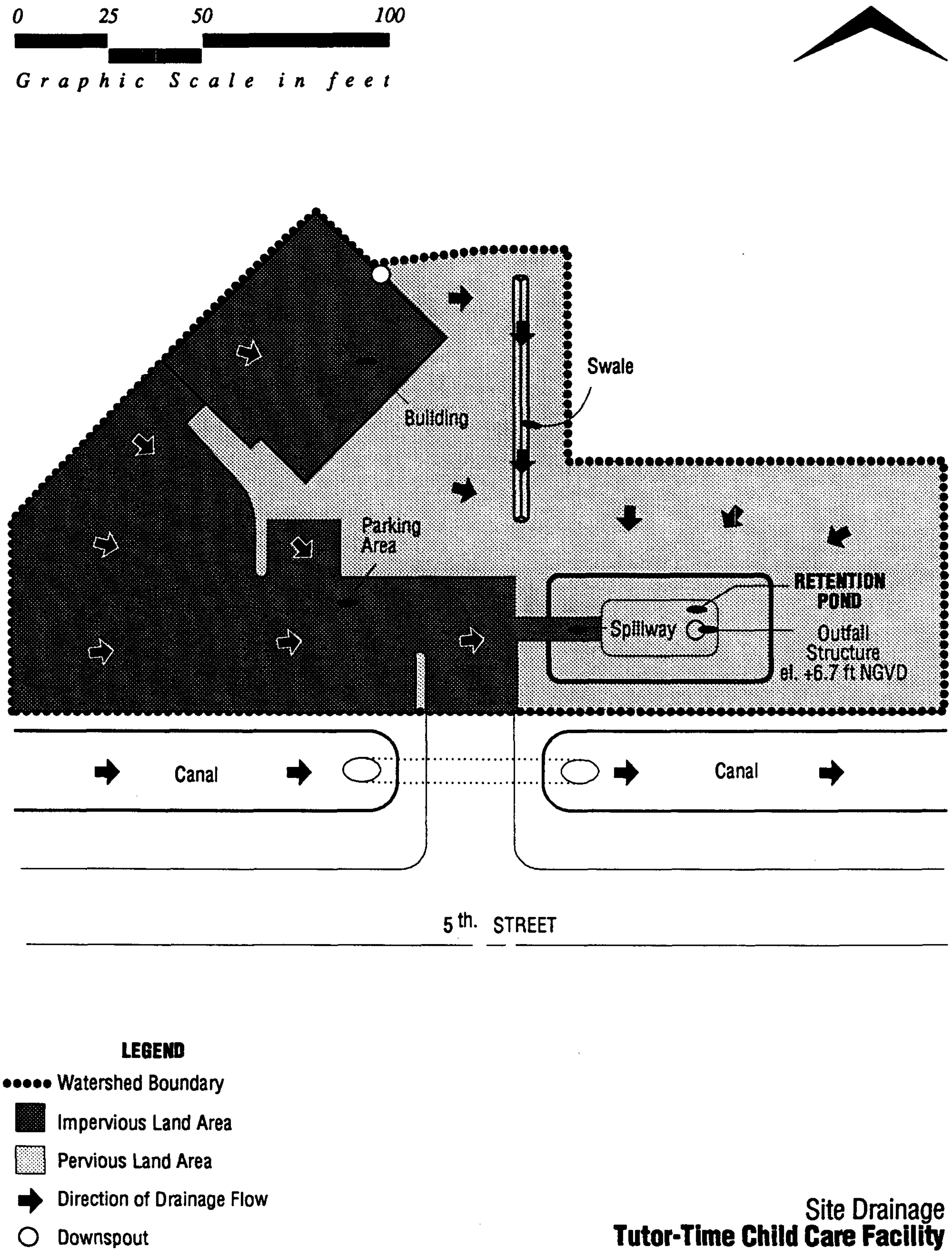
An east-west aligned roadway drainage ditch is located along 5th Avenue, about 45 feet south of the pond. The invert elevation of this ditch is approximately +4.2 ft NGVD.

The stormwater retention pond at the Tutor-Time Child Care Facility was designed as a dry-bottom pond with a positive outfall at elevation +6.73 ft NGVD. Based on visual observations of its stormwater recovery during field visits, it appears that this pond is operating as intended. Prior to the short-term load testing, the pond bottom was scarified.

The contributing drainage area for this pond comprises approximately 31,100 square feet (0.71 acres) and includes portions of the parking area, building, landscape areas and the pond itself. Figure 19 shows the contributing drainage area boundary for this site. The pond receives runoff from a parking lot and driveway area, and a building via an inflow spillway at the western end of the pond. Although overland flow into the pond is not expected to be a normal occurrence, runoff from pervious areas is likely to occur following periods of heavy or extended rainfall.

Impervious ground cover includes the parking and building areas and accounts for approximately 48 percent of the total contributing drainage area, or 15,000 square feet. Runoff from the parking area drains to a spillway at the west end of the pond. Runoff from the building area is channeled to a downspout at the back of the building which discharges across the east lawn toward a 130 feet long swale. A portion of this runoff seeps into the ground before reaching the swale.

The pervious area of the contributing drainage area comprises the remaining 52 percent, or approximately 16,100 square feet. The pervious areas include landscape areas, an undeveloped area to the north of the pond, and the pond itself. In addition, the swale referenced to above, stages and overflows to the pond during heavy rainfall events.



Site Drainage
Tutor-Time Child Care Facility

Figure: 19

The advantages in selecting the Tutor Time pond were as follows:

- good access and ability to install observation wells on all sides
- adequate water source is available for a load test
- very good security
- the pond geometry and stage-storage relationship are well-defined
- the soil stratigraphy is typical
- there is a single inflow and a single overflow structure
- a roadway ditch system is located nearby

The main disadvantage of this site is that the pond is relatively small.

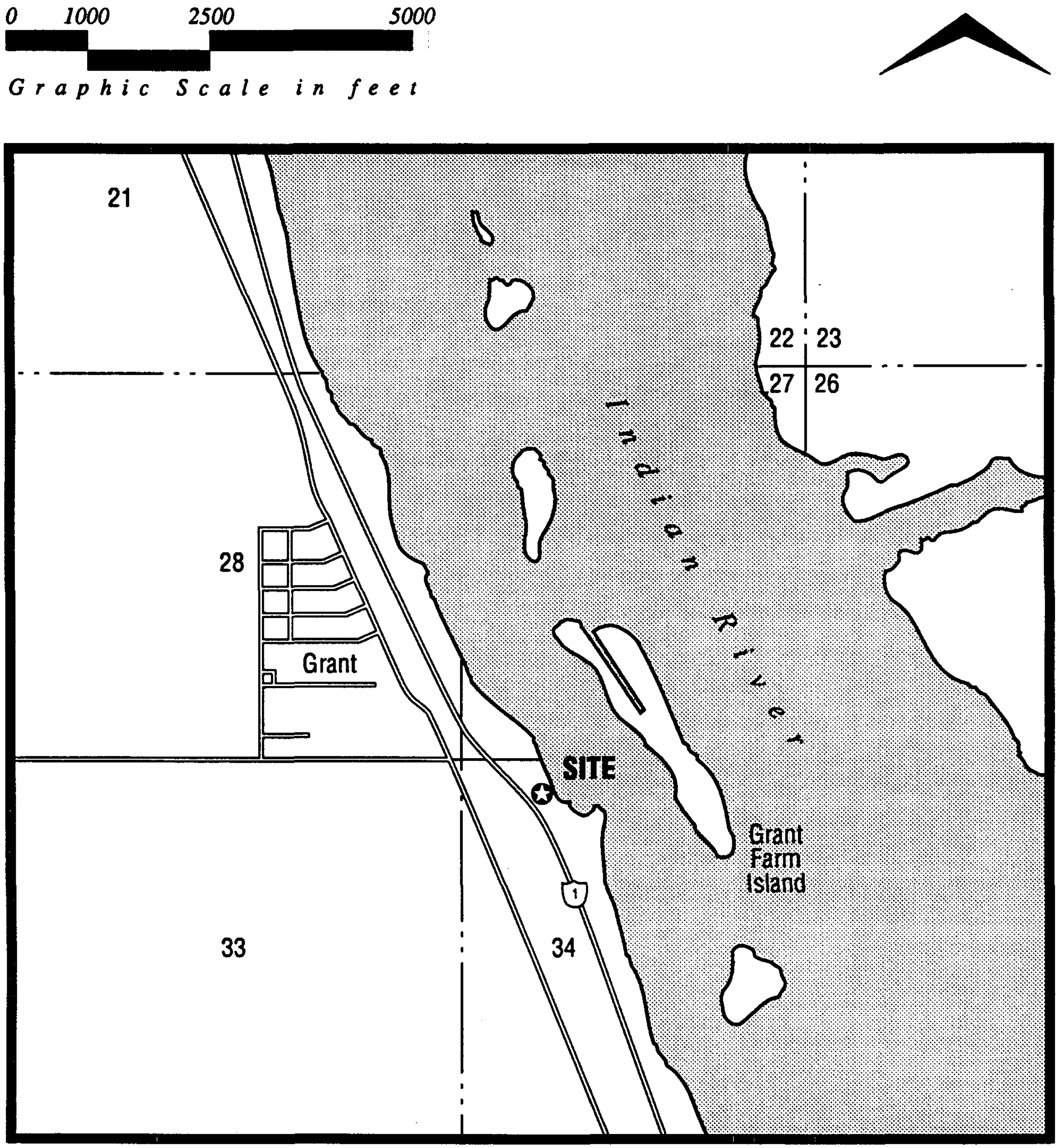
3.3.3 Fisherman's Landing

The Fisherman's Landing study site is located between US Highway 1 and the west bank of the Indian River just south of Grant, in Brevard County, Florida. A site vicinity map is presented on Figure 20 and a layout of the site itself is presented on Figure 21.

There are two (2) retention ponds within the Fisherman's Landing site and the southernmost pond was selected for our study. As noted on Figure 21, the pond is located approximately 90 feet from the Indian River and approximately 30 feet north of a mounded septic tank/drainfield.

This pond was designed as a dry bottom pond with a positive outfall, although, based on visual observations, its bottom normally remains wet since the water level does not fully recover. Prior to the short-term load test, however, the vegetation within the pond was mowed and the pond bottom scarified.

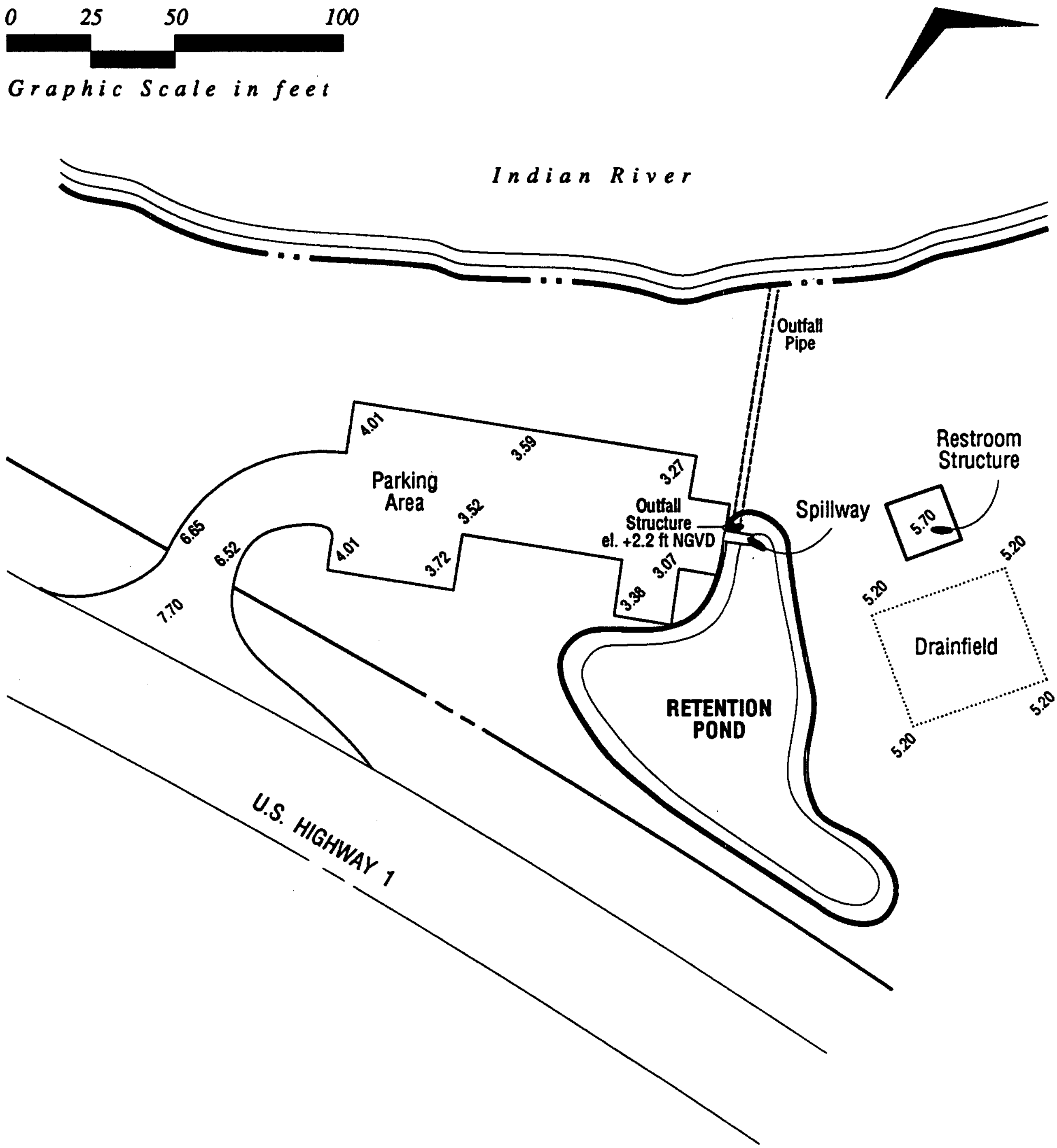
The pond is irregularly shaped with dimensions of approximately 80 $\times$ 120 feet at the top of the berm and a bottom area of approximately 5,600 square feet. The side slopes are approximately 4 horizontal to 1 vertical. On average, the pond bottom elevation is +1.5 ft NGVD and the top of berm ranges from elevation +3.0 to +3.5 ft NGVD. The discharge elevation is at +2.2 ft NGVD. The stage versus storage relationship of the pond is tabulated below:



Adapted from USGS "Grant, Florida" quadrangle map
issued 1949, photorevised 1980
Site located within Section 27, Township 29 south, Range 38 east

Site Location
Fisherman's Landing

Figure: 20



LEGEND

8.77 Finished Grade Elevation in ft NGVD

Site Features
Fisherman's Landing

Figure: 21

Stage (ft NGVD)	Cumulative Storage (ft^3)
+1.0	0
+1.2	200
+1.4	654
+1.6	1,361
+1.8	2,322
+2.0	3,536
+2.2	4,968

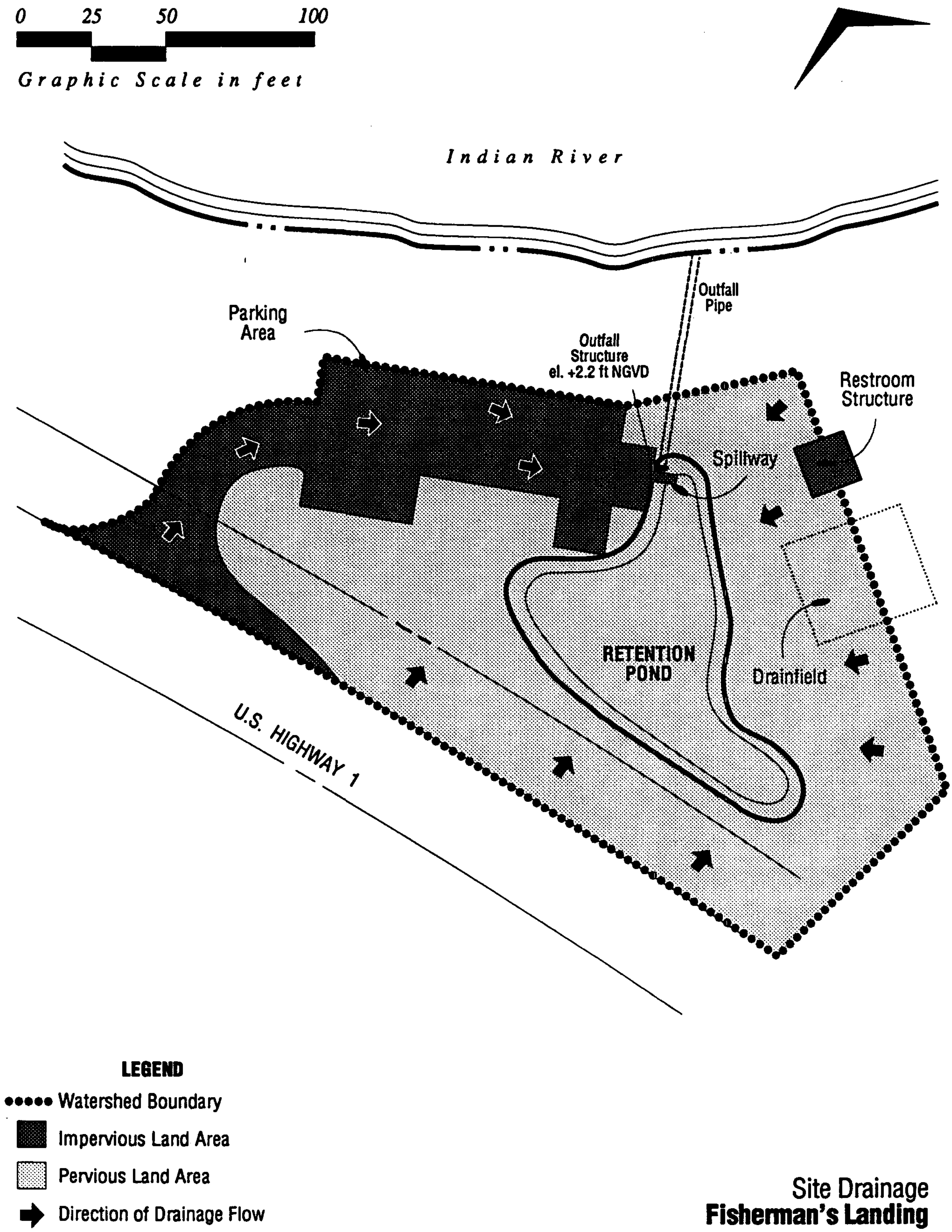
The contributing drainage area for this pond comprises approximately 52,800 square feet (1.21 acres) and includes the parking area, restrooms, septic tank and drainfield system, and the pond itself. Figure 22 depicts the contributing drainage area boundary for this site.

Impervious ground cover, which includes the parking area and restrooms, accounts for approximately 14.5 percent of the total contributing drainage area or 7,650 square feet. Runoff from the parking area drains into the pond via spillway at the northeast corner. The U.S. Highway 1, located west of the site, slopes westward, away from the pond, and is not considered part of the pond's contributing drainage area.

The remaining 85.5 percent of the pond contributing drainage area consists of pervious ground cover and includes landscape (lawn) areas around the pond, the septic tank drainfield, and the pond itself. The drainfield is located approximately 30 feet southeast of the retention pond and is expected to have some effect on stormwater recovery.

The advantages of this site included:

- good access and ability to instrument the pond on all sides
- the pond is close to the Indian River
- adequate source of water is available for a load test
- the soil stratigraphy is typical
- the pond does not recover as intended and would, therefore, constitute a good test case



Site Drainage
Fisherman's Landing

Figure: 22

The disadvantages of this site include:

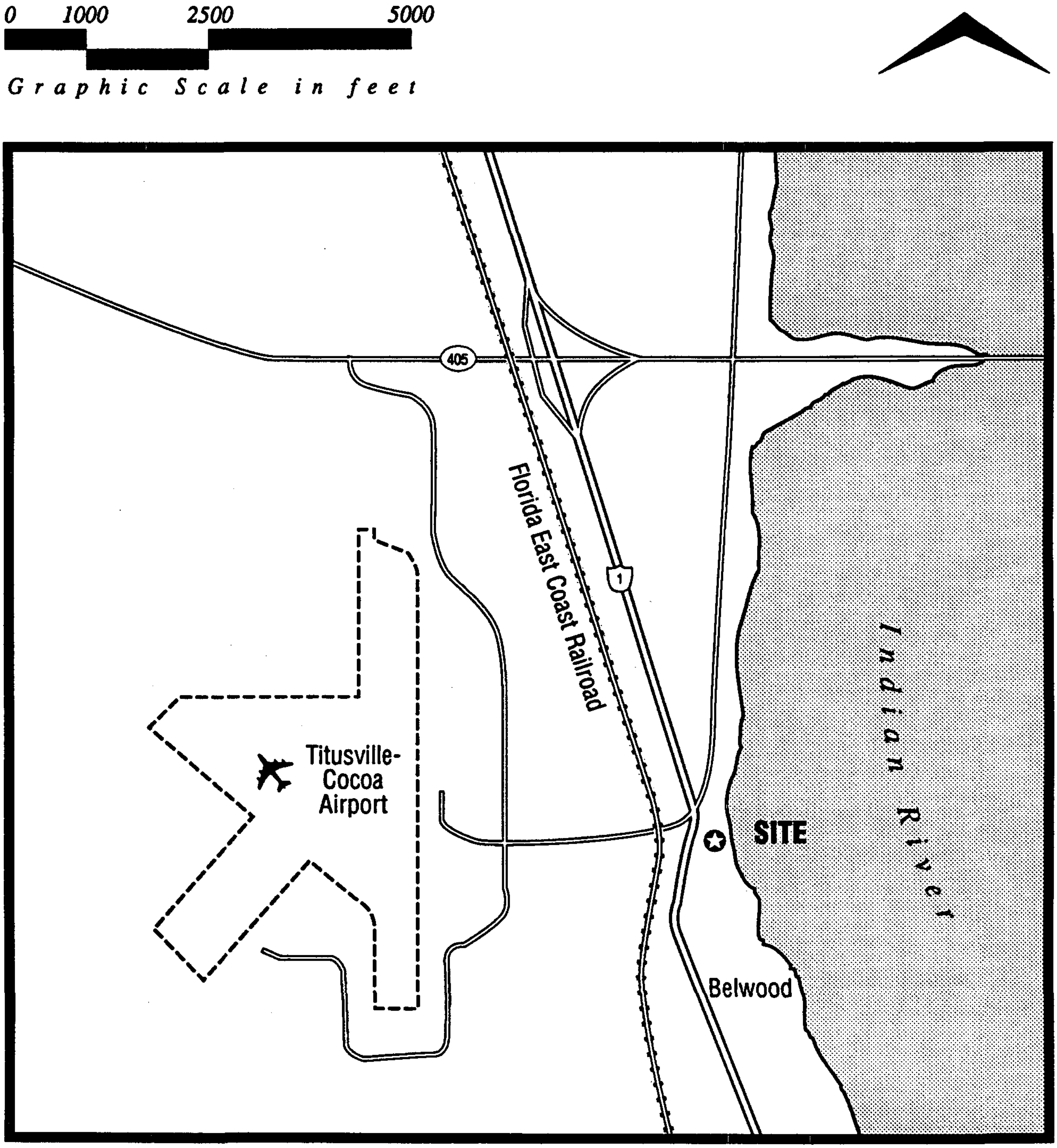
- the pond geometry was not well-defined due to the vegetation overgrowth in the pond
- existence of a nearby septic drainfield from the park washroom
- being a public access area, the site would not be suitable for long-term monitoring from the point of view of security

3.3.4 Tom Statham Park

The Tom Statham Park study site is located just east of the Titusville-Cocoa Airport in Brevard County, Florida. The park is located on the west side of the Indian River between Indian River City and Cocoa, Florida near Bellwood, south of the NASA Causeway. A site vicinity map is presented on Figure 23 and a layout of the site itself is presented on Figure 24.

The stormwater retention pond at Tom Statham Park was designed as a dry-bottom pond with a positive outfall. However, based on visual observations of performance, the pond does not recover as intended and remains wet most of the time with typical emergent vegetation (i.e., cattails, etc.). Because of the heavy vegetation within the pond and the continual wet conditions, it was not possible to remove the vegetation or any pond bottom sediments prior to this study.

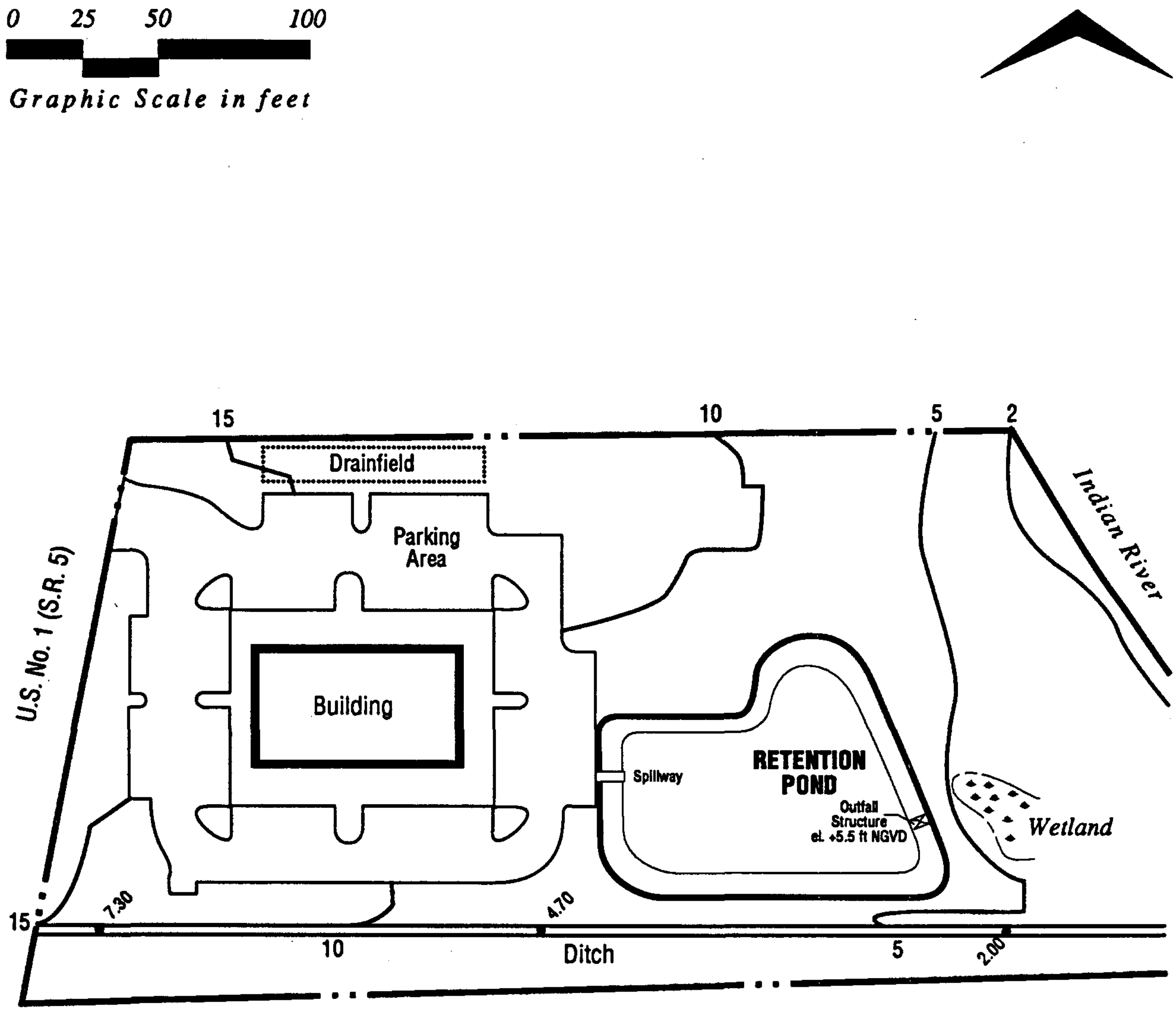
The pond is of approximate dimensions 150 × 180 feet at the top of slope with a bottom area of approximately 18,000 square feet. The side slopes are approximately 4 horizontal to 1 vertical. The average pond bottom elevation is approximately +4.5 feet NGVD, and the top of berm ranges from elevation +5.8 to +8.6 feet NGVD. The pond outfalls through a concrete spillway on the eastern side of the pond with a crest elevation of +5.5 ft NGVD. Water overflowing from the pond via the spillway is directed eastward, away from the pond. A canal is located approximately 30 feet south of the pond. The water in this canal flows eastward into the Indian River, which itself is located about 200 feet east of the pond.



Adapted from USGS "Titusville, Florida" quadrangle map
issued 1949, photorevised 1980
Site located within Section 1, Township 22 south, Range 35 east

Site Location
Tom Statham Park

Figure: 23



LEGEND

—15— Finished Grade Elevation in ft NGVD

4.70 Ditch Bottom Elevation in ft NGVD

Site Features
Tom Statham Park

Figure: 24

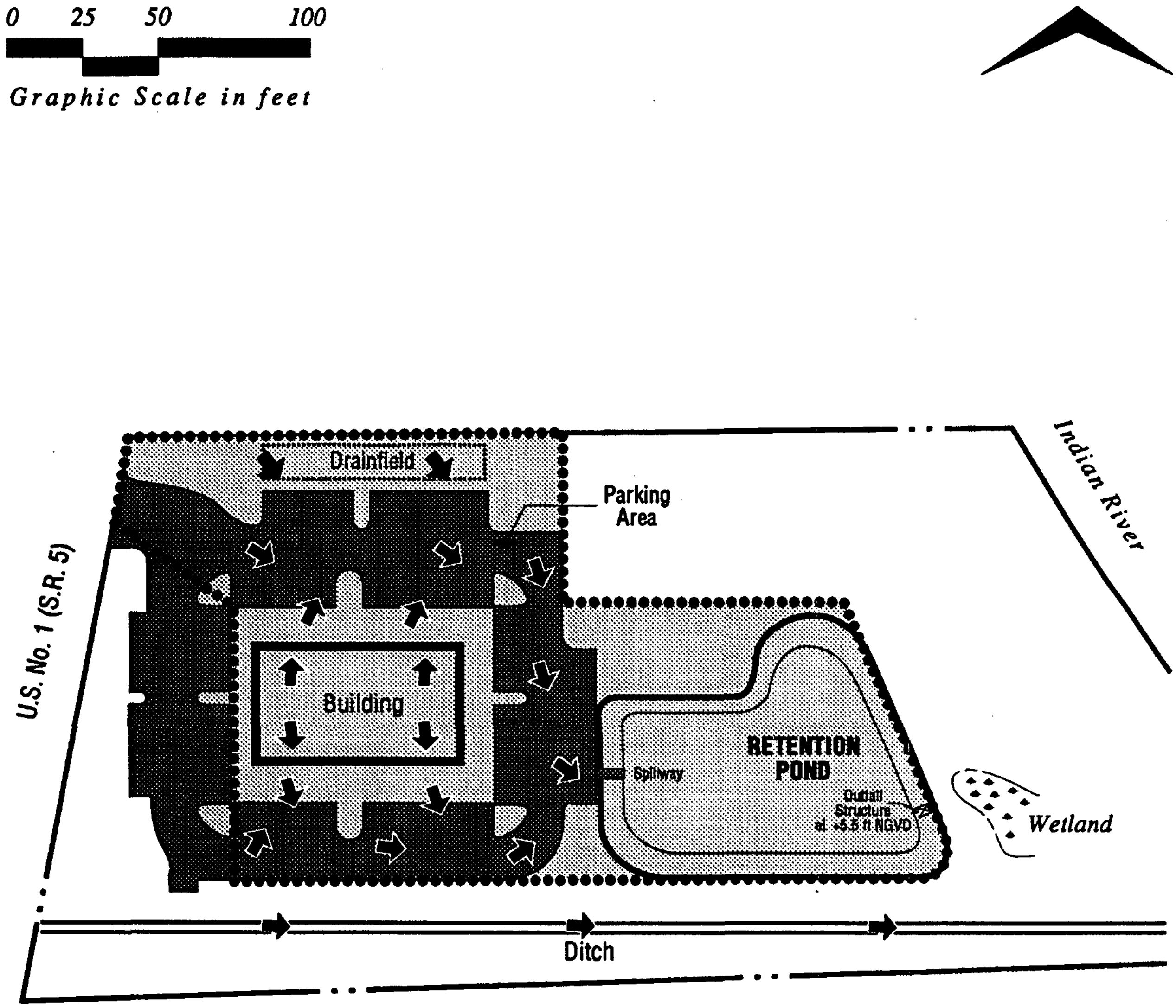
The stage versus storage relationship of the pond is tabulated below:

Stage (ft NGVD)	Cumulative Storage (ft^3)
+4.0	0
+4.2	385
+4.4	1,409
+4.6	3.071
+4.8	5,371
+5.0	8,311
+5.2	11,691
+5.4	15,311
+5.6	19,171

The pond receives runoff from one building, parking lot and driveway areas, and from lawn areas through a 4.3 foot wide inflow flume. Some overland flow into the pond may occur from the north side during heavy or extended rainfall events, although it is not expected to be significant in terms of the overall percentage of inflow. This pond is equipped with an overflow spillway having a crest elevation of +5.5 feet NGVD.

The total area of Tom Statham Park is approximately 5 acres and includes the one-story building, parking area, picnic and recreation areas, and the retention pond. The contributing drainage area for the pond primarily includes the building and parking areas. The contributing drainage area is approximately 94,100 square feet (2.16 acres) and the contributing drainage area boundary is denoted by the heavy line on Figure 25.

Impervious ground cover is limited to the building and parking areas and accounts for about 33.7 percent of the total contributing drainage area or 31,700 square feet. The western portion of the parking area discharges a certain amount of stormwater overland to a canal south of the park and is not considered part of the pond contributing drainage area. Runoff from the remaining parking area drains to a spillway located on the west side of the retention pond. The building area is equipped with downspouts which direct runoff over the lawn area to the north and south and onto the parking lot. A portion of this runoff eventually seeps into the ground before reaching the parking lot.



LEGEND

- Watershed Boundary
- Impervious Land Area
- Pervious Land Area
- Direction of Drainage Flow

Site Drainage
Tom Statham Park

Figure: 25

The pervious area of the pond contributing drainage area is approximately 62,400 square feet or 66.3 percent of the total contributing drainage area, including the retention pond. The northern edge of the pond is not bermed and receives overland flow from the adjacent pervious ground surface. The landscape area at the northern portion of the site drains into the parking area and then into the pond.

Figure 25 denotes the impervious and pervious ground cover within the pond contributing drainage area and indicates the general direction of surface water flow.

The advantages in selecting this site included:

- proximity to Indian River
- good access and ability to instrument the pond on all sides
- pond geometry and stage-storage relationship are well-defined
- the pond is located adjacent to a canal which flows to the Indian River
- the pond has one inflow and one overflow structure
- the pond does not recover as intended via natural exfiltration, and therefore represents a good test case
- the soil conditions were typical

On the other hand, the disadvantages of this site included:

- lack of security for the instrumentation
- difficulty in instrumenting the overflow structure

SECTION 4.0: AQUIFER CHARACTERIZATION AT STUDY SITES

4.1 GENERAL

A comprehensive review of the field and laboratory test methods available for estimating representative aquifer parameters for retention pond recovery analysis can be found in the report titled *Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers* (Jammal & Associates, Inc., 1989). Typically, the following list of aquifer parameters are required for input into the groundwater flow pond recovery models:

- Thickness or elevation of base of mobilized (effective) aquifer
- Weighted horizontal hydraulic conductivity of mobilized aquifer
- Fillable porosity of mobilized aquifer
- Ambient water table elevation which, for design purposes, is usually the normal seasonal high water table

The field and laboratory test programs were tailored to accomplish three primary objectives:

1. Collect the necessary soil and groundwater data, using conventional test methods, to estimate the site-specific aquifer parameters for use in comparing the predictive capabilities of selected recovery analysis models.

2. Conduct a suite of typical insitu and laboratory hydraulic conductivity tests for evaluating the sensitivity and correlations of the measured hydraulic conductivity value to the selected test method.

3. Install observation wells and staff gauges for short-term and/or long-term monitoring of the groundwater levels adjacent to the ponds, and the pond water levels. The instrumentation for the "continuous" recording of water levels is described in detail later in Section 5 of this report.

The hydrogeologic exploration to characterize the site-specific uppermost aquifer at each pond location included the following:

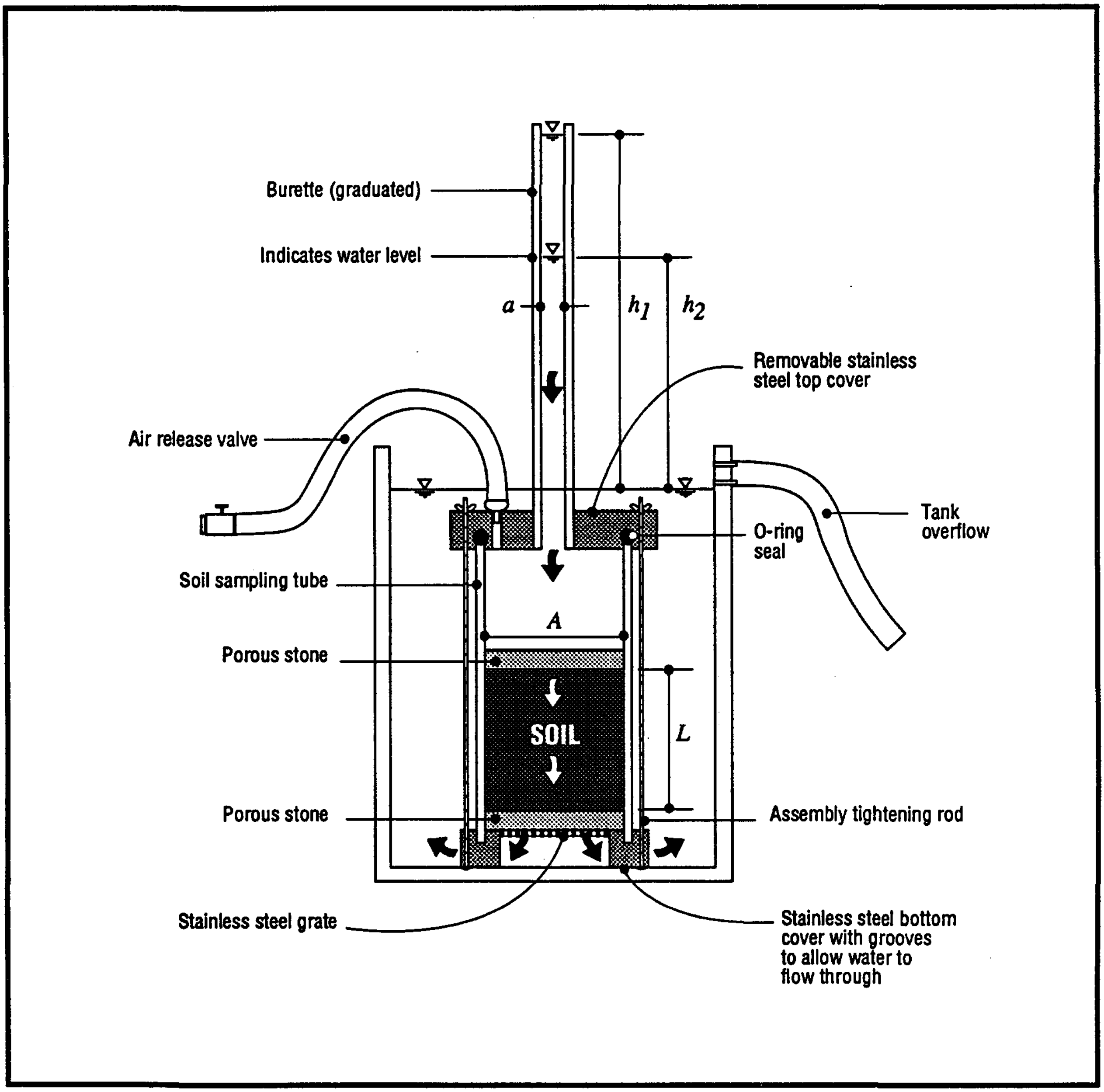
- Standard Penetration Test (SPT) borings (as per ASTM D-1586): in this test, borings are advanced by a rotary drilling technique using a heavy viscous drilling fluid to stabilize the hole and flush out the cuttings. At regular intervals, the drilling tools are removed and soil samples are obtained with a standard 1.4 inch internal diameter, 2 inch outside diameter split-spoon sampler. The sampler is first seated 6 inches to penetrate any loose cuttings and then driven an additional foot. The number of blows required to drive the final foot are recorded and designated as the "standard penetration resistance" or "N" value. The SPT "N" value is an index of the relative density/consistency and soil strength. Representative portions of the disturbed soil sample from the split-spoon are retained in moisture-proof containers for laboratory testing.

- Auger borings (as per ASTM D-1452),
- 2-inch diameter surficial aquifer observation wells , and
- Field and laboratory hydraulic conductivity tests as outlined in Table 1.

Table 1. Field and Laboratory Hydraulic Conductivity Test Methods

Test Method	Orientation	Reference
Laboratory Permeameter (falling head)	k_h (undisturbed sample)	Figure 26
Laboratory Permeameter (falling head)	k_v (undisturbed sample)	Figure 26
Cased hole - soil flush with bottom, falling head	$k_m = \sqrt{(k_h\ k_v)}$	Figure 27
Cased hole - soil flush with bottom, constant head	k_v	Figure 27
Cased hole - with uncased or screened extension, falling head	k_h	Figure 28
Cased hole - with uncased or screened extension, constant head	k_h	Figure 28
Uncased or fully screened auger hole-constant head test	k_h	Figure 29
Uncased or fully screened auger hole-falling head	k_h	Figure 30
Pump test	k_h	Figure 31
Double Ring Infiltrometer Test	Vertical infiltration rate	Figure 32

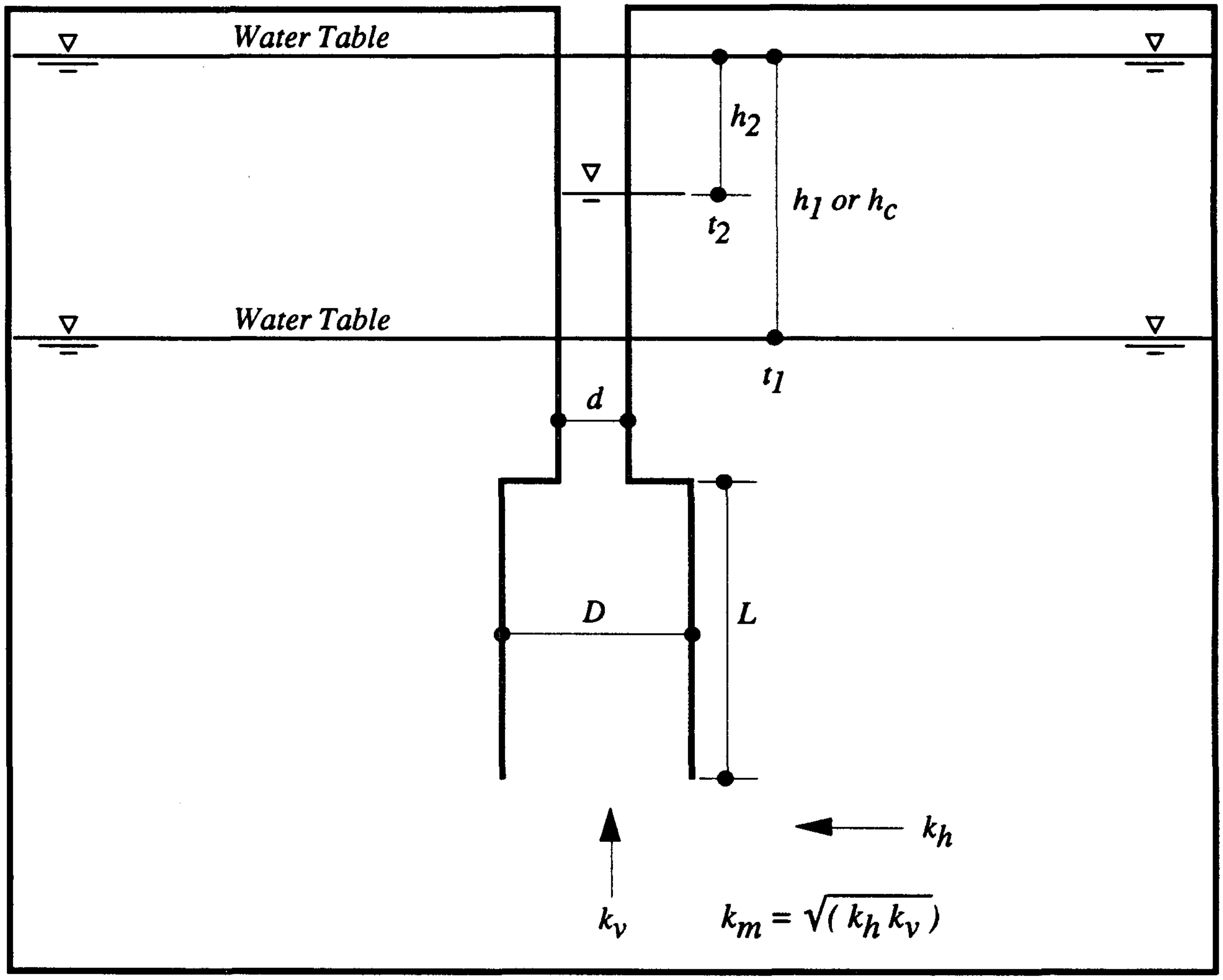
The results of the investigations at each site are documented in the following subsections.



Variable Head $k = \frac{a\,L}{A\,t} \ln \frac{h_1}{h_2}$

Laboratory
PermeameterTest
(PSI/Jammal & Associates Test Equipment)

Figure: 26



Constant Head $k_v = \dfrac{q}{2.75\, D\, h_c}$

Variable Head $k_m = \dfrac{\pi d^2}{11\, D\, (t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ $\quad k_m = \dfrac{\pi D}{11 (t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ *for* $d = D$

NOTE

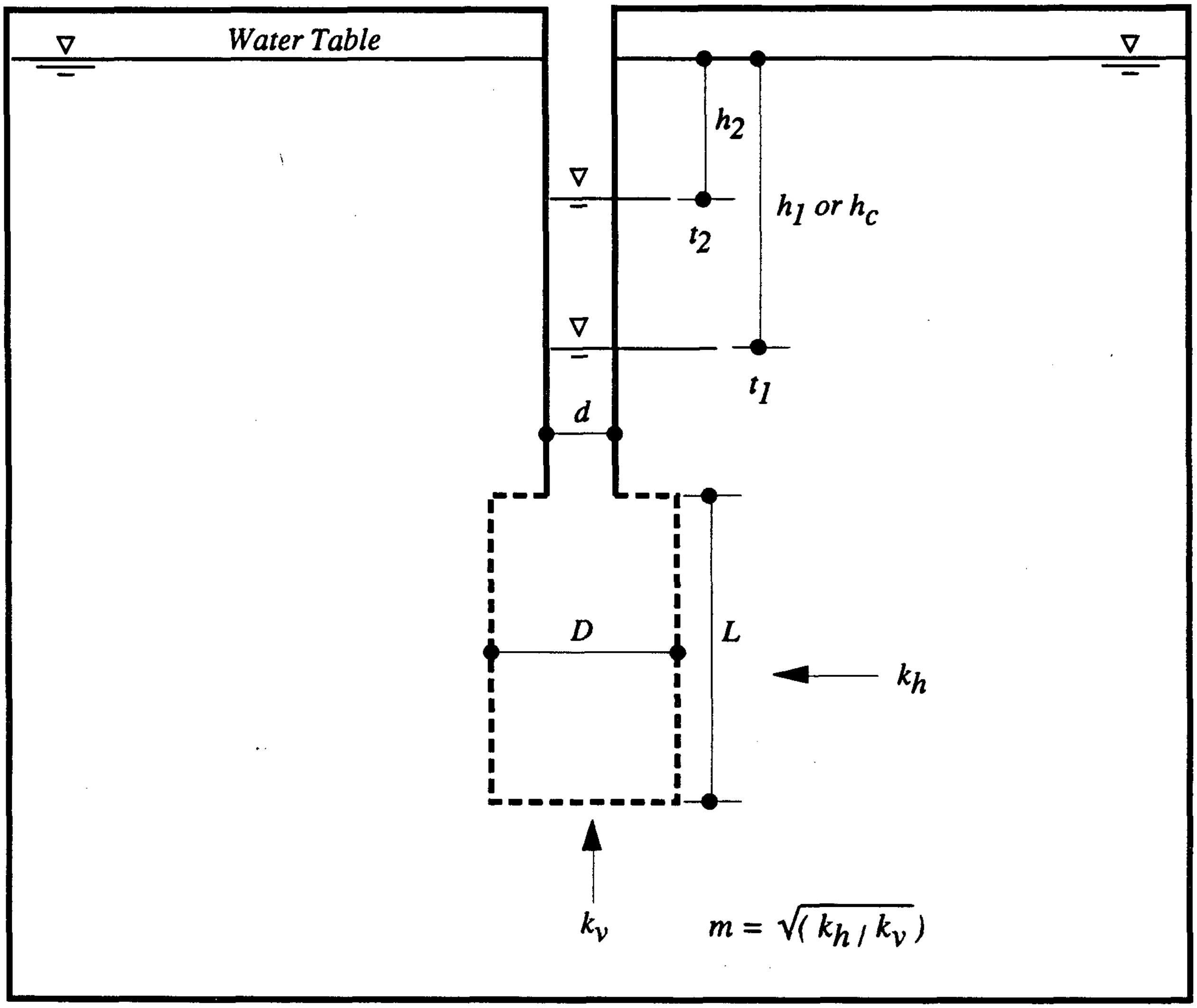
In tests with the bottom of casing above the water table, *h* is the depth of water in hole.

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)

Field Hydraulic Conductivity Test: Cased Hole, Soil Flush with Bottom

Figure: 27



Constant Head $k_h = \dfrac{q \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2\pi L h_c}$

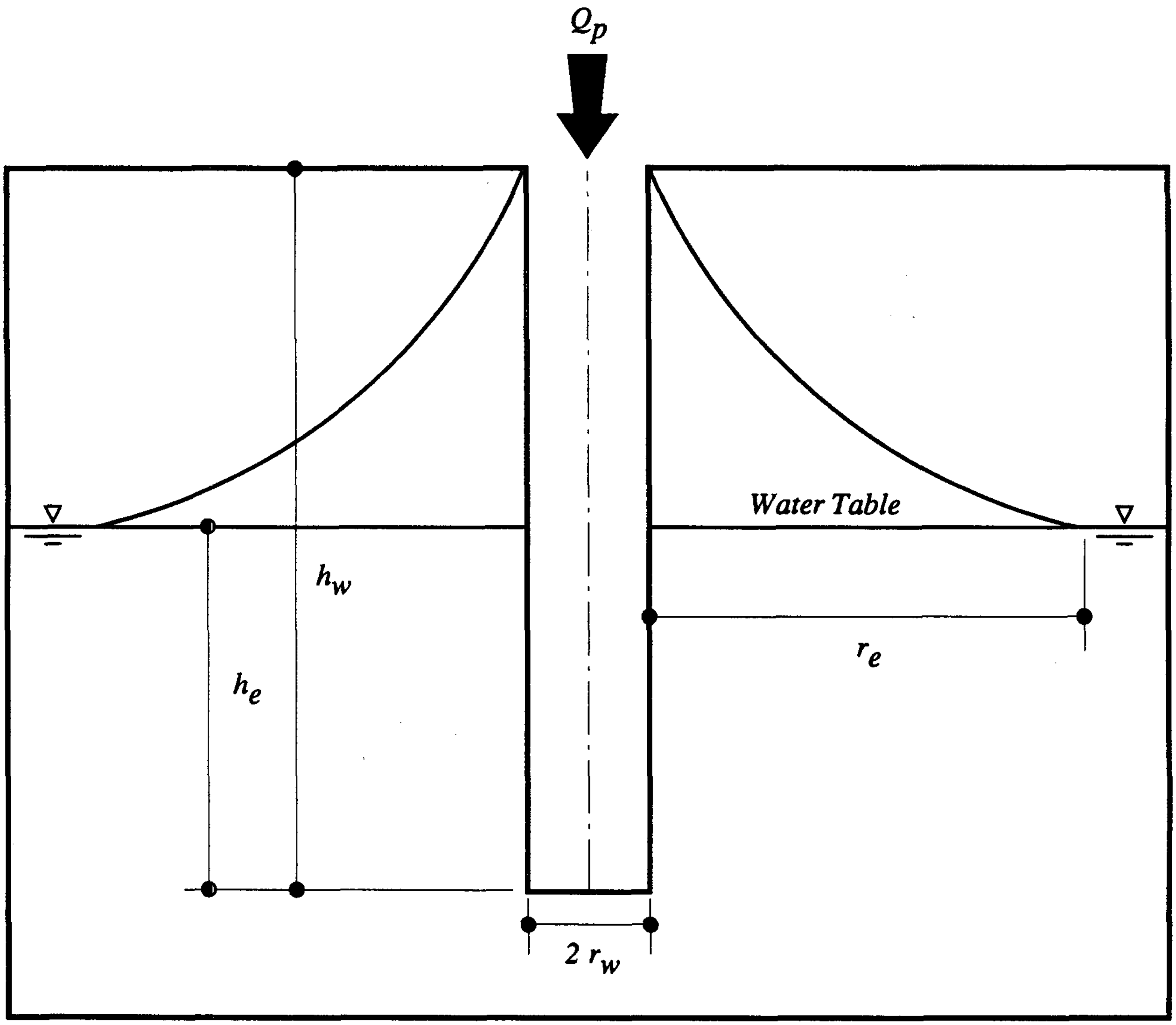
Variable Head $k_h = \dfrac{d^2 \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 L (t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ $\qquad k_h = \dfrac{d^2 \ln \left(\frac{2mL}{D}\right)}{8 L (t_2 - t_1)} \ln \dfrac{h_1}{h_2}$ for $\dfrac{mL}{D} > 4$

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)

Field Hydraulic Conductivity Test: Cased Hole with Uncased or Screened Extension

Figure: 28



NOTE

Use r_e = 20 to 25 ft.

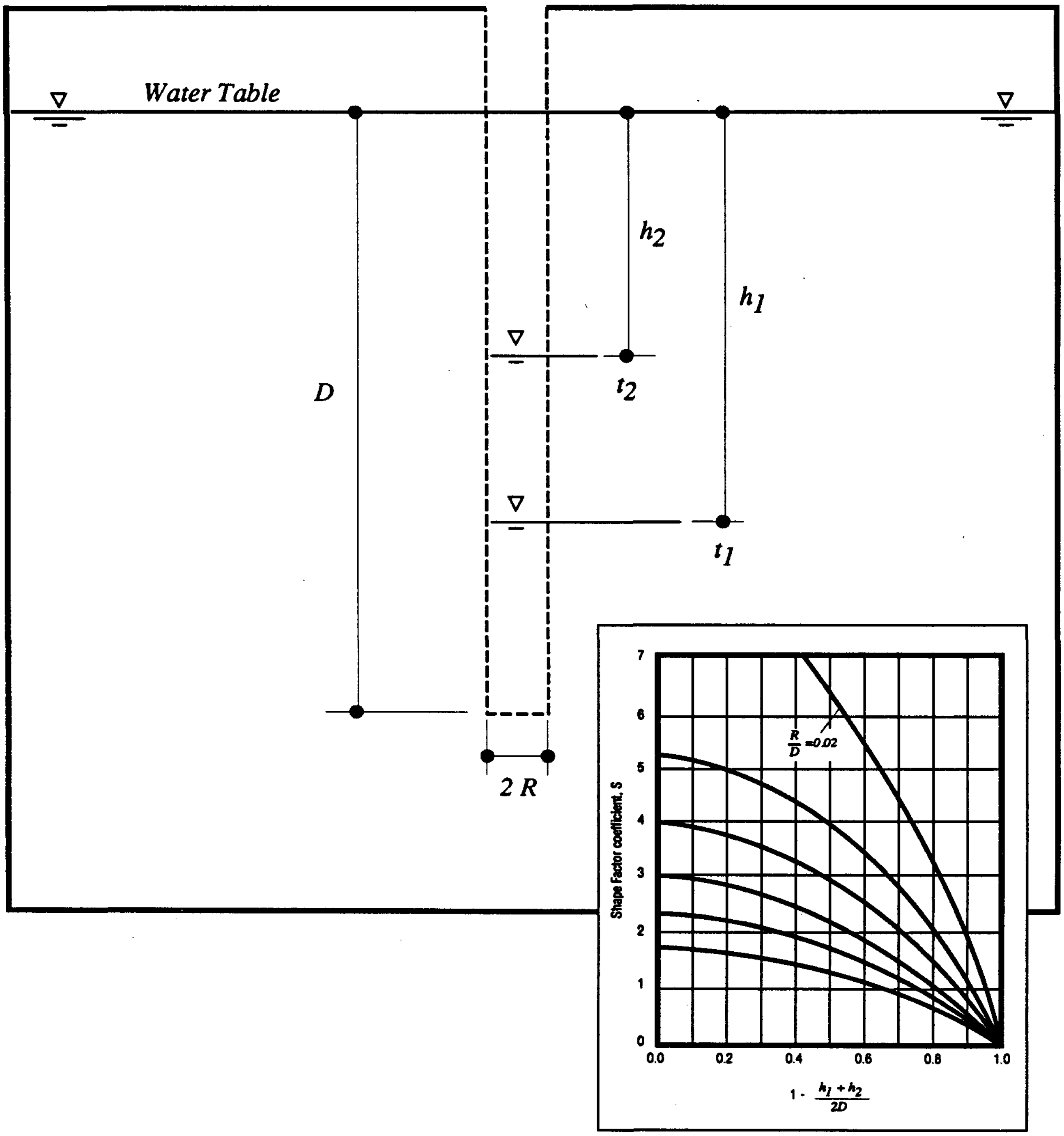
$$K = \frac{Q_p \ln\left(\frac{r_e}{r_w}\right)}{\pi\,(h_w^2 - h_e^2)}$$

Q_p = Steady inflow rate to borehole (cfs)
K = Hydraulic conductivity (ft/sec)
r_e = Radius of influence of borehole (ft)
r_w = Radius of borehole (ft)
h_e = Depth of borehole below water table (ft)
h_w = Total depth of borehole (ft)

Source: South Florida Water Management District, 1987

Field Hydraulic Conductivity Test: Uncased or Fully Screened Auger Hole, Constant Head

Figure: 29

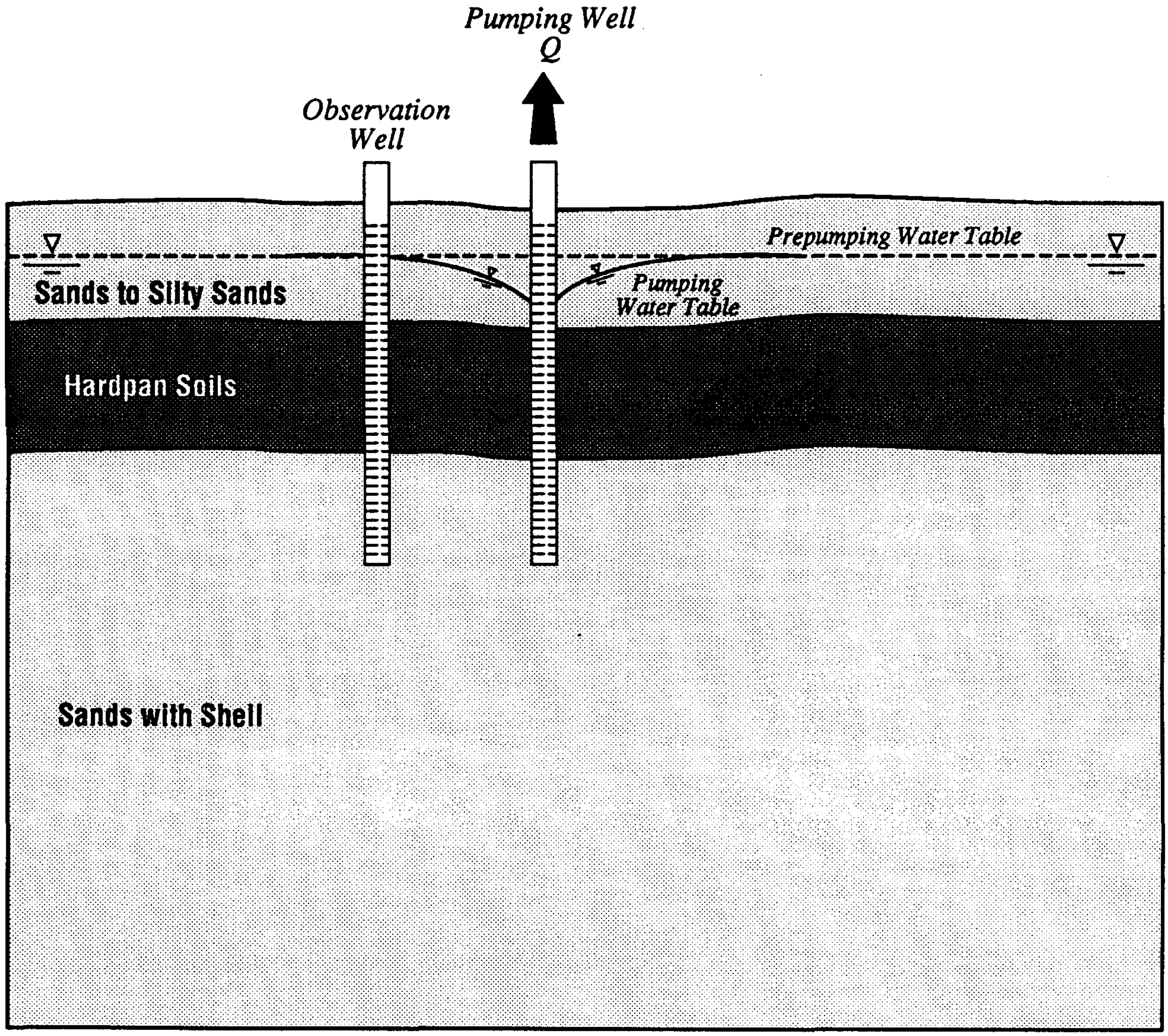


Shape Factor, *F* $F = 16\,\pi DSR$

Permeability, ***k*** **by variable head test** $k_h = \frac{R}{16DS} \times \frac{(h_2 - h_1)}{(t_2 - t_1)}$ *for* $\frac{D}{R} < 50$

Field Hydraulic Conductivity Test: Uncased or Fully Screened Auger Hole, Falling Head

Figure: 30

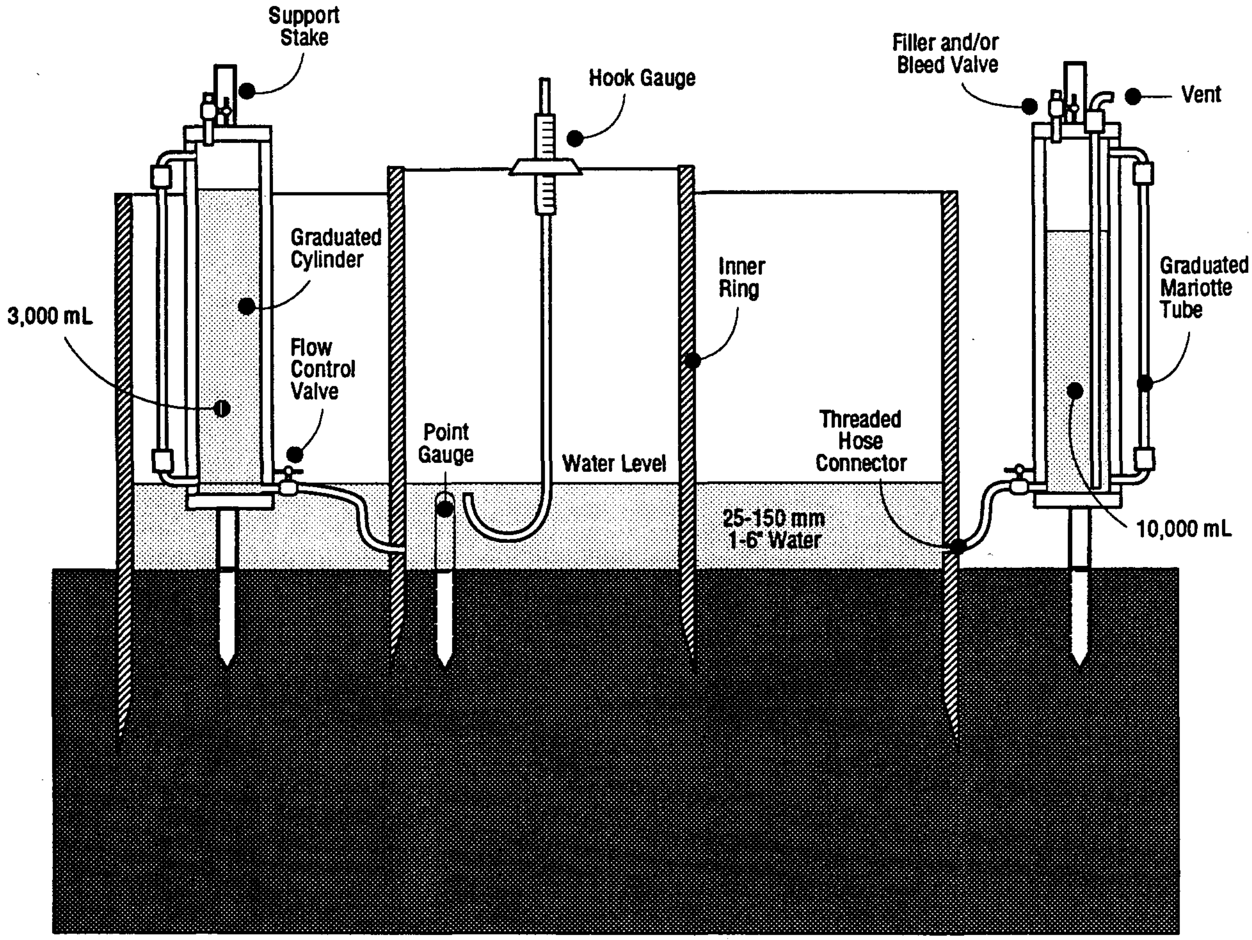


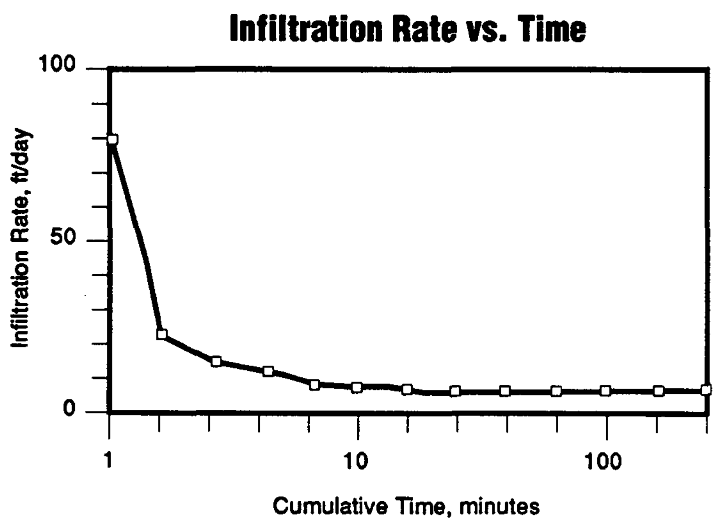
NOTES

1. Pumping well and observation well have similar construction details.
2. Drawdown was only monitored in the observation well using a data logger and probe. Data logger water level readings were checked with a water level indicator at periodic intervals for quality control purposes.
3. Observation well was developed prior to test.
4. A centrifugal pump was used and the pumping rate was set such that a fairly uniform pumping rate was maintained during the test.
5. Flow rate was measured at periodic intervals by the time it took to fill a 30 gallon container.
6. Discharge from the pump was directed at least 100 feet away from the wells.
7. Pump test duration was approximately 8 hours.
8. Pump test data analyzed to obtain transmissivity using Neuman (1975) equation.

Field Hydraulic Conductivity: Pumping Test

Figure: 31





NOTE

Constant-level float valves have been eliminated for simplification of the illustration

Typical Double-Ring Infiltrometer Test

Figure: 32

4.2 AIRPORT WAREHOUSES

4.2.1 Field Investigation & Laboratory Testing - Airport Warehouses

4.2.1.1 Standard Penetration Test and Auger Borings

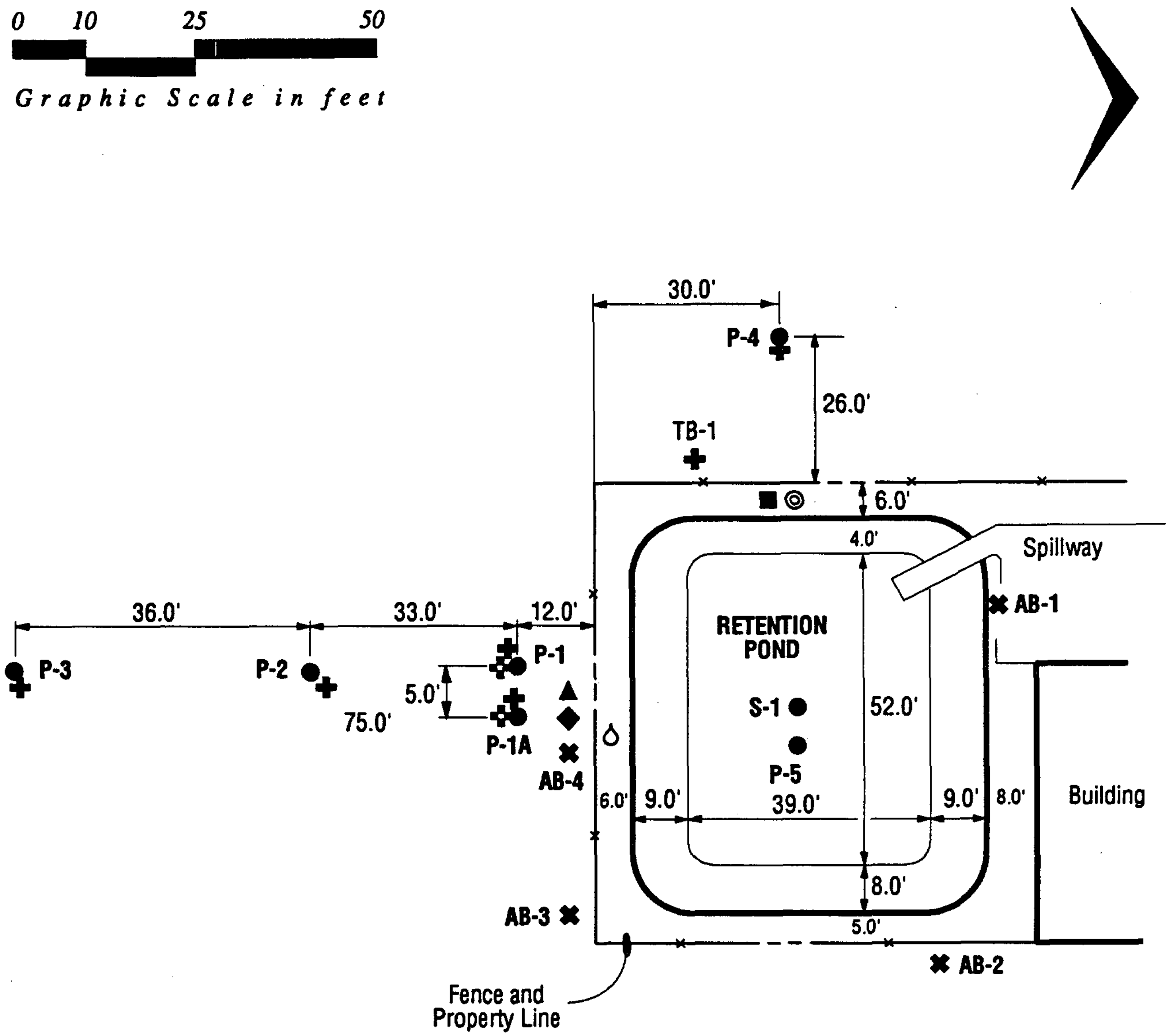
Five (5) SPT borings and four (4) auger borings were performed at the locations outside the limits of the pond shown on Figure 33. On this figure, the SPT borings are identified as P-1 through P-4 and TB-1, while the auger boring locations are labelled as AB-1 through AB-4. Boring depths ranged from 7 to 40 feet below land surface (bls), but were generally in the range 15 to 20 feet bls. The ground surface elevation at each boring location was surveyed.

Standard Penetration Test samples and driving records were obtained continuously in the top 10-12 feet of the soil profile and at 5 foot intervals thereafter. Soil samples were removed from the split-barrel sampler or the power auger and visually classified in the field. Representative portions of soil samples were sealed and packaged for transportation to the laboratory for further examination by a soil scientist/geotechnical engineer. At the time of drilling, the depth to the water table in the boring was also recorded. These test procedures were generic to all the sites.

The results of the test borings and field sampling program are presented in the form of soil boring profiles on Figures 34 and 35. Included on the profiles are the various soil strata, blow count "N" values obtained from the Standard Penetration Testing, the results of grain size analyses to determine the percent by weight finer than the U.S. No. 200 sieve, and the depth to the water table. A complete legend describing the type and in-place density of the soil conditions encountered is also included on Figures 34 and 35.

4.2.1.2 Piezometers

Five piezometers were installed at the locations labelled P-1 through P-5 on Figure 33. Completion details for these piezometers are shown on Figure 36. Piezometer P-1A is an observation well located 5 feet from P-1 for the pump test. All wells were installed using the hollow-stem auger method. The top of casing and ground surface elevations at each piezometer location were surveyed.

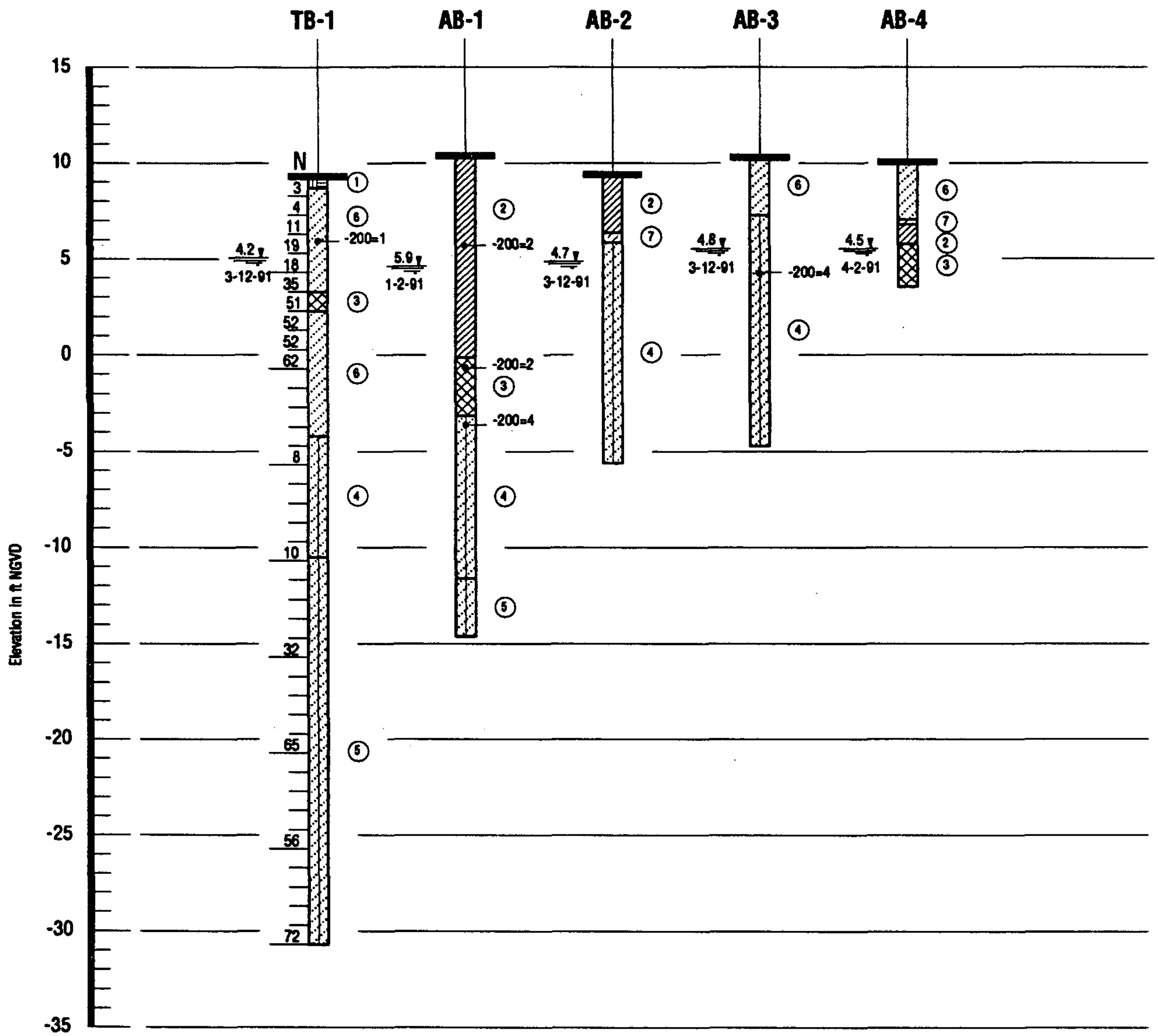


LEGEND

- ✚ Standard Penetration Test Boring Location
- ✖ Auger Boring Location
- ● Piezometer Location
- S-1 ● Staff Gauge Location
- ◎ Double Ring Infiltrometer Test Location
- ✜ Pump Test Location
- ■ Uncased Auger Hole Test Location
- ▲ Cased-hole Hydraulic Conductivity Test Location
- ◆ Undisturbed Sample Location for Laboratory Hydraulic Conductivity Test
- ♁ Rain Gauge Location

Test Locations
Airport Warehouses

Figure: 33



LEGEND

① Grayish-Brown Fine Sand with Roots (Topsoil), (SP), *Very Loose*

② Grayish-Brown to Dark Grayish-Brown Fine Sand with Occasional Rocks, (SP), *Very Loose to Loose*

③ Dark Reddish-Brown Slightly Silty Fine Sand (Hardpan), (SP-SM), *Very Dense*

④ Light Reddish-Brown to Dark Reddish-Brown Fine Sand to Slightly Silty Fine Sand, (SP-SM), *Very Loose to Dense*

⑤ Light Greenish-Gray Fine Sand to Slightly Silty Fine Sand with Occasional Shell, (SP), *Very Loose to Dense*

⑥ Light Gray to Light Brown Fine Sand, (SP), *Very Loose to Very Dense*

⑦ Dark Reddish-Brown Organic Fine Sand, (SP), *Very Loose*

N Standard Penetration Resistance in Blows Per Foot

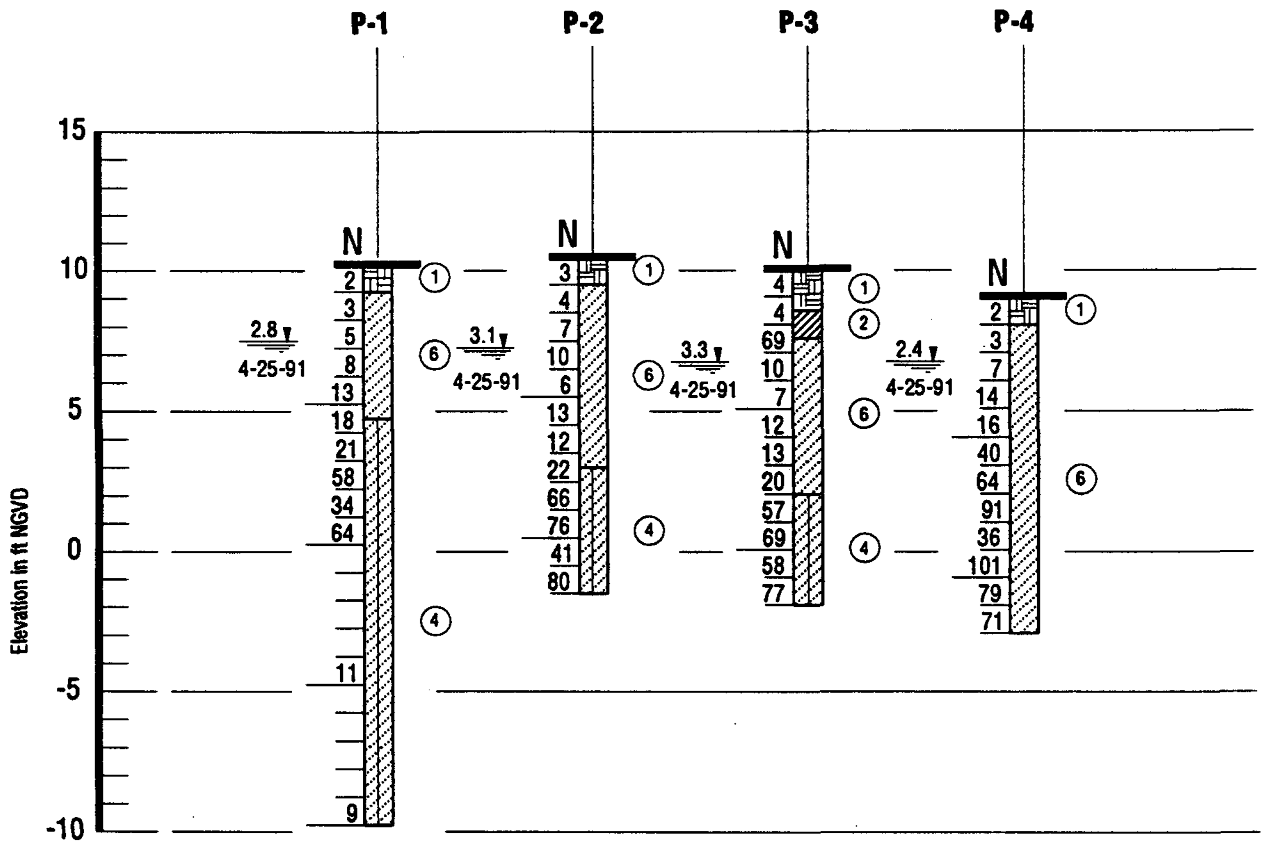
(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

5.9 Depth to Groundwater Level in Feet: Date Noted

-200 Fines Passing No. 200 Sieve (%)

Boring Profiles
Airport Warehouses

Figure: 34



LEGEND

① Grayish-Brown Fine Sand with Roots (Topsoil), (SP), *Very Loose*

② Grayish-Brown to Dark Grayish-Brown Fine Sand with Occasional Rocks, (SP), *Very Loose to Loose*

③ Dark Reddish-Brown Slightly Silty Fine Sand (Hardpan), (SP-SM), *Very Dense*

④ Light Reddish-Brown to Dark Reddish-Brown Fine Sand to Slightly Silty Fine Sand, (SP-SM), *Very Loose to Dense*

⑤ Light Greenish-Gray Fine Sand to Slightly Silty Fine Sand with Occasional Shell, (SP), *Very Loose to Dense*

⑥ Light Gray to Light Brown Fine Sand, (SP), *Very Loose to Very Dense*

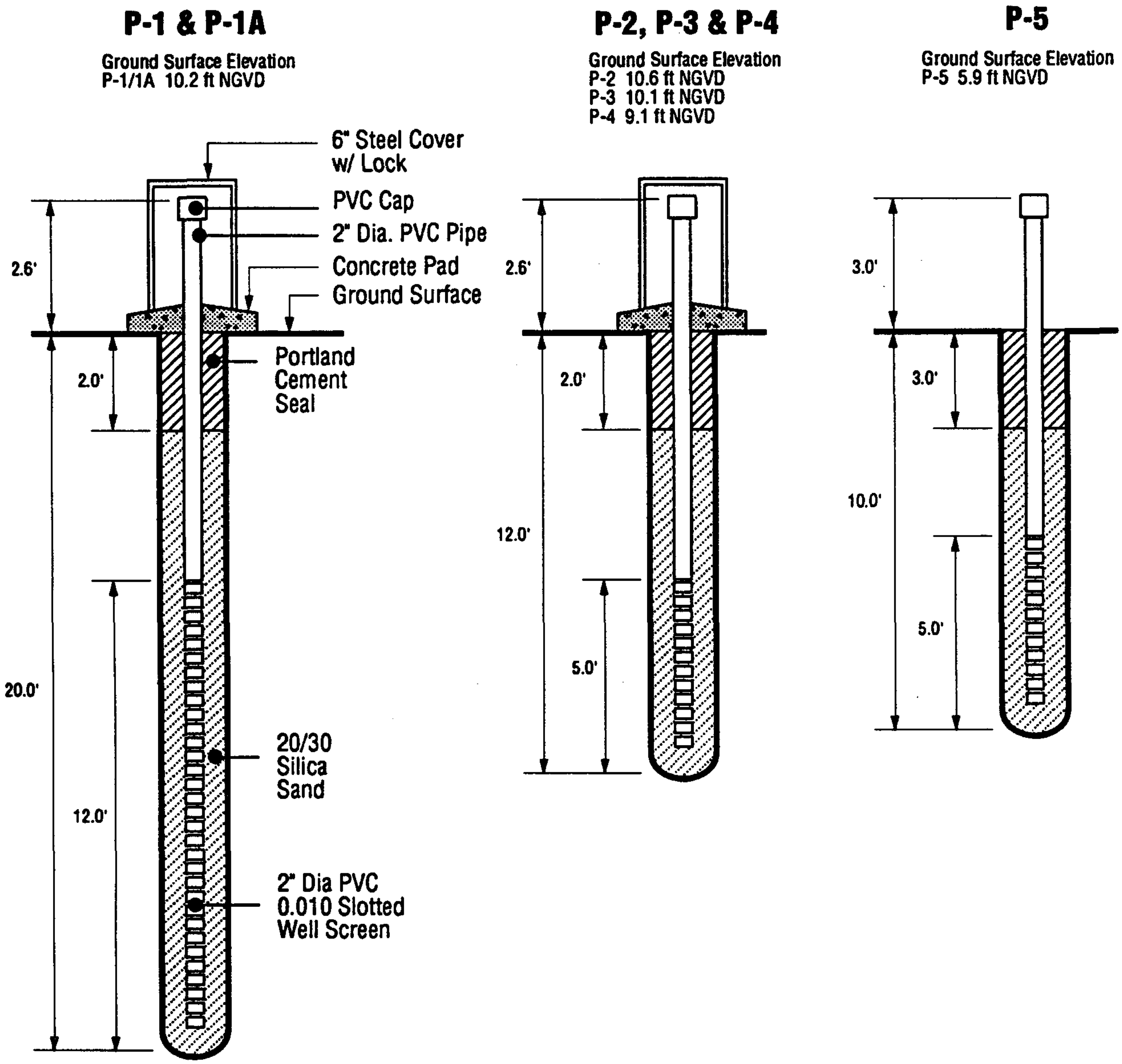
⑦ Dark Reddish-Brown Organic Fine Sand, (SP), *Very Loose*

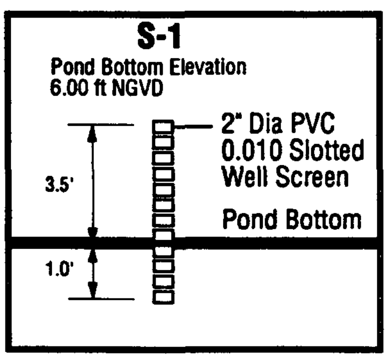
N Standard Penetration Resistance in Blows Per Foot

(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

5.9 Depth to Groundwater Level in Feet: Date Noted

Boring Profiles
Airport Warehouses





Piezometer & Staff Gauge Details
Airport Warehouses

Figure: 36

4.2.1.3 Field and Laboratory Hydraulic Conductivity Tests

Test locations for the series of hydraulic conductivity tests are shown on Figure 33 and results are tabulated on Table 2 with appropriate remarks. With respect to these tests, the following observations should be noted:

- Pump test - As noted on Figure 36, piezometers P-1 and P-1A had identical construction details and were spaced 5 feet apart. The piezometers were screened from just below the groundwater table to a total depth of approximately 20 feet bls. As noted on the soil profile for P-1 (Figure 35), the screened interval included the clean upper mantle of fine sand and the reddish brown slightly silty fine sand (hardpan zone) below a depth of about 5.5 feet. Groundwater was pumped out of one well while the water level in the observation well was monitored at frequent time intervals. The average pumping rate was measured by the time it took to fill a 30 gallon container. Discharge from the pump was directed a minimum of 100 feet away from the wells. The pump test duration was approximately 8 hours.

- For the cased hole (bottom flush) test configuration, the bottom of the casing was seated very close to the hardpan stratum and therefore this boundary condition affected the results.

- Except for one of the falling head laboratory permeameter test, the other tests were performed in the upper fine sand zone above the hardpan.

Table 2: Hydraulic Conductivity Test Results for Airport Warehouses

Test Method	Test Depth Below Ground Surface (ft)	Soil Description	Hydraulic Conductivity Value	
			Constant Head	Falling Head
Cased hole - vertical (Fig. 27)	3.0	light brown fine sand	4 ft/day	4 ft/day
Cased hole - horizontal (Fig. 28)	3.0	light brown fine sand	39 ft/day	14 ft/day
Screened auger hole - horizontal (Fig. 29)	3.0	light brown fine sand	39 ft/day	--
Laboratory (K_h)	2.0	light gray fine sand	--	50 ft/day
Laboratory (K_h)	2.75	light gray fine sand	--	30 ft/day
Laboratory (K_v)	3.0	reddish brown slightly silty fine sand	--	0.3 ft/day
Double Ring Infiltrometer	0.5	light brown fine sand	34 ft/day	--
8 hr Pump Test	2-20	see P-1 on Figure 35	100 ft²/day transmissivity	--

Notes:

1. Test locations shown on Figure 33
2. In the cased borehole (vertical) test, the bottom of the casing was seated close to the hardpan layer thus the low hydraulic conductivity value.
3. The laboratory k_v test was conducted on a sample from the hardpan layer.
4. The value reported for the Double Ring Infiltrometer test is an infiltration rate and not a hydraulic conductivity value
5. The transmissivity value from the pump test is representative of 3 feet of fine sand and 14 feet of hardpan

4.2.2 Hydrostratigraphic Characterization - Airport Warehouses

4.2.2.1 SCS Map Unit

The USDA Soil Conservation Service (SCS) Soil Survey for Brevard County classifies the surface soil (less than 80 inches bls) at the study site as Myakka-Urban land complex. This map unit is composed of nearly level, poorly drained soils formed in beds of marine sands, with a minor dark reddish brown organic component and a slightly cemented layer. The seasonal high groundwater table is estimated by the SCS to be generally between 20 and 40 inches below the ground surface.

4.2.2.2 Soil Stratigraphy

Stratification of the soils encountered is based upon an examination of the recovered samples, and interpretation of the field boring logs by a soil scientist/geotechnical engineer. The stratification lines represent approximate boundaries between soil types of significantly different engineering properties, although the actual transition between layers may be gradual. The following discussion describes the general subsurface conditions revealed by the borings at this site.

The soil stratigraphy at the study site is fairly uniform and can be generalized as follows:

Stratum No	Thickness	Soil Description
1	1 to 1½ ft	Grayish-brown fine sand with roots
6	4 to 6 ft	Grayish-brown to dark grayish-brown fine sand
3 & 4	14 to 15 ft	Reddish-brown fine sand to slightly silty fine sand
5	> 20 ft	Greenish-gray fine sand to slightly silty fine sand with shell fragments

Note that the top of the reddish brown fine sands to slightly silty fine sands (Strata 3 and 4) which are dense and/or cemented ("hardpan" type) are manifested on the SPT boring profiles at the depth where there is a marked increase in "N" value.

4.2.2.3 Groundwater Table.

At the time the soil borings were drilled (March and April, 1991), the groundwater table was encountered at depths ranging from 2.5 to 6 feet below ground surface or at an approximate elevation of +5 to +7 feet NGVD.

4.2.2.4 Hydraulic Conductivity

Test results presented on Table 2 indicate that the horizontal hydraulic conductivity of the Stratum 6 fine sands is in the range of 30 to 50 ft/day. These values are applicable for the test depth of 2 to 3 feet below ground surface and may not be representative of the entire 4 to 6 foot thickness of the Stratum 6 soils.

The vertical hydraulic conductivity of the "hardpan" type soil (Strata 3 and 4) is approximately 0.3 ft/day which is 100 times less permeable than the upper Stratum 6 zone.

As mentioned previously, the screened interval for the pump test well included the clean upper mantle of fine sand and the reddish brown slightly silty fine sand (hardpan zone) below a depth of about 5.5 feet (i.e., approximately 3 feet of fine sand and 14 feet of hardpan).

4.3 TUTOR TIME

<u>4.3.1 Field Investigation & Laboratory Testing - Tutor Time</u>

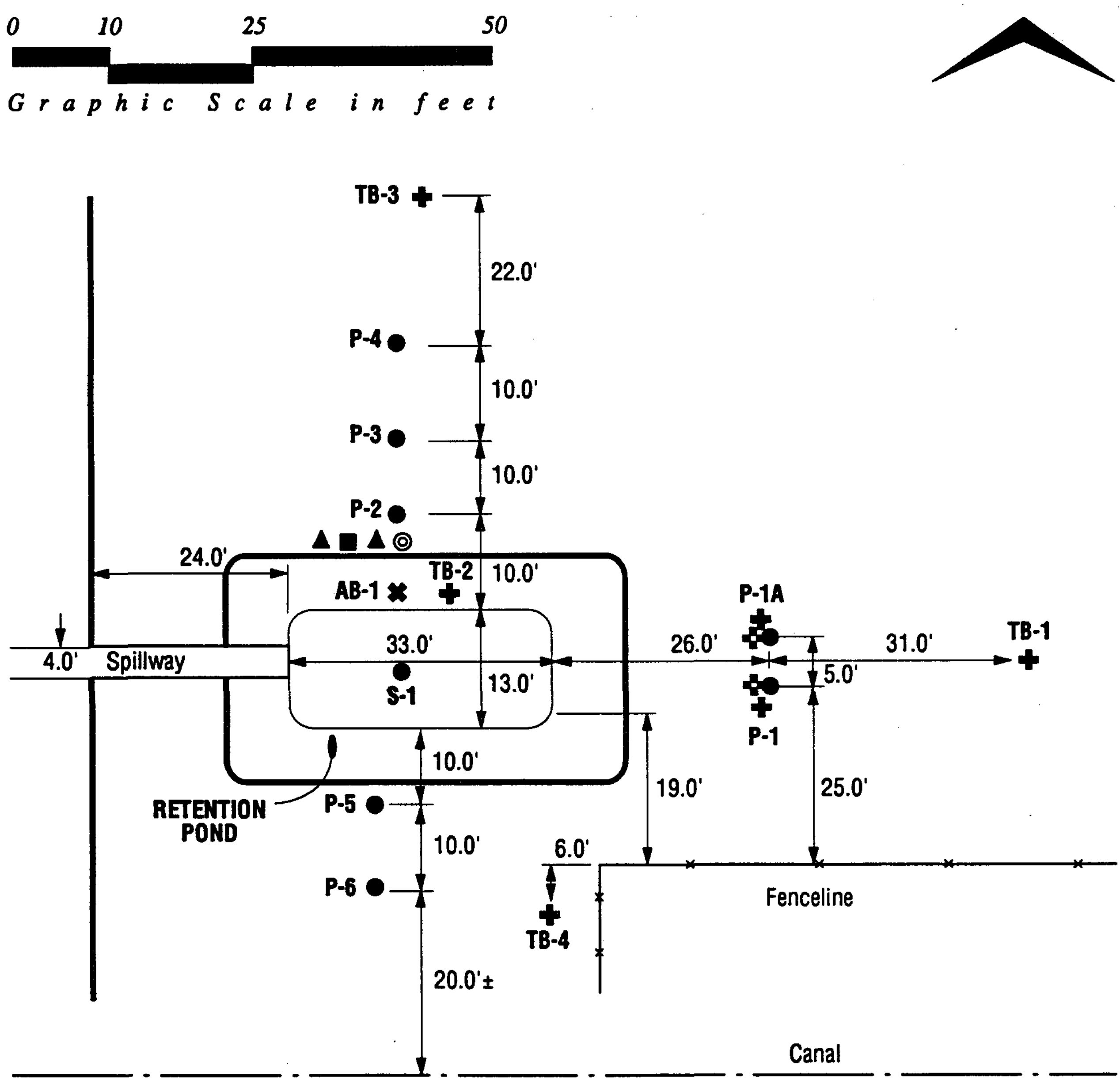
4.3.1.1 Standard Penetration Test and Auger Borings

Five (5) SPT borings and one (1) auger boring were performed at the locations outside the limits of the pond shown on Figure 37. On this figure, the SPT borings are identified as P-1 through P-5, while the auger boring location is labelled as AB-1. Boring depths ranged from 15 to 30 feet below land surface (bls), but were generally in the range 15 to 20 feet bls. The ground surface elevation at each boring location was surveyed.

The results of the test borings and field sampling program are presented in the form of soil boring profiles on Figure 38. Included on the profiles are the various soil strata, blow count "N" values obtained from the Standard Penetration Testing, and the depth to the measured water table. A complete legend describing the type and relative density/consistency of the soil conditions encountered is also included on Figure 38.

4.3.1.2 Piezometers

Five piezometers were installed at the locations labelled P-1 through P-5 on Figure 37. The construction details for these piezometers are shown on Figure 39. Piezometer P-1A is an observation well located 5 feet from P-1 for the pump test. All wells were installed using the hollow-stem auger method. The top of casing and ground surface elevations at each piezometer location were surveyed.

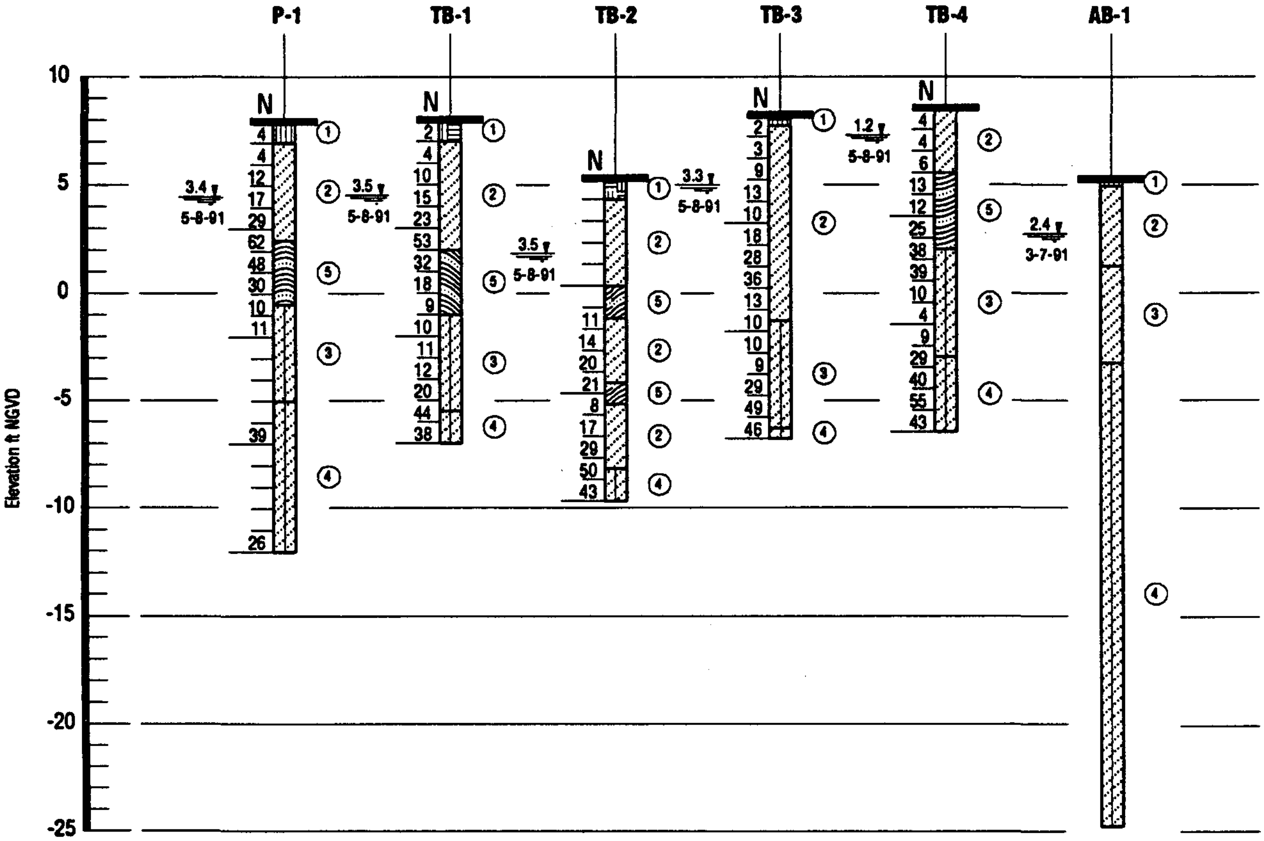


LEGEND

- ✚ Standard Penetration Test Boring Location
- ✖ Auger Boring Location
- ● Piezometer Location
- S-1● Staff Gauge Location
- ◎ Double Ring Infiltrometer Test Location
- ✣ Pump Test Location
- ■ Uncased Auger Hole Test Location
- ▲ Cased-hole Hydraulic Conductivity Test Location
- ◆ Undisturbed Sample Location for Laboratory Hydraulic Conductivity Test

Test Locations
Tutor-Time Child Care Faciltity

Figure: 37



LEGEND

① Grayish-Brown Fine Sand with Roots (Topsoil), (SP), *Very Loose*

② Light Brown to Gray Fine Sand, (SP), *Very Loose to Dense*

③ Dark Reddish-Brown Fine Sand to Slightly Silty Fine Sand, (SP)(SP-SM), *Loose to Dense*

④ Light Greenish-Gray Fine Sand to Slightly Silty Fine Sand with Shell Fragments, (SP)(SP-SM), *Medium Dense to Dense*

⑤ Dark Reddish-Brown Slightly Organic to Organic Silty Fine Sand, (SM), Stif to Very Stiff

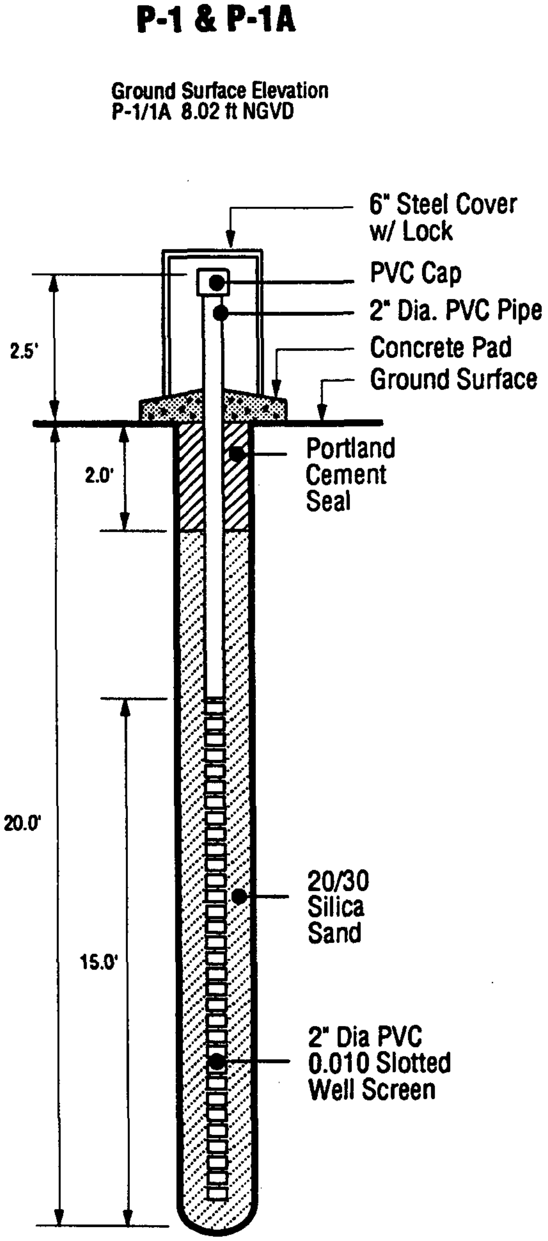
(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

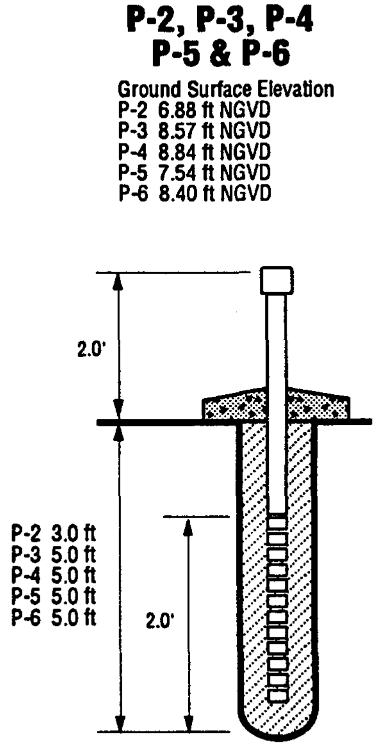
3.4 Depth to Groundwater Levels in Feet: Date Noted

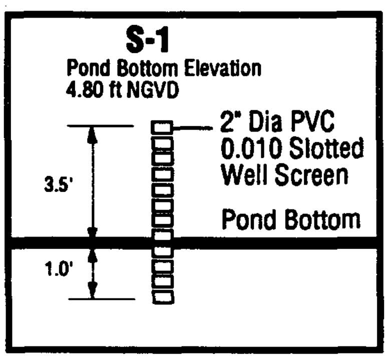
N Standard Penetration Resistance in Blows Per Foot

Boring Profiles
Tutor-Time Child Care Facility

Figure: 38







Piezometer & Staff Gauge Details
Tutor-Time Child Care Facility

Figure: 39

4.3.1.3 Field and Laboratory Hydraulic Conductivity Tests

Test locations for the series of hydraulic conductivity tests are shown on Figure 37 and results are tabulated on Table 3 with remarks where appropriate. With respect to these tests, the following observations are made:

- Pump test - As noted on Figure 39, piezometers P-1 and P-1A had identical construction details and were spaced 5 feet apart. The piezometers were screened from just below the groundwater table to a total depth of approximately 20 feet bls. As noted on the soil profile for P-1 (Figure 38), the soil profile in the screened interval can be generalized as follows:

Depth Below Grade	Soil Type
0 to 5.5 ft	Light brown to gray fine sand
5.5 to 13 ft	Dark reddish-brown slightly silty fine sand
13 to 20+ ft	Greenish-gray fine sand & slightly silty fine sand

  The intermediate dark reddish brown slightly silty fine sand zone is hydraulically more restrictive than the upper mantle of fine sand and the lower zone. The typical pump test details for this study are described on Figure 31.

- For the cased hole (bottom flush) test configuration, the bottom of the casing was seated very close to the hardpan stratum and therefore this boundary condition influenced the results.

- Except for one of the falling head laboratory permeameter test, the other tests were performed in the upper fine sand zone above the hardpan.

Table 3: Hydraulic Conductivity Test Results for Tutor Time

Test Method	Test Depth Below Ground Surface (ft)	Soil Description	Hydraulic Conductivity Value	
			Constant Head	Falling Head
Cased hole - vertical (Fig. 27)	2.5	gray fine sand	1.4 ft/day	1.0 ft/day
Cased hole - horizontal (Fig. 28)	3.0	gray fine sand	18 ft/day	34 ft/day
Uncased auger hole (Fig. 29)	3.0	gray fine sand	32 ft/day	--
Laboratory (K_h)	2.0	gray fine sand	--	36 ft/day
Laboratory (K_h)	2.0	gray fine sand	--	32 ft/day
Laboratory (K_v)	3.0	gray fine sand	--	33 ft/day
Double Ring Infiltrometer	0.5	gray fine sand	19.5 ft/day	--
8 hr Pump Test	2-20	see soil profile for P-1 on Figure 38	1800 ft^2/day transmissivity	--

Notes:
1. Test locations shown on Figure 37
2. In the cased borehole (vertical) test, the bottom of the casing was seated close to the hardpan layer thus the low hydraulic conductivity value.
3. The value reported for the Double Ring Infiltrometer test is an infiltration rate and not a hydraulic conductivity value
4. The transmissivity value from the pump test is representative of the soil profile for P-1 on Figure 38

4.3.2 Hydrostratigraphic Characterization - Tutor Time

4.3.2.1 SCS Map Unit

The USDA Soil Conservation Service (SCS) Soil Survey for Brevard County classifies the shallow soil (less than 80 inches) at the study site as Immokalee sand and St. John's soils, ponded, composed of nearly level, poorly drained sandy soils with a minor organic component and a weakly cemented layering in the St. John's soil. The seasonal high groundwater table is estimated by the SCS to be generally between 10 and 40 inches below the ground surface and infrequently to occasionally flooded after heavy rains.

4.3.2.2 Soil Stratigraphy

As disclosed by the borings, the soil stratigraphy at the study site is fairly uniform and can be generalized as follows:

Stratum No	Thickness	Soil Description
1	< 1 ft	Grayish-brown fine sand with roots
2	3 to 5.5 ft	Light brown to gray fine sand
3 & 5	7 to 9.5 ft	Dark reddish-brown fine sand to slightly silty fine sand
4	> 27 ft	Greenish-gray fine sand to slightly silty fine sand with shell fragments

The Strata 3 and 5 soils are less permeable than the suprajacent Stratum 2 fine sands as well as the subjacent greenish-gray fine sand with shell fragments. The top of this hydraulically restrictive unit (of Strata 3 and 5 soils) can be identified on the SPT boring profiles at the depth interval where there is a marked increase in "N" value. From the borings, the elevation of the top of this unit is in the range of +1 to +3 ft NGVD.

4.3.2.3 Groundwater Table

At the time of the drilling operations (May 1991), the groundwater table was encountered at a depth of 1.2 to 3.5 feet below ground surface. The groundwater levels were influenced by the stormwater surcharge in the pond.

4.3.2.4 Hydraulic Conductivity

Field and laboratory test results presented on Table 3 indicate that the horizontal hydraulic conductivity of the Stratum 2 fine sands is in the range of 32 to 36 ft/day. On the other hand, the vertical hydraulic conductivity of the "hardpan" type reddish brown fine sands (Strata 3 and 5) is less than 1 ft/day. From the pump test data, it also appears that the underlying Stratum 4 layer of fine sand to slightly silty fine sand with shell fragments is a very transmissive zone (transmissivity = 1800 ft^2/day).

4.4 FISHERMAN'S LANDING

4.4.1 Field Investigation & Laboratory Testing - Fisherman's Landing

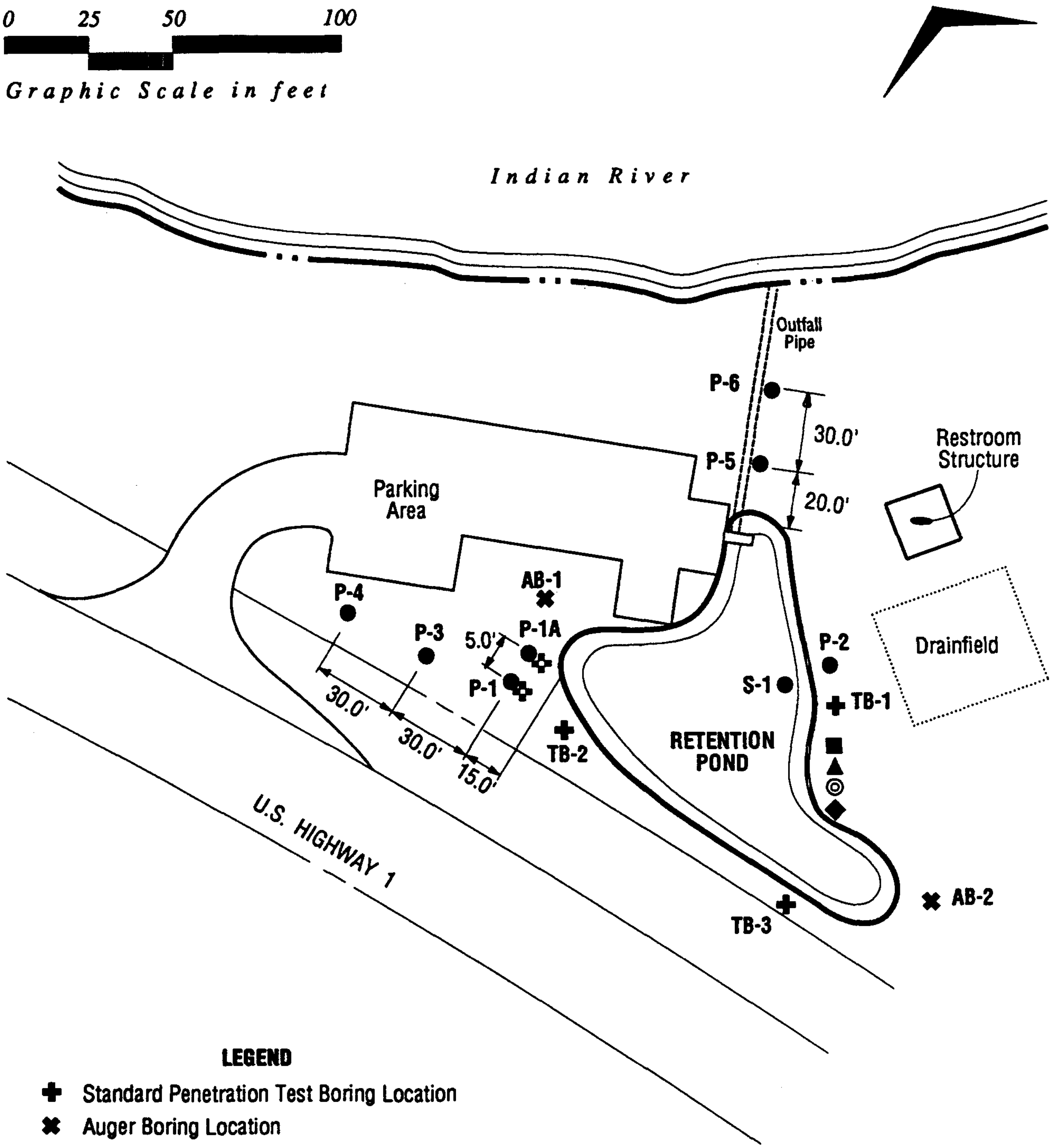
4.4.1.1 Standard Penetration Test and Auger Borings

Three (3) SPT borings and nine (9) auger borings were performed at the locations within and adjacent to the limits of the pond shown on Figure 40. On this figure, the SPT borings are identified as TB-1 through TB-3, while the auger boring locations are labelled as P-1 through P-7, AB-1, and AB-2. Boring depths ranged from 7 to 40 feet below land surface (bls), but were generally in the range 15 to 20 feet bls. The ground surface elevation at each boring location was surveyed.

The results of the test borings and field sampling program are presented in the form of soil boring profiles on Figures 41 and 42. Included on the profiles are the various soil strata, blow count "N" values obtained from the Standard Penetration Testing, results of grain size analyses to determine the percent by weight finer than the U.S. No. 200 sieve, and the depth to the measured water table. A complete legend describing the type and relative density/consistency of the soil conditions encountered is also included on Figures 41 and 42.

4.4.1.2 Piezometers

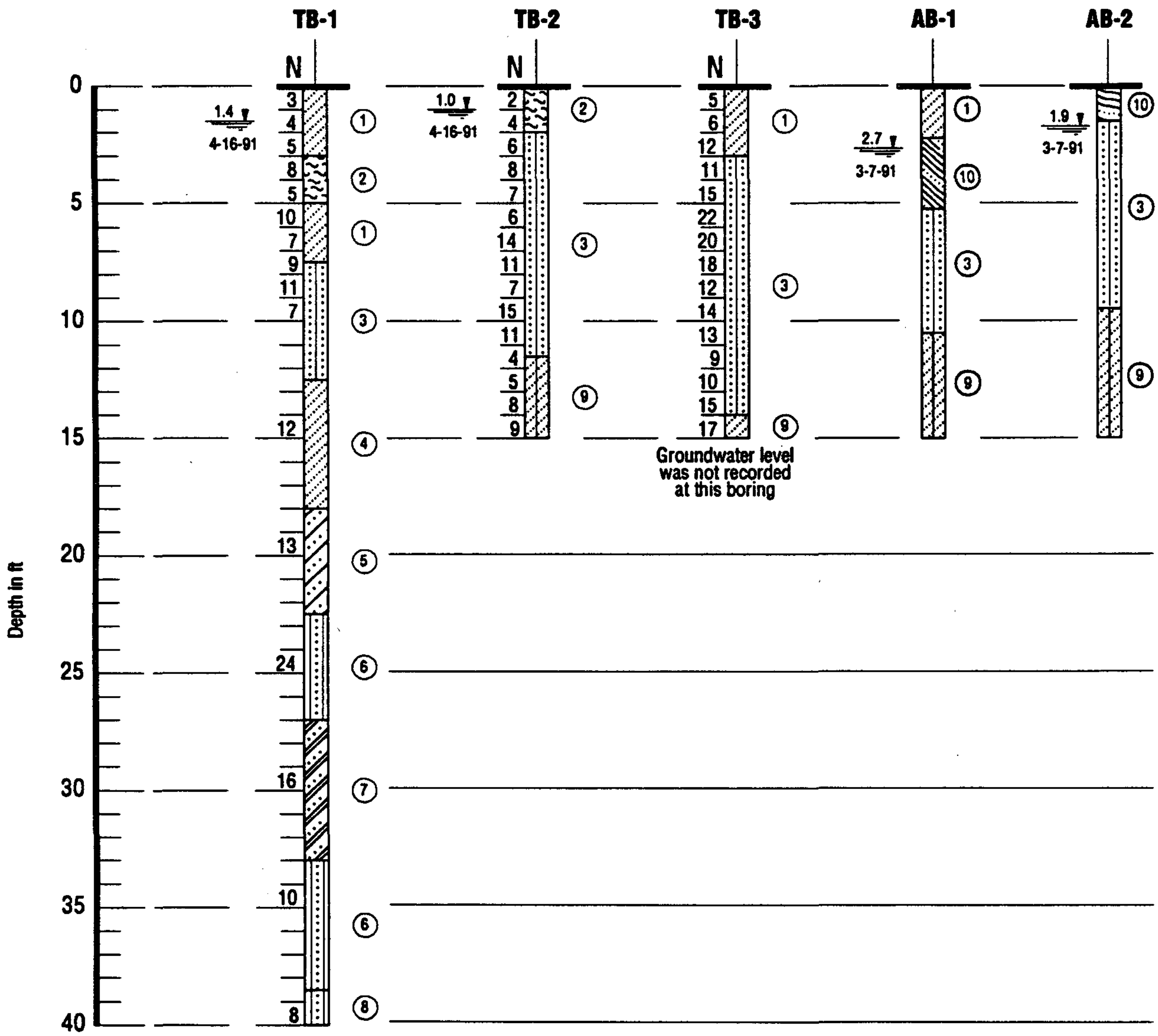
Five piezometers were installed at the locations labelled P-1 through P-5 on Figure 40. Construction details for these piezometers are shown on Figure 43. Piezometer P-1A is an observation well located 5 feet from P-1 for the pump test. All wells were installed using the hollow-stem auger method. The top of casing and ground surface elevations at each piezometer location were surveyed.



LEGEND

- ✚ Standard Penetration Test Boring Location
- ✖ Auger Boring Location
- ● Piezometer Location
- S-1● Staff Gauge Location
- ◎ Double Ring Infiltrometer Test Location
- ✜ Pump Test Location
- ■ Uncased Auger Hole Test Location
- ▲ Cased-hole Hydraulic Conductivity Test Location
- ◆ Undisturbed Sample Location for Laboratory Hydraulic Conductivity Test

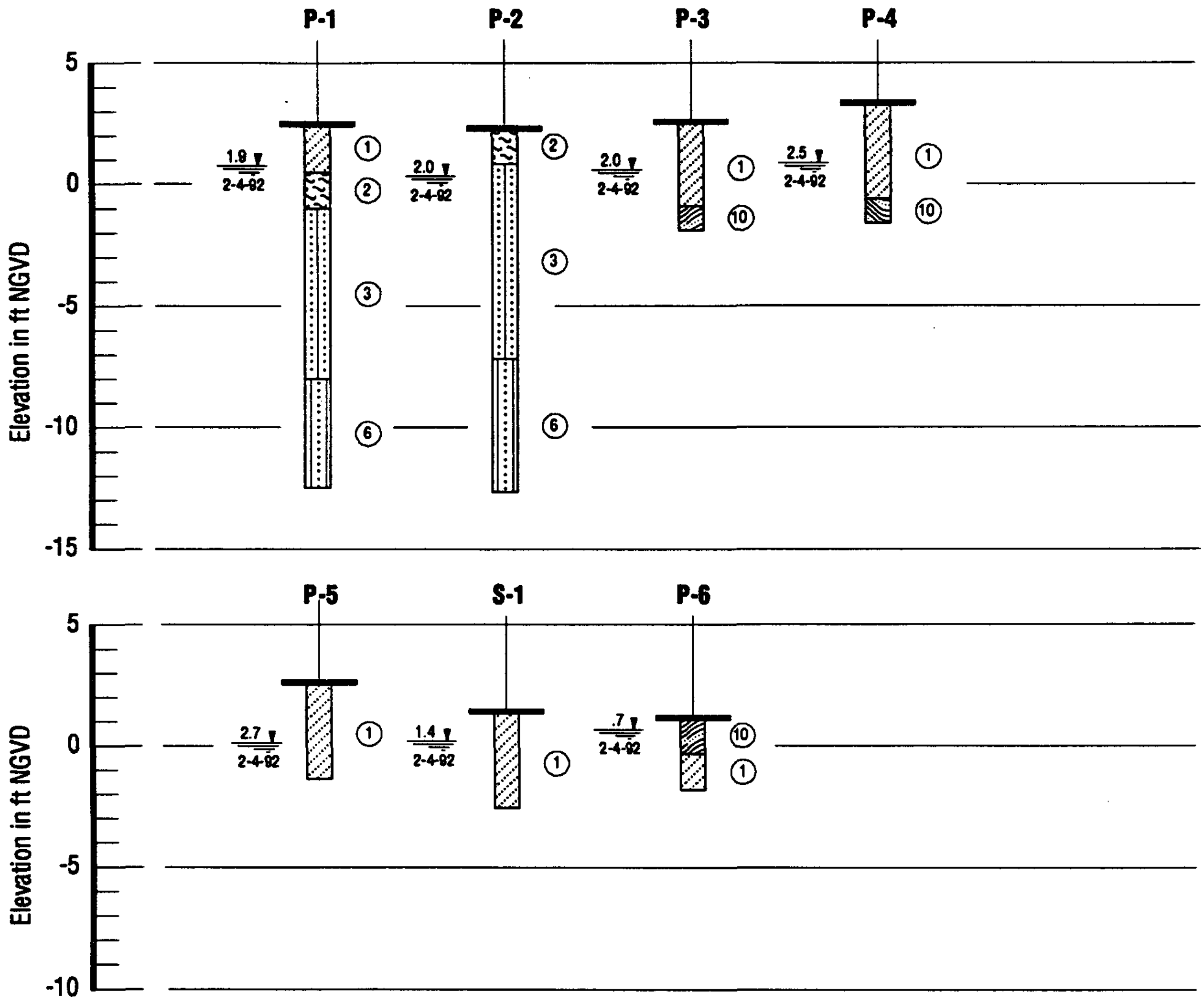
Test Locations
Fisherman's Landing



LEGEND

- ① Grayish-Brown to Dark Gray Fine Sand, (SP), *Very Loose to Loose*
- ② Dark Reddish-Brown Sandy Peat to Peat, (PT), *Soft to Medium Stiff*
- ③ Reddish-Brown to Dark Reddish-Brown Slightly Silty Fine Sand, (SP-SM), *Loose to Medium Dense*
- ④ Gray Fine Sand with Shell Fragments, (SP), *Medium Dense*
- ⑤ Greenish-Gray Clayey Fine Sand, (SC), *Medium Dense*
- ⑥ Greenish-Gray Silty Fine Sand with Parts of Greenish-Gray Clayey Fine Sand with Occasional Shell Fragments, (SM-SC), *Medium Dense*
- ⑦ Greenish-Gray Sandy Clay with Occasional Shell Fragments, (CL), *Very Stiff*
- ⑧ Greenish-Gray Silty Fine Sand with Shell Fragments, (SM), *Loose*
- ⑨ Light Grayish-Brown to Grayish-Brown Fine Sand to Slightly Silty Fine Sand Occasionally with Shell Fragments and Limestones, (SP)(SP-SM), *Very Loose to Medium Dense*
- ⑩ Dark Reddish-Brown Organic Medium Sand, (SP), *Very Loose to Loose*
- (SP) Unified Soil Classification Group Symbol as Determined by Visual Examination
- 5.9 Depth to Groundwater Level in Feet: Date Noted
- N Standard Penetration Resistance in Blows Per Foot

Boring Profiles
Fisherman's Landing



LEGEND

① Grayish-Brown to Dark Gray Fine Sand, (SP), *Very Loose to Loose*

② Dark Reddish-Brown Sandy Peat to Peat, (PT), *Soft to Medium Stiff*

③ Reddish-Brown to Dark Reddish-Brown Slightly Silty Fine Sand, (SP-SM), *Loose to Medium Dense*

④ Gray Fine Sand with Shell Fragments, (SP), *Medium Dense*

⑤ Greenish-Gray Clayey Fine Sand, (SC), *Medium Dense*

⑥ Greenish-Gray Silty Fine Sand with Parts of Greenish-Gray Clayey Fine Sand with Occasional Shell Fragments, (SM-SC), *Medium Dense*

⑦ Greenish-Gray Sandy Clay with Occasional Shell Fragments, (CL), *Very Stiff*

⑧ Greenish-Gray Silty Fine Sand with Shell Fragments, (SM), *Loose*

⑨ Light Grayish-Brown to Grayish-Brown Fine Sand to Slightly Silty Fine Sand Occasionally with Shell Fragments and Limestones, (SP)(SP-SM), *Very Loose to Medium Dense*

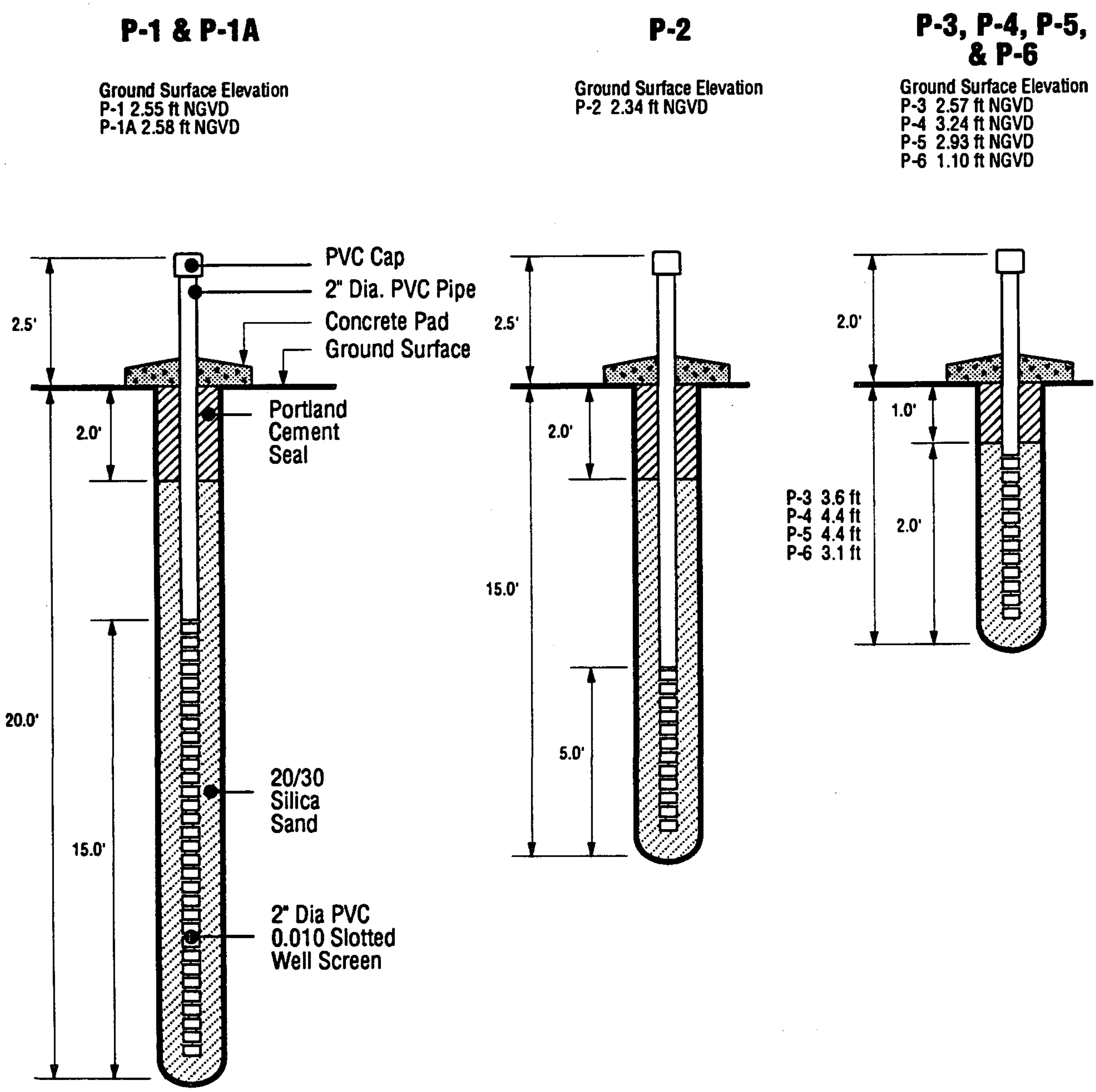
⑩ Dark Reddish-Brown Organic Medium Sand, (SP), *Very Loose to Loose*

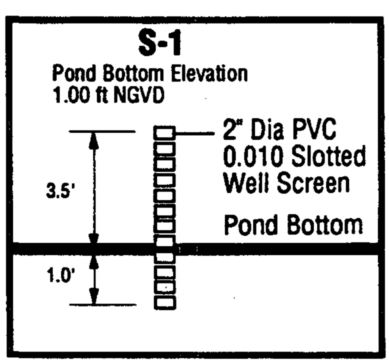
(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

5.9 Depth to Groundwater Level in Feet: Date Noted

Boring Profiles
Fisherman's Landing

Figure: 42





Piezometer & Staff Gauge Details
Fisherman's Landing

Figure: 43

4.4.1.3 Field and Laboratory Hydraulic Conductivity Tests

Test locations for the series of hydraulic conductivity tests are shown on Figure 40 and results are tabulated on Table 4 with appropriate remarks. With respect to these tests, the following points should be noted:

- Pump test - As noted on Figure 43, piezometers P-1 and P-1A had identical construction details and were spaced 5 feet apart. The piezometers were screened from just below the groundwater table to a total depth of approximately 20 feet bls. As noted on the soil profile for P-1 (Figure 42), the screened interval included the clean upper mantle of fine sand and the reddish brown slightly silty fine sand (hardpan zone) below a depth of about 5.5 feet. Groundwater was pumped out of one well while the water level in the observation well was monitored at frequent time intervals. The average pumping rate was measured by the time it took to fill a 30 gallon container. The pump test duration was a minimum of 8 hours.

- For the cased hole (bottom flush) test configuration, the bottom of the casing was seated very close to the hardpan stratum and therefore this boundary condition affected the results.

- Except for one of the falling head laboratory permeameter test, the other tests were performed in the upper fine sand zone above the hardpan.

Table 4: Hydraulic Conductivity Test Results for Fisherman's Landing

Test Method	Test Depth Below Ground Surface (ft)	Soil Description	Hydraulic Conductivity Value	
			Constant Head	Falling Head
Cased hole - vertical (Fig. 27)	4.0	dark reddish brown silty fine sand	2	2
Cased hole - horizontal (Fig. 28)	4.0	dark reddish brown silty fine sand	5	11
Uncased auger hole (Fig. 29)	2.0	dark gray fine sand	7	--
Laboratory (K_h)	1.5	grayish-brown slightly silty fine sand	--	8
Laboratory (K_v)	2.0	dark gray silty fine sand	--	0.7
Double Ring Infiltrometer	0.5	dark gray fine sand	5.2	--
8 hr Pump Test	2-20	see soil profile for P-1 on Figure 42	400 ft^2/day transmissivity	--

Notes:

1. Test locations shown on Figure 40
2. In the cased borehole (vertical) test, the bottom of the casing was seated close to the hardpan layer thus the low hydraulic conductivity value.
3. The laboratory k_v test was conducted on a sample from the hardpan layer.
4. The value reported for the Double Ring Infiltrometer test is an infiltration rate and not a hydraulic conductivity value
5. The transmissivity value from the pump test is representative of the soil profile for P-1 on Figure 42

4.4.2 Hydrostratigraphic Characterization - Fisherman's Landing

4.4.2.1 SCS Map Unit

The USDA Soil Conservation Service (SCS) Soil Survey for Brevard County classifies the shallow soil at the study site as Anclote sand composed of nearly level, very poorly drained sandy soil in marshy depressions or flood plains consisting of fine sand, loamy sand or loamy fine sand in all horizons. The seasonal high groundwater table is estimated by the SCS to be generally between 10 and 40 inches below the ground surface, but is occasionally subject to inundation after heavy rains.

4.4.2.2 Soil Stratigraphy

Stratification of the soils encountered is based upon an examination of the recovered samples, and interpretation of the field boring logs by a soil scientist/geotechnical engineer. The stratification lines represent approximate boundaries between soil types of significantly different engineering properties, although the actual transition between layers may be gradual. The following discussion describes the general subsurface conditions revealed by our borings at this site.

As disclosed by the borings, the soil stratigraphy at the study site is not very uniform. In general, it comprises a surficial layer of grayish brown to dark gray fine sand (Stratum 1), 5 to 7 feet thick within which are layers of sandy peat to peat (Stratum 2) about 2 feet thick. Underlying this upper zone are the typical reddish brown "hardpan" type slightly silty fine sand (Stratum 3) extending to depths up to 14 feet below land surface. Underlying this less permeable intermediate zone are gray fine sands with shell fragments (Strata 4 and 9) with interbeds of clayey fine sand and clay (Strata 5 and 7) to the maximum boring termination depth of 40 feet bls.

4.4.2.3 Groundwater Table

At the time the soil borings were drilled in February 1992, the groundwater table was measured at depths ranging from 1.4 to 2.7 feet bls at the boring locations outside the pond.

4.4.2.4 Hydraulic Conductivity

Test results presented on Table 5 indicate that the horizontal hydraulic conductivity of the Stratum 1 fine sands is in the range 7 to 11 ft/day.

The vertical hydraulic conductivity of the "hardpan" type soil (Stratum 3) is less than 1 ft/day. The results of the pump test which is screened into the underlying Stratum 4 fine sands with shell indicate a relatively high transmissivity value of 400 ft^2/day.

4.5 TOM STATHAM PARK

4.5.1 Field Investigation & Laboratory Testing - Tom Statham Park

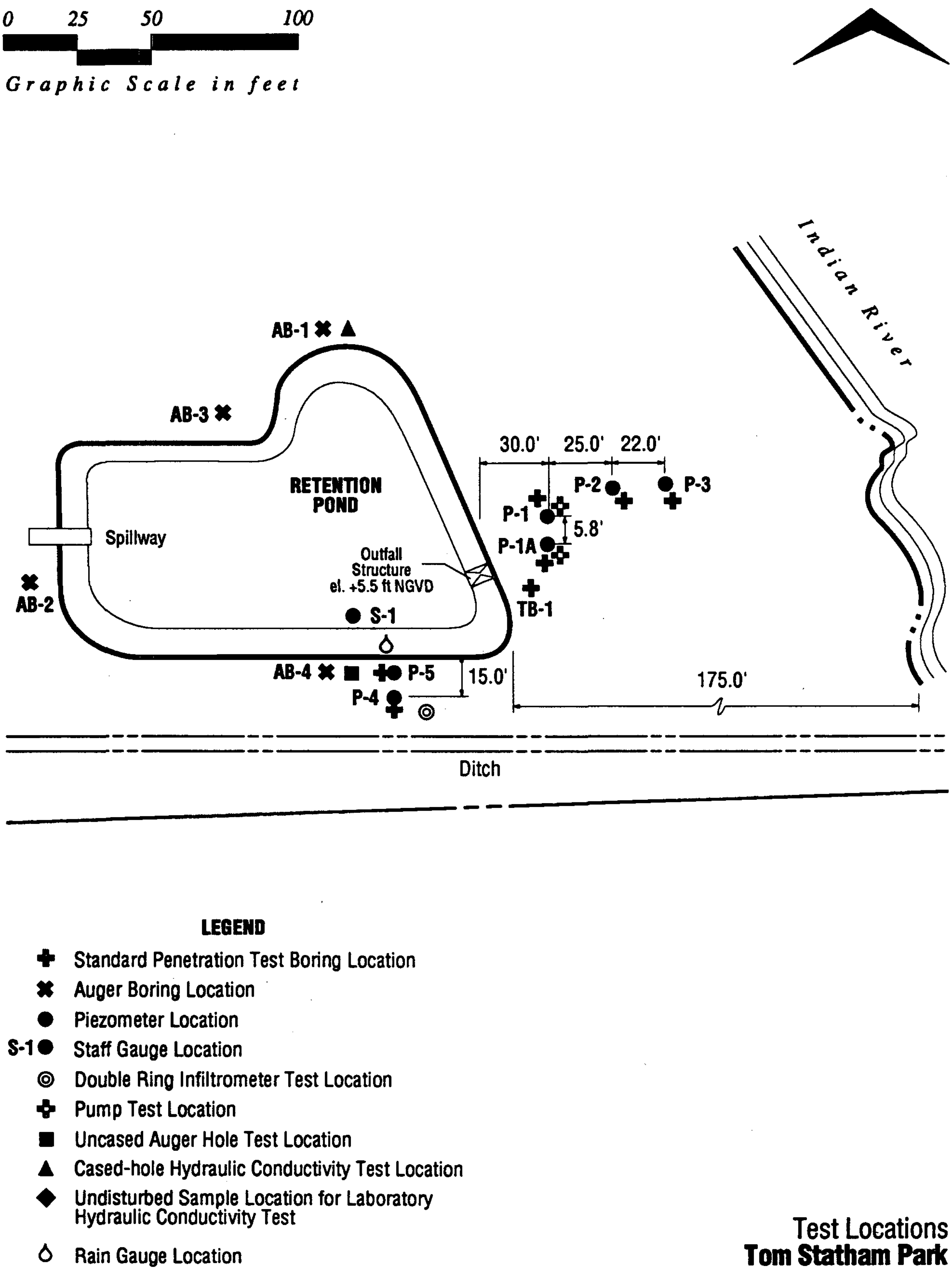
4.5.1.1 Standard Penetration Test and Auger Borings

Six (6) SPT borings and four (4) auger boring were performed at the locations outside the limits of the pond shown on Figure 44. On this figure, the SPT borings are identified as P-1 through P-5 and TB-1, while the auger boring locations are labelled as AB-1 through AB-4. Boring depths ranged from 12 to 40 feet below land surface (bls), but were generally in the range 12 to 20 feet bls. The ground surface elevations at some of the boring locations were surveyed.

The results of the test borings and field sampling program are presented in the form of soil boring profiles on Figures 45 and 46. Included on the profiles are the various soil strata, blow count "N" values obtained from the Standard Penetration Testing, and the depth to the measured water table. A complete legend describing the type and relative density/consistency of the soil conditions encountered is also included on Figures 45 and 46.

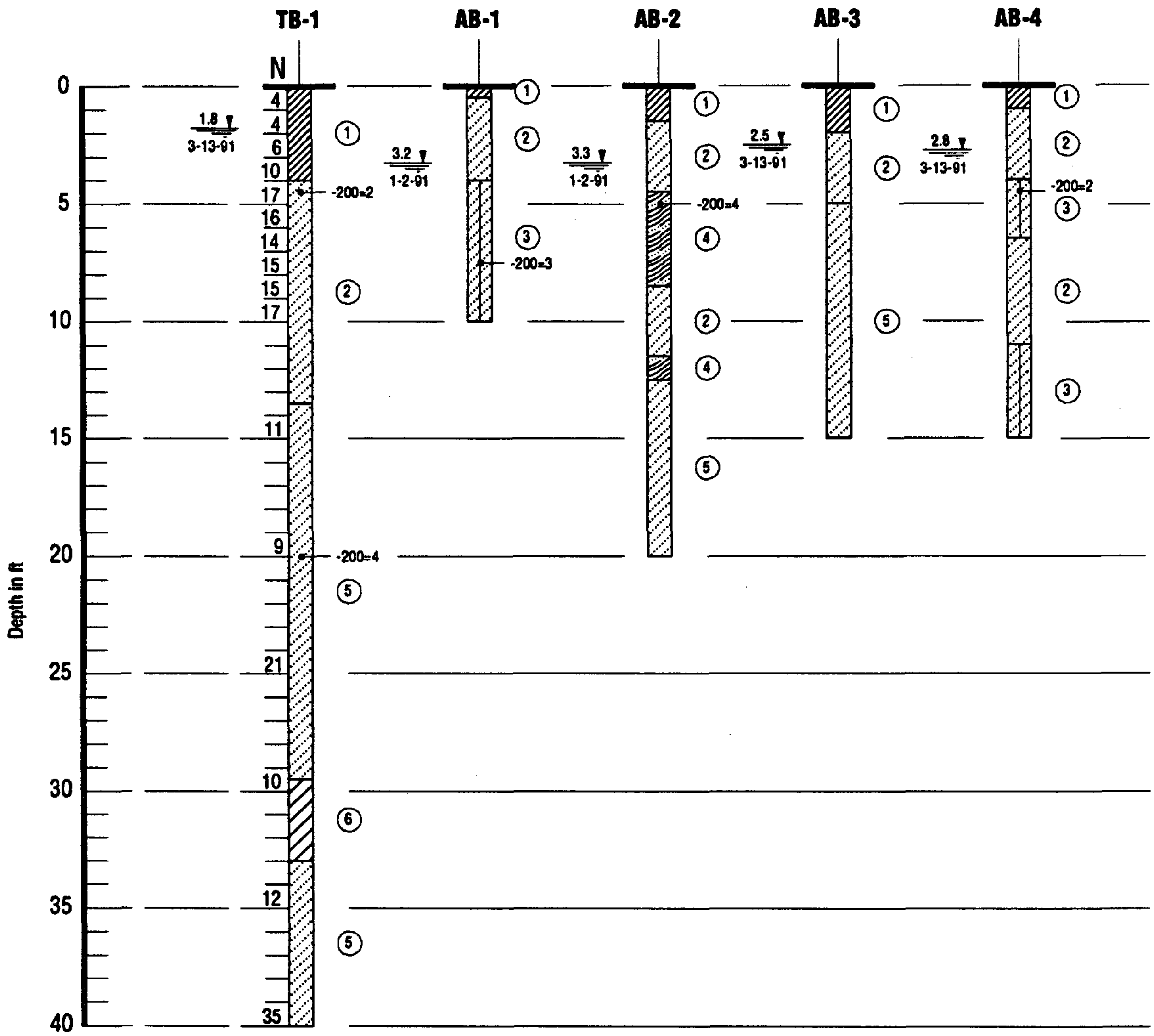
4.5.1.2 Piezometers

Five piezometers were installed at the locations labelled P-1 through P-5 on Figure 44. The construction details for these piezometers are shown on Figure 47. Piezometer P-1A is an additional observation well located 5 feet from P-1 for the pump test. All wells were installed using the hollow-stem auger method. The top of casing and ground surface elevations at each piezometer location were surveyed.



Test Locations
Tom Statham Park

Figure: 44



LEGEND

① Grayish-Brown Fine Sand to Slightly Silty Fine Sand with Occasional Broken Shells (Fill), (SP), *Very Loose to Loose*

② Light Grayish-Brown to Grayish-Brown Fine Sand, (SP), *Medium Dense*

③ Brown to Reddish-Brown Fine Sand to Slightly Silty Fine Sand, (SP)(SP-SM), *Loose to Medium Dense*

④ Dark Reddish-Brown Organic Silty Fine Sand, (SM), Very *Loose*

⑤ Light Grayish-Brown Fine Sand with Occasional Broken Shell, (SP), *Loose to Medium Dense*

⑥ Greenish-Gray Clay, (CH), *Stiff to Very Stiff*

⑦ Dark Reddish-Brown Silty Fine Sand, (SM), *Loose*

⑧ Greenish-Gray Fine Sand with Small Roots, (SP), *Medium Dense*

⑨ Light Grayish-Brown Slightly Silty to Silty Fine Sand, (SP-SM), (SM), *Medium Dense*

(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

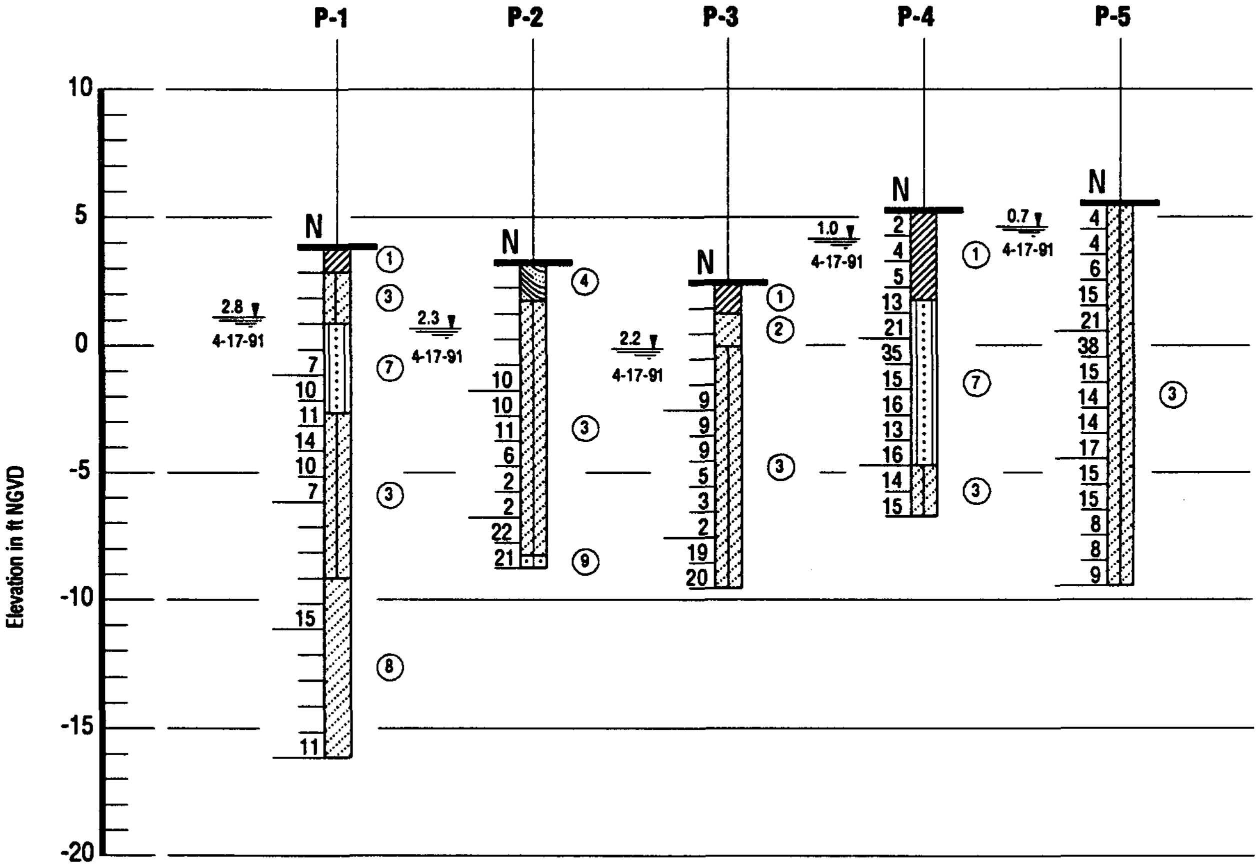
5.9 Depth to Groundwater Level in Feet: Date Noted

N Standard Penetration Resistance in Blows Per Foot

-200 Fines Passing No. 200 Sieve (%)

Boring Profiles
Tom Statham Park

Figure: 45



LEGEND

① Grayish-Brown Fine Sand to Slightly Silty Fine Sand with Occasional Broken Shells (Fill), (SP), *Very Loose to Loose*

② Light Grayish-Brown to Grayish-Brown Fine Sand, (SP), *Medium Dense*

③ Brown to Reddish-Brown Fine Sand to Slightly Silty Fine Sand, (SP)(SP-SM), *Loose to Medium Dense*

④ Dark Reddish-Brown Organic Silty Fine Sand, (SM), Very *Loose*

⑤ Light Grayish-Brown Fine Sand with Occasional Broken Shell, (SP), *Loose to Medium Dense*

⑥ Greenish-Gray Clay, (CH), *Stiff to Very Stiff*

⑦ Dark Reddish-Brown Silty Fine Sand, (SM), *Loose*

⑧ Greenish-Gray Fine Sand with Small Roots, (SP), *Medium Dense*

⑨ Light Grayish-Brown Slightly Silty to Silty Fine Sand, (SP-SM), (SM), *Medium Dense*

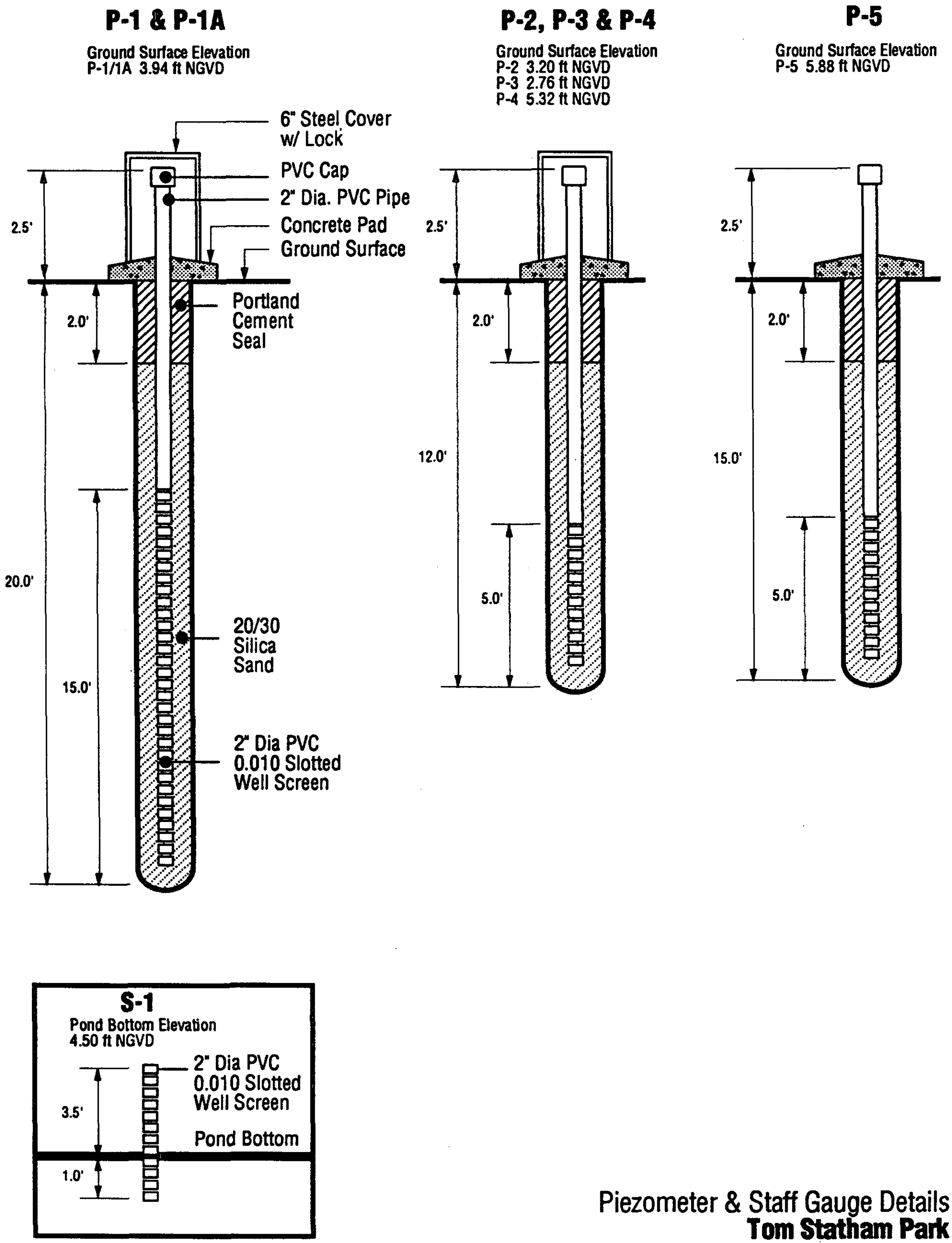
(SP) Unified Soil Classification Group Symbol as Determined by Visual Examination

5.9 Depth to Groundwater Level in Feet: Date Noted

N Standard Penetration Resistance in Blows Per Foot

-200 Fines Passing No. 200 Sieve (%)

Boring Profiles
Tom Statham Park



Piezometer & Staff Gauge Details
Tom Statham Park

Figure: 47

4.5.1.3 Field and Laboratory Hydraulic Conductivity Tests

Test locations for the series of hydraulic conductivity tests are shown on Figure 44 and results are tabulated on Table 5 with remarks where appropriate. With respect to these tests, the following observations should be noted:

- Pump test - As noted on Figure 47, piezometers P-1 and P-1A had identical completion details and were spaced 5 feet apart. The piezometers were screened from just below the groundwater table to a total depth of approximately 20 feet bls. As noted on the soil profile for P-1 and TB-1 (Figures 45 and 46), the soil profile in the screened interval can be generalized as follows:

Depth Below Grade	Soil Type
0 to 4 ft	Grayish-brown fine sand fill with shells
4 to 13 ft	Brown to reddish-brown slightly silty fine sand
13 to 20+ ft	Greenish-gray fine sand

  The intermediate dark reddish brown slightly silty fine sand zone is hydraulically more restrictive than the upper mantle of fine sand and the lower zone. The typical pump test details for this study are described on Figure 31.

- For the cased hole (bottom flush) test configuration, the bottom of the casing was seated very close to the hardpan stratum and therefore this boundary condition influenced the results.

Table 5: Hydraulic Conductivity Test Results for Tom Statham Park

Test Method	Test Depth Below Ground Surface (ft)	Soil Description	Hydraulic Conductivity Value	
			Constant Head	Falling Head
Cased hole - vertical (Fig. 27)	2.5	grayish-brown fine sand	0.5 ft/day	1.5 ft/day
Cased hole - horizontal (Fig. 28)	3.0	grayish-brown fine sand	2.0 ft/day	0.5 ft/day
Uncased auger hole (Fig. 29)	2.0	grayish-brown fine sand	15 ft/day	--
Laboratory (K_h)	1.5	grayish-brown fine sand	--	15 ft/day
Laboratory (K_v)	1.0	grayish-brown fine sand	--	7 ft/day
Double Ring Infiltrometer	0.5	grayish-brown fine sand	.86 ft/day	--
8 hr Pump Test	2-20	see soil profile for P-1 on Figure 46	1,300 ft^2/day transmissivity	--

<u>Notes:</u>

1. Test locations shown on Figure 44
2. In the cased borehole tests (vertical & horizontal), the bottom of the casing was seated close to the hardpan layer thus the low hydraulic conductivity value.
3. The value reported for the Double Ring Infiltrometer test is an infiltration rate and not a hydraulic conductivity value
4. The transmissivity value from the pump test is representative of the soil profile for P-1 on Figure 46

4.5.2 Hydrostratigraphic Characterization - Tom Statham Park

4.5.2.1 SCS Map Unit

The USDA Soil Conservation Service (SCS) Soil Survey for Brevard County classifies the soil at the study site as Anclote sand and Satellite sand composed of nearly level, somewhat to very poorly drained soils, with a minor component of roots and organics in some layers. The seasonal high groundwater table is estimated by the SCS to be generally between 10 and 40 inches below the ground surface.

4.5.2.2 Soil Stratigraphy

As disclosed by the borings, the soil stratigraphy at the study site consists of a grayish brown fine sand with fill containing shell (Stratum 1) at the surface underlain by grayish-brown fine sand (Stratum 2). The combined thickness of the upper zone of Strata 1 and 2 soils is 4 to 5 feet. Below this upper zone are the reddish brown slightly silty to silty fine sands (Strata 3, 4, and 7) which extend to a depth of 12 to 13 feet below land surface (bls). This intermediate layer of less permeable soil is underlain by light grayish brown fine sand with shell fragments (Strata 5 and 8) which extend to depths of over 40 feet bls.

4.5.2.3 Groundwater Table

As noted on the soil profiles (Figures 45 and 46), the groundwater table measured during January through March, 1991, was encountered at depths ranging from .8 to 3.3 feet bls.

4.5.2.4 Hydraulic Conductivity

Field and laboratory test results presented on Table 5 indicate that the horizontal hydraulic conductivity of the Stratum 2 fine sands is approximately 15 ft/day, while the underlying reddish-brown "hardpan" type soils (Strata 3, 4, and 7) are less than 1 ft/day. From the pump test data, it also appears that the underlying Strata 5 and 8 fine sands with shell fragments is a very transmissive zone (transmissivity = 1300 ft^2/day).

SECTION 5.0: HYDROLOGIC MONITORING & LOAD TESTING OF PONDS

5.1 HYDROLOGIC MONITORING PROGRAMS

5.1.1 Parameters Monitored and Frequency of Readings

Hydrologic monitoring instruments were installed at the long-term monitoring sites--Airport Warehouses and Tom Statham Park--to monitor the following hydrologic parameters:

- the surface water/groundwater elevation in the pond at hourly intervals,
- groundwater elevations in observation wells adjacent to the pond at hourly intervals, and
- rainfall measurements at 10 minute intervals from an onsite rain gauge.

The instrumentation for the short-term sites was similar to the long-term sites except that:

- The data logging equipment was set to take readings at 10 minute intervals instead of at 1 hour intervals, and
- Rainfall was not monitored.

A detailed equipment list used for hydrologic monitoring at the short-term and long-term sites is included in Appendix A.

5.1.2 Groundwater/Pond Water Level Monitoring Locations

Table 6 lists the locations adjacent to each pond where groundwater levels were monitored. This table also presents the respective monitoring periods for each pond.

The observation wells were located outside the limits of the ponds and were screened such that groundwater levels in the uppermost aquifer were measured. As an exception, piezometer P-5 at Airport Warehouses was located adjacent to the staff gauge within the pond bottom and had its screened interval grouted into the "hardpan" layer beneath the uppermost aquifer.

Table 6: Ground Water & Surface Water Monitoring Locations and Monitoring Periods

POND	Water Level Monitoring Locations	Piezometer Details	Duration of Monitoring	Monitoring Period From	Monitoring Period To
Airport Warehouses	S-1, P-1, P-2, P-3, and P-4 (Figure 33)	Figure 36	11 months	Nov. 1991	Sept. 1992
Tutor Time	S-1, P-2, P-3, P-4, P-5, and P-6 (Figure 37)	Figure 39	28 days	April 13 1992	May 10 1992
Fisherman's Landing	S-1, P-2, P-3, P-4, P-5, and P-6 (Figure 40)	Figure 43	12 days	May 21 1992	June 1 1992
Tom Statham Park	S-1, P-1, P-2, P-3, P-4, and P-5 (Figure 44)	Figure 47	11 months	Nov. 1991	Sept. 1992

5.1.3 Quality Assurance

Once the instruments were installed and calibrated, and initial readings were recorded, technicians visited the sites a minimum of twice per week during the monitoring period. During each site visit, the equipment was checked to ensure it was functioning properly and that groundwater levels and rainfall data were being recorded. The test data was down-loaded at least once per week and batteries were changed as necessary. Required maintenance to the system was also performed.

For quality assurance purposes, manual measurements of surface water and groundwater levels were made during each site visit and compared to those recorded by the instruments. Observed discrepancies were noted and the equipment recalibrated as necessary.

5.2 PRESENTATION & DISCUSSION OF HYDROLOGIC MONITORING DATA

5.2.1 Airport Warehouses

Figures 48 through 58 show line graphs of the recorded water levels and the corresponding daily rainfall data in bar chart form for the Airport Warehouses pond for each month of the 11-month monitoring period spanning November 1991 through September 1992. Equipment malfunction resulted in missing data for certains wells or the rain gauge and these period of missing data are noted at appropriate points on Figures 48 through 58. The data for the calendar year 1992 is considered very reliable.

From review of the monitoring data, the following key observations are made:

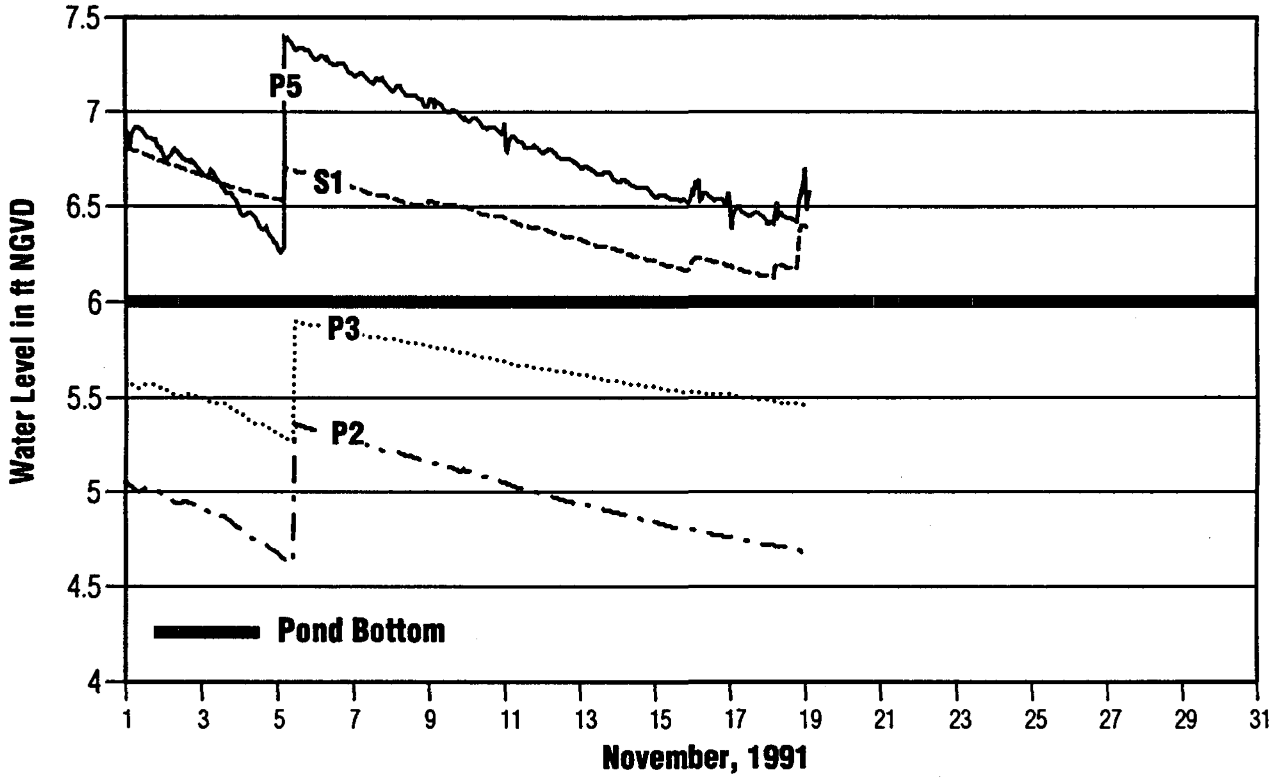
- Rainfall: The total rainfall measured at the site for the period January 1, 1992 through September 30, 1992 was 31.3 inches, of which 10.6 inches fell in the month of June. The highest daily rainfall (4.24 inches) during the monitoring period was recorded on June 26. These rainfall conditions are typical of normal wet season conditions in this vicinity.

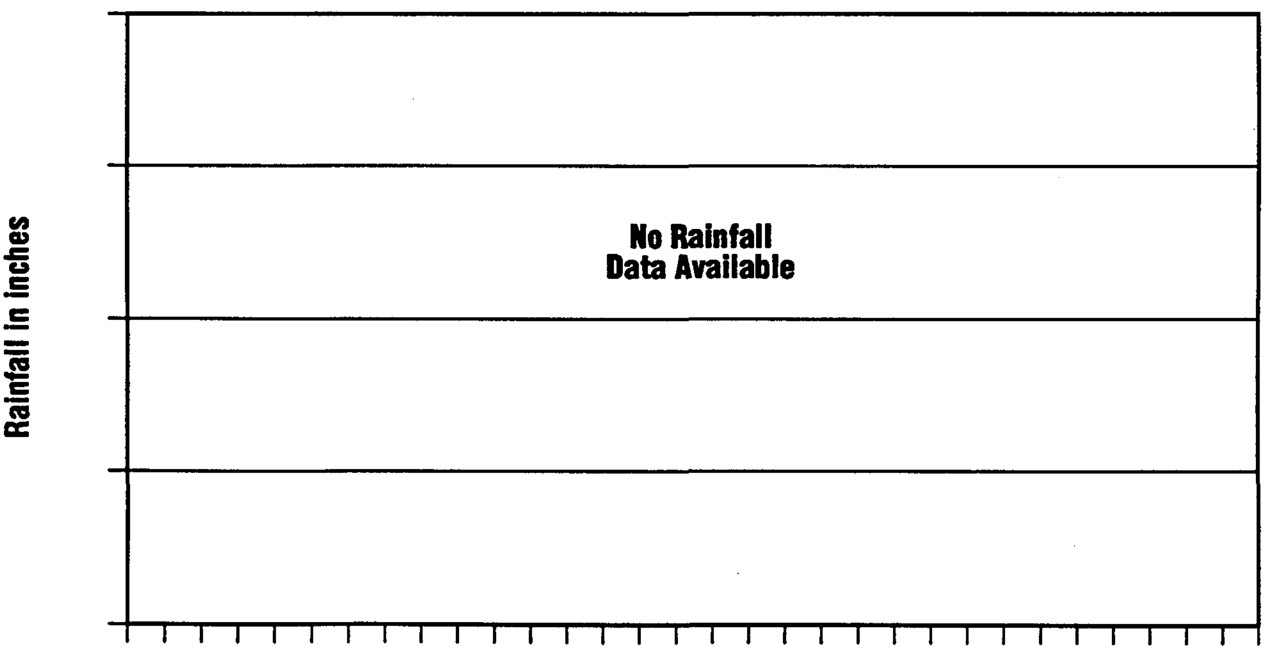
- Pond Stage vs. Rainfall: As expected, the pond stage reacts to the runoff generated by rainfall. During the study period, the maximum pond stage recorded was approximately +9 ft NGVD on June 27 following two days of a cumulative rainfall of 5.92 inches (refer to S-1 readings on Figure 55). Although only 1.81 inches of rainfall was recorded during July, the recovery of this peak pond stage to the pond bottom elevation of +6 ft NGVD took almost the entire month of July. For this particular site, there was no measurable increase in the pond water level when daily rainfall totals were less than 0.1 inch [as an example, see the impact of the 0.11 inch of rainfall on May 7 (see Figure 54)].

- Lateral Spreading Of Groundwater Mound During Recovery: Figure 59 shows plots of the groundwater profiles along the transect line P-1, P-2, and P-3 (Figure 33) during recovery following the 0.7 inches of rainfall on March 25, 1992 (Figure 52). These profiles illustrate the cross-sectional, transient evolution of the groundwater profiles as recovery progresses. Note that there was no additional rainfall during this recovery which could have resulted in vertical recharge to the aquifer and could have been the source for the increase in groundwater levels at P-1, P-2, and P-3. However, most of the rise in groundwater level at the piezometers P-2 and P-3 indicated 3 hours after the peak stage could have resulted from direct rainfall recharge to the uppermost aquifer.

Based on the buildup of the mound observed at P-3 during this recovery event, it is apparent that the radius of influence of the recharge loading in the pond extends to a distance greater than 100 feet from the edge of the pond.

- Inter-aquifer Head Difference: The water level in the pond bottom well P-5 screened only in the "hardpan" layer does not equilibrate rapidly with the water level (S-1) in the more permeable fine sand aquifer above the "hardpan". This suggests that the "hardpan" does indeed act as a restrictive layer to vertical flow.

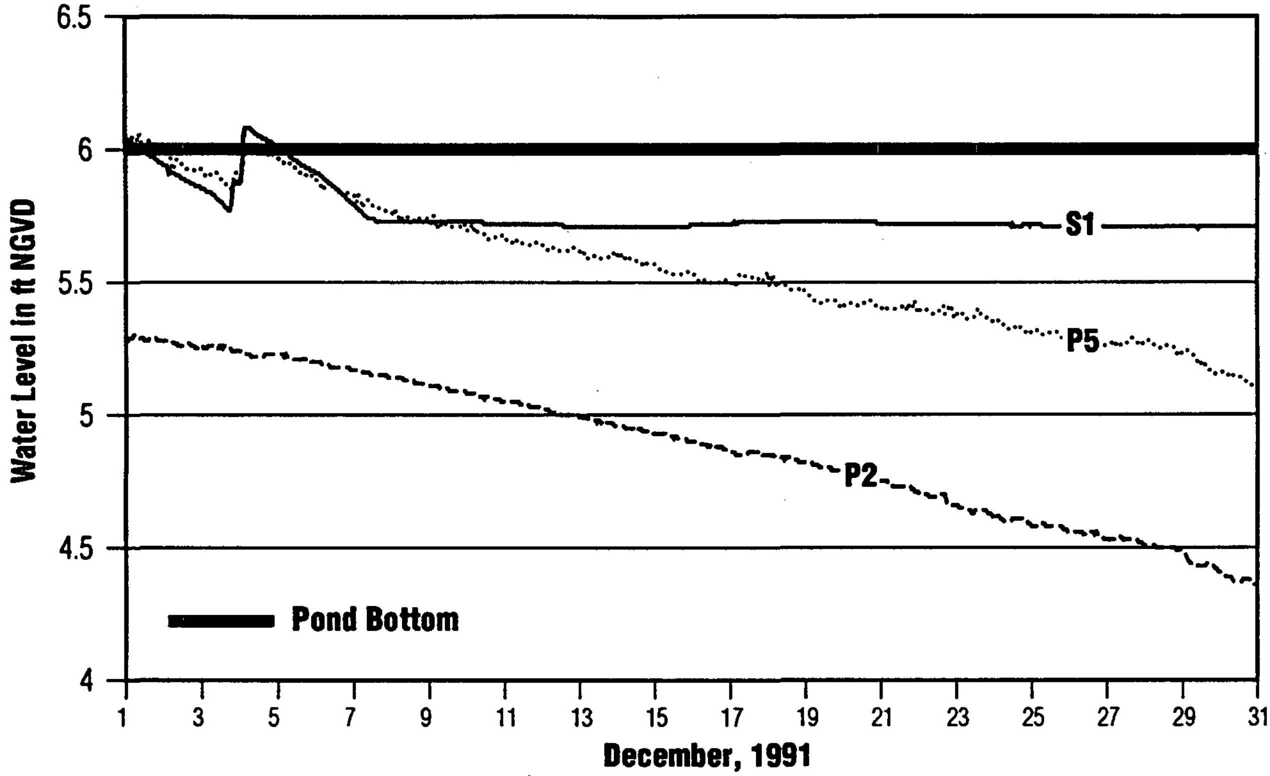


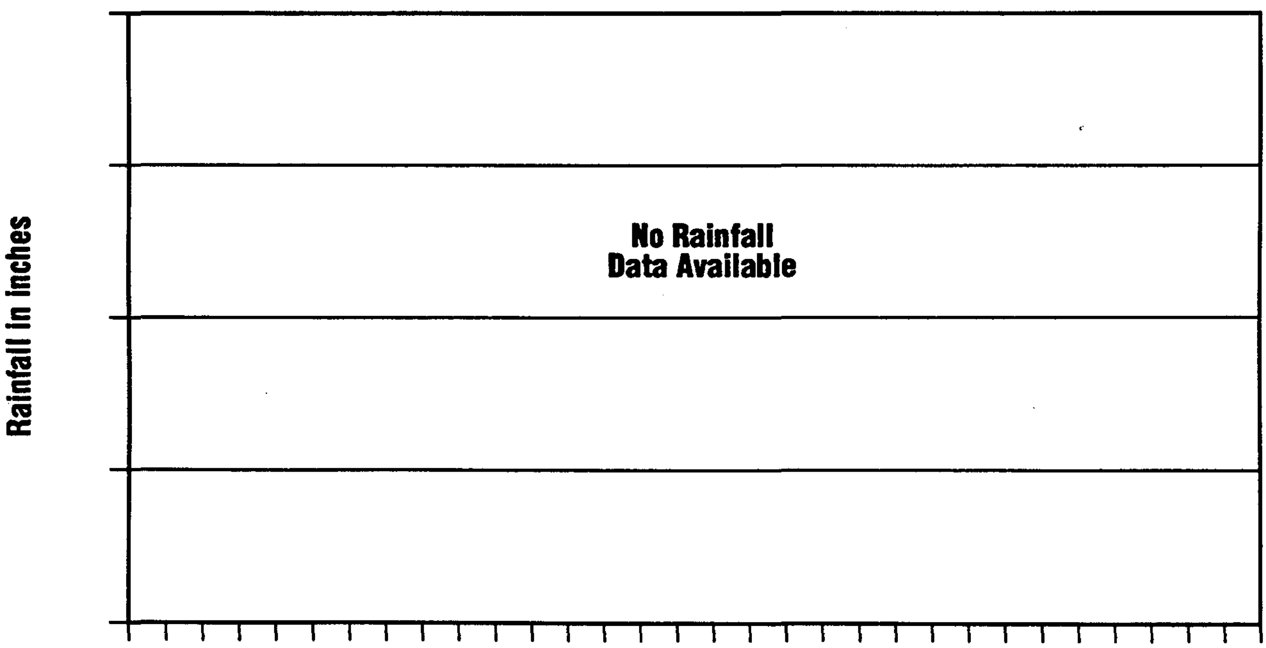


Note: Gaps in line graphs represent periods of equipment malfunction

Hydrologic Monitoring Data
November-1991
Airport Warehouses

Figure: 48

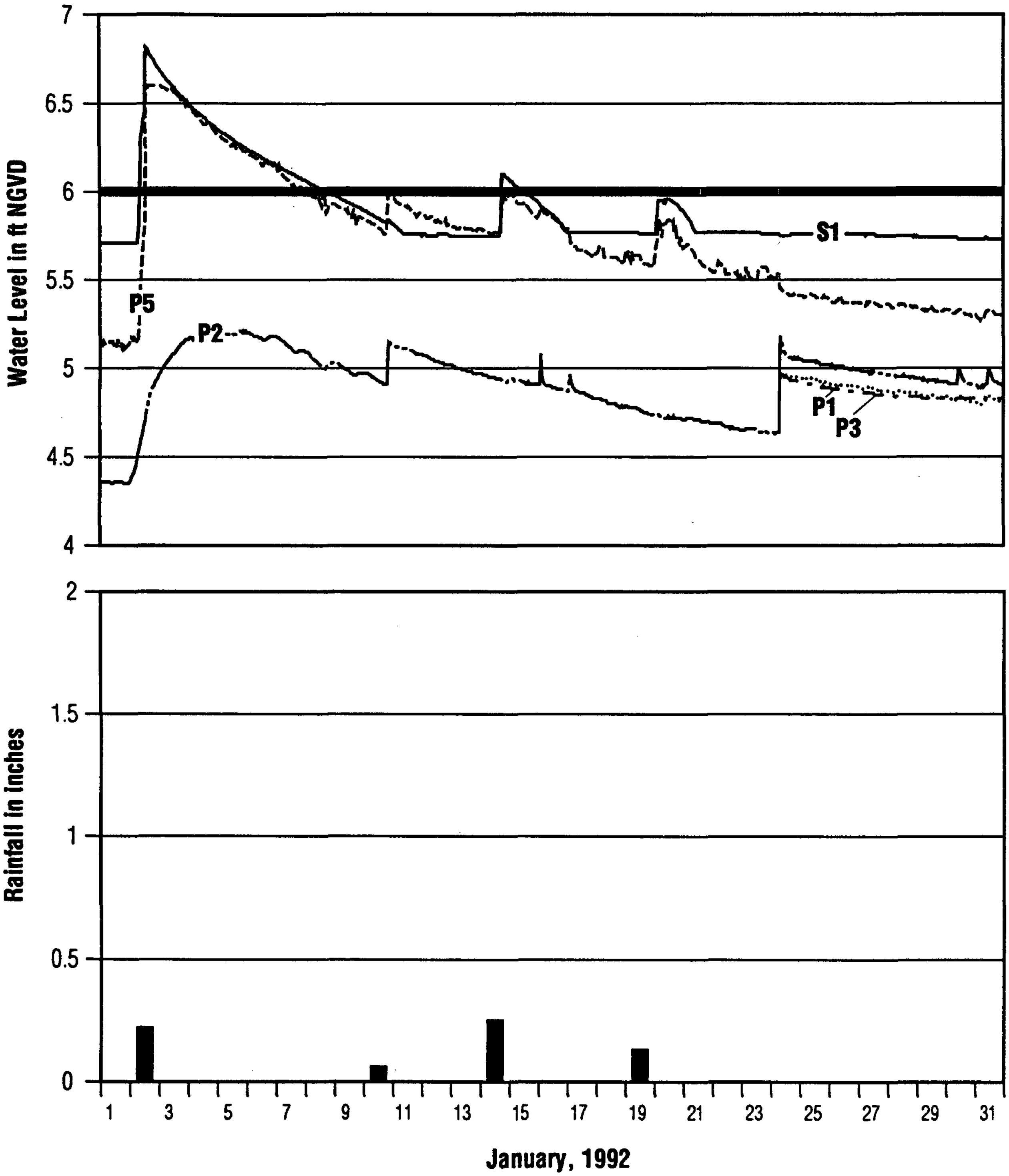




Note: Gaps in line graphs represent periods of equipment malfunction

Hydrologic Monitoring Data
December-1991
Airport Warehouses

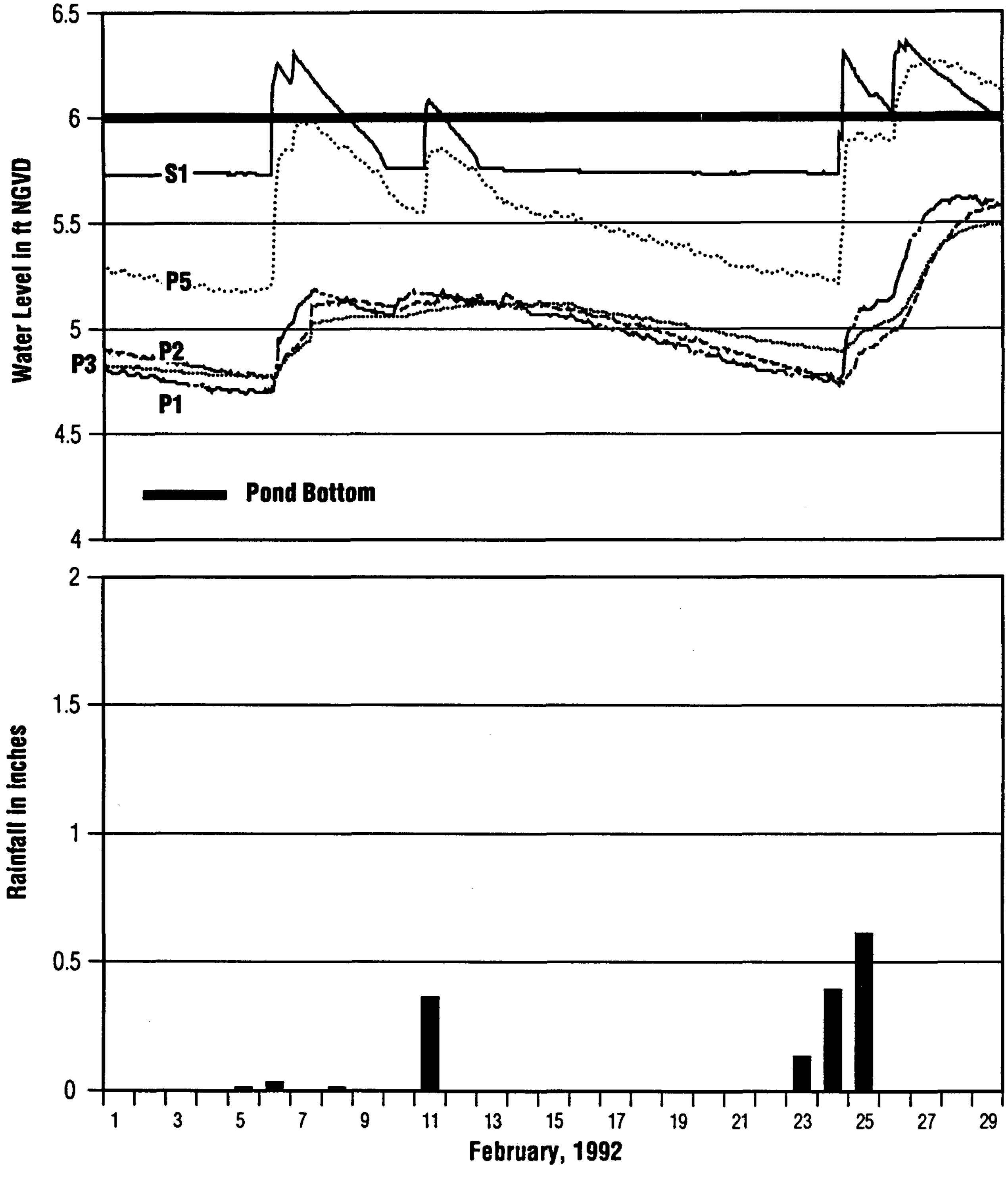
Figure: 49



Note: Gaps in line graphs represent periods of equipment malfunction

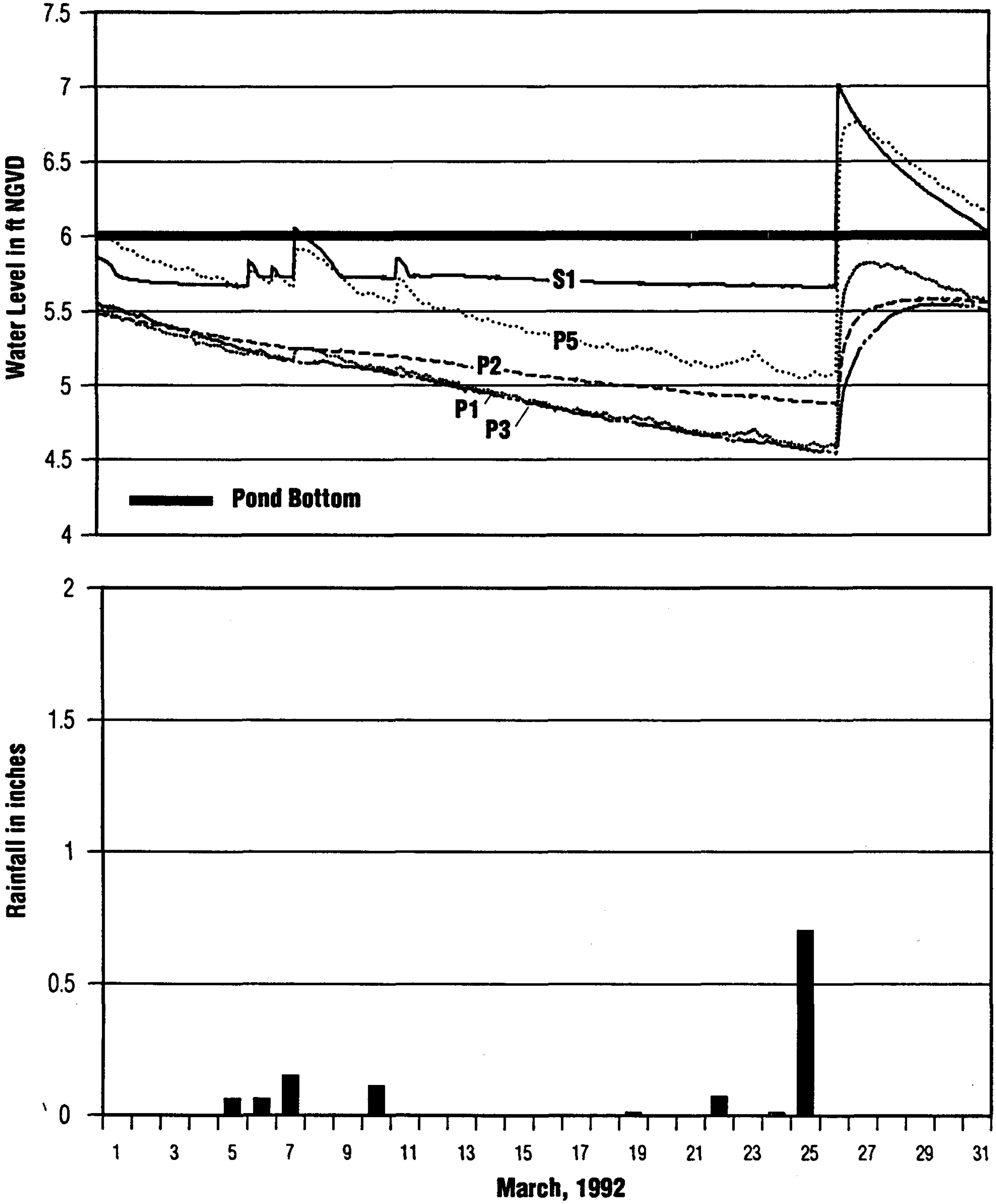
Hydrologic Monitoring Data
January-1992
Airport Warehouses

Figure: 50



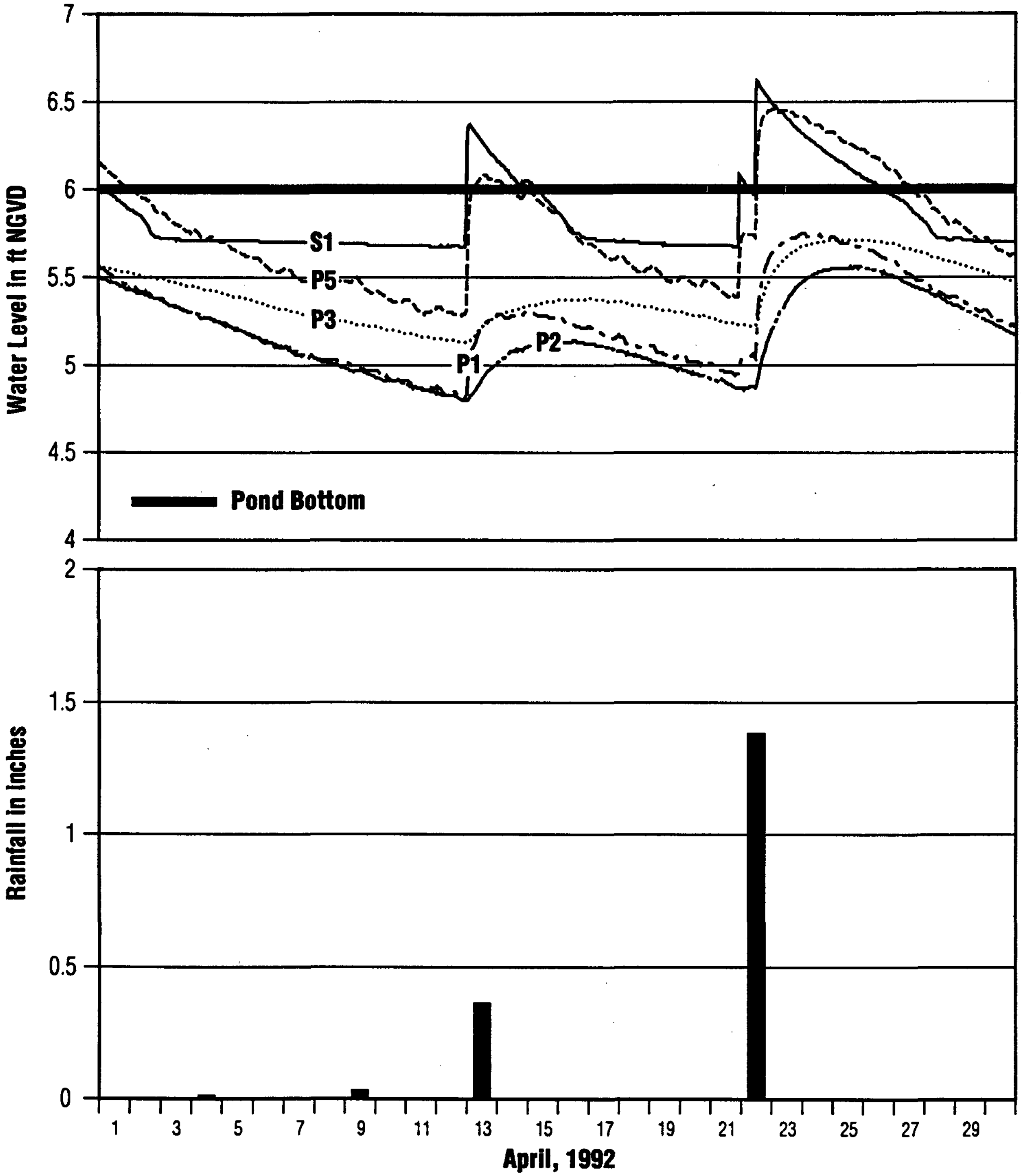
Hydrologic Monitoring Data
February-1992
Airport Warehouses

Figure: 51



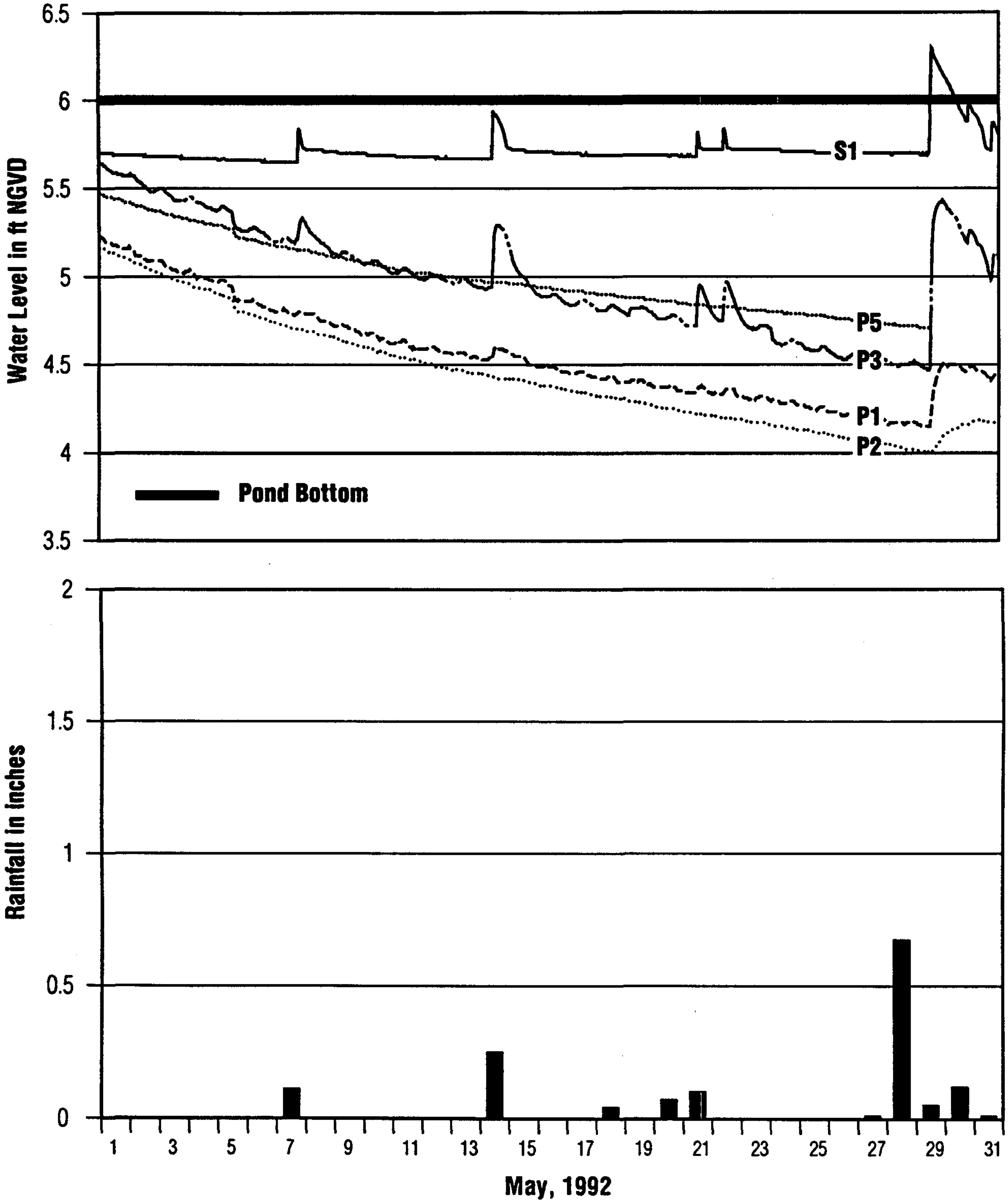
Hydrologic Monitoring Data
March-1992
Airport Warehouses

Figure: 52



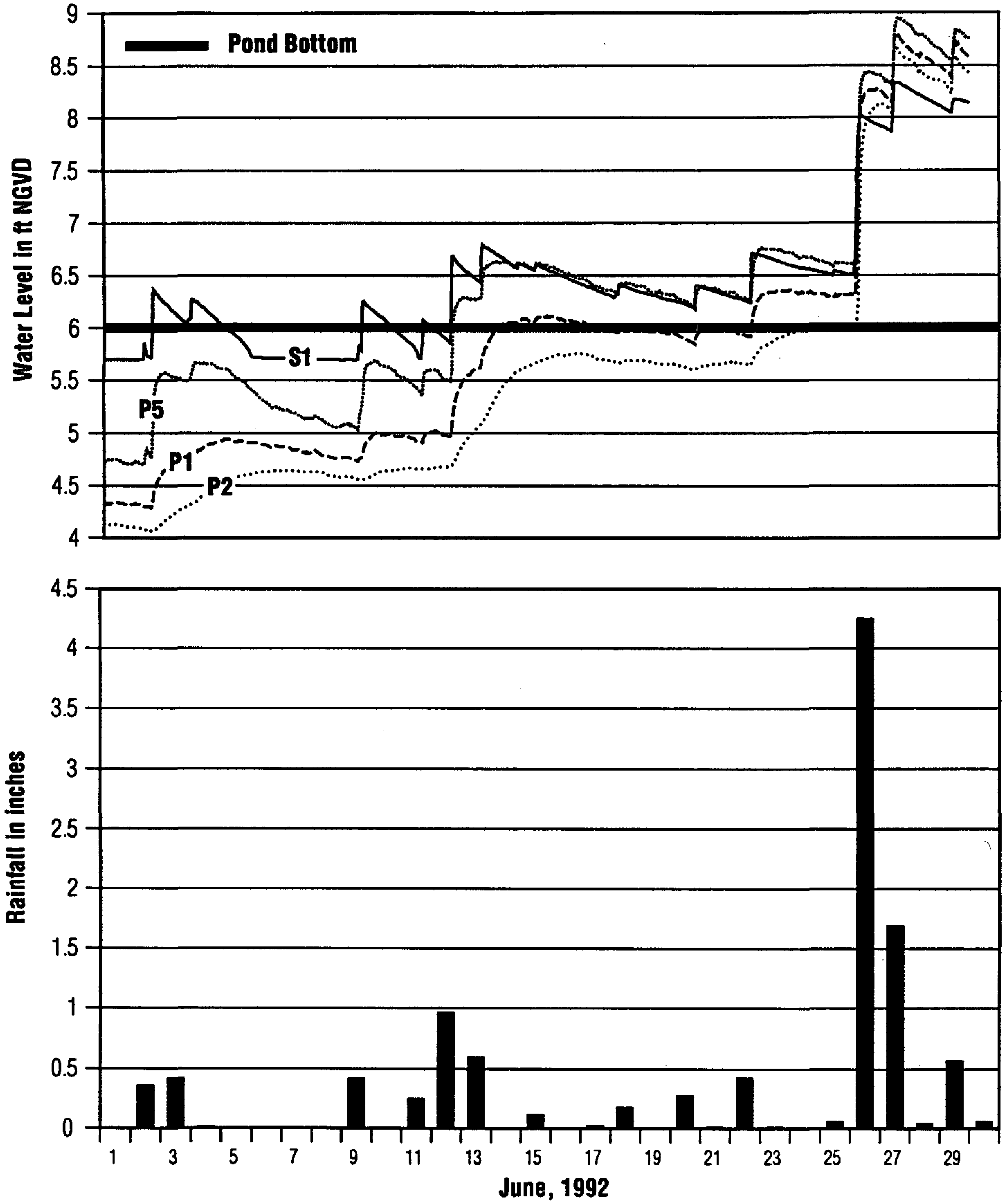
Hydrologic Monitoring Data
April-1992
Airport Warehouses

Figure: 53



Hydrologic Monitoring Data
May-1992
Airport Warehouses

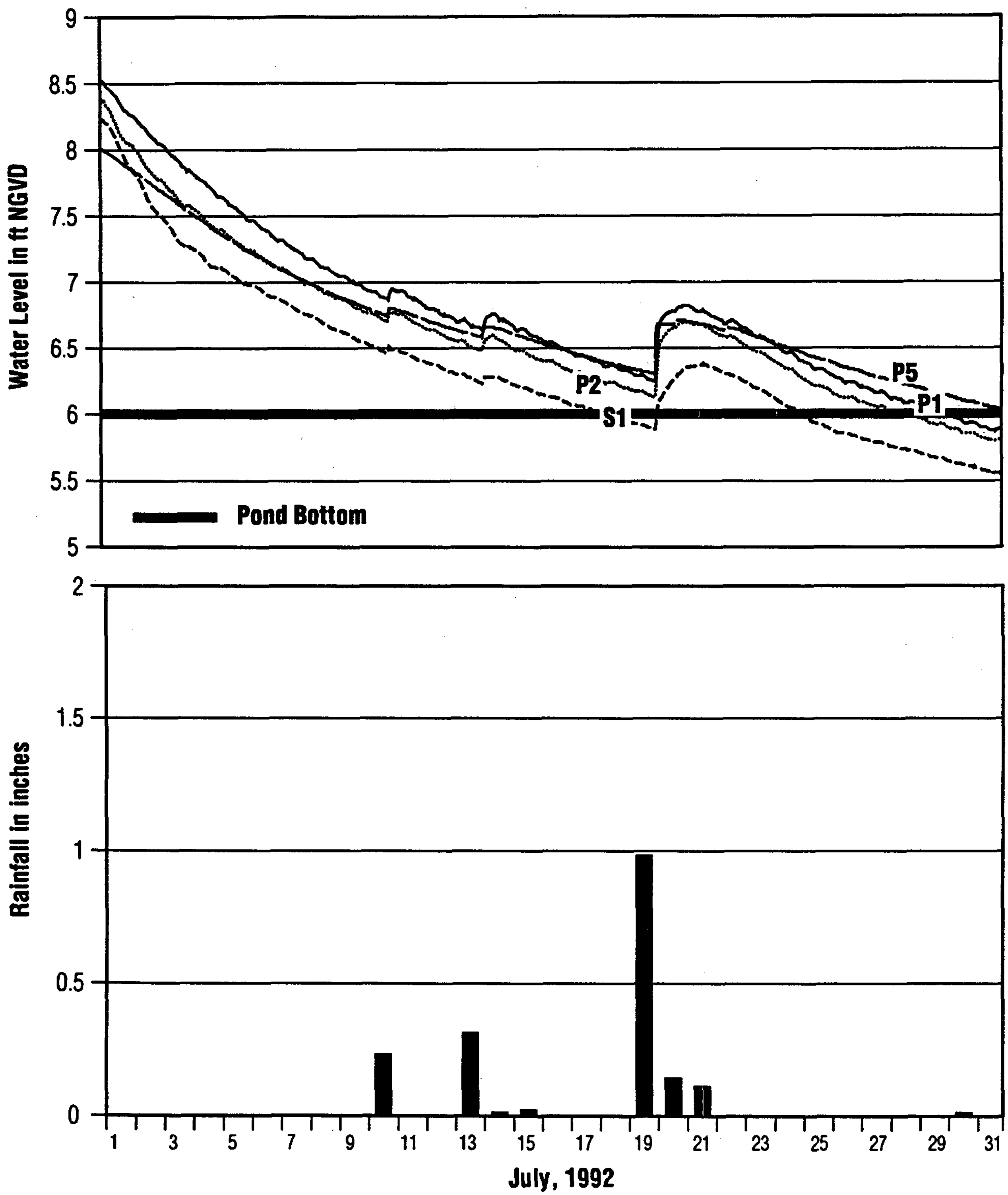
Figure: 54



Note: Gaps in line graphs represent periods of equipment malfunction

Hydrologic Monitoring Data
June-1992
Airport Warehouses

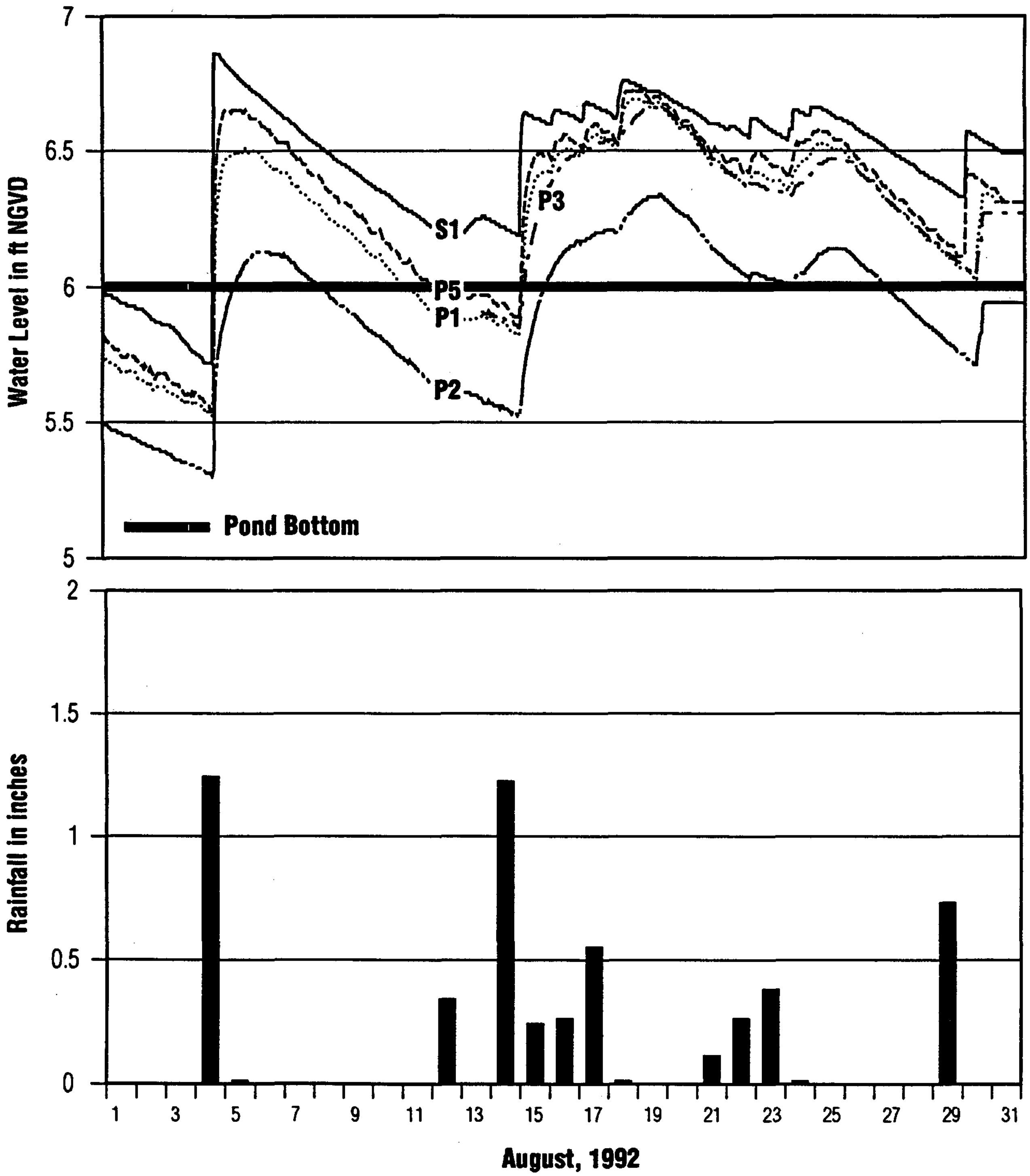
Figure: 55



Note: Gaps in line graphs represent periods of equipment malfunction

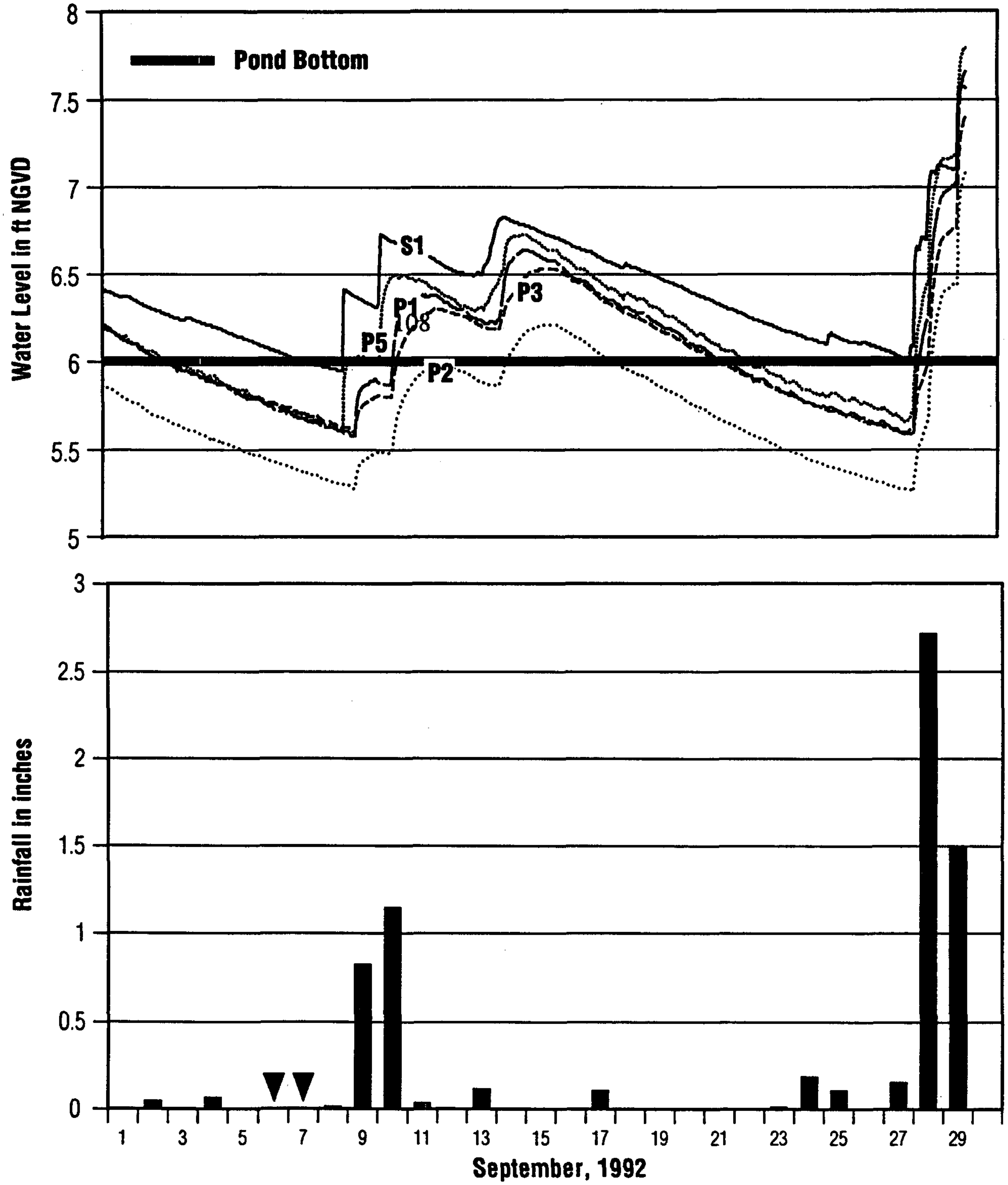
Hydrologic Monitoring Data
July-1992
Airport Warehouses

Figure: 56



Hydrologic Monitoring Data
August-1992
Airport Warehouses

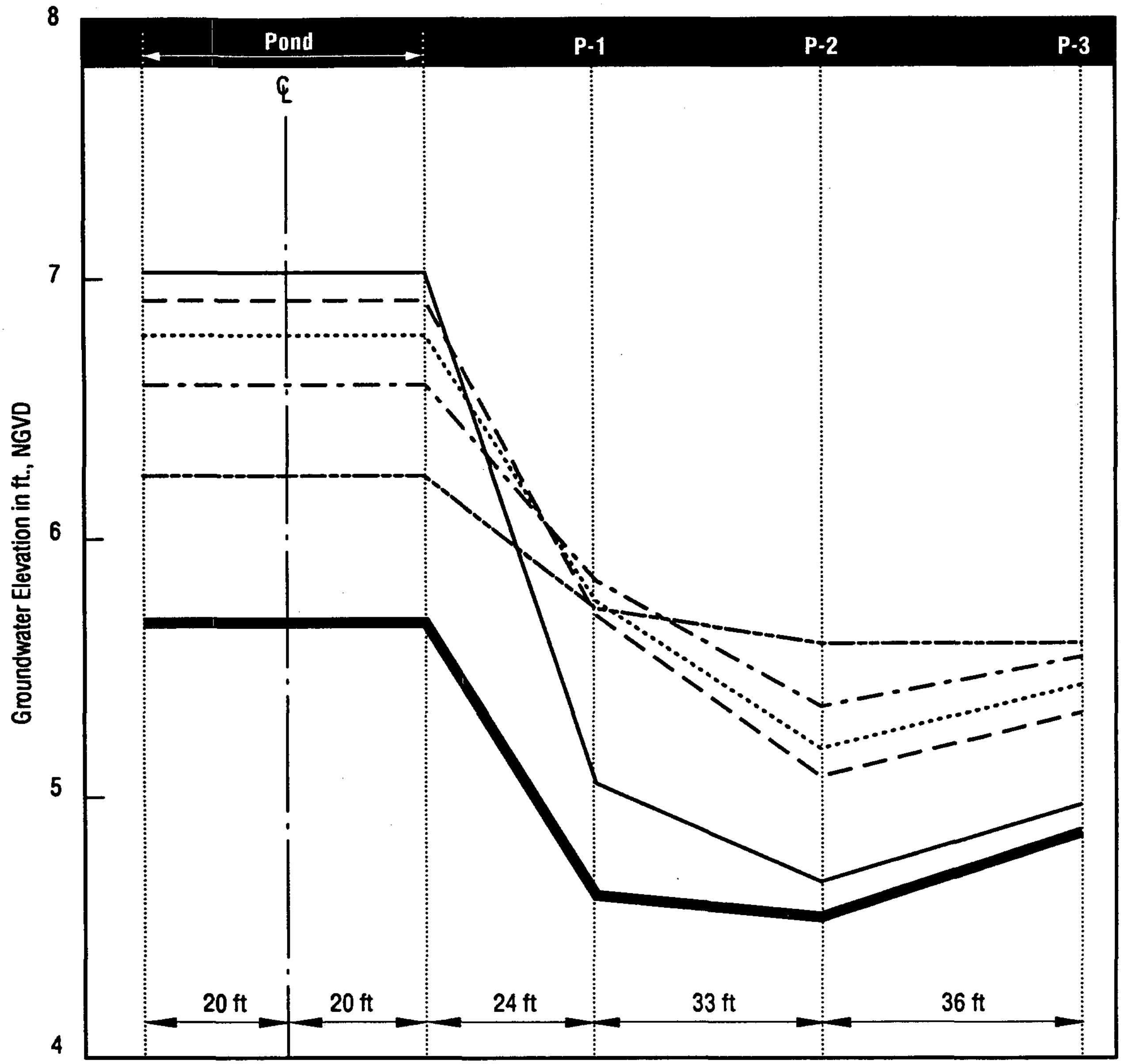
Figure: 57



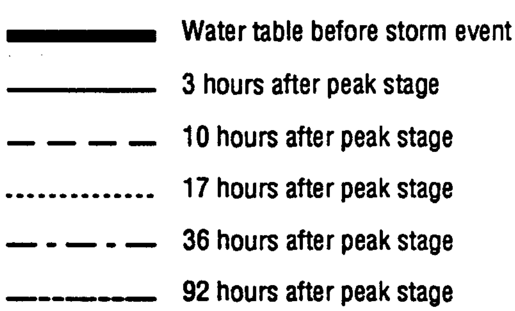
▼ No rainfall data available for this date due to equipment malfunction

Hydrologic Monitoring Data
September-1992
Airport Warehouses

Figure: 58



NOTE: Storm event occured March 25, 1992



Ground Water Profiles
Airport Warehouses

Figure: 59

5.2.2 Tutor Time

Figure 60 shows the stage versus time groundwater and pond water level readings for the Tutor Time load test covering the period April 13, 1992 through May 10, 1992. The following were the sequence of events during this load test:

1. Baseline water level readings were taken just prior to this initial loading on April 13, 1992.

2. On this day, the pond was loaded to near its overflow elevation using water from a nearby fire hydrant. The time for filling the pond was approximately 1 hour. The rate at which and volume of water intorduced into the pond was not measured.

3. On two occasions during recovery from the initial loading, it rained as noted from the gentle spikes in the staff gauge reading (S-1 on Figure 60).

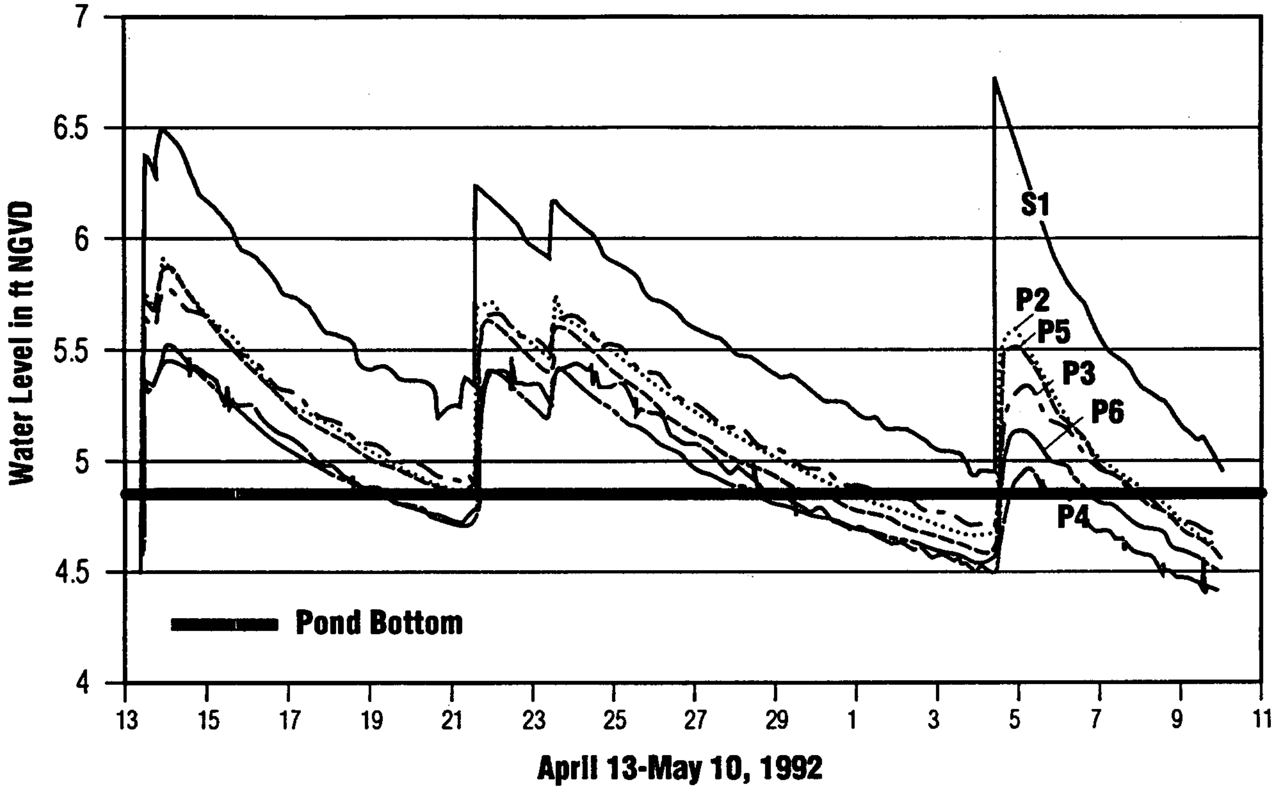
4. The pond was loaded again on April 21 in a similar manner to the initial slug loading on April 13. The pond was loaded to an elevation of +6.3 ft NGVD in 1.2 hours and allowed to recover.

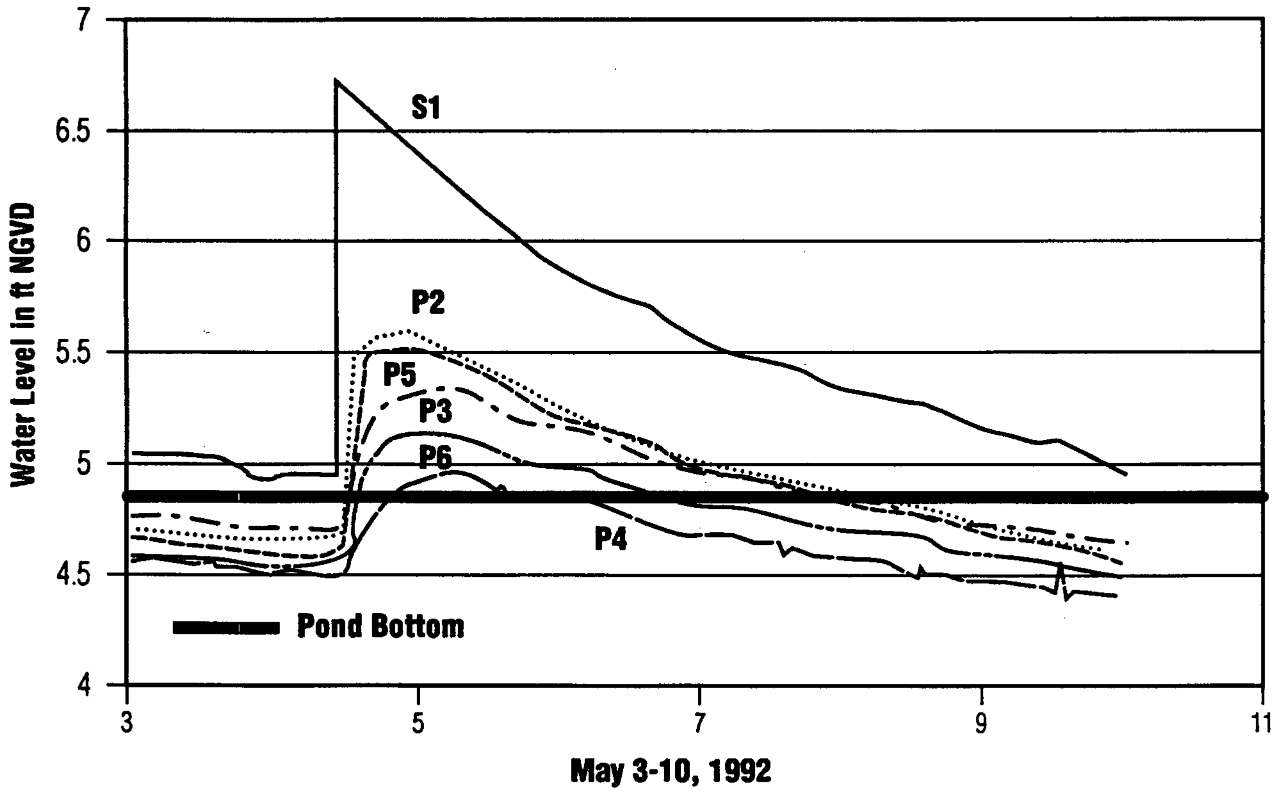
5. On April 23, it rained as noted by the rise in the staff gauge reading on Figure 60.

6. After allowing the pond to recover until the 4th of May, it was loaded again to its overflow elevation of +6.73 ft NGVD in approximately 1 hour and water levels during recovery were monitored until May 10. There was no significant rainfall (if any) during this period of recovery. At the bottom of Figure 60 is a more detailed plot of the stage recovery after this final loading.

From review of the monitoring data for this short-term load test, the following key observations can be made:

- Recovery Time: The peak stage achieved during the final load test of May 4 was +6.73 ft NGVD. The pond water level dropped 1.7 ft to an approximate elevation of +5 ft NGVD in 6 days. Baseline water levels prior to this final slug loading reflected slight mounding from the previous hydraulic loading.



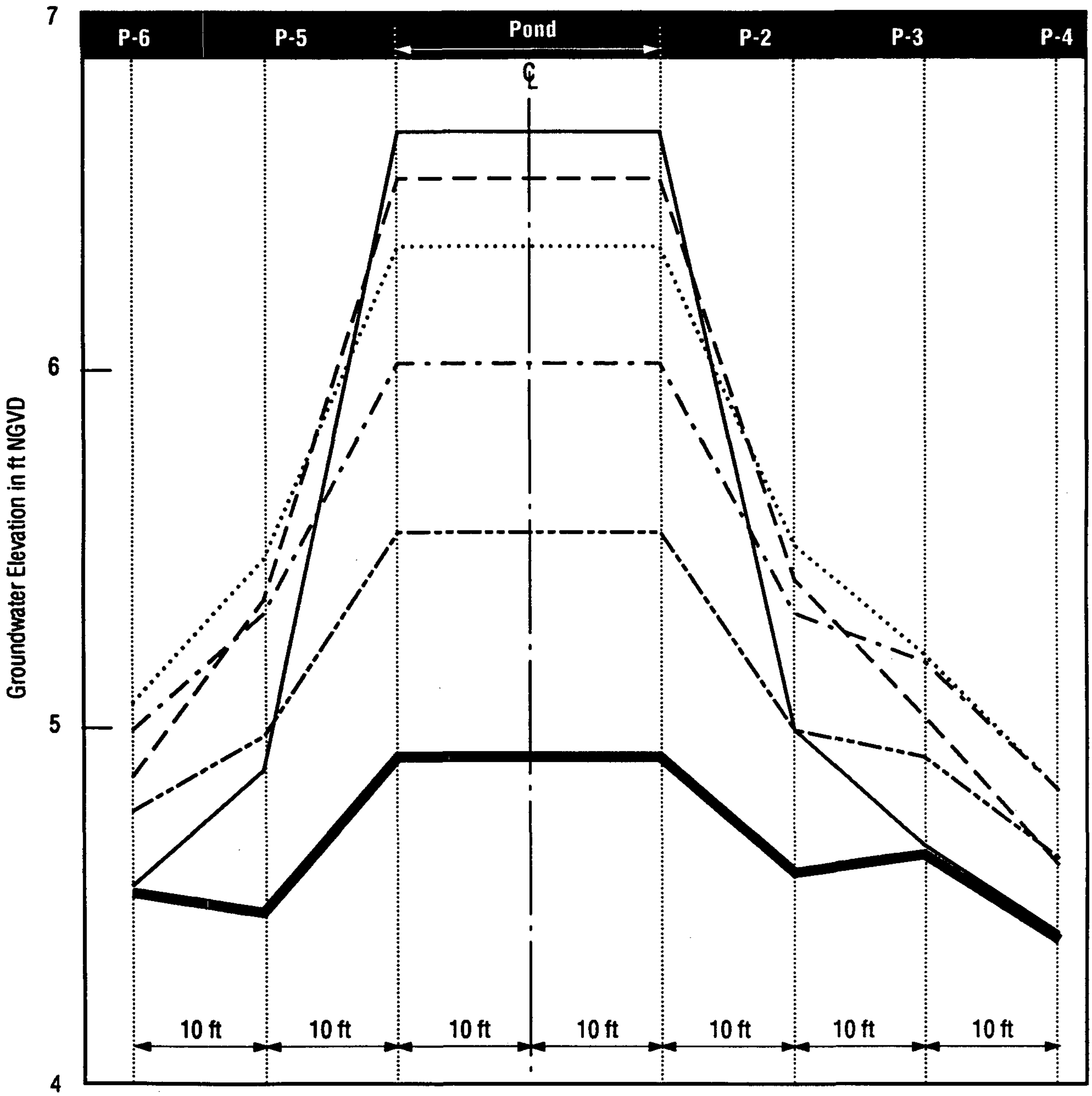


Hydrologic Monitoring Data
Tutor-Time Child Care Facility

Figure: 60

- <u>Influence of Ditch:</u> Note from Figure 37 that piezometers P-2 and P-5 are symmetrically located 10 feet from the edge of the pond, and P-3 and P-6 are are also similarly located at a distance of 20 feet from the edge of the pond. Although the placement of these piezometers may be symmetrical from a geometrical standpoint, they are not necessarily symmetrical from the standpoint of subsurface hydrology because of the presence of the ditch 18 to 20 feet south of P-6. As noted on Figure 60, the water level response in piezometers P-2 and P-5 are identical for all practical purposes. However, the groundwater level in P-6, however, is approximately 0.15 feet below P-3, manifesting the relatively slight influence of the ditch for short-term recovery.

- <u>Lateral Spreading Of Groundwater Mound During Recovery:</u> Figure 61 shows a cross-sectional view of the evolution of the groundwater mound following the slug loading of May 4, 1992. There was little or no rainfall during this recovery period so the measured groundwater rise can be attributed solely to the lateral spread of the mound.



NOTE: Graph of results for load test performed May 4, 1992

Water table before load test

1 hour after peak stage

4 hours after peak stage

11 hours after peak stage

26 hours after peak stage

59 hours after peak stage

Groundwater Profiles

Tutor-Time Child Care Facility

Figure: 61

5.2.3 Fisherman's Landing

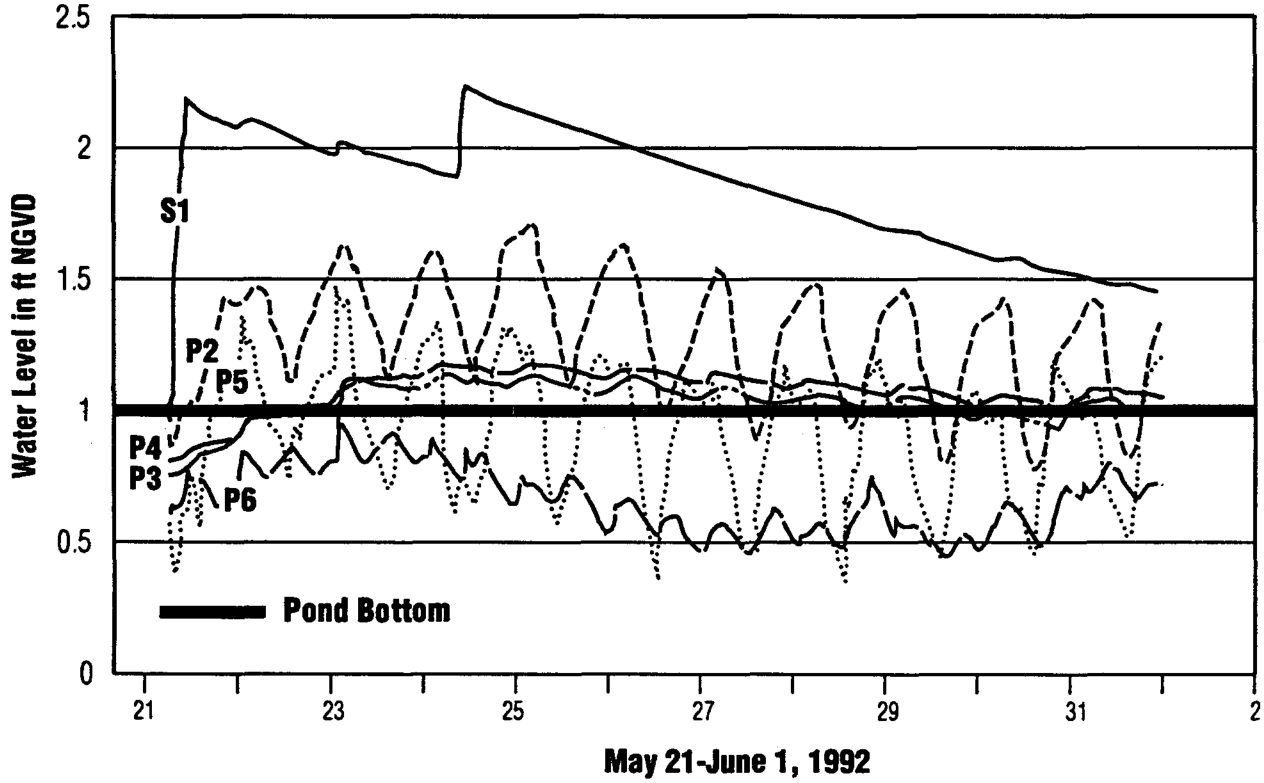
The short-term hydraulic load test at Fisherman's Landing commenced on May 21, 1992 wherein water was pumped from the adjacent Indian River into the pond. The pond was filled in a period of approximately 2 hours. The pond was allowed to recover for 3 days (i.e., the current SJRWMD recovery time crieria for the pollution abatement volume) and a similar slug of water was introduced into the pond again on May 24. Recovery from this second loading was monitored until June 1. The water level readings during this test are shown on Figure 62. Note that during the recovery following the second introduction of the slug volume, little or no rainfall occured.

From review of the monitoring data for this short-term load test, the following key observations can be noted:

- Recovery Time: The peak stage achieved during the final loading on May 24, 1992 was +2.2 ft NGVD. The pond water level dropped 0.8 ft to an approximate elevation of +1.4 ft NGVD in about 8 days.

- Hardpan As A Confining Layer: The borings identified an upper zone of fine sand underlain by "hardpan" type soils, which in turn are underlain by a more transmissive zone of silty fine sand with shell. The effective screened interval of piezometer P-2 extends below the hardpan layer into the subjacent silty fine sand with shell zone. Unlike the response in P-2, the water level in the pond (S-1 on Figure 62) does not show tidal fluctuation which suggest that the pond water is not connected to the lower transmissive zone and therefore the hardpan acts as an effective confining unit.

- Influence of Tidal Fluctuation on Groundwater Levels: Piezometers P-2 and P-5 show daily groundwater fluctuations on the order of 0.6 to 0.7 feet due to tidal movement in the adjacent Indian River. At the other extreme, there was no perceptible fluctuation in the pond water level (S-1) as a result of tidal fluctuation. Piezometers P-3 and P-4 show only very slight effects of tidal movement on groundwater fluctuation in the uppermost aquifer. There is no obvious reason why piezometer P-6, which is closer to the river and screened in the same zone of the uppermost aquifer, did not show a similar amplitude of tidal fluctuation as P-5.



2.5
2
1.5
1
0.5
0
Water Level in ft NGVD
S1
P2
P5
P4
P3
P6
Pond Bottom
21
23
25
27
29
31
2
May 21-June 1, 1992

Figure: 62

- General Trends in Groundwater Levels: If the oscillation in groundwater levels due to tidal fluctuation is ignored, the following general trends in groundwater levels were observed in response to the load test:

Piezometer	Observed Trend
P-2	water table rose only about 0.15 ft following the slug loading (which increased the pond level by 0.8 ft) and declined gradually to its preloading level after 8 days. The response in this well appears to be controlled more by the water level in the underlying, more transmissive silty fine sand with shell zone.
P-3 and P-4	The water levels in these two piezometers follow the same trend. After the first slug of water was introduced into the pond on May 21, the water levels in P-3 and P-4 rose about 0.3 feet. Some of this rise is attributable to the two intervening rainfall events. Following the introduction of the second slug of water on May 24, 1992, groundwater levels at these two piezometers rose very slightly and also receded only very slightly during the recovery period.
P-5 and P-6	The groundwater levels at these wells did not show any perceptible mound buildup due to the loading in the pond.

5.2.4 Tom Statham Park

Figures 63 through 73 show line graphs of the measured water levels and the corresponding daily rainfall data in bar chart form for the Tom Statham Park for each month of the 11-month monitoring period spanning November 1991 through September 1992. Equipment malfunction resulted in missing data for certains wells or the rain gauge and these period of missing data are noted at appropriate points on Figures 63 through 73. The data for the calendar year 1992 is considered very reliable.

From review of the monitoring data, the following key observations are made:

- Rainfall: During the monitoring period, the wettest month was June 1992 with 7.8 inches of rainfall. The wettest day of the monitoring period also occured in June with 2.9 inches of rain on June 6. The next wettest day was April 22, 1992 with 2.41 inches of rainfall. This rainfall distribution is typical of a normal wet year in this area. Note that the rain gauge was not functional for most of August (Figure 72).

- Perched Surface Water/Groundwater Table: The most conspicious aspect of the data is that the water level in the pond (as measured by S-1) is always above the adjacent water table as measured in P-4 and P-5. Even after extended periods of little or no rainfall [see April 12, 1992 for example (Figure 68)], the water level in the pond is perched 1.5 to 2 feet above the adjacent groundwater level in P-4 and P-5. Two possible explanations for this observation are:

    i) since this particular pond bottom was not cleaned of herbaceous wetland type vegetation or scarified prior to the start of hydrologic monitoring, pond bottom sediments may be acting as a natural liner hindering infiltration and mitigating drawdown; and

    ii) the piezometers P-4 and P-5 are screened to depths of 12 to 15 feet below grade and may have penetrated the dense "hardpan" zone into the more transmissive underlying layer of fine sand with broken shell (see soil profile for TB-1 on Figure 45) where the water levels do not reflect perching above the hardpan and may be dewatered to some extent by the adjacent ditch.

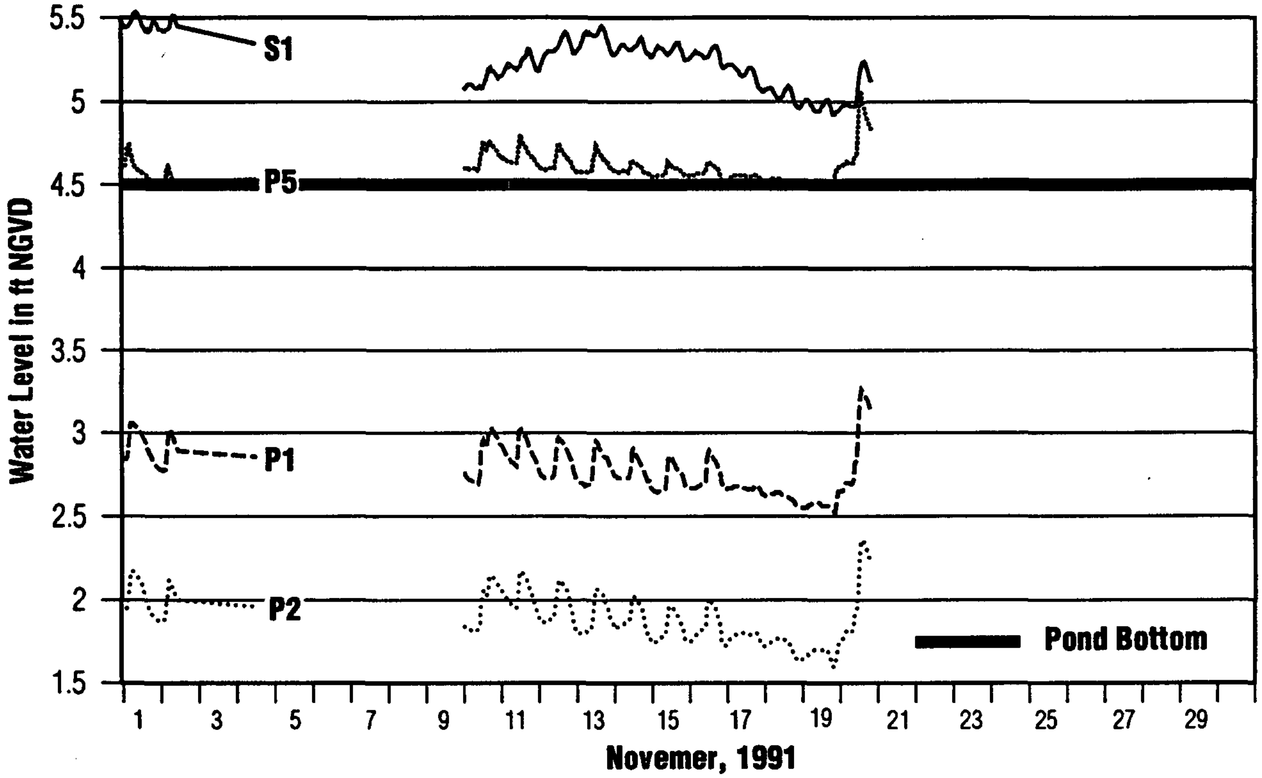
- Tidal Influence on Groundwater Levels: All of the measured water levels fluctuate daily as a result of tidal influence from the adjacent Indian River. Piezometers P-1, P-2, and P-3 show about the same range of daily fluctuation (0.45 feet), while, just perceptible on the plots, the range of fluctuation in the piezometers further inland (P-4 and P-5) is much less pronounced. Careful review also shows an almost imperceptible tidal influence on the pond water levels.

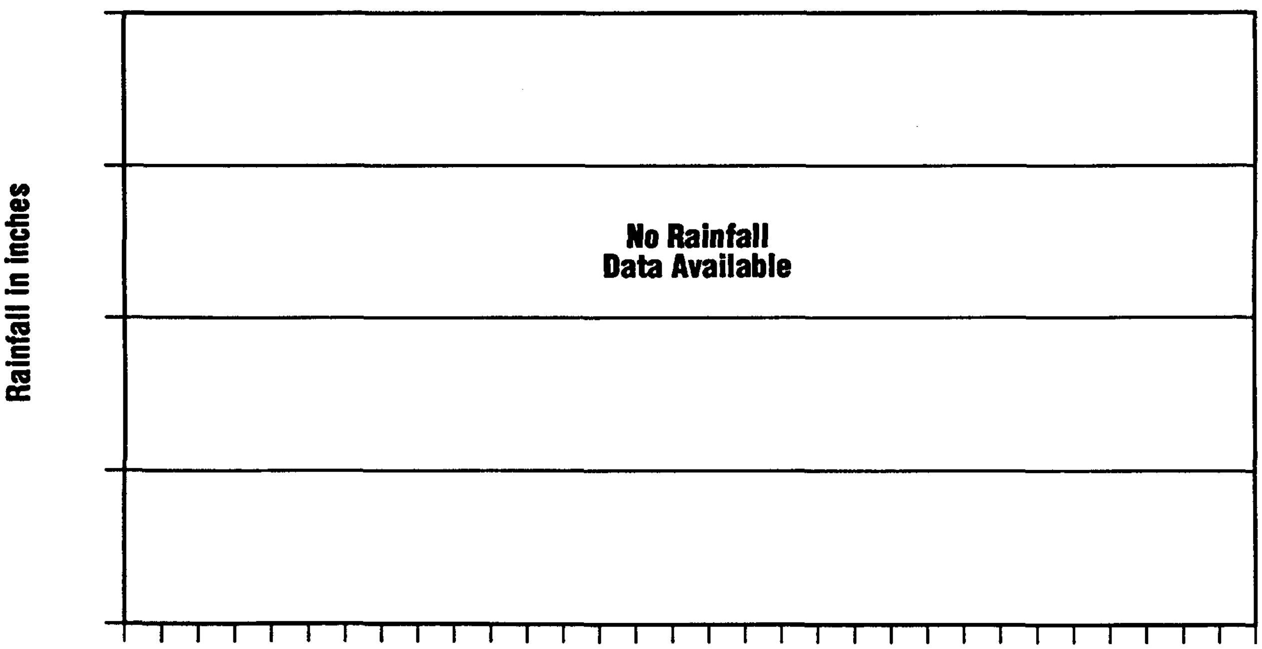
- Rainfall Influence on Groundwater Levels: The groundwater levels rise primarily with rainfall [see for example the noticeable spikes on June 6 (Figure 70)] and decline fairly rapidly, especially P-4 and P-5 which are adjacent to the ditch.

- Rate of Recovery of Pond Stage/Volume: Pond recovery is slow. For example, following the staging to +5.22 ft NGVD on April 23, 1992 (Figure 68), the pond water level dropped 0.65 feet to +4.57 ft NGVD in about 2 weeks (May 7, 1992, Figure 69). There was little or no rainfall during this recovery period and the evapotranspiration in this herbaceous wetland/pond probably accounted for 0.28 feet of this 0.65 foot decline [note: Dolan et al. (1984) reports that the evapotranspiration for a freshwater marsh in central Florida is approximately 6.8 inches for the month of May].

  Mounding effects in the adjacent water table due to recharge within the pond is not noticeable.

The digital printouts of the hydrologic monitoring data is presented in Volume II of this report.

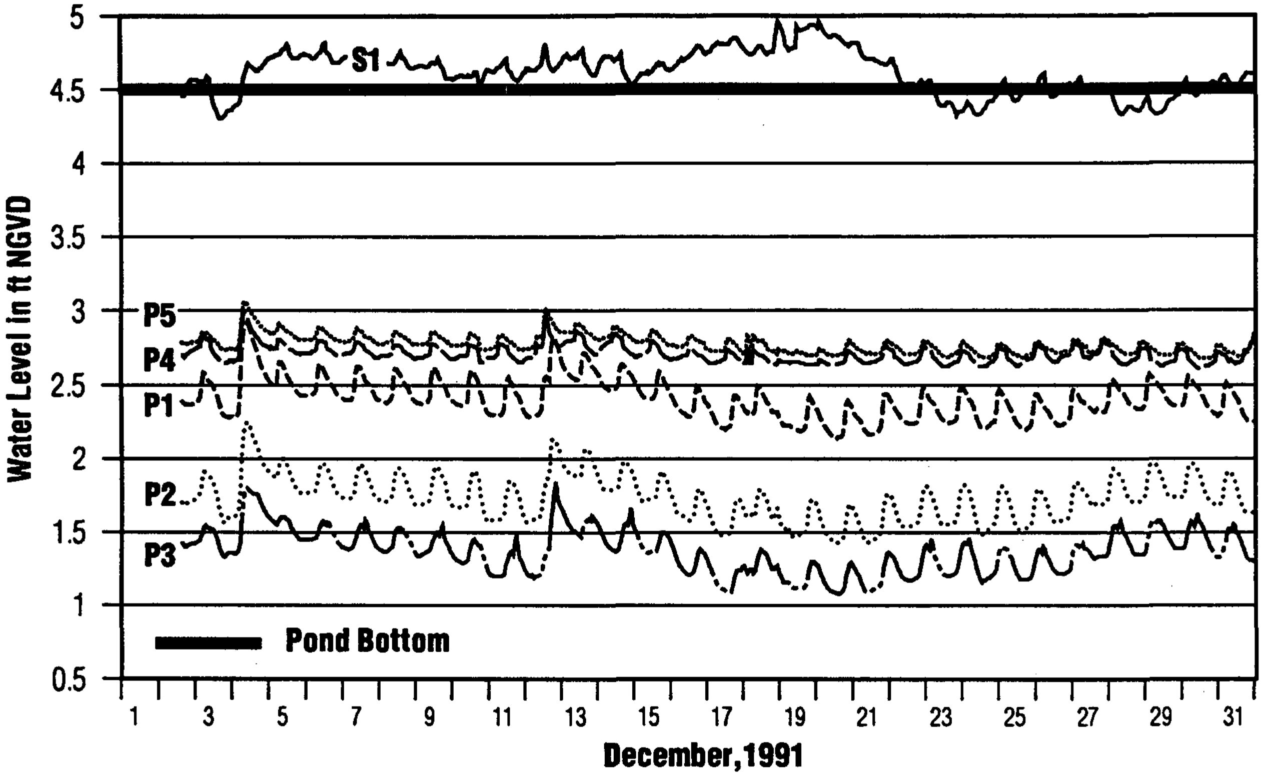


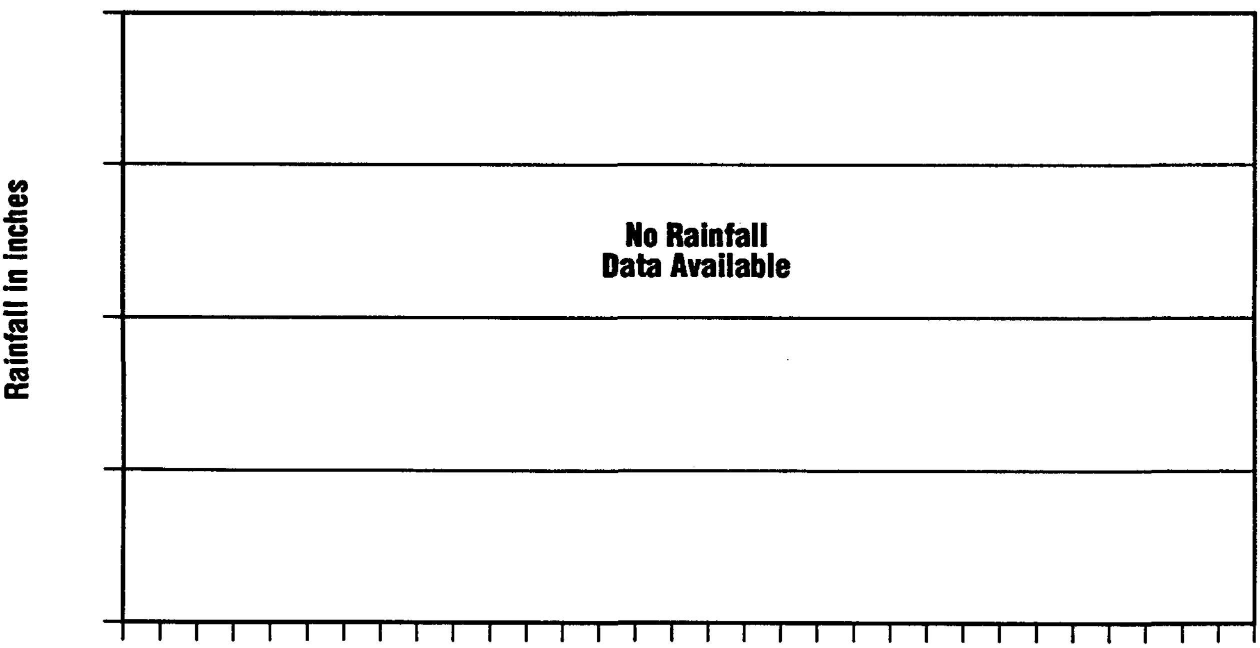


Note: Gaps in line graphs represent periods of equipment malfunction

Hydrologic Monitoring Data
November-1991
Tom Statham Park

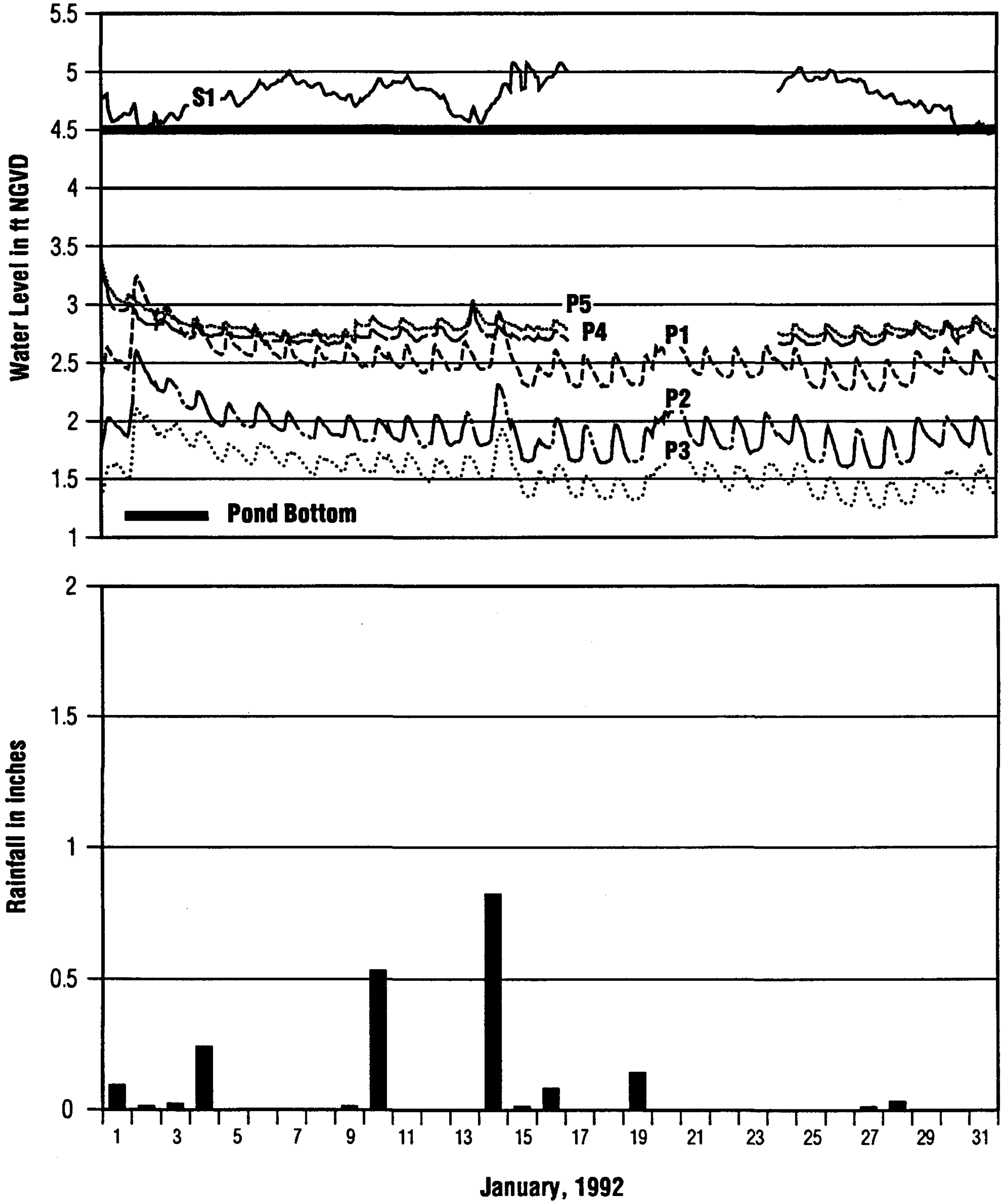
Figure: 63





Hydrologic Monitoring Data
December-1991
Tom Statham Park

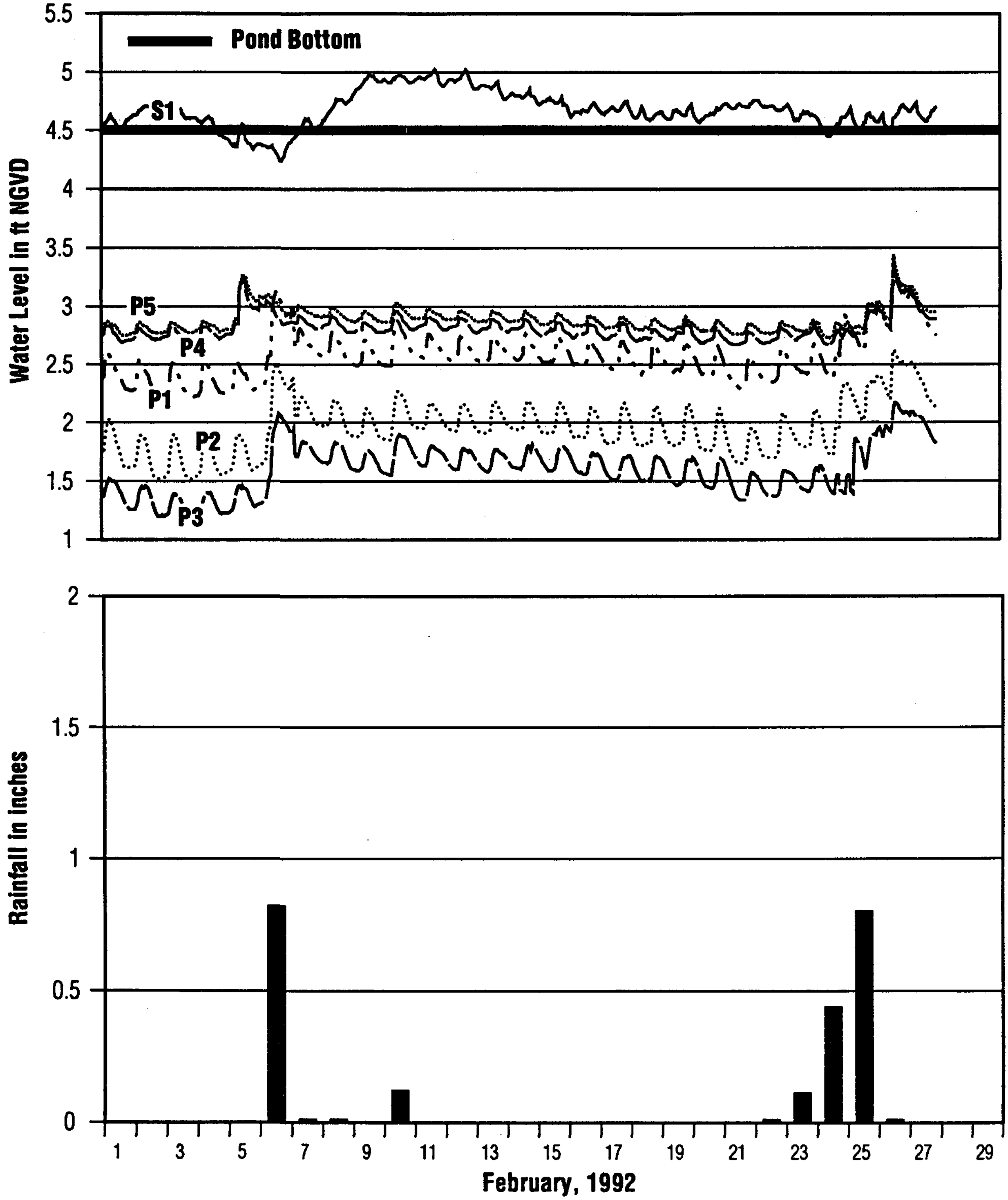
Figure: 64



Note: Gaps in line graphs represent periods of equipment malfunction

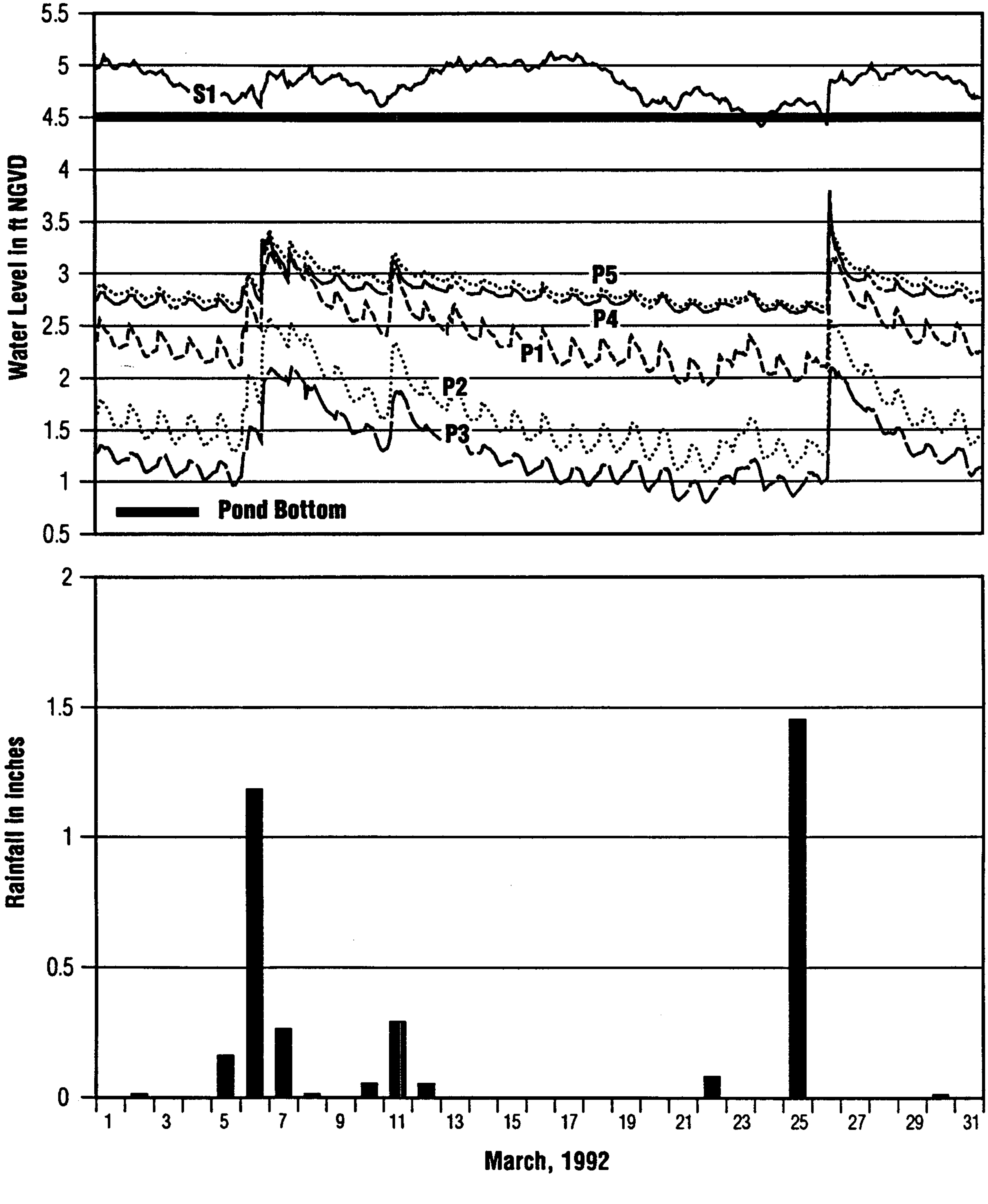
Hydrologic Monitoring Data
January-1992
Tom Statham Park

Figure: 65



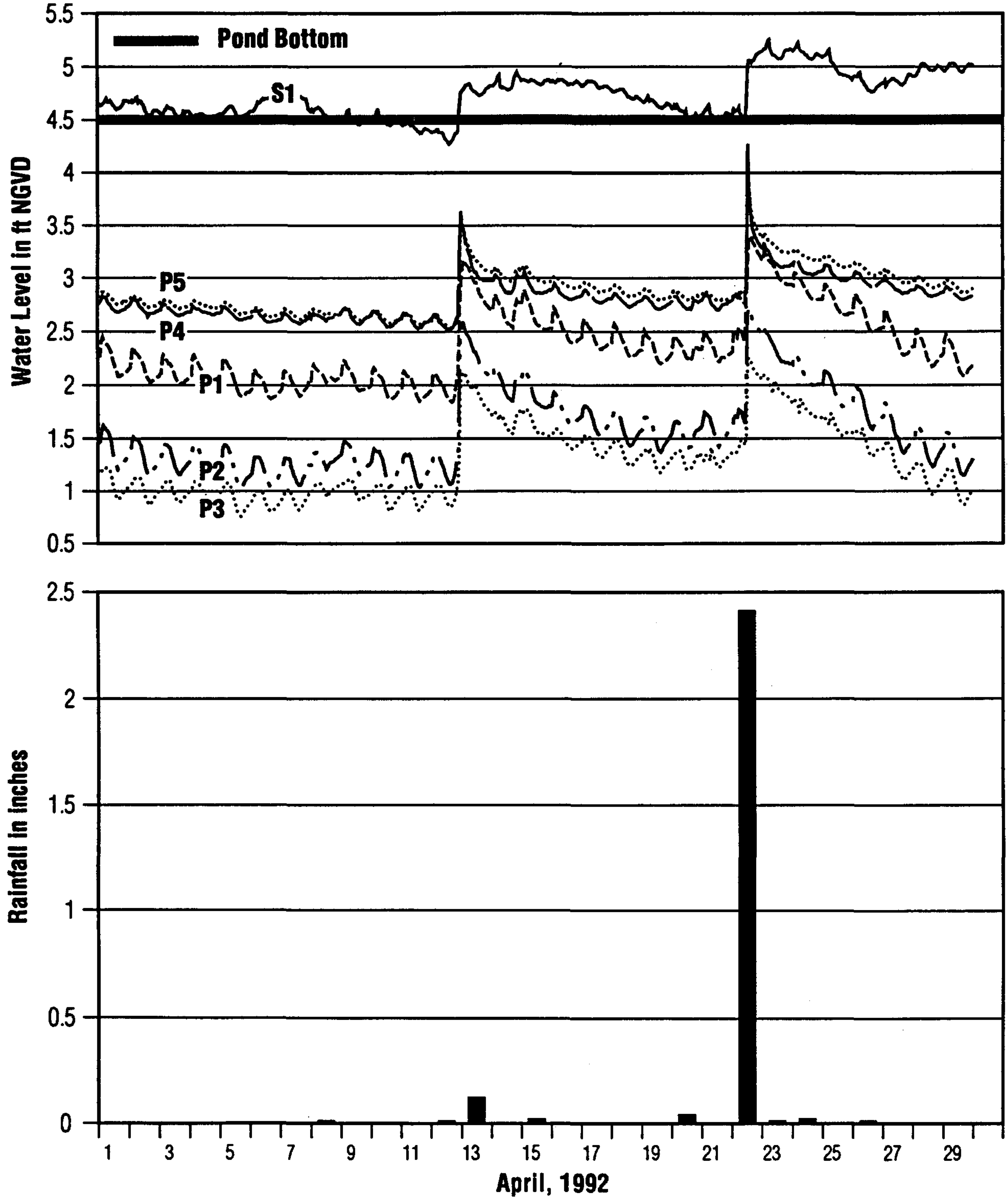
Hydrologic Monitoring Data
February-1992
Tom Statham Park

Figure: 66



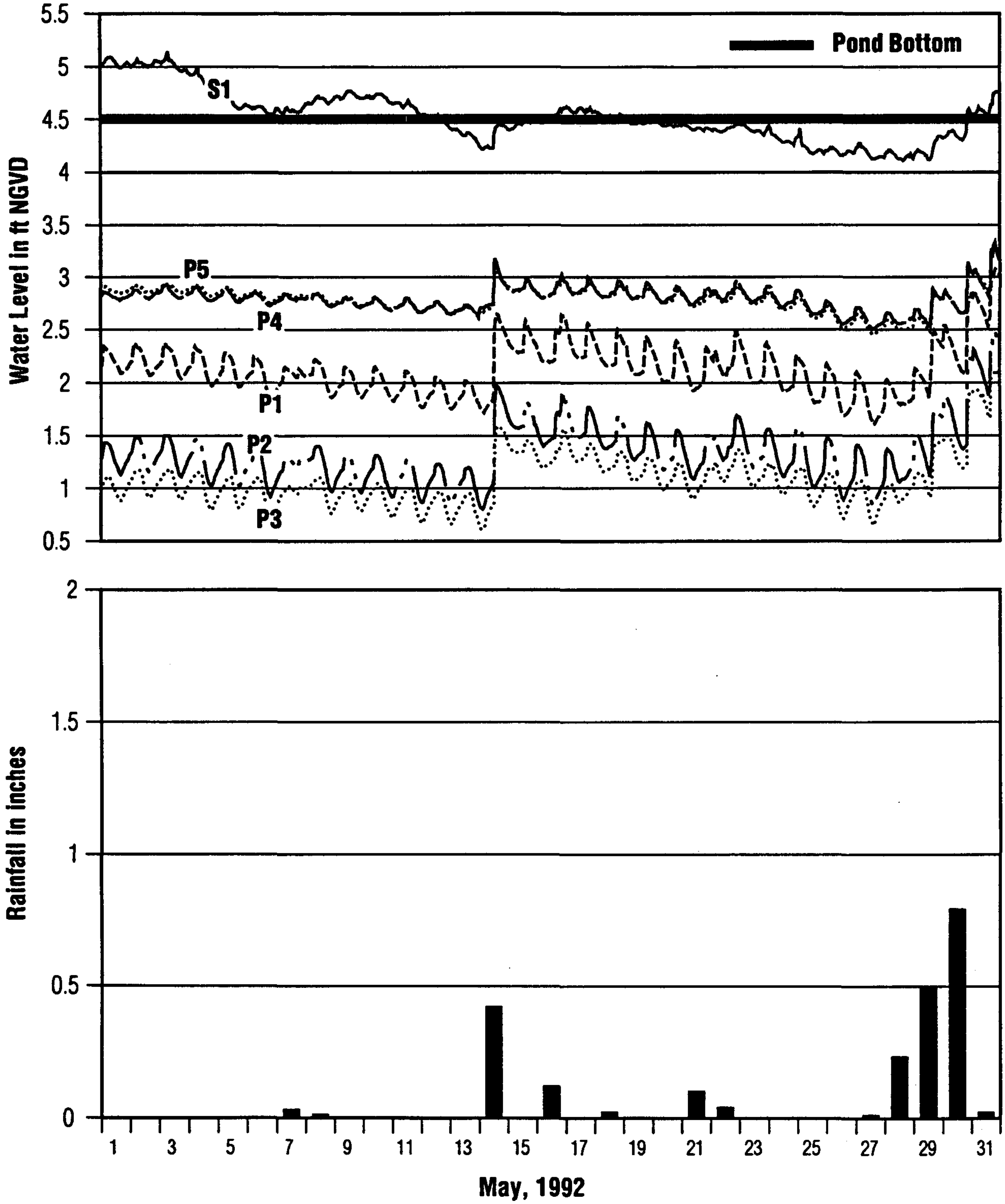
Hydrologic Monitoring Data
March-1992
Tom Statham Park

Figure: 67



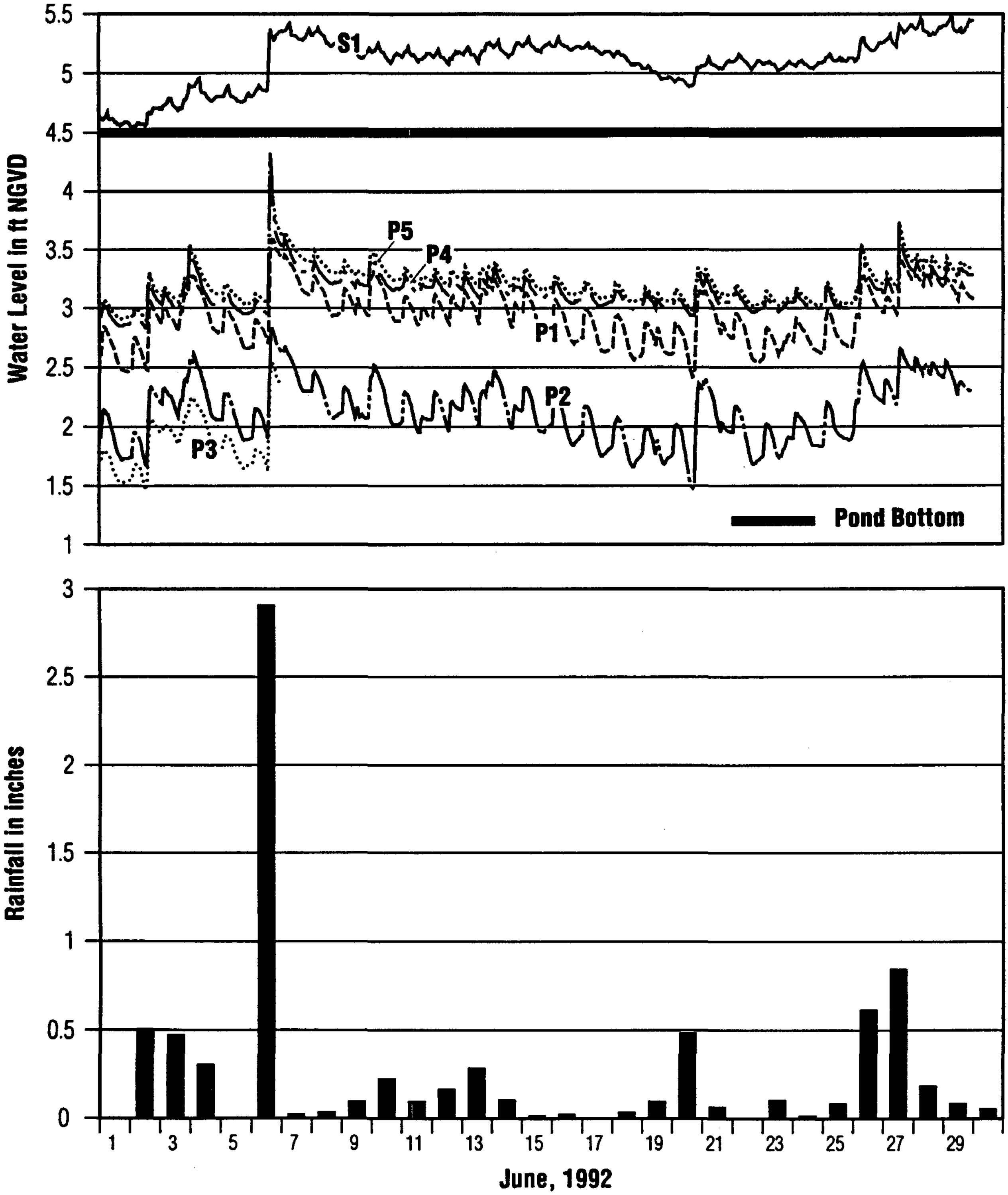
Hydrologic Monitoring Data
April-1992
Tom Statham Park

Figure: 68



Hydrologic Monitoring Data
May-1992
Tom Statham Park

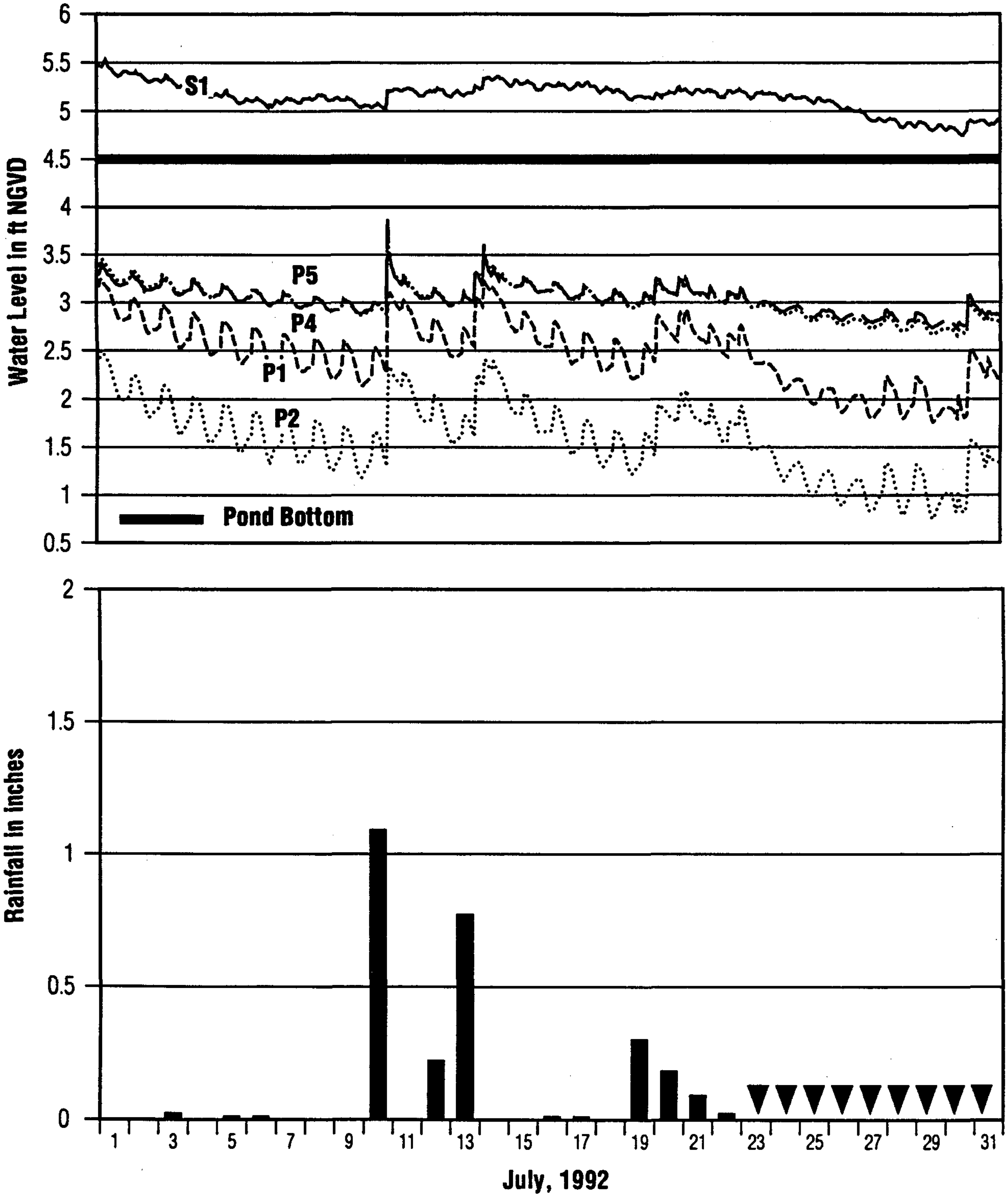
Figure: 69



Note: Gaps in line graphs represent periods of equipment malfunction

Hydrologic Monitoring Data
June-1992
Tom Statham Park

Figure: 70

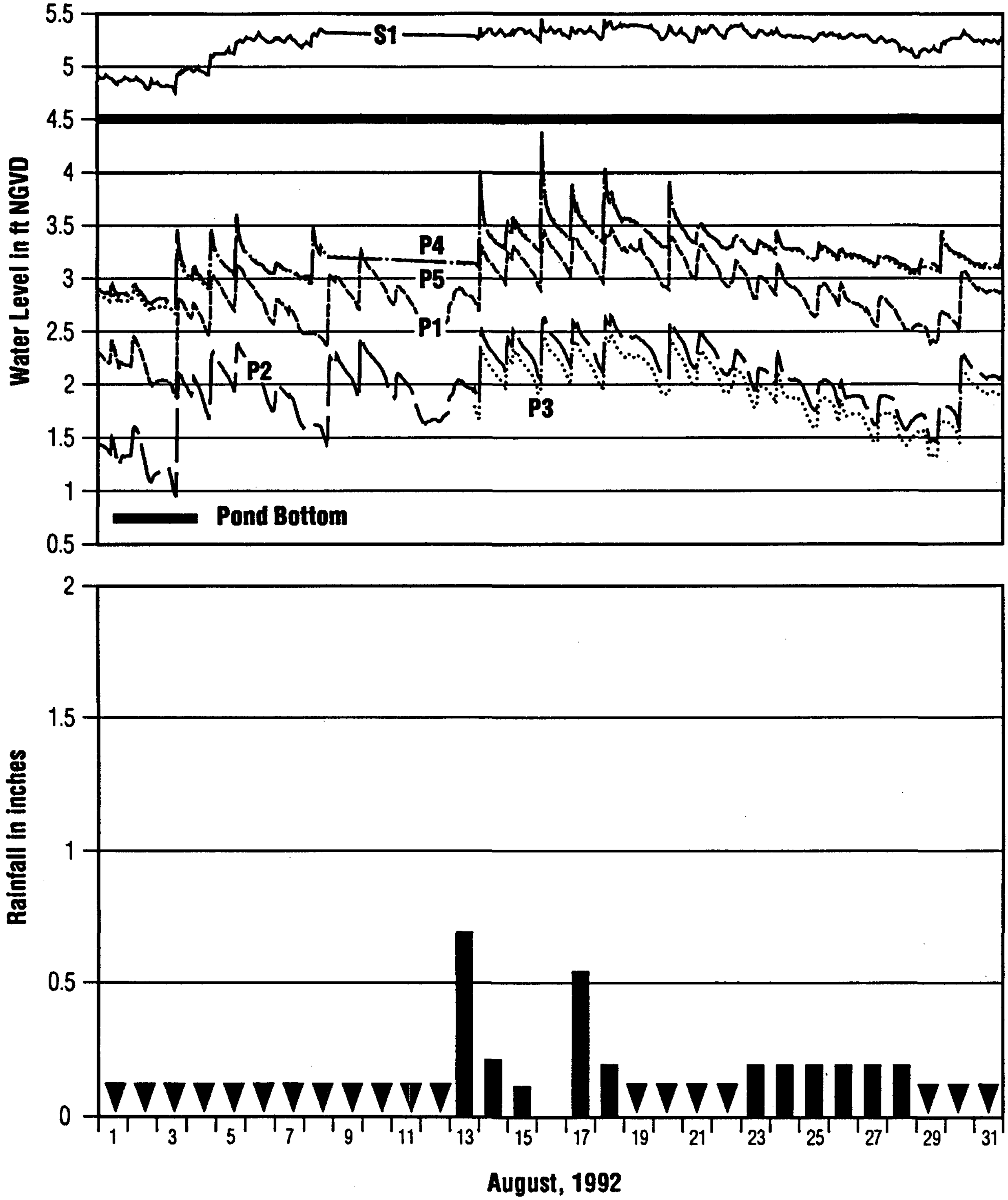


Note: Gaps in line graphs represent periods of equipment malfunction

▼ No rainfall data available for this date due to equipment malfunction

Hydrologic Monitoring Data
July-1992
Tom Statham Park

Figure: 71

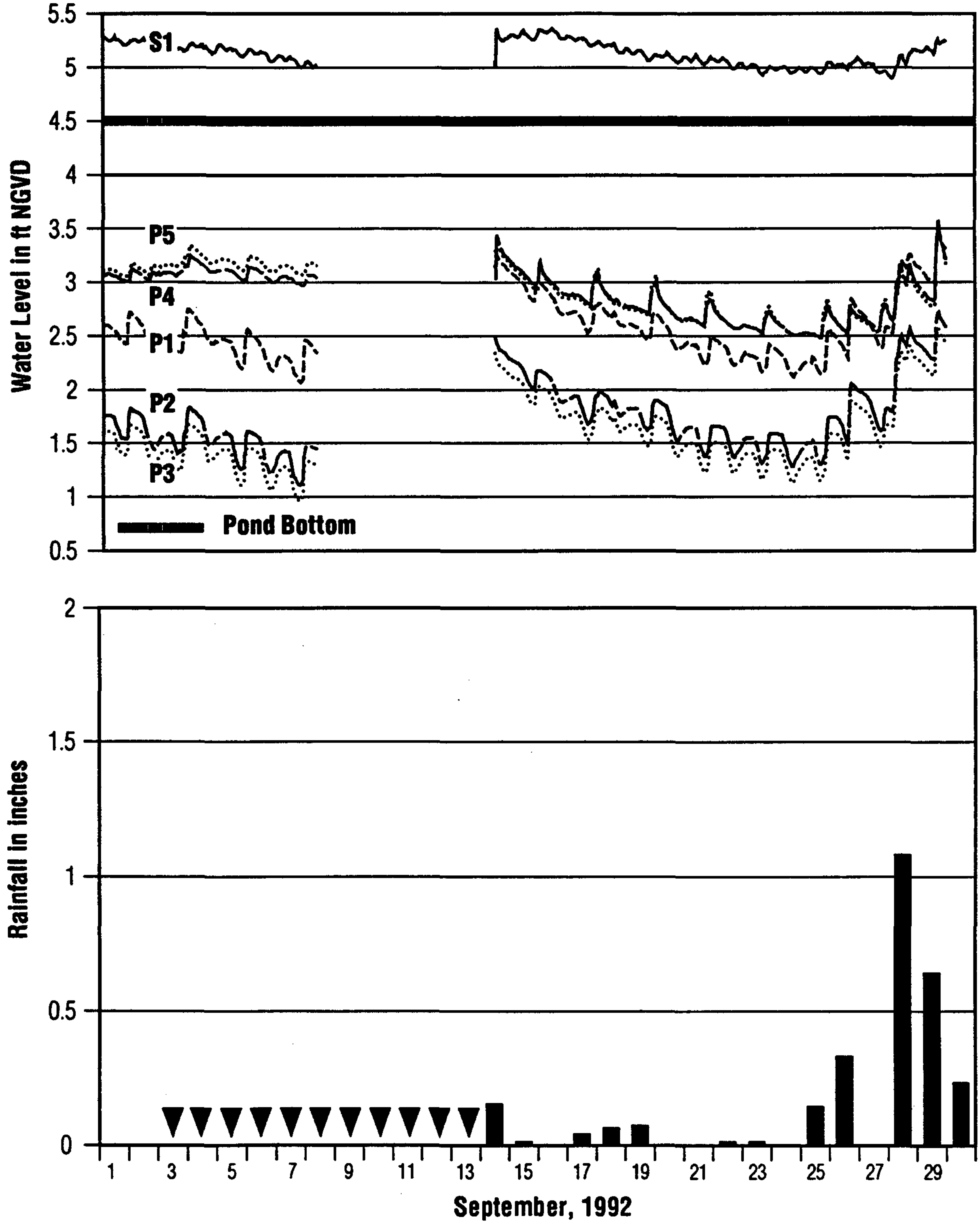


Note: Gaps in line graphs represent periods of equipment malfunction

▼ No rainfall data available for this date due to equipment malfunction

Hydrologic Monitoring Data
August-1992
Tom Statham Park

Figure: 72



Note: Gaps in line graphs represent periods of equipment malfunction

▼ No rainfall data available for this date due to equipment malfunction

Hydrologic Monitoring Data
September-1992
Tom Statham Park

Figure: 73

SECTION 6.0: COMPARISON OF MEASURED AND PREDICTED RESPONSE

6.1 GENERAL

The purpose of this evaluation is to compare the capabilities of selected mathematical saturated flow models to predict the observed drawdown of the pool elevations in the instrumented stormwater retention ponds. Since volumetric flow rates into the ponds were not measured, the evaluation is restricted to modeling recovery of pond water levels after significant rainfall events.

Aquifer parameters for input in the groundwater flow models are estimated from the site-specific field and laboratory geotechnical tests and the recovery analyses are performed for the set of hydraulic conductivity values which correspond to the results of the field and laboratory tests. The recovery times predicted by the models are compared to the measured recovery times. This comparison provides some insight into the applicability, reasonableness, and sensitivity of the models and the representativeness of the measured hydraulic conductivity values.

6.2 BRIEF DESCRIPTIONS OF SELECTED SATURATED GROUNDWATER FLOW MODELS

6.2.1 Background on Selected Models

Five groundwater flow models that are commonly used in Florida were selected for this study, namely

Model	Description	Reference(s)
#1	Simplified Analytical Method	See Figure 74
#2	Glover's Line Source Theory	Glover (1974); Figure 75
#3	MODRET	Andreyev (Jammal & Assoc., 1989); Figure 76
#4	PONDFLOW	Kuhns (1990)
#5	Hantush Equation	Hantush (1967); Figure 77

This selection of pond recovery models is representative of the current state of the geotechnical engineering practice in Florida, except for the simplified analytical model (Model #1) which was developed for this study. Concise descriptions of each of the models are presented in subsequent sections. All of the chosen models, except for Model #1, are well documented in the references cited above and are therefore not described in detail in this report. These models are all similar in that the receiving aquifer system is idealized as a laterally infinite, single-layered, homogenous, isotropic water table aquifer of uniform thickness, with a horizontal pre-loading phreatic surface. The three dimensional shape of the pond is assumed to be that of a rectangular trench. Unlike the other models, the PONDFLOW and MODRET models both have the additional capabilities to:

i) simulate unsaturated vertical flow prior to saturated lateral flow, and

ii) input a runoff hydrograph to simulate infiltration from the pond during the storm event.

6.2.2 Model #1 - Simplified Analytical Method

Figure 74 depicts the basic elements of this simplified theory of transient saturated infiltration from a retention pond excavated into a homogenous water table aquifer. The key assumptions of the simplified analytical model are as follows:

- Darcy's Law is the governing equation for saturated ground water flow.

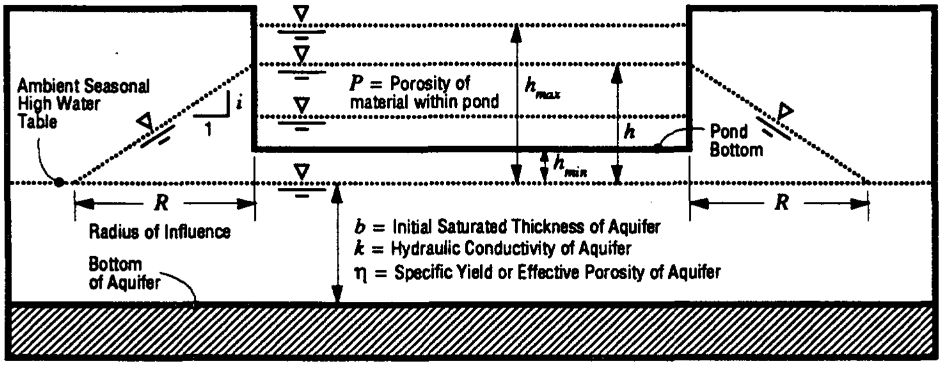
- Dupuit-Forchheimer assumptions are applicable; i.e.,
  1. Flow is considered to be purely horizontal
  2. Flow is assumed to be uniformly distributed with depth

- The moving zone of saturation (or transitory ground water mound) is idealized as a series of triangular prisms adjacent to the pond perimeter. At the corners of the rectangular pond, the triangular prisms assume the shape of a quadrant of a solid cone. The lateral extent of the mound (or radius of influence) increases as recovery progresses.

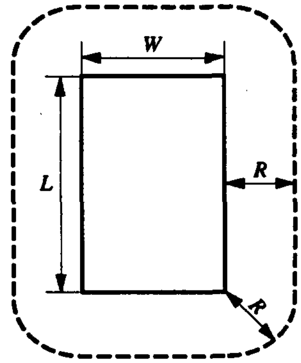
- From the law of conservation of mass, it is assumed that the volume of water which infiltrated out of the pond during this short duration event is equal to the volume of water in soil storage in the triangular saturated prism at any instant.

A similar approach is described by Cedergren (1977), using transient flow nets, to study the spread of water into an unconfined aquifer from the sudden rise of rivers in flood stage. This model also provides a reasonable estimate of the radius of influence during recovery. Using the above assumptions, the recovery time can be solved for as shown on Figure 74. The integral for recovery time may be solved numerically or using commercially available software such as Mathcad. A Mathcad file for solving this equation has been provided to SJRWMD as part of this research project and a hardcopy of the data file is presented in Appendix B.

The simplified analytical model is somewhat conservative since it assumes that, for a prescribed runoff volume, the rise in the pond stage occurs instantaneously and there is no credit for seepage during the storm event.



Profile View



Plan View

Required to find: Time for recovery from h_{max} to h_{min}

Solution: Assumes the volume that infiltrates the aquifer fills a triangular wedge above the water table, adjacent to the pond perimeter. For volume balance, therefore:

Volume Recovered from Pond = Volume in saturated triangular prism adjacent to pond and conical fans around edges

$$PLW\,(h_{max} - h) = \eta\, h \left\{ R\,(L + W) + \frac{\pi}{3} R^2 \right\} \tag{1}$$

Solving equation (1) for radius of influence:

$$R = \frac{\left\{ (L + W)^2 + \frac{4\pi}{3} \frac{PWL}{\eta} \frac{(h_{max} - h)}{h} \right\}^{1/2} - (L + W)}{\frac{2\pi}{3}} \tag{2}$$

$$\text{Therefore gradient } i = \frac{h}{R} = \frac{\frac{2\pi}{3} h}{\left\{ (L + W)^2 + \frac{4\pi}{3} \frac{PWL}{\eta} \frac{(h_{max} - h)}{h} \right\}^{1/2} - (L + W)} \tag{3}$$

$$\text{Seepage Face Area } A = (h + b)\,(2L + 2W) \tag{4}$$

$$\text{From Darcy's Law: Infiltration Rate } q = k\,i\,A = \frac{\frac{4\pi}{3} k\,(L + W)\,h\,(h + b)}{\left\{ (L + W)^2 + \frac{4\pi}{3} \frac{PWL}{\eta} \frac{(h_{max} - h)}{h} \right\}^{1/2} - (L + W)} \tag{5}$$

Incremental recovered volume
$PWL\,dh = q\,dt;$

$$dt = \frac{PWL}{q}\,dh$$

$$\text{Recovery Time } t = \int_{h_{min}}^{h_{max}} PWL \frac{\left[\left\{ (L + W)^2 + \frac{4\pi}{3} \frac{PLW}{\eta} \frac{(h_{max} - h)}{h} \right\}^{1/2} - (L + W) \right]}{\frac{4\pi}{3} k\,h\,(L + W)\,(h + b)}\,dh \tag{6}$$

Simplified Analytical Method

6.2.3 Model #2 - Glover's Line Source Theory

South Florida Water Management District Technical Publication 87-5 (SFWMD, 1987) describes the implementation of Glover's line-source theory (Glover, 1974) to model exfiltration from subsurface trenches. Since the mechanics of the infiltration process from exfiltration trenches is similar to that from retention ponds, it is appropriate to compare the results of this methodology in this research effort.

Figure 75 presents the derivation of the recovery time using the line-source theory. The resulting equation for recovery time is closed form.

6.2.4 Model #3 - MODRET

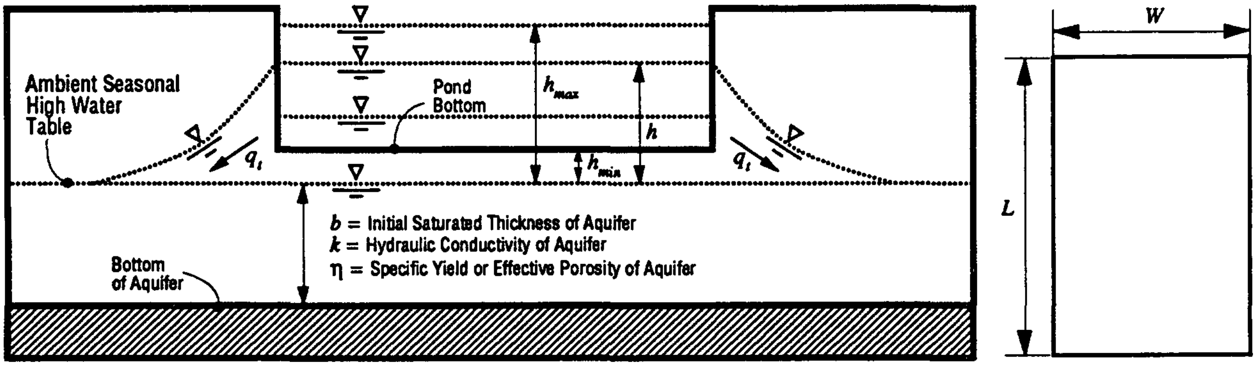
MODRET is currently the most popular computer program used in the St Johns River Water Management District for stormwater retention pond infiltration analysis. This computer program was developed by Andreyev (Jammal & Associates 1989) for the Southwest Florida Water Management District.

The saturated analysis module of MODRET is essentially a pre- and post-processor for the USGS finite difference groundwater flow model MODFLOW (McDonald and Harbaugh, 1984). The MODRET pre-processor also has the capability to calculate the unsaturated flow using the modified Green and Ampt (1911) equation. Unsaturated flow takes place prior to the groundwater mound intersecting the pond bottom.

Input parameters in the pre-processor includes the following:

<u>Saturated Analysis</u>

- The length and width of a rectangular pond which fully penetrates a laterally "infinite" homogenous isotropic water table aquifer.
- The base elevation of the unconfined (water table) aquifer and the initial water table altitude (assumed flat)
- The hydraulic conductivity and fillable porosity (or specific yield) of the homogenous aquifer.
- Time varying vertical recharge to the pond and to the aquifer outside the pond (if applicable). Time variation is discretized into a number of computational stress periods.



Profile View   Plan View

Required to find: Time for recovery from h_{max} to h_{min}

Glover's Line Source Theory:

$$q_t = \frac{\pi^{1/2} k\, b\, h}{\left(\frac{k\, b}{\eta}\right)^{1/2} t^{1/2}} (W + L) \tag{1}$$

At time t, $$WL\, dh = (W + L)\, q_t dt = \frac{\pi^{1/2} k\, b\, (W + L)}{\left(\frac{k\, b}{\eta}\right)^{1/2}} \frac{h}{t^{1/2}} dt \tag{2}$$

Rearranging equation (2)

$$\frac{dh}{h} = \frac{\pi^{1/2} k\, b\, (W + L)}{W\, L\, \left(\frac{k\, b}{\eta}\right)^{1/2}} t^{-1/2} dt \tag{3}$$

$$Let\; F = \frac{W\, L \left(\frac{k\, b}{\eta}\right)^{1/2}}{\pi^{1/2} k\, b\, (W + L)} \tag{4}$$

Therefore, $$F \frac{dh}{h} = t^{-1/2} dt \tag{5}$$

Integrating equation (5) gives:

$$\int_{o}^{T} t^{-1/2} dt = F \int_{h_{min}}^{h_{max}} \frac{dh}{h} \tag{6}$$

$$2\, T^{1/2} = F\, ln \left(\frac{h_{max}}{h_{min}}\right) \tag{7}$$

Recovery Time $$T = 1/4 \left\{F\, ln \left(\frac{h_{max}}{h_{min}}\right)\right\}^2 \tag{8}$$

Glover's Line Source Theory

Figure: 75

Unsaturated Analysis

- The length and width of the pond bottom.
- The separation distance between the pond bottom and the groundwater table.
- Unsaturated vertical hydraulic conductivity.
- Fillable porosity.

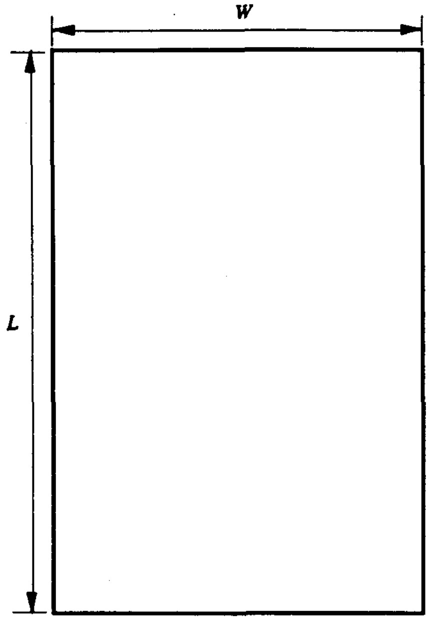
The input parameters in the MODRET pre-processor are then used to create MODFLOW input files. After the MODFLOW program is executed, the MODRET post-processor extracts and prints the relevant information from the MODFLOW output file including the pond stage, infiltration rate, and groundwater profiles at the end of each stress period.

Since the MODRET program allows the user to input time-varying recharge (such as a discretized inflow hydrograph for a 24 hour storm), it allows the calculation of saturated seepage out of the pond during recharge (i.e., during the storm event).

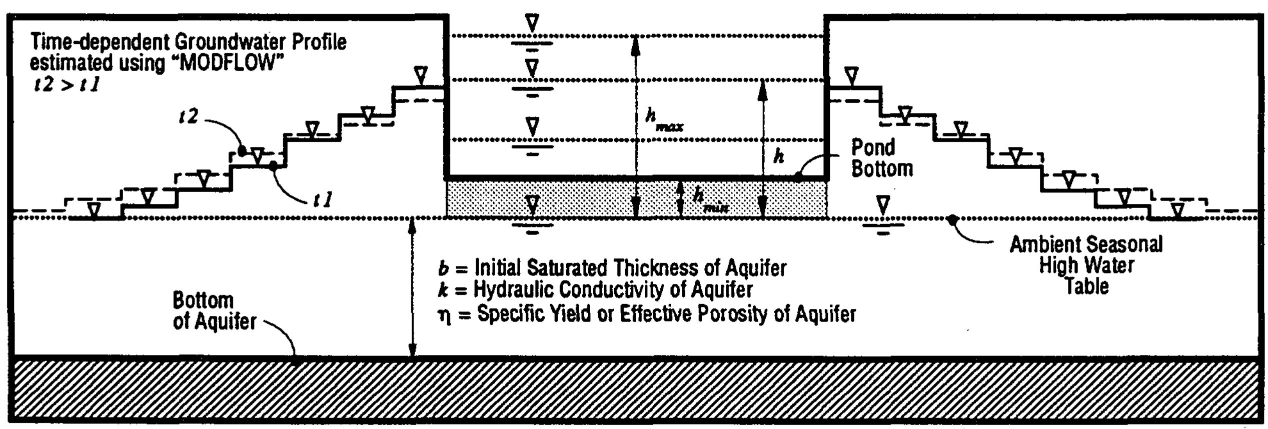
Figure 76 is a representation of the saturated recovery analysis assumptions for ponds using MODRET.

6.2.5 Model #4 - PONDFLOW

PONDFLOW is a stormwater pond recovery analysis developed by Kuhns (1990). It is similar to MODRET in that it uses a finite difference numerical technique (Prickett and Lonnquist, 1971) to approximate the time varying groundwater profile adjacent to the pond. Also, like MODRET, it can accommodate a time-varying recharge to the pond and account for seepage during the storm and it also allows for unsaturated flow.



Plan View



Zone assumed saturated

Profile View

Schematic Representation of "MODRET" Assumptions for Saturated Infiltration Analysis

Figure: 76

6.2.6 Model #5 - Hantush Equation

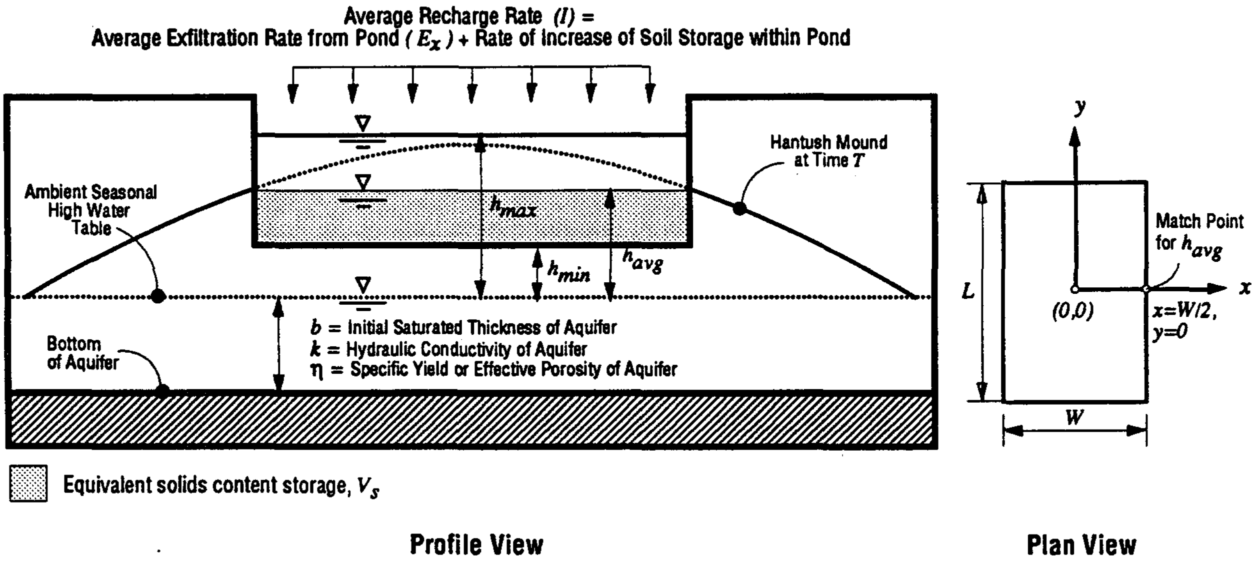
The original Hantush (1967) equation was developed to predict the rise and fall of a groundwater mound at any point of an X and Y coordinate system beneath a rectangular recharge basin. This equation is not ideally suited for modeling the recovery of stormwater retention ponds for a number of reasons including the following:

1. The Hantush equation assumes that the slope of the phreatic surface is small and therefore the streamlines are horizontal and the equipotentials are vertical (i.e., the Dupuit assumption). This assumption is violated in most retention ponds since the recharge rate is usually high during and following a design storm event.

2. The Hantush equation does not account for the hydraulic conductivity and effective porosity of the open space within the pond.

3. The Hantush equation loses validity when the groundwater mound height exceeds 50% of the initial saturated aquifer thickness.

In spite of these limitations, this "slow, uniform recharge rate" equation has been implemented to predict the volume recovery of retention ponds. Based on correlations with model tests and full-scale field tests on retention ponds with relatively high recharge rates, Andreyev (1985) has demonstrated that the exfiltration rate can be approximated using Hantush's equation in the manner described on Figure 77. An example of this method of retention pond recovery analysis has more recently been presented by Yovaish (1990).



Required to find: Time for recovery (T) from h_{max} to h_{min}

Procedure:

(1) As a first trial, assume a recovery time (T) which is used as the time to end of infiltration in the Hantush mounding equation.

(2) Compute the average recharge rate (I)which produces a mound that matches a mound height of h_{avg} at $x = w/2, y = 0$. Note that (I) shall not exceed (k).

(3) Compute the average infiltration rate from the pond (E_x) = recharge rate (I) - rate of increase of soil storage within the pond $E_x = I - \left[\left(\frac{h_{avg} - h_{min}}{T} \right) \eta \right]$

(4) Compute the average recovery rate $\frac{h_{max} - h_{min}}{T}$ with the average exfiltration rate E_x computed in step (3).

(5) Adjust recovery time (T), if necessary, and repeat steps (2) through (4) until a reasonable match is obtained between (E_x) and the average recovery rate. If $\frac{h_{max} - h_{min}}{T}$ is less than (E_x), decrease the recovery time or vice-versa.

Adaptation of Hantush's Equation for Retention Pond Recovery Analysis

Figure: 77

6.3 SELECTION OF MEASURED RECOVERY EVENTS FOR MODELING

In addition to testing the predictive capabilities of the selected models for reproducing the recovery after the short duration load tests at the Tutor Time and Fisherman's Landing ponds, events were extracted for modeling from the data sets for the long-term sites. In selecting these events from Airport Warehouses and Tom Statham Park, the following factors were considered:

1. Rainfall Intensity - a rainfall event of relatively high intensity is considered desirable since it generates a high runoff volume that will stress the pond to near its design level.

2. Preceding and Subsequent Rainfall - an event which is preceded and followed by several days of no rainfall is desirable since, for the purpose of this study, it avoids having to consider a pronounced mounded ambient condition and the potential distortions arising from inflow to the pond during recovery.

3. Equipment Reliability - events where the all of the site-specific data logging equipment was functional was also a very important factor in selecting an event.

Based our review of the hydrologic data and the criteria outlined above, the following events were selected:

Site		Event(s)
Airport Warehouses	1.	March 25 → March 31, 1992: Recovery from stage +7.0 ft NGVD to +6.0 ft NGVD following 0.7 inches of rainfall on March 25.
	2.	June 29 → July 10, 1992: Recovery from stage +8.89 ft NGVD to +6.92 ft NGVD in 10.5 days, following 6.28 inches of rainfall in the period June 26 through June 29.
	3.	August 4 → August 11, 1992: Recovery from stage +6.86 ft NGVD to +6.23 ft NGVD in 7.0 days, following 1.24 inches of rainfall on August 4.
Fisherman's Landing	1.	Load Test on May 24, 1992: Recovery from stage elevation of +2.2 ft NGVD to +1.4 ft NGVD in 7.8 days.

Site		Event(s)
Tutor Time	1.	Load Test on May 4, 1992: Recovery from +6.73 ft NGVD to +5.27 ft NGVD in 4 days.
	2.	Load Test on April 13, 1992: Recovery from +6.50 ft NGVD to +5.20 ft NGVD in 6.81 days.
Tom Statham Park	1.	April 23 → May 12: recovery following 2.41 inches of rain on April 22 ; stage dropped from +5.25 ft NGVD to +4.50 ft NGVD in 19 days.

6.4 HYDROSTRATIGRAPHIC IDEALIZATIONS & RETENTION POND DATA

The receiving aquifer system for each pond was idealized based on the site-specific field and laboratory test data presented in Section 4 of this report. Table 7 presents a summary of the idealized 1-layered water table aquifer parameters; the following points are pertinent to the estimation of these parameters:

- The base of the uppermost aquifer was defined as the depth where either the SPT "N" values increased markedly in the reddish brown "hardpan" type fine sands to slightly silty fine sands or the soil classification was silty sand.
- The ambient water table prior to the selected storm event or the load test was taken as the average water table within the estimated radius of influence of the slug loading in the pond.
- The fillable porosity of the soil was estimated to be 20% in all cases, which is in the range of typical values for poorly graded fine sands in Central Florida. A subsequent section of this report discusses the sensitivity of the computed recovery time to the assumed magnitude of fillable porosity.

Table 7: Retention Pond Data & Hydrostratigraphic Idealizations

PARAMETER DESCRIPTION	UNIT	Airport Warehouse	Tutor Time	Fisherman's Landing	Tom Statham Park
A. POND GEOMETRY:					
A.1 Equivalent Pond Width	ft	40	20.5	55	120
A.2 Equivalent Pond Length	ft	53	48.5	120	170
A.3 Pond bottom elevation	ft NGVD	6	4.8	1	4.5
A.4 Discharge elevation	ft NGVD	None	6.73	2.2	5.5
B. AQUIFER PARAMETERS:					
B.1 Base of Aquifer elevation	ft NGVD	3.5	1.5	-12.0	0.0
B.2 Hydraulic Conductivity Values:					
Laboratory falling head - horizontal	ft/day	30	36	8	15
Laboratory falling head - vertical	ft/day	0.3	33	1	7
Fig. 28 (falling head)	ft/day	14	18	5	0.5
Fig. 27 (falling head)	ft/day	4	1	2	1.5
Fig. 28 (constant head)	ft/day	39	34	11	2
Fig. 27 (constant head)	ft/day	4	1	2	0.5
Fig. 29 (open hole - constant head)	ft/day	39	32	7	15
B.3 Fillable porosity	%	20%	20%	20%	20%
B.4 Double Ring Infiltration Rate	ft/day	33.8	19.5	5.23	0.86
STORM EVENT MODELED:					
Ambient water table	ft NGVD	4.7	4.7	0.8	3.5
Maximum stage	ft NGVD	7.0	6.73	2.2	5.25
Minimum stage	ft NGVD	6.0	5.27	1.4	4.50
Observed recovery time	day	5.30	4.00	7.8	19.00
Volume of water to be recovered	cu. ft.	2,120	1,452	5,280	15,300

6.5 PRESENTATION AND DISCUSSION OF MODEL SIMULATIONS

The model simulations were executed using the aquifer parameters on Table 7 for the recovery events identified in subsection 6.3. Independent simulations were performed for each of the measured hydraulic conductivity values in Table 7, except for the transmissivity values determined from the pump tests since they were not representative of the uppermost aquifer. To facilitate an unbiased comparison of the five selected models, the stormwater volume was assumed to fill the pond instantaneously to the maximum stage, which means that the models allowed for no seepage out of the ponds during the storm event/slug loading.

During our evaluation, it was discovered that the MODRET model was producing unstable MODFLOW solutions when modeling the recovery of some of the sites. This problem generally occurs when one or a combination of the following is true:

- the pond dimensions are relatively large (greater than 100 feet)
- the aquifer is relatively thin (less than 5 feet)
- the hydraulic conductivity is relatively low (less than 5 ft/day)

Upon further review, the MODRET model was modified to correct this instability problem by changing the head change criterion for convergence (in the Strongly Implicit Procedure module) to .001 ft from .01 ft. The original MODRET model with this modification is therefore referred to herein as "Modified MODRET".

Tables 8 through 11 present the computed recovery times using seven hydraulic conductivity values for the five different models. The corresponding measured recovery time is noted on the respective tables. However, before reviewing the results in Tables 8 through 11, the following observations and comments are noteworthy:

A. <u>Hydraulic Conductivity Data</u>

A.1 Fairly comparable hydraulic conductivity measurements were obtained at all sites from the laboratory permeameter test (horizontal sample), the cased hole with uncased or screened extension (constant head and falling head), and the uncased or fully screened auger hole constant head test. There are, however, some unexplained discrepancies between the constant head and falling head tests for the cased hole with uncased or screened extension.

A.2 The cased borehole, bottom flush configuration for determination of k_v gave measured hydraulic conductivity values which were consistently lower than the similar test with the wellpoint filter. This suggests that the subjacent hardpan confining layer influenced the flow regime for this test configuration.

A.3 The pump test was performed over a screened interval which included the transmissive zone of fine sand with shell beneath the hardpan layer. This transmissivity value therefore does not reflect solely the effective uppermost aquifer which receives the water seeping out of the ponds. It was therefore deemed inappropriate to use these high transmissivity values to model pond recovery.

B. Model Assumptions

B.1 The models were set up in a common manner to simulate recovery following the instantaneous introduction of a slug of water into the pond. In reality, water enters the ponds in a few hours and some lateral spread of the groundwater mound takes place during this loading period. The implication here is that the models will predict initial recovery rates which are somewhat faster than the measured rates since some mounding has already occurred when the peak stage is achieved.

B.2 Unsaturated flow is not considered in all cases since the water table was mounded close to or above the pond bottoms prior to the recovery event.

B.3 At Tom Statham Park, water losses due to evapotranspiration were not considered during the 19 days of recovery in April-May, 1992. This pond is an unintentionally created herbaceous wetland and the losses due to evapotranspiration in such an ecosystem is estimated to be .35 feet (Dolan et al., 1984) or 46% of the total stage recovery. In addition, this pond was not scarified or cleared of vegetation prior to the test so bottom sediments and root matter may be acting as a liner, reducing infiltration through the pond bottom and sides.

Table 8: Comparison of Measured & Predicted Recovery Times - AIRPORT WAREHOUSES

Hydraulic Conductivity Test Method	Measured Hydraulic Conductivity (ft/day)	Recovery Times (in days) Predicted by Various Models				
		Simplified Analytical Method	Modified MODRET	Hantush	Glover's Equation	Pond Flow
Laboratory Falling Head - k_h	30	2.2	2.5	1.7	1.9	2.3
Laboratory Falling Head - k_v (Hardpan)*	0.3	219	>200	165	187	214
Cased Hole Falling Head - k_h (Fig. 28)	14	4.7	4.5	3.5	4.0	4.8
Cased Hole Falling Head - k_v (Fig. 27)	4	16.5	20.0	12.5	14.0	15.5
Cased Hole Constant Head - k_h (Fig. 28)	39	1.7	2.0	1.3	1.4	1.8
Cased Hole Constant Head - k_v (Fig. 27)	4	16.5	20.0	12.5	14.0	15.5
Open Hole Constant Head (Fig. 29)	39	1.7	2.0	1.3	1.4	1.8

ACTUAL RECOVERY TIME = 5.3 DAYS

Selected Event: March 25, 1992

Recovery From Elevation +7.0 ft NGVD To +6.0 ft NGVD

Table 9: Comparison of Measured & Predicted Recovery Times - TUTOR TIME

Hydraulic Conductivity Test Method	Measured Hydraulic Conductivity (ft/day)	Recovery Times (in days) Predicted by Various Models: Simplified Analytical Method	Modified MODRET	Hantush	Glover's Equation	Pond Flow
Laboratory Falling Head - k_h	36	2.5	2.3	1.5	1.2	2.2
Laboratory Falling Head - k_v	33	2.7	2.5	1.65	1.3	2.3
Cased Hole Falling Head - k_h (Fig. 28)	18	5.0	4.5	3.1	2.3	4.3
Cased Hole Falling Head - k_v (Fig. 27)	1	90	79	55	42	76
Cased Hole Constant Head - k_h (Fig. 28)	34	2.6	2.4	1.6	1.2	2.3
Cased Hole Constant Head - k_v (Fig. 27)	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Open Hole Constant Head (Fig. 29)	32	2.8	2.5	1.7	1.3	2.4

ACTUAL RECOVERY TIME = 4 DAYS

Selected Event: Short Term Load Test on May 4, 1992
Recovery From Elevation +6.73 ft NGVD to +5.27 ft NGVD
N.A. = Not Available

Table 10: Comparison of Measured & Predicted Recovery Times - FISHERMAN'S LANDING

Hydraulic Conductivity Test Method	Measured Hydraulic Conductivity (ft/day)	Recovery Times (in days) Predicted by Various Models: Simplified Analytical Method	Modified MODRET	Hantush	Glover's Equation	Pond Flow
Laboratory Falling Head - k_h	8	10.2	9	7.2	4.0	7.2
Laboratory Falling Head - k_v	1	82	67	57.5	31.7	57.5
Cased Hole Falling Head - k_h (Fig. 28)	5	16.4	14	11.6	6.3	11.5
Cased Hole Falling Head - k_v (Fig. 27)	2	41	33	28.8	15.9	28.8
Cased Hole Constant Head - k_h (Fig. 28)	11	7.5	6.5	5.2	2.9	5.3
Cased Hole Constant Head - k_v (Fig. 27)	2	41	33	28.8	15.9	28.8
Open Hole Constant Head (Fig. 29)	7	11.7	9.5	8.2	4.5	8.2

ACTUAL RECOVERY TIME = 7.8 DAYS

Selected Event: Short Term Load Test on May 24, 1992

Recovery From Elevation +2.2 ft NGVD to +1.4 ft NGVD

Table 11: Comparison of Measured & Predicted Recovery Times - TOM STATHAM PARK

Hydraulic Conductivity Test Method	Measured Hydraulic Conductivity (ft/day)	Recovery Times (in days) Predicted by Various Models: Simplified Analytical Method	Modified MODRET	Hantush	Glover's Equation	Pond Flow
Laboratory Falling Head - k_h	15	23.8	22	15.4	11.7	18.3
Laboratory Falling Head - k_v	7	51	47	33	25	39.3
Cased Hole Falling Head - k_h (Fig. 28)	0.5	712	630	460	352	> 500
Cased Hole Falling Head - k_v (Fig. 27)	1.5	237	220	154	117	183
Cased Hole Constant Head - k_h (Fig. 28)	2	178	160	116	88	137
Cased Hole Constant Head - k_v (Fig. 27)	0.5	712	630	460	352	> 500
Open Hole Constant Head (Fig. 29)	15	23.8	22	15.4	11.7	18.3

ACTUAL RECOVERY TIME = 19 DAYS

Selected Event: April 23 (6:00 a.m.) to May 12 (9:00 a.m.), 1992
Recovery From Elevation +5.25 ft NGVD to +4.50 ft NGVD

The following is a summary review of the results in Tables 8 through 11:

General Observations

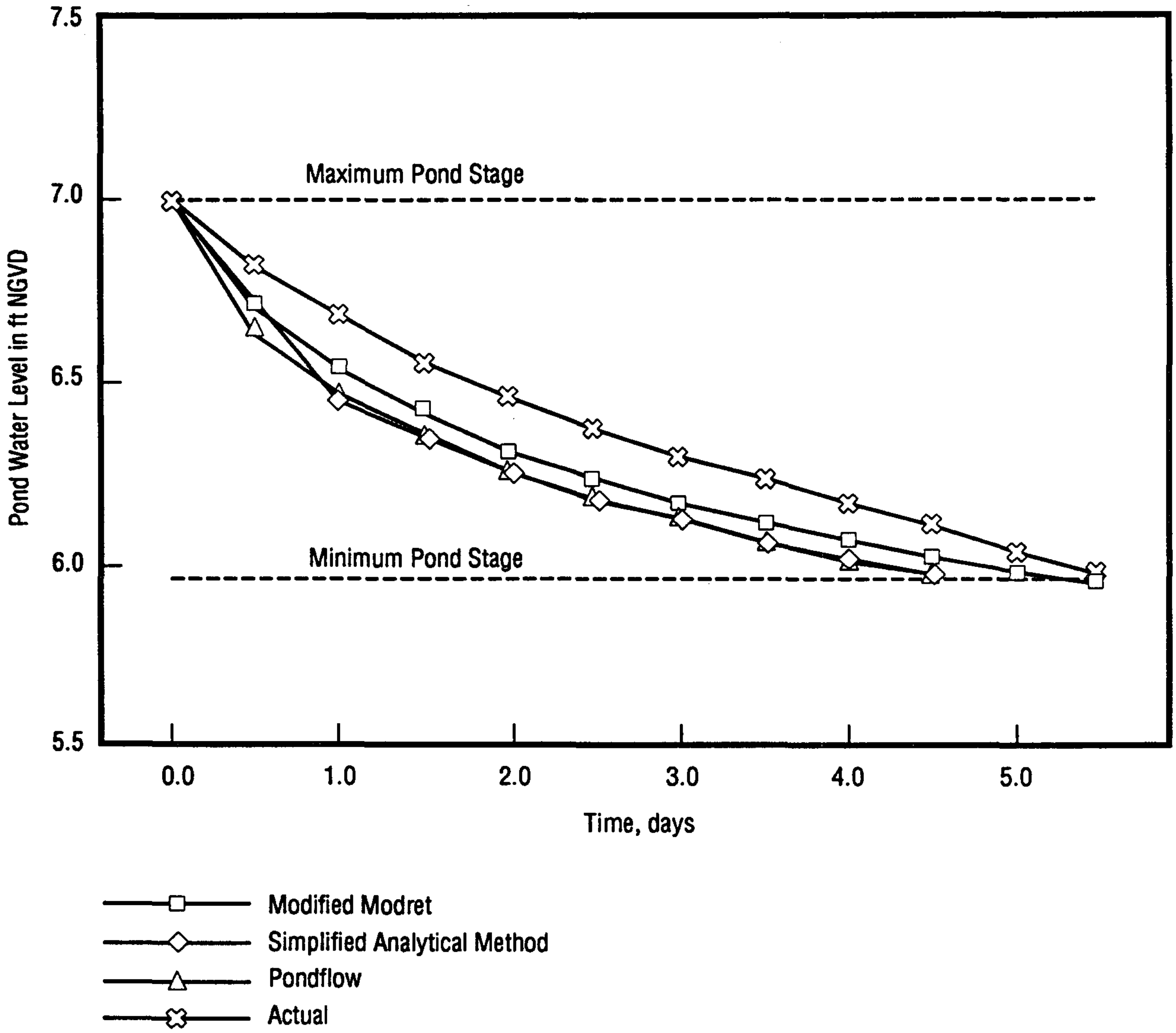
- The recovery times predicted by the Simplified Analytical Model, PONDFLOW, and Modified MODRET are relatively close in all cases. As anticipated, the simplified analytical model is slightly more conservative than the Modified MODRET or PONDFLOW models. Figures 78 to 82 show predictions of pond stage versus elapsed time for 5 events using these 3 models. As noted, the predicted response for the three models are close, and the predicted initial rates of recovery are slightly faster than the measured response, as expected (see item B.1 above).

- For the most part, the recovery times predicted using Glover's Line Source Theory is half the time predicted by the three models discussed above. Although not as unconservative as Glover's Line Source Theory, Hantush's equation also predicts faster recovery times than the three models. Based on their limitations arising from the assumptions used in their formulation and their unconservative predictions of recovery time, both the Glover and Hantush models appear to be inappropriate for predicting pond recovery.

Site-Specific Observations

- For Airport Warehouses (Table 8), a hydraulic conductivity value of approximately 14 to 15 ft/day provides the best match between the measured and predicted response. This is about 50% of the measured horizontal hydraulic conductivity from the permeameter test on the horizontal sample. Note that this sample was extracted at a depth of 2 feet below ground surface while the aquifer depth is 6.5 feet. This suggests that the weighted hydraulic conductivity of this 6.5 foot thick uppermost fine sand aquifer is about 50% of the 30 ft/day value measured at a depth of 2 feet.

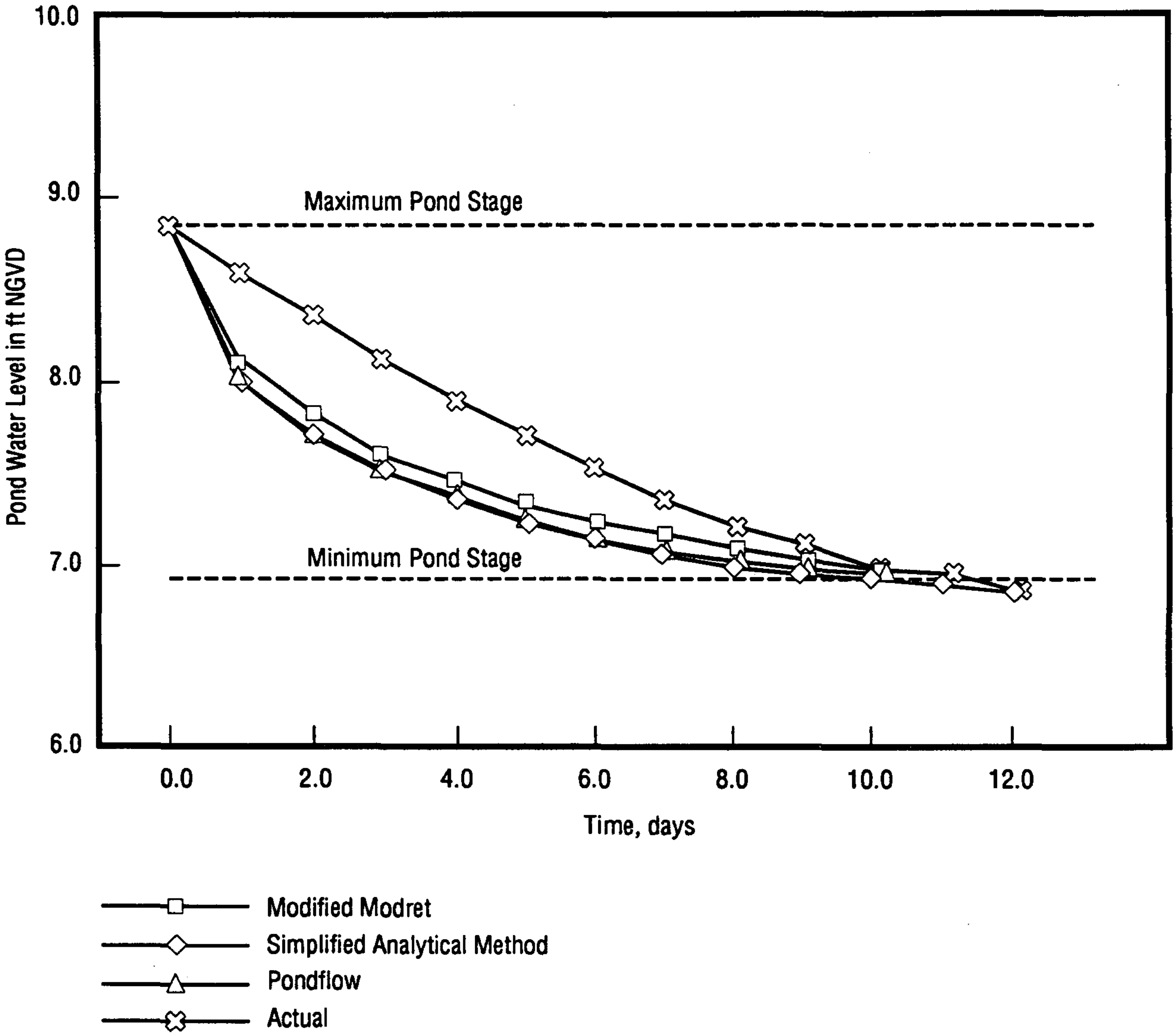
- The same comment as for Airport Warehouses applies for the Tutor Time pond (Table 9). The hydraulic conductivity measurement at a depth of 2 feet appears to represent 50% of the weighted hydraulic conductivity of this 5 to 6 ft thick upper mantle of fine sand. The fact that, for both Aiport Warehouses and Tutor Time, the weighted hydraulic conductivity of the mobilized aquifer appears to be approximately equal to 50% of the hydraulic conductivity of the upper portion of the aquifer should not be taken as a general rule. This finding could be purely coincidental.



NOTE: Aquifer parameters used in simulations above are identical for all models (see Table 7).
k_h = 15.5 ft/day

Measured vs. Predicted Response
Pond Recovery Following
Storm Event of March 25, 1992
Airport Warehouses

Figure: 78

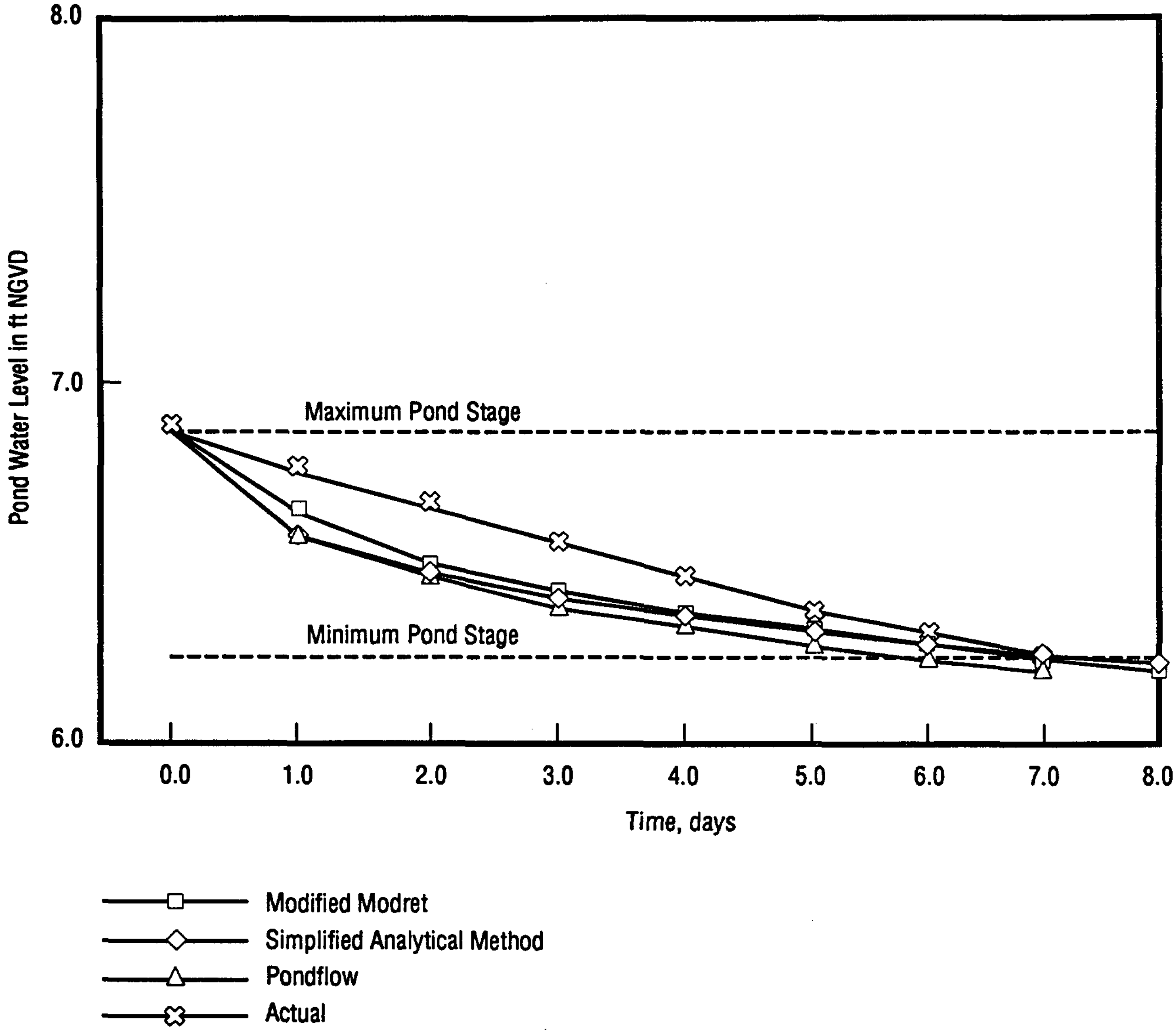


NOTES: Aquifer parameters used in simulations above are identical for all models and consistent with Table 7 except for k_h = 15.5 ft/day

Values associated with the "Storm Event Modeled" differ from those presented on Table 7

Measured vs. Predicted Response
Pond Recovery Following
Storm Event of June 29, 1992
Airport Warehouses

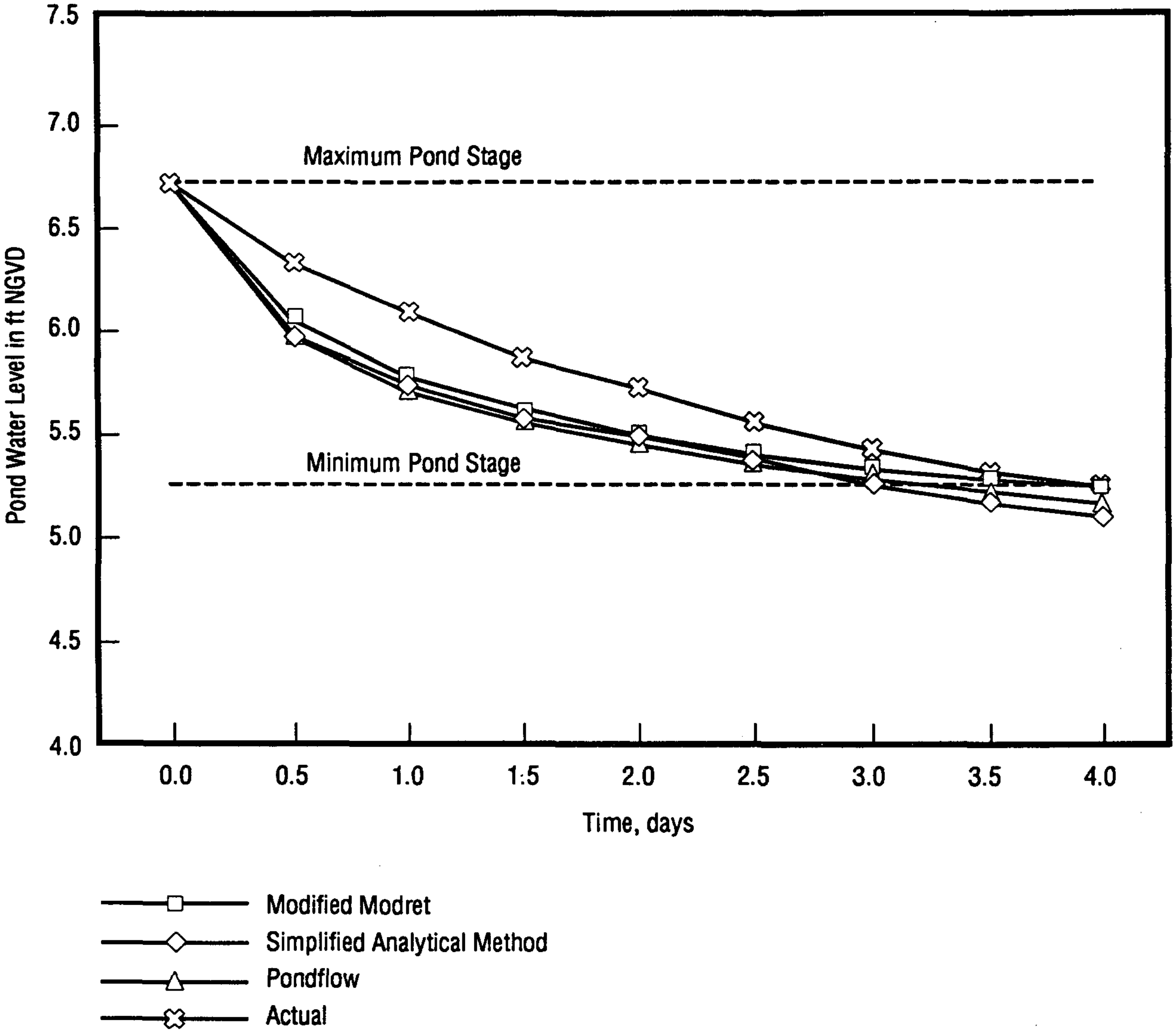
Figure: 79



NOTE: Aquifer parameters used in simulations above are identical for all models (see Table 7).
k_h = 15.5 ft/day

Measured vs. Predicted Response
for Pond Recovery Following
Storm Event of August 4, 1992
Airport Warehouses

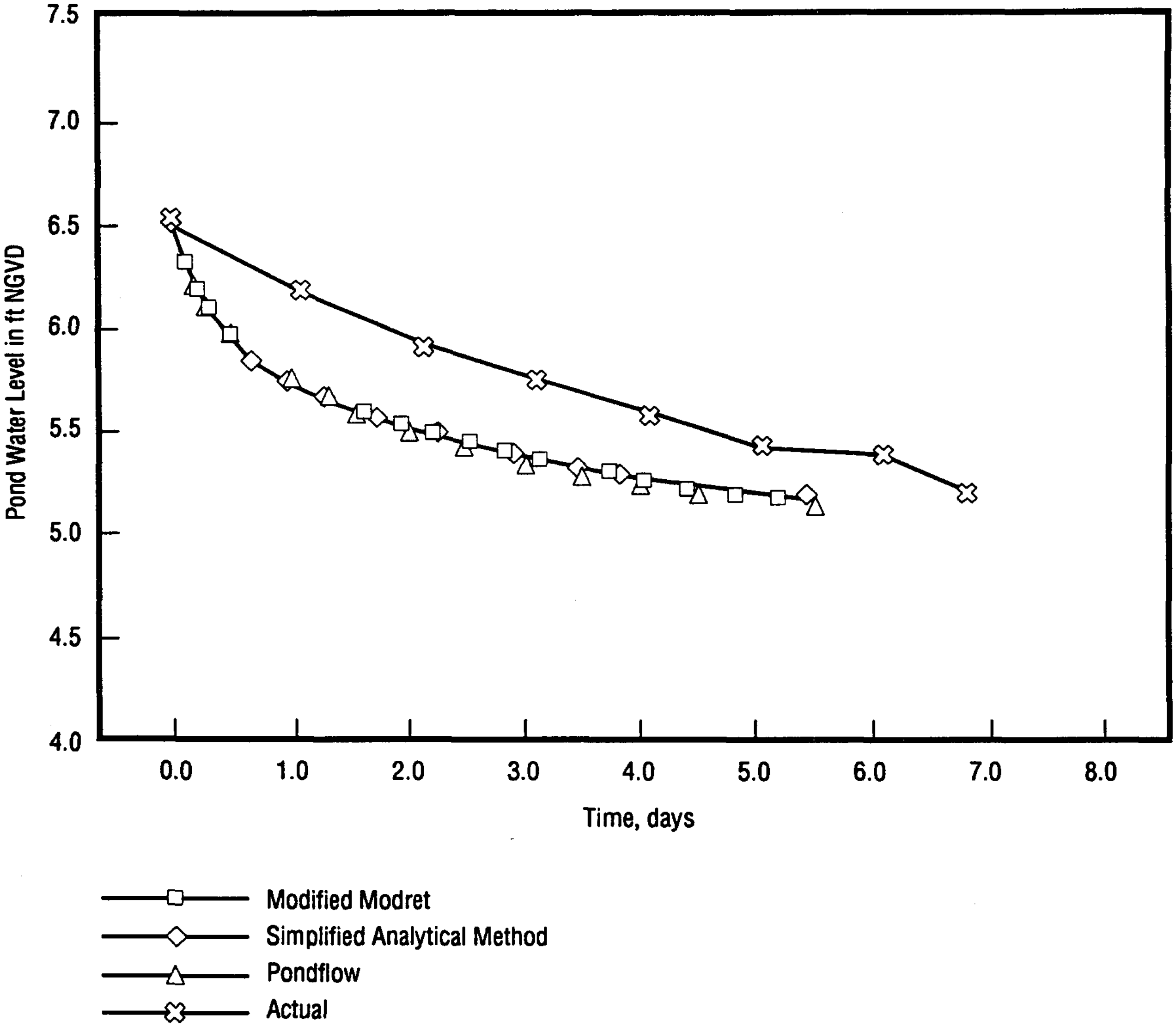
Figure: 80



NOTE: Aquifer parameters used in simulations above are identical for all models (see Table 7).
kh = 15.5 ft/day

Measured vs. Predicted Response
Pond Recovery Following
Load Test of May 4, 1992
Tutor-Time Child Care Facility

Figure: 81



NOTE:
Aquifer parameters used in simulations above are identical for all models and consistent with Table 7 except for kh = 15.5 ft/day and ambient water table = +4.653 ft NGVD

SELECTED TIME:
Recovery from stage elevation +6.50 ft NGVD at 21.5 hr on April 13 to stage elevation +5.20 ft NGVD at 17.0 hr on April 20. Duration = 6.81 days.

Measured vs. Predicted Response
Pond Recovery Following
Slug Loading of April 13, 1992
Tutor-Time Child Care Facility

Figure: 82

- The predicted recovery times at the Fisherman's Landing pond reasonably matches the measured recovery time for the hydraulic conductivity test results in the range 7 to 11 ft/day. This range of hydraulic conductivity measurements was obtained from the laboratory permeameter test (horizontal sample), the cased borehole constant head test (k_h configuration), and the open hole (or screened hole) constant head test.

- At the Tom Statham Park pond, a good match between the theoretical prediction and the measured data was obtained for the hydraulic conductivity test values obtained from the laboratory permeameter test (horizontal sample) and the open hole (or screened hole) constant head test.

6.6 SENSITIVITY OF PREDICTED RECOVERY TIME TO ASSUMED VALUE OF FILLABLE POROSITY

The porosity of a soil is the percentage of the total volume of the material that is occupied by pores or interstices. These pores may be filled with water if the material is saturated, or with air and water if it is unsaturated. Typically, for fine sand, the porosity is in the range 40 to 50% (Davis 1969). The amount of water that an unconfined aquifer can store per unit rise in water table and per unit area is called the fillable porosity (Bouwer 1978). The fillable porosity is less than the total porosity because of water in the vadose zone and not all of the unsaturated void space is available for filling. Bodman and Coleman (1943) suggest that 90% of the void space is available for filling. In addition, Mausbach (1992) reports an estimated capillary fringe thickness of one to four inches for fine sands. Using these relationships, the fillable porosity for poorly graded fine sands can be expected to vary with distance above the water table as follows:

Distance above Water Table	Fillable Porosity
0.0 to 0.5 ft	0%
0.5 to 1.0 ft	10%
1.0 to 1.5 ft	20%
1.5 to 2.0 ft	27%
2.0+ ft	30%

For fine sand aquifers, it is therefore recommended that a fillable porosity in the range 20 to 30% be used in infiltration calculations. The higher values of fillable porosity will apply to the deep, well drained, SCS hydrologic group "A" soils. If there is a reason to obtain a more precise estimate of fillable porosity, it is recommended that the equation presented in Jammal & Associates (1989) be used to compute the fillable porosity; i.e., Fillable porosity, η = $(0.9\ f) - (\omega\ \gamma_d/\gamma_w)$

where
f = total porosity
ω = natural moisture content (as a fraction)
γ_d = dry unit weight of soil
γ_w = unit weight of water

With all other dimensional and aquifer parameters the same, the predicted recovery time decreases as the assumed value of fillable porosity increases. The sensitivity of the predicted recovery times to fillable porosity is shown on Table 12. Based on the results of selected sensitivity analyses presented in Table 12, increasing the fillable porosity from 20 to 30% results in a 15-30 percent reduction in the predicted recovery times for the study ponds.

6.7 REMARKS ON APPLICABILITY OF DOUBLE RING INFILTROMETER TEST RESULTS TO RETENTION POND RECOVERY ANALYSIS

One of the earlier methods used to predict the recovery of retention ponds simply involved taking 50 percent of the Double Ring Infiltrometer Test infiltration rate as the average percolation rate during recovery. The inappropriateness of this earlier method is demonstrated below by comparing the results from this calculation method to the actual recovery times for the events referenced previously on Tables 8 through 11:

Pond	Recovery Time Predicted by Double Ring Infiltrometer Test Method (days)	Actual Recovery Time (days)
Airport Warehouses	0.06	5.3
Tutor Time	0.15	4
Fisherman's Landing	0.31	7.8
Tom Statham Park	1.74	19

Table 12: Sensitivity of Predicted Recovery Times To Fillable Porosity (η)

POND	MODEL	Hydraulic Conductivity (ft/day)	Predicted Recovery Time (T_1) η = 20% (day)	Predicted Recovery Time (T_2) η = 30% (day)	T_2/T_1
Airport Warehouses	Simplified Analytical	39	1.7	1.2	0.72
Airport Warehouses	Pondflow	39	1.8	1.4	0.77
Airport Warehouses	Modified Modret	39	2.0	1.5	0.75
Airport Warehouses	Simplified Analytical	.3	219	159	0.73
Airport Warehouses	Pondflow	.3	214	168	0.78
Tutor Time	Simplified Analytical	36	2.5	1.9	0.76
Tutor Time	Pondflow	36	2.2	1.9	0.86
Tutor Time	Simplified Analytical	1	90	67	0.74
Tutor Time	Pondflow	1	76	64	0.84
Fisherman's Landing	Simplified Analytical	11	7.5	5.5	0.73
Fisherman's Landing	Simplified Analytical	1	82	60	0.73
Tom Statham Park	Simplified Analytical	.5	712	515	0.72
Tom Statham Park	Simplified Analytical	15	24	17	0.71

SECTION 7.0: RECOMMENDED DESIGN GUIDELINES

7.1 GENERAL

In analyzing stormwater retention ponds, a geotechnical engineer systematically performs the tasks outlined below.

STEP #	TASK DESCRIPTION
1	Review the engineering plans and runoff calculations provided by the drainage engineer to understand: i) the performance expectations of the stormwater management system from both a regulatory and practical standpoint, and ii) the implications of failure of the system. Larger ponds which discharge into sensitive ecosystems may warrant more elaborate study than smaller ponds in less sensitive areas. Closed systems (i.e., ponds without a positive outfall) also require more attention since these systems are not forgiving when they fail.
2	After the preliminary review in Step #1, the geotechnical engineer may choose to interact with the drainage engineer to address any potential performance limitations which may be obvious from the available data (including published soils information).
3	Plan and perform a site-specific field and laboratory test program to obtain representative parameters of the receiving aquifer system, including: ■ Effective (or mobilized) thickness ■ Weighted horizontal hydraulic conductivity ■ Fillable porosity ■ Estimated normal seasonal high water table which will represent design ambient conditions.
4	Interpret and evaluate the field and laboratory data and check for reasonableness of results based on experience and his local knowledge of an area. Experience and judgement both play key roles in reviewing such geotechnical data.

STEP #	TASK DESCRIPTION
5	Select and utilize a groundwater flow model to analyze the infiltration rates and/or volume recovery of the pond for the design storm event(s) to check for compliance with the recovery period(s) stipulated by the regulatory agencies. The seepage model is also sometimes used to address potential operational constraints, such as the recession of the mound in closed systems or potential adverse mounding in areas adjacent to the pond.
6	Provide recommendations on construction and maintenance requirements to ensure long-term performance of the retention pond.

This study focused specifically on providing testing and design guidelines for Steps #3 and #5 above, with special emphasis on the Indian River Lagoon Basin of the St. Johns River Water Management District, Florida.

7.2 RECOMMENDED METHODOLOGIES FOR AQUIFER CHARACTERIZATION

Based on the findings of this study as well as the economic and practical considerations of the current state of the geotechnical engineering practice, the following field and laboratory investigation and testing guidelines are recommended for aquifer characterization:

7.2.1 Definition of Aquifer Thickness

Standard Penetration Test (SPT) borings (ASTM D-1586) or auger borings (ASTM D-1452) are typically used to define the thickness of the mobilized aquifer.

Standard Penetration Test (SPT) borings are recommended for definition of the aquifer thickness especially where the ground water table is high. This type of boring provides a continuous measure of the relative density/consistency of the soil (as manifested by the SPT "N" values) which is important in the Indian River Lagoon (IRL) Basin for detecting the top of cemented or very dense "hardpan" type layers. Such layers restrict the vertical movement of groundwater and are typical of the IRL Basin (excluding the relic sand dunes). If carefully utilized, manual "bucket" auger borings can also be used to define the thickness of the uppermost aquifer (i.e., depth to "hardpan" or restrictive layer), especially for small ponds and swales. Power flight auger borings may also be used, although this method may result in some mixing of soil from a given level with soils from strata above, thus masking the true thickness of the aquifer. To avoid this problem, technical guidelines for continuous flight auger borings are included in Appendix C.

The number of borings required to characterize the receiving aquifer of a retention pond depends on the anticipated areal and vertical variability of the aquifer. The local experience of the geotechnical engineer also plays an important role in the selection of the number of borings. As a guide, Jammal & Associates, Inc. (1989) suggested the following empirical equation to estimate the number of exploratory borings required:

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

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

Preferably, the SPT borings should be continuously sampled at least 2 feet into the top of the hydraulically restrictive layer. If a restrictive layer is not encountered, the boring should be extended to at least 10 feet below the bottom of the pond. As a minimum, the depth of the exploratory borings should extend to the base elevation of the aquifer assumed in analysis, unless nearby deeper borings or well logs are available. Electric analog studies (Bouwer 1978) indicate that the maximum depth of the mobilized aquifer is about equal to the width of the pond for isotropic aquifers. Based on Bouwer's study, it is recommended that the aquifer thickness used in analysis not be greater than the width of the pond.

Ground surface elevations at the boring locations should be surveyed if there is significant relief in the locality of the borings.

7.2.2 Estimated Normal Seasonal High Groundwater Table

The contemporaneous measurements of the water table should be adjusted to arrive at an estimate of the normal seasonal high groundwater table considering the factors outlined by Seereeram (1993). These key factors include:

1. Antecedent rainfall

2. Soil map unit descriptions published by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS)

3. Examination of the soil profile, including redoximorphic features (Vepraskas 1992, and Watts and Hurt 1991), SPT "N" values, depth to "hardpan" or other impermeable horizons (such as clayey fine sands and clays), etc.

4. Consistency of water levels with adjacent surface water bodies and knowledge of typical hydraulic gradients (water table slopes).

5. Vegetative indicators

6. Effects of existing and future development, including drainage ditches, modification of land cover, subsurface drains, irrigation, septic tank drainfields, etc.

7. Hydrogeologic setting including potentiometric surface of Floridan aquifer and degree of connection between the water table aquifer and the Floridan aquifer.

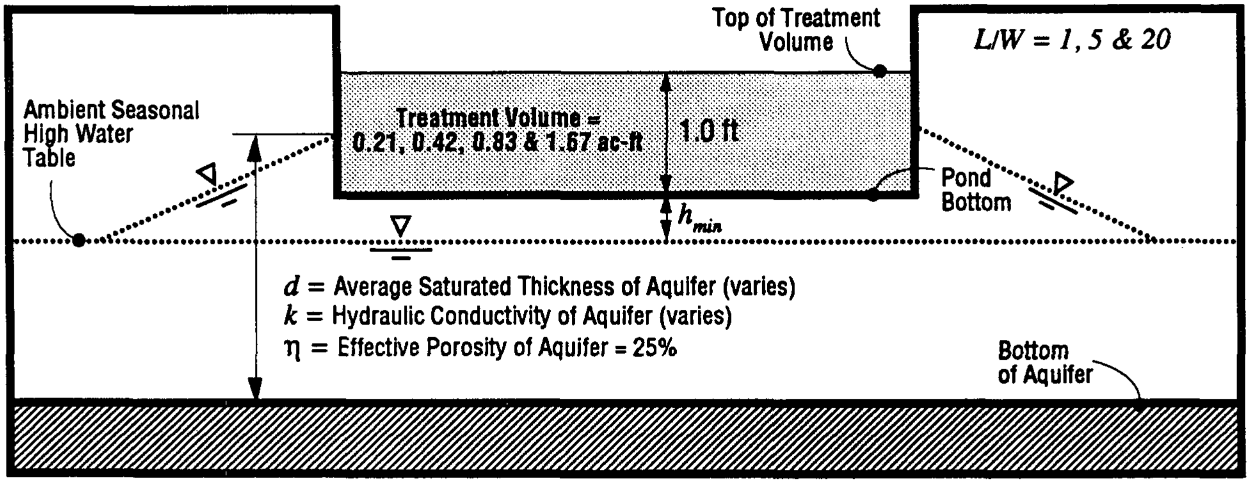
8. Soil Morphological Features

The application of the above factors in estimating the seasonal high water table (SHWT) requires considerable experience. However, the scope of the present study did not include development or evaluation of the methodologies for estimating the SHWT.

In general, the measurement of the depth to the groundwater table is less accurate in SPT borings when drilling fluids are used to maintain an open borehole. Therefore, when SPT borings are drilled, it may be necessary to drill an auger boring adjacent to the SPT to obtain a more precise stabilized water table reading. In poorly drained soils, the auger boring should be left open long enough (at least 24 hours) for the water table to stabilize in the open hole.

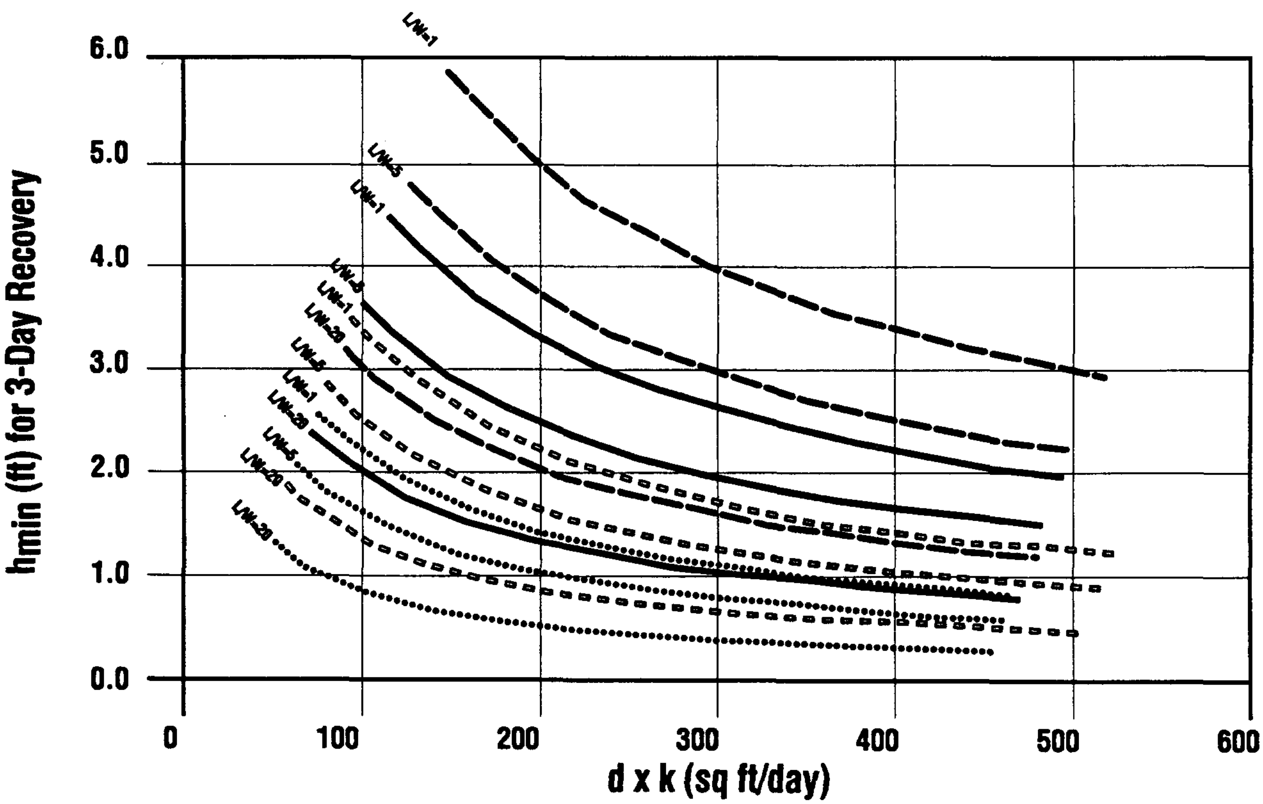
The required separation between the retention pond bottom and the seasonal high water table depends on the length/width ratio of the pond, the actual width of the pond, the average transmissivity of the mobilized aquifer, and the depth of the treatment volume within the pond. Figure 83 shows a family of curves relating this required minimum separation for a typical 3-day recovery period. This family of curves was developed using the Simplified Analytical Method. As noted on Figure 83, establishing the pond bottom 2 to 4 feet above the estimated SHWT covers a wide range of practical cases.

If there is groundwater relief within the footprint of the pond, the average groundwater contour should be considered representative of the pond.



Notes: 1. Unsaturated vertical infiltration not considered
2. h_{min} computed using Simplified Analytical Method

Profile View



Treatment Volume = 0.21 ac-ft
Treatment Volume = 0.42 ac-ft
Treatment Volume = 0.83 ac-ft
Treatment Volume = 1.67 ac-ft

Sensitivity of Required Separation Between Pond Bottom and Water Table for 3-day Revovery Period

Figure: 83

7.2.3 Estimation of Horizontal Hydraulic Conductivity of Aquifer

Based on the findings of this study as well as practical and economic considerations, the following hydraulic conductivity tests are recommended for retention ponds in the IRL basin:

i) Laboratory hydraulic conductivity test (k_h) on undisturbed sample (Figure 26)

ii) Uncased or fully screened auger hole using the equation shown on Figure 29.

iii) Cased hole with uncased or screened extension with the base of the extension at least 1 foot above the confining layer (see Figure 28).

iv) Pump test, when accuracy is important and hydrostratigraphy is conducive to such a test method.

Of the above methods, the most cost-effective is the laboratory hydraulic conductivity test on an undisturbed horizontal sample. However, it becomes difficult and expensive to obtain undisturbed hydraulic conductivity tube samples under the water table or at depths greater than 5 feet below ground surface. In such cases--where the sample depth is over 5 feet below ground surface or below the water table--it is more appropriate to use the insitu uncased or fully screened auger hole method (Figure 29) or the cased hole with uncased or screened extension (Figure 28).

As observed in this study, the pump test results reflected transmissivity values for the combined thickness of the thin upper layer of fine sand, the more restrictive hardpan zone, and, in some cases, the lower transmissive zone of fine sand with shell fragments. These transmissivity values do not represent the mobilized (or effective) aquifer since the relatively low hydraulic conductivity of the hardpan layer retards vertical flow between the upper and lower transmissive zones. In addition, pump tests are the most expensive of the recommended hydraulic conductivity test methods. Therefore, based on the above discussion, it is recommended that pump tests be used in cases where the effective aquifer is relatively thick (greater than 10 feet) and where the environmental, performance, or size implications of the system justifies the extra cost of such a test.

The main limitation of the laboratory permeameter test on a tube sample is that it represents the hydraulic conductivity at a point in the soil profile which may or may not be representative of the entire thickness of the mobilized aquifer. In most cases, the sample is retrieved at a depth of 2 to 3 feet below ground surface where the soil is most permeable, while the aquifer thickness may be 5 to 6 feet. It is therefore important to use some judgement and experience in reviewing the soil profile to estimate the weighted hydraulic conductivity of the mobilized aquifer. It is not practical or economical to obtain and test permeability tubes at each point in the soil profile where there is a change in density, degree of cementation, or texture. Some experience must therefore be used to estimate representative hydraulic conductivities of the less permeable zones of the mobilized aquifer. For the uninitiated, valuable insight into the variation of saturated hydraulic conductivity with depth in typical Florida soils can be gleaned from the many soil characterization reports published by the Soil Science Department at the University of Florida (see, for example, Sodek et al. 1990). As an additional guide, Figure 84 presents the results of over 500 laboratory permeameter tests on poorly graded fine sands performed in the PSI/Jammal & Associates laboratory. On this figure, hydraulic conductivity is plotted versus the percent by dry weight passing the U.S. No. 200 sieve.

When the aquifer is layered, it is possible to combine several layers and consider the resulting medium as homogenous. If the flow through such layers is mainly horizontal, the arithmetic mean of the hydraulic conductivity estimates of the individual layers should be used to obtain the weighted horizontal hydraulic conductivity of the mobilized aquifer as follows:

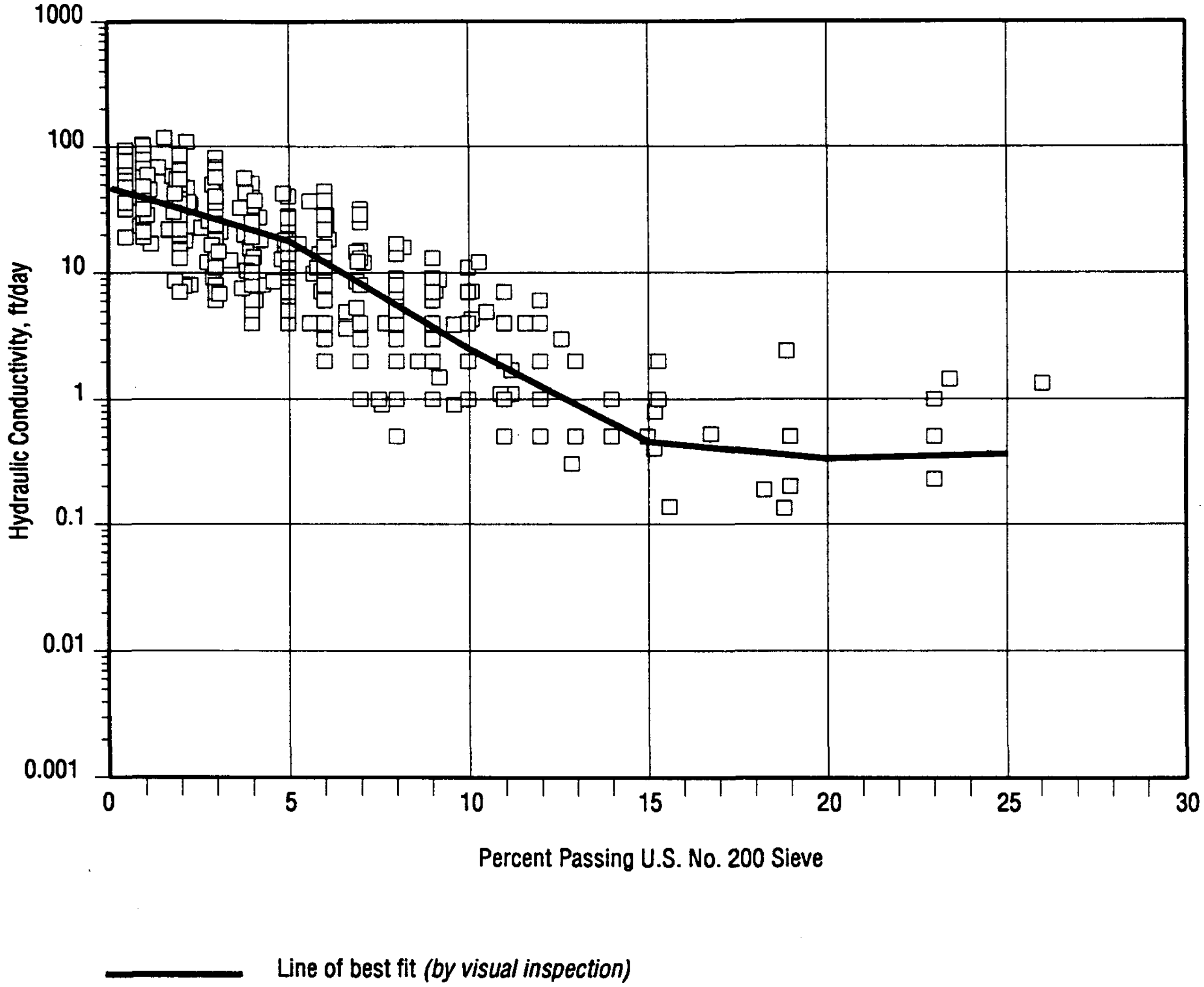
$$k_h = \frac{k_1\, z_1 + k_2\, z_2 + \ldots + k_n\, z_n}{Z}$$

where the formation consists of n horizontal isotropic layers of different thickness z, and Z is the combined thickness. Note that these layers are above the restrictive layer of hardpan or clayey material. Since the most permeable layer will control the value of the weighted hydraulic conductivity, it is important that the hydraulic conductivity of this layer be tested.

The uncased or fully screened auger hole or cased hole with uncased or screened extension hydraulic conductivity test methods are suitable for use where the mobilized aquifer is stratified and there is a high water table. Ideally, these tests should be screened over the entire thickness of the mobilized aquifer to obtain a representative value of the weighted horizontal hydraulic conductivity. Tests performed below the water table avoid the need to saturate the soil prior to testing. If the mobilized aquifer is thick with substantial saturated and unsaturated zones, it is worthwhile to consider performing a laboratory permeameter test on an undisturbed sample from the upper unsaturated profile and performing one of the insitu tests to characterize the portion of the aquifer below the water table.

The measured hydraulic conductivity value should be checked for reasonableness with the typical values published in Table 3-2 of Jammal & Associates, Inc. (1989) and Figure 84. For design purposes, a hydraulic conductivity value over 40 ft/day should not be used for fine-grained sands and 60 ft/day for medium-grained sands.

The selection of the number of hydraulic conductivity tests for a specific project depends on the local experience and judgement of the geotechnical engineer. Jammal & Associates, Inc. (1989) recommends one hydraulic test plus one more test for every four soil borings.



NOTES: Hydraulic conductivity also depends on cementation, roots, gradation, compaction, remolding, density, and other factors.

Based on permeameter tests conducted on poorly graded fine sands in PSI/Jammal & Associates (Winter Park, FL) Laboratory.

Correlation of Hydraulic Conductivity with Fraction by Weight Passing the U.S. No. 200 Sieve
(Poorly Graded Fine Sands in Florida)

Figure: 84

7.2.4 Estimation of Fillable Porosity

In Florida, the receiving aquifer system for retention ponds predominantly comprises poorly graded (i.e., relatively uniform particle size) fine sands. In these materials, the water content decreases rather abruptly with the distance above the water table and thus have a well-defined capillary fringe.

Unlike the hydraulic conductivity parameter, the fillable porosity of the poorly graded fine sand aquifers in Florida are in a narrow range (20 to 30 percent) and can be estimated with much more reliability. For fine sand aquifers, it is therefore recommended that a fillable porosity in the range 20 to 30 percent be used in infiltration calculations. The higher values of fillable porosity will apply to the well- to excessively-drained, hydrologic group "A" fine sands, which are generally deep, contain less than 5 percent by weight passing the U.S. No. 200 (0.074 mm) sieve, and have a natural moisture content of less than 5 percent.

No specific field or laboratory testing requirement is recommended, unless there is a reason to obtain a more precise estimate of fillable porosity. In such a case, it is recommended that the equation presented in Jammal & Associates (1989) be used to compute the fillable porosity; i.e.,

$$\text{Fillable porosity} = (0.9\,\eta) - (\omega\,\gamma_d/\gamma_w)$$

where
η = total porosity
ω = natural moisture content (as a fraction)
γ_d = dry unit weight of soil
γ_w = unit weight of water

7.3 RECOMMENDED METHODOLOGIES FOR RECOVERY ANALYSIS

Based on the comparison of measured and predicted response of the ponds in this study, an assessment of the rationality of the formulations, the tractability of the mathematical models, and the current state of the practice of retention pond infiltration analysis, the following three methodologies are recommended:

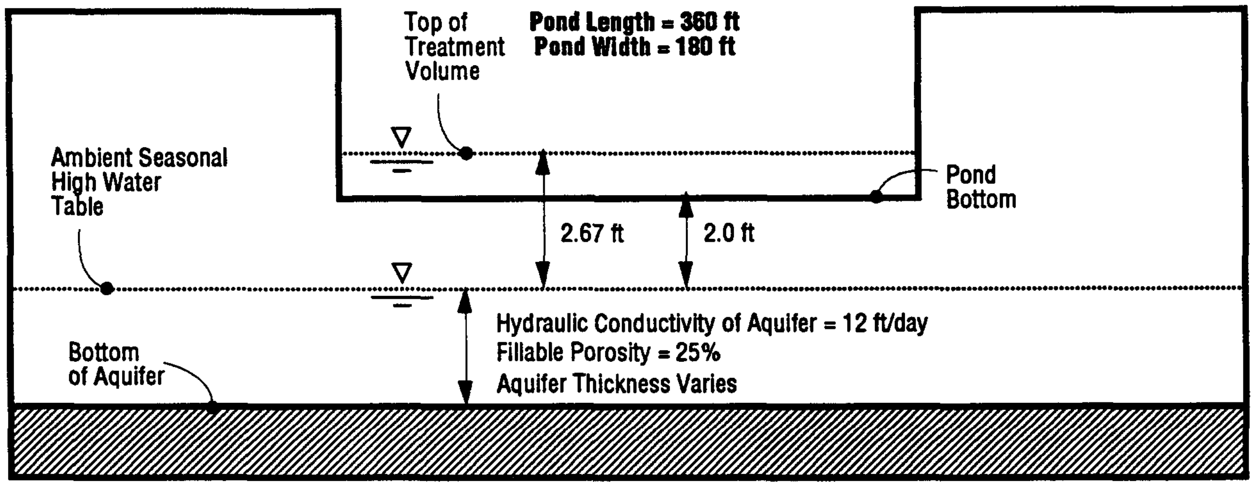
1. Simplified Analytical Method (Figure 74)
2. PONDFLOW
3. Modified MODRET

Figure 85 shows a comparison of predicted recovery times for the three recommended models for a hypothetical, although not unrealistic, pond over a range of aquifer thicknesses. As noted, the three models predict remarkably close results over a wide range of aquifer thickness and recovery periods. The PONDFLOW model predicts the fastest recovery time.

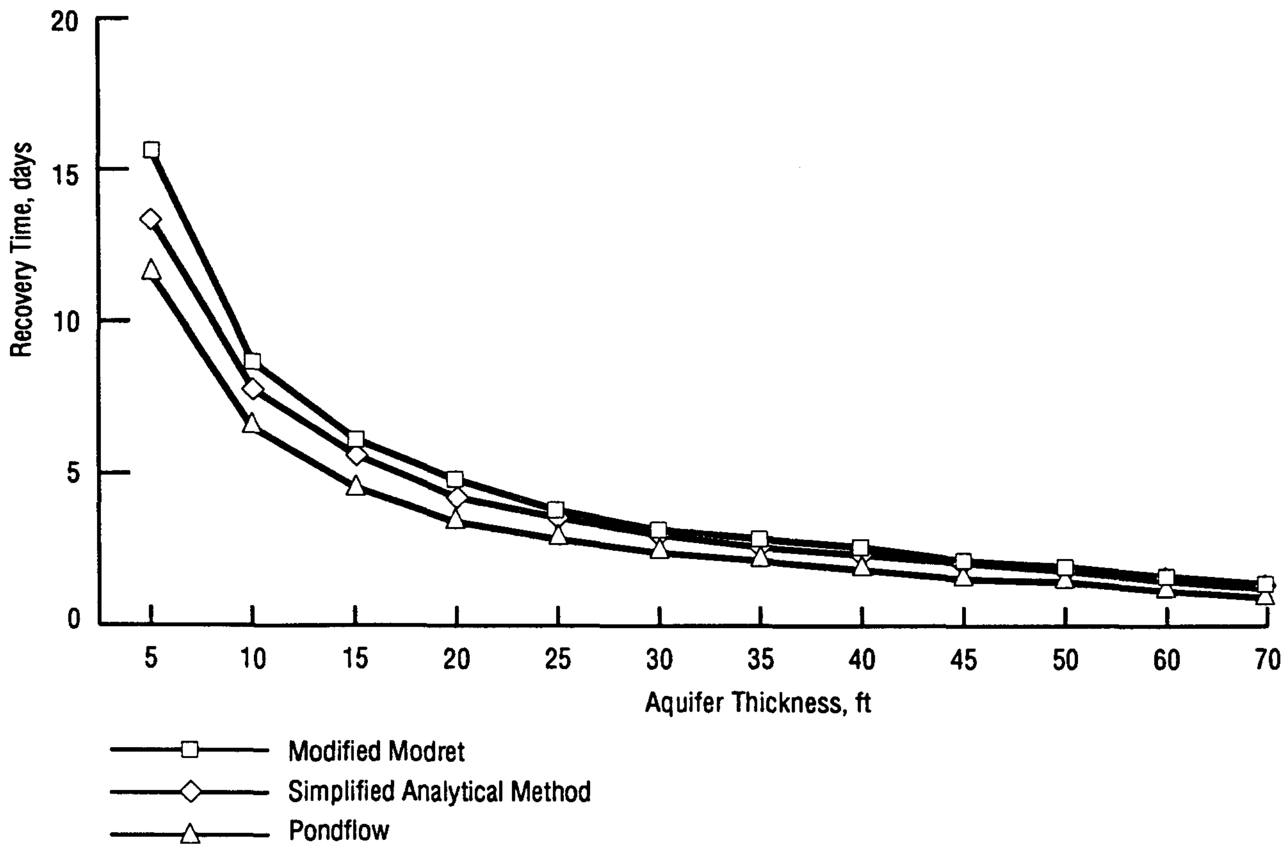
All three models can take into account unsaturated vertical flow prior to saturated lateral flow. However, the PONDFLOW and Modified MODRET models can also allow input of a discretized runoff hydrograph which allows for simulation of infiltration during a storm event. It is recommended that, unless the normal seasonal high water table is over 2 feet below the pond bottom, unsaturated flow prior to saturated lateral mounding be conservatively ignored in recovery analyses. In other words, there should be no credit for soil storage immediately beneath the pond if the seasonal high water table is within 2 feet of the pond bottom. This is not an unrealistic assumption since the height of capillary fringe in fine sand is on the order of 6 inches and a partially mounded water table condition may be remnant from a previous storm event.

It is also recommended that the filling of the pond with the pollution abatement (or treatment) volume be simulated as a "slug" loading (i.e., treatment volume fills the pond within an hour). Note that the same recommendation does not apply for the recovery of the design runoff volume for closed ponds (i.e., with no positive outfall) where the design storm events can be 24 to 96 hours long and infiltration during such storm events can be signficant.

In situations where the water table is deep and the ground water mound is not anticipated to rise above the pond bottom, the Hantush mounding equation may be applied. In addition to these 1-layered, uniform aquifer idealization models, more complicated fully three dimensional models with multiple layers (such as MODFLOW, McDonald & Harbaugh 1988) may be used where:



Profile View



Comparison of Predicted Recovery Times for a Hypothetical Pond

Figure: 85

i) the aquifer is markedly heterogeneous and non-uniform within the radius of influence of the transient hydraulic loading in the pond (such as cases with strongly sloping water tables or aquifer bases, hydraulic barriers adjacent to one side of the pond, etc.), or

ii) the receiving aquifer system cannot be idealized as a 1-layered water table aquifer (such as when sand-filled trenches are cut into the pond bottom to access lower transmissive confined aquifer zones which necessitate a 2-layered or even a 3-layered model), or

ii) the potential detrimental impacts of pond failure warrant the extra effort in obtaining a more refined estimate of infiltration.

In order to use such three dimensional models, however, much more field data is necessary to characterize the three dimensional nature of the aquifer.

7.4 RECOMMENDATIONS FOR FURTHER STUDY

Of the model input parameters and the recommended methodologies for pond recovery analysis, the largest potential source of error lies in the estimation of the weighted horizontal hydraulic conductivity of the receiving water table aquifer. Predicted recovery times are virtually inversely proportional to this input parameter. Additional research is required to develop a correlation among the results of the standard insitu hydraulic conductivity tests for typical shallow aquifer hydro-stratigraphic settings within the St Johns River Water Management District. Significant variations in hydraulic conductivity are inherent within and among the various soil horizons that comprise the receiving aquifer system for stormwater retention ponds. Further study of the applicability and limitations of the hydraulic conductivity test methods is warranted. Guidelines for assessing the reasonableness of the saturated hydraulic conductivity parameter used in models should be developed, since model predicted recovery times are virtually linearly related to this parameter. One approach would be to develop correlations between hydraulic conductivity and more economical soil tests such as particle size distribution analyses and other classification tests.

The other potential source of error lies in the estimation of the normal seasonal high groundwater table and an effort should be made to formalize this procedure.

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APPENDIX A: EQUIPMENT LIST FOR SITE INSTRUMENTATION

Long-Term Monitoring Sites: Airport Warehouses and Tom Statham Park

The following is a list of the specific equipment name, type, manufacturer, and model number used for instrumentation of the long-term monitoring sites -- Airport Warehouses and Tom Statham Park.

1. Pressure Transducers for surface water and groundwater level measurements:

Brand Name:	Delta 590 Pressure Transducer, 15 psi
Manufactured by:	Delta Controls Corporation
Model no.:	590-G2-P for pond water level Range = 6.6 feet; Accuracy = 0.5%; Repeatability = 0.1%
	590-G4-P for groundwater level Range = 33 feet; Accuracy = 0.5%; Repeatability = 0.1%
Support Equipment:	Site specific sensor cable length for each piezometer. Dryer box to reduce humidity and clogging of transducer vent tube.

2. Rain Gauge to record precipitation at the site:

Brand Name:	WEATHERtronics Rain Gauge
Manufactured by:	WeatherMeasure Division of Qualimetrics, Inc.
Model No.:	6011-A Tipping Bucket Rain Gauge Sensitivity = 0.01 inch; Accuracy = 0.5% at 0.5 in/hr.; Resolution = 0.1 inch; Calibrated at factory.

3. Data Loggers to record and store water level measurement and rainfall data:

Brand Name: Easy Logger 800

Manufactured by: Omnidata

Model No.: EL-824-GP

Support Equipment:
Easy Logger reader, EL-800
Multiplexer Model EA-110
128k EPROM Storage Pack, EM-128
EPROM eraser, 110VAC, EA-265T
NEMA 4X Enclosure

Short-Term Load Test Sites

The following is a list of the specific equipment names, type, manufacturer, and model number used for instrumentation of the short-term load test sites -- Tutor Time Child Care Facility and Fisherman's Landing.

1. Pressure Transducer

Brand Name: Electronic Pressure Transducer, 10 psi
Manufactured by: Thor International, Inc.
Model No.: DXPE-01A

2. Data Logger to record water levels.

Brand Name: Datalogger
Manufactured by: Thor International, Inc.
Model No.: SDEE-03A
Support equipment: PSION Pocket Computer, Model TCEE
Thorsoft downloading software
32K Rampak
32K Datapak

APPENDIX B: MATHCAD™ FILE FOR SIMPLIFIED ANALYTICAL MODEL

SIMPLIFIED ANALYTICAL METHOD FOR RETENTION POND & EXFILTRATION TRENCH RECOVERY ANALYSIS

by Devo Seereeram, Ph.D.

Input Parameters for Airport Warehouses, SJRWMD Research Project:

$l := 53$	ft	...................POND LENGTH
$w := 40$	ft	...................POND WIDTH
$B := 3.5$	ft datum	........ELEVATION OF BASE OF AQUIFER
$WT := 4.7$	ft datum	........AMBIENT WATER TABLE ELEVATION
$k := 30$	ft/dav	............HYDRAULIC CONDUCTIVITY OF AQUIFER
$p := 1.0$		.......................POROSITY OF MATERIAL WITHIN POND OR TRENCH
$n := .20$		.......................FILLABLE POROSITY OF AQUIFER
$PB := 6.0$	ft datum	..........BOTTOM ELEVATION OF VOLUME TO BE RECOVERED
$PT := 7.0$	ft datum	..........TOP ELEVATION OF VOLUME TO BE RECOVERED

CALCULATIONS:

$WT - B = 1.2$ ft..........Initial Saturated Thickness of Aquifer

$PB - WT = 1.3$ ft..........Minimum Drivinq Head

$PT - WT = 2.3$ ft..........Maximum Driving Head

Compute Constants:

$b := WT - B$ $\qquad a := l + w$ $\qquad c := \dfrac{w \cdot p \cdot l}{4 \cdot \frac{\pi}{3} \cdot k \cdot (l + w)}$

$d := 4 \cdot \dfrac{\pi}{3} \cdot w \cdot p \cdot \dfrac{l}{n}$ $\qquad hmin := PB - WT$ $\qquad hmax := PT - WT$

COMPUTE MAXIMUM RADIUS OF INFLUENCE:

$$\frac{\left(a^2 + d \cdot \frac{hmax - hmin}{hmin}\right)^{.5} - a}{\left(\frac{2 \cdot \pi}{3}\right)} = 54.379$$

ft...........RADIUS OF INFLUENCE

COMPUTE RECOVERED VOLUME:

$(hmax - hmin) \cdot (l \cdot w) \cdot p = 2120$ cubic feet................RECOVERED VOLUME

COMPUTE RECOVERY TIME:

$$c \cdot \int_{hmin}^{hmax} \frac{\left[\left(a^2 + d \cdot \frac{hmax - h}{h}\right)^{.5} - a\right]}{h \cdot (h + b)} \, dh = 2.193$$

days.............RECOVERY TIME

APPENDIX C: RECOMMENDED PROCEDURE FOR AUGER BORINGS

Auger types include bucket augers, continuous flight augers, worm augers, etc., each of which has its application in soil sampling. The general problem with auger equipment is that they do not perform well under the water table. However, the continuous flight auger works better than the other "short length" samplers like the bucket auger or worm auger. The bucket auger has to be removed quite frequently, at least every 6 inches to clear the soil before inserting and taking the next 6-inch sample. Once the water table is encountered, the possibility of cave-in or sloughing is much greater. The continuous flight auger maintains a column of soil throughout the drilled depth. This helps to support the walls of the borehole and minimize sloughing during drilling operations. The hole walls are only without support during withdrawal of the auger for collecting soil samples.

A typical operation for a 4-inch flight auger requires at least a "fishtail" or "clay" bit for the tip of the lead auger. This type of bit protects the auger from excessive wear since it cuts a slightly larger hole, reducing friction on the auger flights. In addition, the "fishtail" bit smoothly cuts and moves through the soil, feeding the soil into the flights.

Some operators use the auger to transport soil from the tip of the auger to the ground surface, resulting in a mixed sample which can only be used for rough classification. During the drilling of such a hole, augers are added periodically as the tip is advanced and the soil is cleared and transported upward to the ground surface via the auger flights. These auger holes are primarily used for obtaining ground water table measurements, but are not recommended for characterization of the soil profile.

The correct procedure is for the driller to "screw" the auger into the soil and shear the soil momentarily (at least ¼ to ½ turn of the auger) during advancement. This action virtually shears a "block" of soil on the auger and reduces friction for auger advancement. The determination of when to shear the auger can be judged from the "lugging down" or slowing of the engine RPM under load. Shearing may have to be done from 1 to 3 times per five-foot run. A "run", in this sense, refers to going into the hole and then retracting the tools. The auger should be removed after a 5-foot length is advanced. The soil profile is recorded and representative samples collected for laboratory classification and testing. The remaining soils on the auger flights is either cleared by hand or by throwing the rig clutch in and out and "shaking" the auger. The latter operation should only be performed by an experienced operator making sure all personnel are clear of the machine.

After the soils are cleared from the auger flights, the auger is reinserted into the borehole to the depth previously drilled. If the auger does not return completely, it indicates that there has been some sloughing. This will require turning the auger until the pre-drilled depth is reached. At this depth, the auger should be allowed to turn for a short period of time to remove all previous cuttings from the borehole. When the soil cuttings stop flowing out at the ground surface, the auger may be advanced the next 5-foot increment. Again, the auger is advanced by turning the auger into the formation. A visual check on the auger flight as drilling progresses should show the auger flights appearing stationary during advancement. If the flights are moving slightly upwards, it indicates that there is not enough down pressure ("pull down") to advance the turning flight into the soil. This will cause excessive mixing of the soil and travel of the soil up the auger flights giving imprecise depth readings of the various soil strata. On the other hand, feeding the auger flights with excessive down pressure will visibly show the flights moving slightly downward. This will cause a punching effect instead of advancement due to the cutting action of the bit. In such a case, the auger is pushing through the soil profile faster than the soil can be fed into the auger flights. Errors in interpreting the soil profile due to excess down pressure can also result in imprecise depth measurements of the soil strata.

In clayey (cohesive) soils, it may be necessary to drill only two feet or so at a time and remove the auger to record and classify the soils. At the other extreme, in saturated, transmissive layers of clean sand, sloughing and caving may occur, and each clearing and advancement of the auger may remove excessive soil to the point where a small surface collapse or subsidence occurs. Such occurences are rare, but important to keep in mind.

Drilling equipment must have sufficient torque and pulldown/pullback to effectively advance and retract the flight auger. Trailer-mounted auger rigs usually do not have the pulldown needed for augers. In addition, a "fishtail" bit is recommended for cutting the hole, although a carbide tooth rock bit may be required for advancing the auger through hardpan, rock, and other very dense material.