

# **PONDS Version 2.26**

## **USER MANUAL**

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*by*

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### **Ground Water / Surface Water Models For Stormwater Ponds Applications**

- ① Transient Infiltration Analysis of Dry or Wet Bottom Retention Ponds, Exfiltration Trenches, and Swales (with & without discharge structures)
- ② Recovery Analysis of Retention Ponds With Standard Side Bank or Bottom Filtration Systems
- ③ Recovery Analysis of Retention Ponds With Vertical Hollow Cylindrical Sand Filters (aka VVRS)
- ④ Recovery Analysis of Retentions Ponds With Subsurface Drainage Systems (aka "Underdrained Retention Ponds")
- ⑤ Ground Water Baseflow Calculations & Drawdown Analysis For Wet Detention Ponds.
- ⑥ Calculation of Dewatering Rates from and Water Table Drawdown Profiles Due to Road Underdrains, Interceptor Trenches, Ditches, Canals, Borrow Pits, and Other Dewatering Systems.
- ⑦ Hydrograph Generation (SCS Unit Hydrograph)

June 1995

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**VERSION 2.26**

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## **GETTING STARTED**

## I. GETTING STARTED

### 1.1 REVIEW OF LICENSE AGREEMENT

Before you begin, it is extremely important that you read and agree to the terms of the license agreement bound in this document. You should pay particular attention to the "Limited Warranty" section. If you do not agree to the terms and conditions of the license, please return all documentation and diskettes (unopened) and your money will be refunded.

### 1.2 HARDWARE REQUIREMENTS

PONDS™ (Version 2.26) is available on 3½" or 5¼" diskettes for the IBM-PC family and other fully compatible machines. It requires at least 570 kilobytes RAM memory. Although the program will run on any monitor, a VGA monitor is required to use the online graphics help screen and to view graphical display of results. The following special hardware requirements apply to specific modules:

Retention Pond - Refined Method:	DOS 3.0 or higher; 80386/80486 CPU, Intel Math Coprocessor; at least 1MB of extended memory
Ground water Baseflow:	DOS 2.0 or higher; 8086, 80286, 80386, & 80486 with numeric coprocessor
All Other Modules:	DOS 2.0 or higher; 8086, 80286, 80386, & 80486.

### 1.3 PROGRAM INSTALLATION

PONDS is shipped in either a 5¼" or 3½" high density disk format. Follow the steps below to install the program:

1. Place the program disk in drive "A" or "B"
2. Log on to the "A" drive (or "B")
3. Type INSTALL (enter) and follow the instructions

The "INSTALL" program will automatically set up a subdirectory for you and decompress all program and support files for you.

## 1.4 PROGRAM SECURITY

The PONDS security device is called a hardware lock. The hardware lock attaches directly to any parallel port on the back of your PC. The device protects PONDS from unauthorized use.

## 1.5 PROGRAM EXECUTION

You are now ready to run the program. To do this return to the program directory

```
A>C:      (change to "C" drive)
```

```
C>CD\PONDS226 (change to program directory)
```

Before executing the program, you may want to set up a separate data directory. This can be on any valid disk drive (preferably the hard disk) and can be under a directory or subdirectory. For example, you can set up a subdirectory under \PONDS226 called DATA. This is accomplished as follows:

```
C>MD \PONDS226\DATA      (this creates a subdirectory)
```

When reading and saving data files, the user should specify the data subdirectory (if one is used).

To execute the program type PONDS and press the ENTER key.

## 1.6 COMMON PROBLEMS

### 1.6.1 Hardware

1. If running the ground water baseflow module or the refined retention pond recovery module, ensure the computer has a math co-processor.
2. A 386 or 486 computer with at least 1 MB of extended memory is required to run the refined retention pond recovery module.

3. Check available memory on computer using, for example, the MEM command. The maximum executable program size shall be at least 570K.

### 1.6.2 Install Problems

1. If you are using DOS 5.0 and get the following error message:

**"PACKED CORRUPT FILE"**

Then reinstall using the following command:

**"LOADFIX INSTALL"**

2. If you do not have a math co-processor, the install program may return a syntax error message. You need a math coprocessor to run two modules in PONDS: ground water baseflow and the refined retention pond recovery analysis.

### 1.6.3 Copy Protection Problems

1. Make sure the provided hardware lock is firmly attached to the parallel port on the back of your PC or the program will issue a "Hardware Lock" message at execution time.
2. The hardware lock must be attached to the parallel port for the duration of the PONDS session. If the lock is disconnected during a model run, the program will halt execution until the lock is reattached. No message is displayed under these conditions.

### 1.6.4 Missing/Inadvertently Deleted Files Problem

Please check to ensure the files listed in Table 1-1 are installed in the main program directory and the graphics subdirectory.

<b>Table 1-1: List of Files</b>	
<b>File Name</b>	<b>Description</b>
<b>PONDS.EXE</b>	<b>Main Program Executable</b> This file is located in the main program (PONDS226) directory.
<b>MODFLOW.EXE</b> <b>ROUTE.EXE</b> <b>ROUTE2.EXE</b> <b>ROUTE3.EXE</b> <b>ROUTE4.EXE</b> <b>DRAWDOWN.EXE</b> <b>FILTER.EXE</b> <b>SEEP.EXE</b> <b>UNDER.EXE</b> <b>VVRS.EXE</b> <b>READDATA.EXE</b> <b>SAVEDATA.EXE</b> <b>CLEARDATA.EXE</b> <b>MFA.EXE</b> <b>BRUN45.EXE</b> <b>SWFWMD.EXE</b>	<b>Subprogram files called by PONDS.EXE</b>  These files are located in the main program (PONDS226) directory.
<b>UGH256.UHG</b> <b>UGH323.UHG</b> <b>UGH484.UHG</b> <b>USER.UHG</b>	<b>Shape Factor files used by the Hydrograph Generation Module</b> These files are located in the main program (PONDS226) directory.
<b>CONTINUED ON NEXT PAGE</b>	

<b>Table 1-1: List of Files</b>	
<b>File Name</b>	<b>Description</b>
<p>RAIN2MOD.RAI                      SFWMD72.RAI                      SJRWMD96.RAI                      SUWANEE.RAI                      ORAN10.RAI                      ORAN25.RAI                      ORAN100.RAI                      FDOT1.RAI                      FDOT2.RAI                      FDOT4.RAI                      FDOT8.RAI                      FDOT24.RAI                      FDOT72.RAI                      FDOT168.RAI                      FDOT240.RAI                      USER.RAI</p>	<p><b>Rainfall Distribution Files used by the Hydrograph Generation Module</b></p> <p>These files are located in the main program (PONDS226) directory.</p>
<p>PDS_1.HLP                      PDS_2.HLP                      PDS_3.HLP                      PDS_4.HLP                      PDS_5.HLP                      PDS_6.HLP                      PDS_7.HLP                      PDS_8.HLP                      PDS_9.HLP</p>	<p><b>General Help Files for the PONDS program.</b></p> <p>These files are located in the main program (PONDS226) directory.</p>
<p><b>CONTINUED ON NEXT PAGE</b></p>	

<b>Table 1-1: List of Files</b>	
<b>File Name</b>	<b>Description</b>
<p>EXAMPLE1.DAT                      EXAMPLE2.DAT                      EXAMPLE3.DAT                      EXAMPLE4.DAT                      EXAMPLE5.DAT                      EXAMPLE6.DAT                      EXAMPLE7.DAT                      EXAMPLE8.DAT                      EXAMPLE9.DAT                      EXAMPL10.DAT                      EXAMPLE3.INP                      EXAMPLE3.HYD</p>	<p>Data files for Example Problems presented in the User Manual.</p> <p>Note that data files for example problems for the Retention Pond Recovery - Refined Method module are listed below.</p> <p>These files are located in the main program (PONDS226) directory.</p>
<p>SWFWMD.HYD                      SWFWMD.INP                      SWFWMD1.DAT                      SWFWMD2.DAT                      SWFWMD3.DAT                      SWFWMD4.DAT                      SWFWMD5.DAT                      SWFWMD6.DAT                      SWFWMD7.DAT                      SWFWMD8.DAT                      SWFWMD9.DAT                      SWFWMD10.DAT                      SWFWMD11.DAT</p>	<p>Data files for example problems for the Retention Pond Recovery - Refined Method module.</p> <p>These files are located in the main program (PONDS226) directory.</p>
<p><b>CONTINUED ON NEXT PAGE</b></p>	

<b>Table 1-1: List of Files</b>	
<b>File Name</b>	<b>Description</b>
<p> <b>_BJ200.BGI</b>  <b>_BJC600.BGI</b>  <b>_CANON.BGI</b>  <b>_DJ.BGI</b>  <b>_DJC.BGI</b>  <b>_DJC550.BGI</b>  <b>_FX. BGI</b>  <b>_GENERIC.BGI</b>  <b>_HGL2.BGI</b>  <b>_HP7470.BGI</b>  <b>_HP7475.BGI</b>  <b>_HP7550. BGI</b>  <b>_HP7585.BGI</b>  <b>_IBMQ.BGI</b>  <b>_LJ.BGI</b>  <b>_LJ3R.BGI</b>  <b>_LJ4.BGI</b>  <b>_LQ.BGI</b>  <b>_PJ.BGI</b>  <b>_PP24.BGI</b>  <b>_PS.BGI</b> </p>	<p><b>Printer Graphic Drivers</b></p> <p>These files are located in the \PONDS226\GRAPHICS subdirectory</p>
<p><b>EGAVGA .BGI</b></p>	<p><b>VGA Screen Driver</b></p> <p>These files are located in the \PONDS226\GRAPHICS subdirectory</p>
<p> <b>BOLD.CHR</b>  <b>EURO.CHR</b>  <b>GOTH.CHR</b>  <b>LCOM.CHR</b>  <b>LITT.CHR</b>  <b>SANS.CHR</b>  <b>SCRI.CHR</b>  <b>SIMP.CHR</b>  <b>TRIP.CHR</b>  <b>TSCR.CHR</b> </p>	<p><b>Borland Stroke Fonts</b></p> <p>These files are located in the \PONDS226\GRAPHICS subdirectory</p>
<b>CONTINUED ON NEXT PAGE</b>	

<b>Table 1-1: List of Files</b>	
<b>File Name</b>	<b>Description</b>
PLOT.CFG PRINTER CFG SCREEN CFG	PONDS configuration files These files are located in the \PONDS226\GRAPHICS subdirectory
_LJ.INI _LJ3R.INI _LJ4.INI _PS.INI	Font information files for LaserJet & Postscript printers. These files are located in the \PONDS226\GRAPHICS subdirectory
_PS.PRO	Postscript header file These files are located in the \PONDS226\GRAPHICS subdirectory
SCREEN.INI LASER.INI DJ.INI DM.INI PLOTTER.INI	Plot initialization files used by PONDS to control plot attributes (such as colors, line styles, line thicknesses, etc.) These files are located in the \PONDS226\GRAPHICS subdirectory

# **OVERVIEW OF STORMWATER MANAGEMENT SYSTEMS**

## **2. OVERVIEW OF STORMWATER MANAGEMENT SYSTEMS**

### **2.1 GENERAL**

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 WMDs 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.

### **2.2 DESCRIPTION OF SYSTEMS**

Some of the more common BMPs recommended by the St. Johns River Water Management District (SJRWMD) are described in the subsequent sections.

#### **2.2.1 Retention Systems**

These 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.

#### **2.2.2 Wet Detention Ponds**

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 may 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. *Ground water inflow (baseflow)* must be considered when the control elevation is set below the normal ground water table elevation or the project utilizes road underdrains to control ground water conditions on the site. The design of the outfall structure must provide for the discharge of *ground water baseflow* at the design normal water level in the pond. *Baseflow rates* must also be included in the drawdown calculations for the outfall structure and must be considered in the permanent pool residence time design.

### 2.2.3 Filtration Systems

#### 2.2.3.1 *Side-bank and Pond-bottom Filters*

Filtration systems consist of a perforated pipe which collects and conveys stormwater following infiltration through a sand filter. Filters are generally used where space, hydraulic conductivity of the uppermost aquifer, and/or high water table conditions preclude recovery of the stormwater treatment volume 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 water quality volume flow through at least 2 feet of fine aggregate filter media before discharge into the filter drain pipe. 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 contributing drainage area of less than 10 acres. Until recently, "filters" were commonly known as "underdrains", but with the introduction of the "Underdrain" system in SJRWMD, which differs from filters, it is important to be aware of the correct terminology.

#### 2.2.3.2 *Vertical Hollow Cylindrical Sand Filter (aka VVRS)*

This is an alternative filtration system to the more common side-bank and pond-bottom filters described above. The VVRS consists of two concentric and vertically placed, perforated aluminum pipes which are set on a concrete base. The diameters of the inside and outside pipes are selected to provide a minimum annular space of 2 feet. The annular space is backfilled with fine aggregate filter media (as per SJRWMD criteria) forming a hollow cylindrical filter. VVRSs are located within the pond (not in the side slopes) and the base is usually established 2 to 4 feet below the pond bottom or normal water level. Typical details of a VVRS are included in Appendix E.

VVRSs are advantageous from a maintenance standpoint since their performance can be observed first-hand following storm events, and they can be backflushed if clogged. In

addition, unlike the typical side-bank or bottom filters, the filter media can be periodically replaced without the need to completely replace the system.

#### 2.2.4 Underdrain Systems

These 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.

#### 2.2.5 Exfiltration Trench

An exfiltration trench is a subsurface system consisting of a large diameter perforated pipe (or Infiltrator™ chambers) surrounded by coarse aggregate which temporarily stores and infiltrates stormwater runoff. Stormwater enters the conduit and infiltrates through the base and sides of the trench into the shallow ground water aquifer. The perforated pipes (or Infiltrator™ chambers) increase 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.

#### 2.2.6 Swales

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.

### 2.2.7 Wetlands Stormwater Management System

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.

### 2.2.8 Dry Detention Ponds

Dry detention ponds are normally dry storage areas which are designed to store a defined quantity of runoff and slowly release the collected runoff through an outlet structure to adjacent surface waters. After drawdown of the stored runoff is complete, the storage basin does not hold any water and thus the system is normally dry. Dry detention basins are similar to retention systems in that the basins are normally dry. The main difference between the two systems is that retention systems are designed to percolate the stored runoff into the ground while dry detention systems are designed to discharge the runoff through an outlet structure. Dry detention systems have limited pollutant removal efficiency. Their use is therefore limited to small drainage basins (< 5 acres) where high ground water or slowly permeable soils preclude retention systems.

## **OVERVIEW OF PONDS COMPUTER PROGRAM**

### **3. OVERVIEW OF PONDS COMPUTER PROGRAM**

#### **3.1 GENERAL**

PONDS is an interactive, menu-driven ground water/surface water computer program which was written specifically for analyzing stormwater management and percolation ponds. Interactive graphical and text help screens are provided in all modules. Although there are numerous other engineering applications for the generic modules, they have been tailored to perform design calculations for most of the typical stormwater management systems permitted in the state of Florida.

The methodologies in the PONDS computer program are formally approved by the St. Johns River Water Management District and the program is also used by the Southwest Florida Water Management District for permit review purposes.

#### **3.2 DESCRIPTION OF MODULES**

The program presently has seven (7) modules or menu options. Brief descriptions of the capabilities and applications of each of these modules are described in the following subsections.

##### **3.2.1 Module #1: Retention Pond Recovery Analysis - Simplified Method**

This module calculates the time for recovery of i) dry bottom or wet bottom retention ponds, ii) exfiltration trenches, and iii) swales following a slug or instantaneous filling of the pond. Typical water management district permit criteria require that the water quality (aka pollution abatement or treatment) volume instantaneously fill the pond without credit for ground infiltration during the filling of the pond.

This analytical theory of recovery analysis was developed as part of a research project for the SJRWMD (Florida) titled "Full-scale Hydrologic Monitoring of Stormwater Retention Ponds and Recommended Hydro-geotechnical Design Methodologies" by PSI/Jammal Division (August 1993). An explanation of the theory is provided in Appendix C of this manual. As documented in the SJRWMD-funded research report, this new methodology has proven to be effective and mathematically stable in predicting the volume recovery of retention ponds. It also computes the radius of influence or the distance from the edge of the pond where the water table is unaffected by the recovery of the treatment volume in the pond. To be consistent with water management district permitting criteria for recovery of the water quality volume, the model does not permit ground water or surface water discharge during the storm event that generates the water quality volume.

This module does not have the capability to compute time-varying infiltration during storm events. However, the Retention Pond Recovery Analysis - Refined Method module, which is described in the next section, has this capability.

### 3.2.2 Module #2: Retention Pond Recovery Analysis - Refined Method

This is the most powerful module in the PONDS computer program. It is a true ground water/surface water interaction model which simultaneously computes ground water and surface water discharges during and following transient hydraulic (hydrograph) loading of a stormwater pond. The groundwater component can be deactivated by using a low soil permeability in the routing analysis.

Saturated ground water flow is simulated using a modified (recompiled) version of the USGS MODFLOW computer code. The two primary modifications which have been made to MODFLOW are as follows:

- ① A new subprogram (written by the author) has been included to model non-linear discharge structures such as unsubmerged weirs, orifices, notches, etc., and
- ② Irregular surface area vs. stage relationships for ponds are modeled by changing the storage coefficient in the pond as the water level varies.

Unsaturated flow, which may occur prior to saturated flow, is modeled using an algorithm written by the author and it is uncoupled from MODFLOW. This algorithm has the ability to model perched conditions as well as the increase in the potential ground infiltration rate as the water level stages up and spreads out within the basin. Discharge through overflow structures is also modeled during unsaturated flow.

To the author's knowledge, there is no other commercially available computer program which has the true ground water/surface water interaction modeling capabilities of the Retention Pond Recovery Analysis - Refined Method module.

The unsaturated and saturated flow models are described in Appendix D. PONDS essentially acts as pre- and post-processor to these calculation routines. The following is a list of the key features of this module:

- Although the current version of the program can only simulate a single pond at a time, it can perform multiple interconnected pond routing analysis if it is run in conjunction with adICPR.

- A maximum of 3 discharge structures per pond can be modeled. The types of discharge structures which may be modeled include unsubmerged weirs, orifices, notches, etc., and other free discharge structures.
- It is a true routing model (with or without infiltration credit) and can predict peak flow rates and stage elevations since it models the true stage-area-storage relationship of the pond.
- Hydrographs generated using adICPR can be imported directly into the program and the ground water discharge versus time data (rating curve) can be subsequently exported for use in adICPR. This is also possible for multiple ponds.
- In addition to reading hydrographs created using AdICPR or the SWFWMD hydrograph generation routine included with PONDS (i.e., Module #3), hydrographs can be input manually (in a spreadsheet-type environment), or can be read from ASCII data files.
- The program has the flexibility to simulate complex fluctuations of the water table (outside the pond) during the storm event.
- The program can simulate the effects of adjacent ditches with control levels above or below the ambient water table.
- The results can be displayed and/or printed in report-quality graphical format.
- An option is included to use the refined model for water quality or pollution abatement volume recovery (i.e., slug loading of a known volume of water).
- Ponds which are partially lined can also be modeled.
- Ground water mounding due to hydraulic loading in wastewater percolation ponds and septic drainfields can be modeled.

### 3.2.3 Module #3: SCS Unit Hydrograph Generation Routine

The calculation routines for this SCS unit hydrograph generation module were written by Sam Seebali, P.E., and David Sua, P.E., of the Southwest Florida Water Management District (Brooksville, Florida). This module generates hydrographs for a single drainage basin; it does not allow the user to create composite hydrographs for multiple basins. The hydrograph data generated in this module can be imported for use in Module #2.

### 3.2.4 Module #4: Ground Water Baseflow (or Background Seepage) Calculations

This module calculates the peak ground water baseflow into retention and wet detention ponds in water table (unconfined) aquifers which have their control elevations set below the seasonal high water table. A by-product of this numerical analysis is the induced draw down of the water table as a function of distance from the edge of pond. This module has a multitude of other applications including the ability to simulate borrow pit dewatering and ground water seepage into roadway underdrains, ditches, interceptor trenches, etc. It is also useful for assessing potential wetland dehydration impacts.

The baseflow seepage methodology was developed by the author and is described in Appendix A of this manual. The USGS MODFLOW model performs the transient ground water flow calculations, although the user never interfaces directly with the MODFLOW program. A working knowledge of MODFLOW is NOT required to run the program. All input parameters are comprehensible to a "lay engineer/hydrologist". Users will find a number of real-world applications for this flexible dewatering program, other than those mentioned above.

### 3.2.5 Module #5: Calculation of the Length of Side-Bank & Bottom Filter

This module calculates the length of side-bank or pond-bottom drain filter required for retention ponds that treat stormwater with filtration systems. The theory of the analysis is based on a closed-form, analytical adaptation of the falling head equation derived by the author. The filter detail includes a gravel envelope around the drain pipe which is consistent with the latest St. Johns River Water Management District (SJRWMD, Florida) criteria. For the interested reader, a paper describing the theoretical basis of the side-bank/pond-bottom filtration model is included in Appendix B. This module also allows for the automatic inclusion of ground water baseflow in computing the required length of side-bank or pond-bottom filter. Ground water baseflow is computed using the previously described module which can also be accessed as a subprogram from the filtration sizing menu.

### 3.2.6 Module #6: Vertical Volume Recovery (Filtration) Structures (aka VVRSs)

This module computes the number of Vertical Hollow Cylindrical Sand Filters (also known as Vertical Volume Recovery Structures or VVRS) required for recovery of the treatment volume. The theory for this recovery analysis was developed by the author and is described in Appendix F of this manual. This module also automatically allows for ground water baseflow in the calculations.

### 3.2.7 Module #7: Underdrain Design Calculations

This module determines the spacing, total length, and diameter of subsurface drains for "Underdrained Retention Ponds", a relatively new but increasingly popular Best Management Practice permitted by SJRWMD. The methodology is described in Appendix E and is based on design procedures outlined in the book "Drainage of Agricultural Land: A Practical Handbook For the Planning, Design, Construction, and Maintenance of Agricultural Drainage Systems" by the Soil Conservation Service of the USDA. Ground water baseflow is automatically included in design calculations. Ground water baseflow is computed using the previously described module which can also be accessed as a subprogram from the underdrain menu.

In addition to the above, the main menu has additional file management modules including modules to save data files and read previously saved data files.

## 3.3 SALIENT FEATURES OF PROGRAM

### 3.3.1 Interactive HELP

The input screens are interactive & user-friendly with an online HELP routine to provide a graphic explanation of each input parameter for each input screen. Model inputs may be varied interactively to perform on-screen sensitivity analyses, prior to finalizing and printing results.

### 3.3.2 Report-Quality Output

Results can be printed out in presentation-quality format. Results of time-dependent simulations can be displayed graphically and report-quality graphs can be plotted to a number of standard printers and plotters.

### 3.3.3 Standardized Function Keys

Function keys have been standardized (for the most part) in all modules.

- < F1 >      Help    (graphical screen or text file appears)**
- < F2 >      Start Calculations (after input data sheet is complete)**
- < F4 >      Print Results (after calculations are complete)**
- < F5 >      Clear the current input cell**
- < F8 >      Clear the current input page**
- < F10 >     Access ground water baseflow module from another module**

3.3.4 Request For Geotechnical Services

The input aquifer parameters are simple & straightforward, and are usually contained in site-specific geotechnical reports. If there is any uncertainty, the user can incorporate the text below in a Request For Proposal to the project geotechnical engineer.

**SAMPLE REQUEST FORM FOR GEOTECHNICAL SERVICES**

We are using the Ponds (Version 2.26) computer program to model.....*[the project stormwater management system]*..... Details of the (proposed/existing) stormwater management system are attached. *[User should provide an information package to geotechnical engineer containing the following:*

1. *Site location*
2. *Location of stormwater management system within site, preferably on site plan with all features such as topographic contours, existing drainage ditches, wetlands, pavement, proposed pond elevations, etc.]*

Please provide a proposal for a site-specific investigation to estimate the hydro-geotechnical parameters listed below. Based on interpretation of the results of the site-specific investigation and experience with similar soil and ground water conditions, we will need your report to recommend appropriate values and/or ranges of input values for the following aquifer parameters:

- ① Base elevation of effective or mobilized aquifer (ft datum or feet below land surface)
- ② Weighted horizontal hydraulic conductivity of mobilized aquifer (ft/day), and, *[if the project involves recovery analysis of a dry bottom retention pond]unsaturated vertical infiltration rate (ft/day). [For the underdrain option, sometimes the pond bottom is excavated into a less permeable soil stratum and it is possible to overexcavate this slowly permeable soil and replace it with more freely draining indigenous soil from an overlying layer. If this is the case, ask the geotechnical engineer to measure the hydraulic conductivity of both layers so you can see if the savings in subsurface pipe will justify such an overexcavation and backfill operation.]*
- ③ Specific yield or fillable porosity of mobilized aquifer (%)
- ④ Normal seasonal high water table elevation within pond footprint (ft datum or feet below land surface)
- ⑤ Fluctuation (rise) of the water table during a normal wet season (feet) and duration of normal wet season (days).

**MODULE #1**  
**RETENTION POND RECOVERY ANALYSIS - SIMPLIFIED METHOD**  
**INPUT PARAMETERS & EXAMPLE PROBLEMS**

## 4. RETENTION POND RECOVERY ANALYSIS - SIMPLIFIED METHOD

### 4.1 GENERAL

Conceptual model ..... see Figure 4-1 below  
Description of input parameters ..... see page 4-2  
Theory ..... see Appendix C  
Example Problems ..... pp. 4-5 through 4-8

Figure 4-1: Simplified Retention Pond Recovery Analysis Model

## 4.2 LIST OF INPUT PARAMETERS

1. Equivalent Pond Length (L)
2. Equivalent Pond Width (W)
3. Pond Bottom Elevation (PB)
4. Porosity of Material Within Pond (p in %)
5. Base of aquifer elevation (B)
6. Seasonal high water table elevation (WT)
7. Horizontal hydraulic conductivity of aquifer (Kh)
8. Fillable porosity of aquifer (n)
9. Unsaturated vertical infiltration rate (Iv)
10. Runoff volume (V)
11. Percent of runoff volume to be recovered (PV)

There are two common oversights by users: i) ensuring that the porosity values are input in percent (%) not fractions (i.e., for example, use 25% but not 0.25); and ii) improper calculation of the pond length and width. The equations for computing the equivalent pond length and width is described next.

### 4.3 CALCULATION OF EQUIVALENT POND WIDTH & LENGTH

All of the idealizations assume a rectangular plan configuration for computational purposes. If you are dealing with a pond of irregular shape or non-rectangular shape, a methodology is suggested below for determining an equivalent pond length and equivalent pond width.

In selecting an equivalent pond length and width, two criteria must be satisfied:

- i) the pond stage height ( $h$ ) at the overflow (i.e., discharge elevation minus pond bottom elevation) multiplied by the equivalent pond length ( $L$ ) and equivalent pond width ( $W$ ) must give the storage volume ( $V$ ) from the stage-storage data. Mathematically, this may be stated as follows:

$$h \times L \times W = V \quad (1)$$

Note that if we are dealing with a wet bottom pond, " $h$ " is the discharge elevation minus the initial water table elevation. For dry bottom ponds, " $h$ " is the discharge elevation minus the pond bottom elevation. The volume " $V$ " is the volume between the discharge elevation and the discharge elevation minus " $h$ ".

CONTINUED ON NEXT PAGE

- ii) The effective perimeter of the pond "P" must be equal to the perimeter of the equivalent rectangle (i.e., 2L + 2W). In mathematical terms,

$$(2 \times L) + (2 \times W) = P \tag{2}$$

Note that "P" is the effective perimeter of the pond which may or may not be equal to the actual perimeter of the pond. The user should review the shape of the pond and draw an imaginary line around the pond through which ground water will flow unencumbered away from the pond. The length of this line is the effective perimeter "P". See figure below for an example.

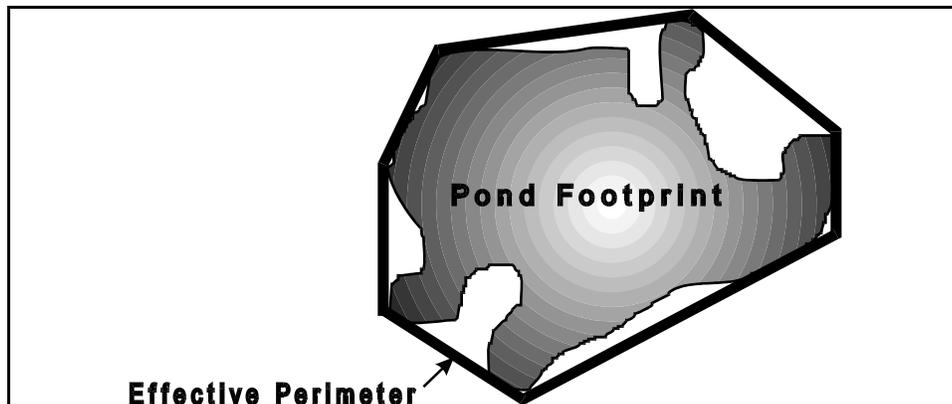
Since h, V, and P are known, equations (1) and (2) above may be solved simultaneously for L and W as follows:

$$L = \frac{\frac{P}{2} + \sqrt{\frac{P^2}{4} - \frac{4 \times V}{h}}}{2}$$

and

$$W = \frac{\frac{P}{2} - \sqrt{\frac{P^2}{4} - \frac{4 \times V}{h}}}{2}$$

Units in the above equation must, of course, be consistent.



#### 4.4 EXAMPLE #1 - EXFILTRATION TRENCH

(contained in data file EXAMPLE1.DAT)

The objective of this example is to analyze the recovery of the treatment volume stored in an exfiltration trench. Based on review of the engineering plans and site-specific geotechnical data, the following input parameters are selected:

##### Pond Parameters

1.	Equivalent Pond (Trench) Length	=	150.00 ft
2.	Equivalent Pond (Trench) Width	=	26.00 ft
3.	Pond (Trench) Bottom Elevation	=	+ 91.50 ft
4.	Porosity of Material Within Pond (Trench)	=	44.5 %

##### Aquifer Parameters

5.	Base of Aquifer Elevation	=	+ 90.00 ft
6.	Seasonal High Water Table Elevation	=	+ 91.00 ft
7.	Saturated Hydraulic Conductivity (horizontal)	=	34.0 ft/day
8.	Fillable Porosity	=	30.0 %
9.	Unsaturated Vertical Infiltration Rate	=	17.0 ft/day

##### Runoff/Treatment Volume

10.	Runoff Volume	=	3176.0 ft <sup>3</sup>
11.	Percent of Runoff Volume To Be Recovered	=	100.0 %

The output results are presented in Exhibit 1. As noted, the predicted recovery time for unsaturated and saturated flow is 3.44 days and the radius of influence is 69.5 feet.

**EXHIBIT 1  
PRINTOUT OF EXAMPLE PROBLEM #1**

4.5 EXAMPLE #2 - DRY BOTTOM RETENTION POND

(contained in data file EXAMPLE2.DAT)

The objective of this example is to select the minimum pond bottom elevation which will allow the treatment volume to recover within 3 days (72 hours) following a storm event. The following are the relevant input parameters:

Pond Parameters

1.	Equivalent Pond Length	=	100.00 ft
2.	Equivalent Pond Width	=	50.00 ft
3.	Pond Bottom Elevation	=	to be determined
4.	Porosity of Material Within Pond (open space)	=	100.0 %

Aquifer Parameters

5.	Base of Aquifer Elevation	=	+ 22.00 ft
6.	Seasonal High Water Table Elevation	=	+ 23.00 ft
7.	Saturated Hydraulic Conductivity (horizontal)	=	12.0 ft/day
8.	Fillable Porosity	=	26.5 %
9.	Unsaturated Vertical Infiltration Rate	=	Not required

Runoff/Treatment Volume

10.	Runoff Volume	=	3750.0 ft <sup>3</sup>
11.	Percent of Runoff Volume To Be Recovered	=	100.0 %

The pond bottom elevation is varied interactively on the screen until the recovery time is less than 3 days. The output results are presented in Exhibit 2. As noted, the predicted recovery time for saturated flow only is 2.99 days for a pond bottom elevation + 24.89 ft. The radius of influence is 39.28 feet.

**EXHIBIT 2  
PRINTOUT OF EXAMPLE PROBLEM #2**

**MODULES #2 & #3**  
**RETENTION POND RECOVERY ANALYSIS - REFINED METHOD**  
**INPUT PARAMETERS & EXAMPLE PROBLEM**

## **5. RETENTION POND RECOVERY ANALYSIS - REFINED METHOD**

### **5.1 CONCEPTUAL MODEL**

Conceptual model ..... see Figure 5-1 below  
Theory ..... see Appendix D

**Figure 5-1: Retention Pond Recovery Model - Refined Method**

5.2 LIST OF INPUT PARAMETERS

Input parameters for this model are summarized in Table 5-1.

Table 5-1: List of Input Parameters		
Input Parameter		Explanation
Page #	Description	
1	Equivalent Pond Length	These parameters control the effective perimeter of the pond through which saturated flow can occur. See the illustration at the bottom of page 4-4 for the physical meaning of the effective perimeter parameter. Note that the length and width do not control the stage vs. area relationship of the pond as this data must be explicitly input.
1	Equivalent Pond Width	
1	Base of aquifer elevation	These aquifer parameters should be obtained from the site-specific geotechnical report. See page 3-7 of this manual.
1	Water table elevation	Guidance for selecting these parameters is provided in Chapter 7 of the SJRWMD publication titled <i>Full-Scale Hydrologic Monitoring of Stormwater Retention Ponds and Recommended Hydro-Geotechnical Design Methodologies</i> . This publication is presently distributed free of charge and may be obtained by calling the SJRWMD library at 904-329-4500 (ask for Special Publication SJ93-SP10).  Note that the fillable porosity parameter is input in % and not in fractions (for example, 25% and not 0.25). This is a common mistake made by first-time users.
1	Horizontal saturated hydraulic conductivity of aquifer	
1	Fillable porosity of aquifer	
2	Vertical unsaturated infiltration rate	

**Table 5-1: List of Input Parameters**

Input Parameter		Explanation
Page #	Description	
2	Maximum area of pond available for unsaturated infiltration	<p>The volume of soil below a pond depends on the instantaneous water surface area. This parameter is the user-specified maximum area for computing the available soil storage volume directly below the pond. Once this soil storage volume is filled, the model simulates saturated lateral flow in which ground water moves laterally away from the pond. The user must ensure that the predicted peak stage is not higher than the stage corresponding to the maximum area.</p> <p>The pond is assumed to be lined up to the first stage elevation in the stage-area data if the input value for the maximum area is arbitrarily less than the pond bottom area. This is important to remember when modeling unsaturated flow in a lined pond. In other words, the area corresponding to the pond bottom elevation is not counted as available soil storage volume.</p>
2	Ground water mound intersection switch	<p>If the water table is very deep and it is not expected that the water table will mound and intersect the pond bottom, set this switch accordingly. This is useful when simulating the hydraulic loading of wastewater percolation ponds.</p>
2	Ditches	<p>This feature allows the user to simulate the impact of adjacent ditches. The input parameters are the distance from the edge of the pond and the water elevation in the ditch. In order to conserve memory and reduce the size of the model, the current version of PONDS assumes that an identical mirror image of the ditch exists on the other side of the pond.</p> <p>For reasons outlined on the interactive HELP screen, it is strongly recommended that this feature not be activated when modeling pond recovery.</p>
3	Discharge structures	<p>Up to three (3) discharge structures (i.e., weirs/orifices) can be modeled. The interactive HELP screen contains coefficients and exponents for typical discharge structures.</p>
4	Stage vs. Area	<p>This is the stage-area data for a pond. If a dry-bottom pond is being modeled, the first stage elevation represents the pond bottom elevation.</p>

Table 5-1: List of Input Parameters		
Input Parameter		Explanation
Page #	Description	
5	Inflow Hydrographs	Each line of data contains 3 parameters: elapsed time (hr), runoff rate (cfs), and recharge rate to the water table outside the limits of the pond (ft/day). Except for complex simulations involving fluctuation of the water table, the recharge rate parameter should be set to zero.
5	Simulation Time	This is the simulation time after the storm event to model recovery of the pond.

### 5.3 MODEL LIMITATIONS

The user should also appreciate the following limitations of this module:

- Surface water discharge is only modeled for free (or unsubmerged) discharge structures such as weirs, orifices, and V-notches. Up to 3 discharge structures can be modeled for a single pond.
- If the ditch feature is used, symmetry is implicit since the model assumes that there is a mirror image of the ditch on the other side of the pond.

### 5.4 SUPPLEMENTAL FUNCTION KEYS

Apart from the standard function keys (see page 3-6), the user should be aware of the supplemental function keys listed in Table 5-2.

**Table 5-2: Special Function Keys for Refined Recovery Analysis Module**

PAGE #	DESCRIPTION	SPECIAL FUNCTION KEYS
1	Pond/aquifer parameters	i) < F3 > shows additional function keys ii) < F11 > exports results to an ASCII text file. iii) < F12 > exports time vs. infiltration data for use as an offsite hydrograph in AdICPR
2	Adjacent ditch data	i) < F1 > general warning about using ditches
3	Weir/orifice data	i) < F1 > help screen provides typical weir & orifice coefficients
4	Hydrograph data	i) < F3 > shows additional function keys ii) < F9 > generates hydrograph for water quality volume iii) < F10 > import hydrograph data file in AdICPR format, Southwest Florida Water Management District/PONDS format, or generic ASCII format iv) < F11 > generate hydrographs to simulate long-term loading of wastewater percolation ponds or septic drainfields.

**5.5 HYDROGRAPH INPUT**

There are five possible ways to input a hydrograph:

- i) The hydrograph may be input manually on the third page (press PageDown or F9 to access) using spreadsheet-like commands. Type the F1 key to see function keys for manual input.

- ii) The user may generate a hydrograph using the SWFWMD/PONDS hydrograph generation routine accessible from the main menu (Module #3). The hydrograph may then be imported by using the F10 function key from the third input page.
- iii) The user may import a hydrograph created using the popular AdICPR surface water model. The hydrograph may then be imported by using the F10 function key from the third input page.
- iv) A user-created ASCII data file in SWFWMD/PONDS format may be imported. The hydrograph may be input from a user-created ASCII data file (F1, F10 keys on third screen) with the following format:

Number of x,y points (n) on the hydrograph (1st line)

$x_1$        $y_1$                       (note:  $x_i$  = time in hours,  $y_i$  = flow rate in cfs)

$x_2$        $y_2$

..

..

..

$x_n$        $y_n$

(free format, with at least one blank space between x and y data values on the same line)

- v) The fifth option is to import a "Generic ASCII File" using the <F10> function key. The format for the ASCII file is the same as that as the SWFWMD/PONDS hydrograph, except it now includes a third column for the outside recharge rate. The latter option is used mainly for performing long-term hydroperiod simulations where there is a need to model the natural fluctuation of the water table outside the pond.

## 5.6 INTERFACING WITH ADICPR

In addition to importing adICPR hydrographs, a rating curve of time vs. ground water discharge can be exported to AdICPR as an equivalent offsite hydrograph. The latter function can be accessed by pressing the <F12> key on the first page of input after the ground water calculations are complete. This function key allows the user to create an adICPR offsite hydrograph file for multiple ponds (nodes) instead of just a single pond.

## 5.7 MODELING OF DISCHARGE STRUCTURES

Table 5-3 presents typical "free discharge" weir equations which can be used in the hydrograph routing model. The formula is general and many other weirs can be modeled using this equation. This table can be viewed on this input screen by pressing the <F1> key when inputting these parameters.

Table 5-3: Typical Weir Equations for Free Discharge

$q \text{ (cfs)} = C D h^n$				
Weir Type	Coefficient (C)	Dimension (D)	Exponent (n)	Discharge Elevation
Rectangular weir - sharp crested	3.13	Length in ft	1.5	weir discharge
Rectangular weir - broad crested	2.861	Length in ft	1.5	weir discharge
Cipoletti trapezoid	3.367	Length in ft	1.5	weir discharge
Ogee spillway	3.97	Length in ft	1.5	weir discharge
V-Notch/triangular	2.5	$\tan(\theta/2)$ $\theta =$ angle of V- notch (in degrees)	2.5	vertex of notch
Orifice - sharp edged	4.9	Area in ft <sup>2</sup>	0.5	orifice centroid
Orifice - rounded	7.9	Area in ft <sup>2</sup>	0.5	orifice centroid
Orifice - short tube	6.4	Area in ft <sup>2</sup>	0.5	orifice centroid
Orifice - Borda	4.1	Area in ft <sup>2</sup>	0.5	orifice centroid

## 5.8 SWFWMD HYDROGRAPH GENERATION MODULE

### 5.8.1 General

The calculation routines for the SCS Unit Hydrograph generation module in PONDS is based on public domain computer code written by Sam Seebali, P.E., and David Sua, P.E., of the Southwest Florida Water Management District (Brooksville, Florida). The hydrograph generator therefore operates somewhat independently of PONDS in terms of data file management. The data file which contain the basin parameters used to generate the hydrograph are saved in a separate file (such as "SWFWMD.INP") while the detailed hydrograph output is saved separately (such as in a file called "EXAMPLE.HYD"). The latter file is the one that can be read into the Refined Method module.

### 5.8.2 User-Specified Rainfall Distribution & Unit Hydrograph

The user may create a customized dimensionless rainfall distribution file using an ASCII text editor. This file must be called "USER.RAI" and must be in the following free format:

Line #1:      N = number of x-y pairs in dimensionless rainfall distribution file  
 Line #2:       $x_1$      $y_1$   
 Line #3:       $x_2$      $y_2$   
 Line #4:       $x_3$      $y_3$   
  
 Line #N + 1    $x_N$      $y_N$

Note:  $x_i$       =    Elapsed Time/Duration of Storm Event  
        $y_i$       =    Fraction of total rainfall for storm event

This user-defined rainfall distribution may be used in the hydrograph generation routine by selecting the last of the the rainfall distribution options.

The "USER.RAI" file must be in the program directory with the other \*.RAI files so it can be accessed by PONDS.

Similarly, the user may also create a customized dimensionless unit hydrograph file using an ASCII text editor. This file must be called "USER.UHG" and must be in the following free format:

**Line #1:** Hydrograph shape factor  
**Line #2:**  $n$  = number of x-y pairs in dimensionless unit hydrograph file  
**Line #3:**  $x_1$   $y_1$   
**Line #4:**  $x_2$   $y_2$   
**Line #5:**  $x_3$   $y_3$   
  
**Line #n + 2**  $x_n$   $y_n$

**Note:**  $x_i$  = Elapsed Time/Time to peak  
 $y_i$  = Runoff rate/peak runoff rate

This user-defined normalized unit hydrograph may be used in the hydrograph generation routine by selecting the last of the the unit hydrograph or shape factor options.

The file "USER.UHG" must be in the program directory with the other \*.UHG files so it can be accessed by PONDS.

## 5.9 GRAPHIC DISPLAY OF RESULTS

The results can be displayed graphically by using the <F5> function key when the tabulated results appear on the screen. Relevant function keys appear at the bottom of the screen. Report quality graphs can be printed out on a number of hardcopy devices, including:

- ① Epson-compatible 9-pin Dot Matrix Printers & IBM Proprinter
- ② Epson-compatible 24-pin Dot Matrix
- ③ IBM Proprinter X24
- ④ IBM Quietwriter
- ⑤ LaserJet II, LaserJet III, LaserJet 4, Canon Laser
- ⑥ DeskJet (monochrome), DeskJet 500C (color cartridge), DeskJet 550C
- ⑦ PaintJet
- ⑧ Postscript
- ⑨ Hewlett-Packard Plotters

From the results screen, the <F8> function key is used to select the hardcopy output device and the port to which it is connected. In addition, the graphic output can be printed

to a file for future printing. To print a graph saved in a file, do not use the DOS "PRINT" command; use instead the DOS "COPY" command as follows: COPY "Filename" PRN /B. In addition to the printer devices listed above, an option has been provided to allow installation of printer drivers which may become available.

Once the graph is displayed, the <F5> function key allows the user to rescale the time axis for detailed viewing/printing of selected portions of the simulation. The <F2> function key gives an interactive tabulated summary of the key results. When printing the graphs, there is an option for using a "data-thinning algorithm" instead of plotting all the points. This is used for distinguishing the line types on the printer when there are many data points; it does not change the results.

The graphics routine is written in Borland Turbo C++ and the graphic drivers are included in the subdirectory "GRAPHICS" under the PONDS226 directory. Drivers can only be accessed if they are located in this subdirectory. Drivers are provided for screen and hardcopy devices. However, only the VGA screen mode is supported at this time.

## 5.10 PARTIALLY LINED PONDS

Unsaturated vertical flow (prior to saturated lateral flow) in partially lined ponds can be modeled as follows:

- ① The first stage in the stage-area data must correspond to elevation of the top of the liner (i.e. assumed pond bottom), while the last stage must correspond to the top of the unlined slope.
- ② The maximum area available for unsaturated flow must be set to some arbitrary positive value less than the surface area at the first stage elevation.
- ③ The model simulates the increase in unsaturated infiltration rate as the water spreads out above the liner elevation. Saturated flow commences when the soil storage volume beneath the unlined slope is consumed during unsaturated flow.

## 5.11 FLOW TYPE SYMBOLS

Table 5-4 lists the flow types that can occur during a simulation. The flow type is printed in the last column of the tabular output.

Table 5-4: Symbols & Description of Flow Types

Flow Type	Description
U	Discharge to ground water is via vertical unsaturated flow through the bottom of the basin. The unsaturated infiltration flow rate changes depending on the stage in the pond and the pond's stage-area characteristics. This flow type means that there is still storage in the soil voids beneath the pond but the inflow rate is not rapid enough to cause perching of the water in the pond.
U/P	Discharge to ground water is via vertical unsaturated flow through the bottom of the basin. Although the storage in the soil voids beneath the pond has not been filled, the inflow rate (in cfs) is larger than the unsaturated infiltration rate (in cfs) which results in perching of water in the pond.
U/S	There is usually only one computational time step which has this flow type. It is the time step where the soil voids beneath the pond become filled and the flow mechanism transitions from unsaturated flow to saturated flow.
S	Ground water discharge is saturated lateral flow computed using MODFLOW. Once the model transitions to saturated flow, it cannot transition back to unsaturated flow.

If there is water in the pond, the program reports the water level in the pond. On the other hand, if there is no water in the pond, the program reports the ground water level. Therefore, as the flow type changes from "U" to "U/P", jump discontinuities may appear in the reported water elevation. This should not be misinterpreted as a problem or an error. As mentioned previously, the model uses a different algorithm which is uncoupled from MODFLOW to simulate the unsaturated flow regime prior to saturated lateral flow. Although volume is preserved during the transition, the model may not report a continuous water level at the transitional time step.

Note that the program models discharge through the overflow structures (if there are any) for any flow type.



## 5.12 EXPLANATION OF CERTAIN ASPECTS OF THE PROGRAM

The following are common oversights made by users when running the model for the first time:

- ① The fillable porosity parameter must be entered in percent not decimals. So, for example, input 25% and not 0.25.
- ② If manually inputting a hydrograph, a value must be entered for the “outside recharge rate” (typically 0) in order for the program to recognize it as a valid line.
- ③ PONDS requires input of stage-area data. There is no explicit pond bottom elevation parameter. The first stage in the stage vs. area data represents the pond bottom elevation. If the water table is above the first stage elevation, the water table elevation is automatically set as the elevation where there is zero accumulated storage volume.
- ④ In situations where the water table is deep and where the ground water mound is not anticipated to intersect the pond bottom, the model setup is different and this toggle must be appropriately set. This is mainly used for modeling mounding beneath wastewater percolation ponds and septic leachfields.
- ⑤ The equivalent pond length and equivalent pond width are used to compute the effective perimeter of the pond. These parameters do not affect the storage volume of the pond as in previous versions of the PONDS computer program. Refer to the figure on pg. 4-4 of the User Manual for the meaning of the effective perimeter parameter.

## 5.13 EXAMPLE PROBLEMS

### 5.13.1 Description of Example Problems

#### *5.13.1.1 General*

There are a multiplicity of practical applications for the Refined Method module in PONDS. However, only some of the more common ones in current use are presented here. Table 5-5 overviews the ten (10) example problems described in this manual. Data files for all ten examples are supplied with the program and file names are included in Table 5-5. To reduce the size of this manual, neither tabular or graphical printouts of the detailed (and sometimes voluminous) PONDS output are presented. However, it is recommended that the user hardcopy the output from the disk files on an as needed basis. The user is also encouraged to experiment with the graphical plots of the results.

#### *5.13.1.2 Examples #3.1a & #3.1b*

As a first step, the hydrograph for a 25 yr-24 hr design storm event is created in the SWFWMD hydrograph generation module (i.e., Example #3.1a). Example #3.1b involves routing this hydrograph through a dry bottom pond which has a 7-ft wide overflow weir together with a 3-inch diameter bleed-down pipe (i.e., a sharp-edged orifice). It is assumed in this example that there are no ditches close enough to the pond to affect its recovery. The objectives of this example are to compute the peak discharge rate and peak stage during the storm event, taking into account ground infiltration during the storm.

**Table 5-5: Example Problems for Retention Pond Recovery - Refined Method**

EXAMPLE	APPLICATION	RESULTS	FILE NAME
3.1a	Generate runoff hydrograph for 25 yr - 24 hr storm event	time vs. inflow rate	SWFWMD.INP
			SWFWMD.HYD
3.1b	Route the hydrograph generated in 3.1a through a dry bottom retention pond which has multiple discharge structures (an overflow weir & a bleeddown pipe), taking infiltration credit into account during the routing.	peak stage	SWFWMD1.DAT
		peak discharge rate	
3.2	Same as Example 3.1b without infiltration credit	peak stage	SWFWMD2.DAT
		peak discharge rate	
3.3	Same as Example 3.1b without discharge structures (i.e., a closed pond). Note that closed ponds are usually designed for a 100 yr - 24 hr storm event, but the 25 yr - 24 hr hydrograph is used here only for illustrative purposes.	peak stage	SWFWMD3.DAT
		volume available in pond 14 days after storm	
3.4	Same as Example 3.1b without discharge structures and infiltration credit. This is a worst-case situation.	peak stage	SWFWMD4.DAT
3.5	Recovery of water quality volume for dry bottom pond in Example 3.1b, excluding unsaturated infiltration	peak stage	SWFWMD5.DAT
		volumes recovered after 36 hr and 72 hr	
3.6	Recovery of water quality volume for dry bottom pond in Example 3.1b, including unsaturated infiltration	peak stage	SWFWMD6.DAT
		volumes recovered after 36 hr and 72 hr	
3.7	Recovery of water quality volume for wet bottom - wet detention pond with bleed-down device. Same as Example 3.1b, but higher water table. Take into account infiltration during recovery.	volumes recovered after 24 hr, 48 hr, 60 hr, and 120 hr	SWFWMD7.DAT
3.8	Recovery of water quality volume for wet bottom - wet detention pond with bleed-down device. Same as Example 3.1b, but higher water table. Do not take into account infiltration during recovery.	volumes recovered after 24 hr, 48 hr, 60 hr, and 120 hr	SWFWMD8.DAT
3.9	Dewatering due to ditching (Cases #1 and #2 on Figure 5-2)	Water table drawdown	SWFWMD10.DAT SWFWMD11.DAT

#### *5.13.1.3 Example #3.2*

Example #3.2 is the same run as Example #3.1b but without the infiltration credit. The infiltration credit is "turned off" during the routing by setting the permeability (aka hydraulic conductivity) of the soil to a relatively low magnitude such as 0.001 ft/day. This allows the design engineer or the permit reviewer to assess the worst-case stage and discharge rate if the pond silted up or if the input permeability value is higher than a representative value for the aquifer.

#### *5.13.1.4 Example #3.3*

Example #3.3 is for a pond without a positive outfall (i.e., a closed pond). This example is the same as Example #3.1b but with both discharge structures "turned off" and the inclusion of a recovery period (i.e., simulation time) of 14 days after the storm. The objectives of this example are to determine the peak stage during the storm and the cumulative volume infiltrated up to 14 days following the storm. Note that some agencies require closed ponds to be able to accommodate a second design storm 14 days following the first one.

#### *5.13.1.5 Example #3.4*

Example #3.4 is the same as the previous example (i.e., Example #3.3) but with no infiltration credit. Like Example #3.2, infiltration credit is deactivated by reducing the aquifer permeability to a small value such as 0.001 ft/day or less. Such an analysis is useful for predicting a worst-case stage during the storm if the design engineer/reviewer suspects the aquifer permeability might be over-estimated or if pond bottom siltation is likely.

#### *5.13.1.6 Examples #3.5 & #3.6*

Example #3.5 models the recovery of the water quality volume for the dry bottom pond described in Example #3.1. Unsaturated infiltration is not considered in Example #3.5 while it is in Example #3.6.

#### *5.13.1.7 Examples #3.7 & #3.8*

Example #3.7 assumes that the pond bottom is wet (i.e., the ambient water table elevation is assumed to be the same as the bleed-down orifice) and treatment of the water quality volume is by wet detention processes. A 3-inch diameter bleed-down pipe is used to recover the water quality volume. Infiltration (positive or negative) credits are allowed during the simulation. Example #3.8 is the same as Example #3.7, but does not allow credit for simultaneous infiltration during recovery of the water quality volume through the orifice.

5.13.1.8 Example #3.9

In addition to the *Background Seepage* module in PONDS, the *Retention Pond Recovery - Refined Method* module can also be used to simulate dewatering due to ditching, borrow pits, stormwater ponds controlled below the seasonal high water table, etc. The numerical techniques used in these modules are different and it is recommended that the *Background Seepage* module be used for modeling situations where the drawdown impact distance is expected to exceed 800 ft.

Example #3.9 illustrates a dewatering application using the Refined Method for simulating the drawdown of the water table adjacent to a ditch for the typical hydrostratigraphic cases in Figure 5-2. A by-product of the analysis is the ground water baseflow into the ditch which is an important result for selecting pump sizes, etc.

Southwest Florida Water Management District (SWFWMD) has, in a previous technical procedure, defined the drawdown impact distance as the distance to the 0.10 foot drawdown contour assuming the following:

- i) no rainfall recharge over a 30 day period, and
- ii) the wet season water table is the initial condition.

On the other hand, South Florida Water Management District (SFWM) specifies the impact distance as the distance to the 1-ft drawdown contour after 90 days of no rainfall recharge using the dry season water table as an initial condition. These criteria are summarized in Table 5-6 below.

Table 5-6: Summary of Regulatory Criteria for Defining Drawdown Impact Distance

Agency	Duration of simulation	Rainfall Recharge	Water Table Drawdown	Initial Water Table
SWFWMD	30 days	None	0.10 ft	wet season
SFWM	90 days	None	1.00 ft	dry season

As mentioned previously, three typical hydro-stratigraphic cases are shown in Figure 5-2. Guidelines for the selection of aquifer parameters and the control elevation for these cases are presented in Table 5-7.

Figure 5-2: Hydrostratigraphic Variations for Ditch Dewatering Impacts

**Table 5-7: Recommendations for Selecting Aquifer Parameters for Dewatering Analysis**

Case # (refer to Fig 5-2)	Objective	Control Elevation	Base of Aquifer	Hydraulic Conductivity
1	compute water table drawdown & seepage rates into ditch	ditch control elevation	base of uppermost aquifer	$k_h$ of uppermost aquifer
2	compute water table drawdown & seepage rates into ditch	0.1 ft above base of uppermost aquifer	base of uppermost aquifer	$k_h$ of uppermost aquifer
3	compute water table drawdown	0.1 ft above base of uppermost aquifer	base of uppermost aquifer	$k_h$ of uppermost aquifer
3	compute seepage rates into ditch	ditch control elevation	base of lower aquifer	weighted average $k_h$ of saturated thickness

The criteria in Table 5-7 apply equally well when modeling the effects of an adjacent ditch on retention pond recovery.

**5.13.2 Input Parameters**

*5.13.2.1 Examples #3.1 through #3.8*

A common set of input data is used for Examples #3.1 through #3.8. These data are presented in Tables 5-8 through 5-13 which are self-explanatory.

*5.13.2.3 Example #3.9*

The objective of this example is to determine the impact distance from a ditch for establishing wetland setbacks, etc. Table 5-14 contains the input parameters used for the 500 ft long ditch in this example. Figure 5-2 shows the three typical cases encountered and Table 5-7 summarizes how the aquifer parameters should be selected for these cases. Data files for cases #1 and #2 are included on the disk. Although not included here, case #3 can be run using the recommendations in Table 5-7.

When printing the output for these examples, please remember to specify in the printing options that the ground water mound should be printed at each time step.

**Table 5-8: Input Data & Results - Postdevelopment Runoff Parameters**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
Area of contributing drainage basin	acre	10
Directly Connected Impervious Area	%	30
Curve Number for Pervious Area	-	70
Time of concentration	minutes	20
Storm Duration	hour	24
Rainfall Depth (for 24 hr, 25 yr return period)	inch	8.6
Unit Hydrograph Shape Factor	-	484
Rainfall Distribution	-	SCS Type II (Fl. Modified)
Calculated peak discharge rate	cfs	37.67
Time to peak discharge rate	hr	12.04
Calculated total runoff volume	ft <sup>3</sup>	217,290

**Table 5-9: Input Data & Computation - Water Quality Volume**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
Area of Contributing Drainage Basin	acre	10
Directly Connected Impervious Area (DCIA)	%	30.00%
Area other than DCIA (pervious area)	acre	7.0
Directly Connected Impervious Area	acre	3.0
Curve Number for Pervious Area	-	70
Rainfall Depth	inch	1
Runoff Depth from Pervious Area	inch	0.005
Runoff Depth from DCIA	inch	0.821
Pond Area	ft <sup>2</sup>	42,336
Treatment Volume for Dry Retention Pond	ft <sup>3</sup>	12,581
Treatment Volume for Wet Detention Pond	ft <sup>3</sup>	36,300

**Table 5-10: Input Data - Pond Dimensional Parameters**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
Equivalent Pond Length	ft	270
Equivalent Pond Width	ft	120
Stage vs. Area data	-	see Table 5-11
Maximum Area Available for Unsaturated Infiltration	ft <sup>2</sup>	32,400

**Table 5-11: Stage Versus Area Data for Retention Pond**

Stage (ft NGVD)	Area (ft <sup>2</sup> )	Cum. Volume (ft <sup>3</sup> )	Remarks
100.0	25,000.0	0	pond bottom
100.4	26,130.2	10,226	within limits of pond  4H:1V up to + 102.5', and then 10H:1V
100.8	27,281.0	20,908	
101.2	28,452.2	32,055	
101.6	29,643.8	43,674	
102.0	30,856.0	55,774	
102.5	32,400.0	71,588	
102.9	35,584.0	85,185	
103.3	38,896.0	100,081	
103.7	42,336.0	116,327	top of pond
104.1	45,904.0	133,975	outside limits of pond
104.5	49,600.0	153,076	
104.9	53,424.0	173,681	
105.3	57,376.0	195,841	
105.7	61,456.0	219,607	

**Table 5-12: Input Data - Discharge Structures**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
<b>Discharge Structure #1: Sharp-Crested Overflow Weir</b>		
Weir Discharge Elevation	ft MSL	102.0
Weir Discharge Coefficient	-	3.13
Weir Length	ft	7
Weir Exponent	-	1.5
<b>Discharge Structure #2: Bleed-down pipe (sharp-edged orifice)</b>		
Orifice Control Elevation	ft MSL	100.5
Orifice Discharge Coefficient	-	4.9
Orifice Diameter	inch	3
Orifice Area	ft <sup>2</sup>	0.0491
Orifice Exponent	-	0.5

**Table 5-13: Input Data - Aquifer Parameters**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
Unsaturated vertical infiltration rate	ft/day	10
Base of uppermost aquifer	ft MSL	+ 92
Horizontal saturated hydraulic conductivity (when using ground infiltration credit) Note: use $k_h = 0.001$ ft/day when not considering infiltration credit.	ft/day	12
Fillable porosity of aquifer	%	25
Seasonal High Water Table (for dry retention pond example)	ft MSL	+ 98
Seasonal High Water Table (for wet bottom pond example)	ft MSL	+ 100.5

**Table 5-14: Input Data - Ditch Dewatering Example**

PARAMETER DESCRIPTION	UNIT	MAGNITUDE
<b>DITCH DIMENSIONAL PARAMETERS</b>		
Length of ditch	ft	500
Width of ditch	ft	10
Side slopes	-	assumed vertical
<b>DISCHARGE STRUCTURE PARAMETERS</b> (ditch control level is modeled using an equivalent sharp-crested weir which is as wide as the ditch)		
Weir discharge elevation	ft MSL	+ 92.0 (for Case #1)
		+ 97.1 (for Case #2)
Weir discharge coefficient	-	3.13
Weir length	ft	10
Weir discharge exponent	-	1.5
<b>AQUIFER PARAMETERS</b>		
Base of uppermost aquifer (see Figure 4 for Case #1 and Case #2)	ft MSL	+ 88.0 (for Case #1)
		+ 97.0 (for Case #2)
Initial water table elevation	ft MSL	+ 96.0 (for Case #1)
		+ 99.0 (for Case #2)
Horizontal saturated hydraulic conductivity	ft/day	10
Drainable porosity of aquifer	%	20
<b>TIME-STEP SIMULATION PARAMETERS</b>		
Computational time step	hr	100
Duration of simulation	hr	2160 (or 90 days)

5.13.3 Results

Detailed results of each of the above simulations can be viewed on the interactive output screen or can be printed out from the data files on disk. Key results for all examples are summarized in Table 5-15.

Table 5-15: Summary of Key Results for Examples #3.1 to #3.9		
EXAMPLE #	APPLICATION	RESULTS
3.1a	Generate runoff hydrograph for 25 yr-24 hr storm event	see Table 5-8 & data file on disk "SWFWMD.HYD"
3.1b	Route the hydrograph generated in 3.1a through a dry bottom retention pond which has multiple discharge structures (an overflow weir & a bleeddown pipe), taking infiltration credit into account during the routing.	peak stage = + 102.82 ft
		peak discharge rate = 15.47 cfs
3.2	Same as Example 3.1b without infiltration credit	peak stage = + 102.99 ft
		peak discharge rate = 20.52 cfs
3.3	Same as Example 3.1b without discharge structures (i.e., a closed pond). Note that closed ponds are usually designed for a 100 yr - 24 hr storm event, but the 25 yr - 24 hr hydrograph is used here for expediency.	peak stage = + 104.92 ft
		cumulative volume infiltrated 14 days after storm = 134,511 ft <sup>3</sup>
3.4	Same as Example 3.1b without discharge structures and infiltration credit. This is a worst-case situation.	peak stage = + 105.66 ft
3.5	Recovery of water quality volume for dry bottom pond in Example 3.1b, excluding unsaturated infiltration	peak stage = + 100.49 ft
		vol. recovered @ 36 hr = 11,064 ft <sup>3</sup> vol. recovered @ 72 hr = 12,584 ft <sup>3</sup>
3.6	Recovery of water quality volume for dry bottom pond in Example 3.1b, including unsaturated infiltration	peak stage = + 100.49 ft
		vol. recovered @ 36 hr = 12,584 ft <sup>3</sup> vol. recovered @ 72 hr = 12,584 ft <sup>3</sup>

Table 5-15: Summary of Key Results for Examples #3.1 to #3.9		
EXAMPLE #	APPLICATION	RESULTS
3.7	Recovery of water quality volume for wet bottom - wet detention pond with bleed-down device. Same as Example 3.1b, but higher water table. Take into account infiltration during recovery.	Ambient stage = +101.50 ft Stage @ 0 hr = +101.78 ft Stage @ 24 hr = +101.06 ft Stage @ 48 hr = +100.69 ft Stage @ 60 hr = +100.59 ft Stage @ 96 hr = +100.50 ft
3.8	Recovery of water quality volume for wet bottom - wet detention pond with bleed-down device. Same as Example 3.1b, but higher water table. Not taking into account infiltration during recovery.	Ambient stage = +101.50 ft Stage @ 0 hr = +101.78 ft Stage @ 24 hr = +101.14 ft Stage @ 48 hr = +100.72 ft Stage @ 60 hr = +100.60 ft Stage @ 96 hr = +100.50 ft
3.9	Drawdown of water table due to ditching. Determination of required setback distance from wetlands.	Case #1 Impact distance = 240 ft (SFWMD) Impact distance = 325 ft (SWFWMD)
		Case #2 Impact distance = 43 ft (SFWMD) Impact distance = 117 ft (SWFWMD)

**MODULE #4**  
**GROUND WATER BASEFLOW (OR BACKGROUND SEEPAGE)**  
**INPUT PARAMETERS & EXAMPLE PROBLEM**

## 6. GROUND WATER BASEFLOW MODULE

### 6.1 GENERAL

Conceptual model ..... see Figure 6-1 below  
Description of input parameters ..... see page 6-2  
Theory ..... see Appendix A  
Example Problems ..... pp. 6-3 through 6-8

Figure 6-1: Conceptual Model For Ground Water Baseflow Module

## 6.2 LIST OF INPUT PARAMETERS

1. Equivalent Pond Length (L)
2. Equivalent Pond Width (W)
3. Control elevation of pond (O)
4. Bottom of aquifer elevation (B)
5. Hydraulic conductivity of aquifer (k)
6. Specific yield of aquifer (S)
7. Seasonal high water table elevation (H)
8. Rise or fluctuation of water table during wet season (F) or any other arbitrary duration (time period) specified by the user
9. Duration for rise of water table (T)

**6.3 EXAMPLE #4 - BASEFLOW INTO POND FOR GRADUAL RISE IN WATER TABLE**

(contained in data file EXAMPLE4.DAT)

The objective of this example is to estimate the peak ground water seepage into a pond for a normal wet season in the Orlando area given the following pond dimensions and aquifer parameters estimated from a site-specific investigation:

- |    |  |   |           |
|----|--|---|-----------|
| 1. | Control elevation of pond (O)                              | = | + 100 ft  |
| 2. | Bottom of aquifer elevation (B)                            | = | + 93 ft   |
| 3. | Seasonal high water table elevation (H)                    | = | + 103 ft  |
| 4. | Rise (or fluctuation) of water table during wet season (F) | = | 4 ft      |
| 5. | Hydraulic conductivity of aquifer (k)                      | = | 10 ft/day |
| 6. | Specific yield of aquifer (S)                              | = | 25%       |
| 7. | Duration of wet season (T)                                 | = | 120 days  |
| 8. | Length of pond (L)   | = | 500 ft    |
| 9. | Width of pond (W)  | = | 200 ft    |

Note that the wet season in the Orlando (Florida) area generally starts in June and extends to the end of September, a period of approximately 120 days. Soil borings drilled within the limits of the ponds and laboratory hydraulic conductivity tests provide estimates of the aquifer parameters.

The results of the model run are presented in Exhibit 4 and show a peak ground water flow rate of 13.6 gallons per minute. As the output also indicates, the drawdown of the water table with distance from the edge of the pond is tabulated as follows:

<u>Distance From Long Edge of Pond</u>	<u>Drawdown or Depression of Water Table</u>
0 ft	3.00 ft
25 ft	2.46 ft
75 ft	1.69 ft
125 ft	1.16 ft
175 ft	0.80 ft
250 ft	0.43 ft
350 ft	0.20 ft
450 ft	0.09 ft

**EXHIBIT 4  
PRINTOUT OF EXAMPLE PROBLEM #4**

**6.4 EXAMPLE #5 - BASEFLOW INTO POND FOR RAPID RISE IN WATER TABLE**

(contained in data file EXAMPLE5.DAT)

Consider now the same pond described in example #4, but it is required to compute the ground water seepage for a 25 year-24 hour design storm event. Assume that the project geotechnical engineer/hydrogeologist estimates that, in the worst case, the water table will rise 2.5 feet to an elevation of + 103.7 ft during a 2 day period for such a design storm. In this example, therefore, all input parameters are the same as example #4 except for:

- |    |  |   |            |
|----|--|---|------------|
| 3. | Seasonal high water table elevation (H)                      | = | + 103.7 ft |
| 4. | Rise (or fluctuation) of water table during study period (F) | = | 2.5 ft     |
| 7. | Duration of model period (T)                                 | = | 2 days     |

The results of this example are included in Exhibit 5 and show a peak ground water inflow of 72.5 gpm. Note that the drawdown of the water table for this short duration event is as follows:

<u>Distance From Long Edge of Pond</u>	<u>Draw Down Or Depression of Water Table</u>
0 ft	3.70 ft
25 ft	1.05 ft
75 ft	0.14 ft
125 ft	0.02 ft

**EXHIBIT 5  
PRINTOUT OF EXAMPLE PROBLEM #5**

**6.5 EXAMPLE #6 - BASEFLOW INTO POND FROM ROAD UNDERDRAINS**

(contained in data file EXAMPLE6.DAT)

Assume that in addition to the ground water seepage coming into the pond of example #4, there is a contribution to the pond's baseflow from roadway underdrains in the development which the pond serves. The road underdrains have the following dimensions and aquifer parameters in the area where they will be installed:

- |    |  |   |          |
|----|--|---|----------|
| 1. | Invert elevation of underdrain pipe (O)                    | = | + 102 ft |
| 2. | Bottom of aquifer elevation (B)                            | = | + 96 ft  |
| 3. | Seasonal high water table elevation (H)                    | = | + 104 ft |
| 4. | Rise (or fluctuation) of water table during wet season (F) | = | 4 ft     |
| 5. | Hydraulic conductivity of aquifer (k)                      | = | 7 ft/day |
| 6. | Specific yield of aquifer (S)                              | = | 25%      |
| 7. | Duration of wet season (T)                                 | = | 120 days |
| 8. | Length of underdrain (L)                                   | = | 600 ft   |
| 9. | Width of underdrain (W)                                    | = | 2.5 ft   |

The results of the background seepage run for this problem are included in Exhibit 6. As noted, the estimated seepage from the road underdrains is 6.67 gpm. Therefore, the total baseflow into the pond is sum of the contribution from background seepage and flow from the road underdrain system or 20.22 gpm. The model predicts a water table drawdown profile as follows:

<u>Distance From Long Edge of Underdrain</u>	<u>Draw Down Or Depression of Water Table</u>
0 ft	2.00 ft
0.3 ft	1.99 ft
1.3 ft	1.97 ft
3.1 ft	1.93 ft
6.9 ft	1.85 ft
14.4 ft	1.70 ft
29.4 ft	1.44 ft
58.1 ft	1.04 ft
108 ft	0.59 ft

**EXHIBIT 6  
PRINTOUT OF EXAMPLE PROBLEM #6**

**MODULE #5**  
**CALCULATION OF THE LENGTH OF SIDE-BANK & BOTTOM FILTER**  
**INPUT PARAMETERS & EXAMPLE PROBLEM**

## **7. SIDE-BANK AND POND BOTTOM FILTRATION SYSTEMS**

### **7.1 GENERAL**

Conceptual model ..... see Figure 7-1 below  
Description of input parameters ..... see page 7-2  
Theory ..... see Appendix B  
Example Problems ..... pp. 7-3

**Figure 7-1: Filtration System Idealization**

## 7.2 LIST OF INPUT PARAMETERS

1. Hydraulic conductivity of filter media (K)
2. Surface area of pool at top of volume to be filtered ( $A_{top}$ )
3. Surface area of pool where last increment of water drains through filter ( $A_{bot}$ )
4. Driving head when water stage in pond at top of volume to be filtered ( $h_{top}$ )
5. Driving head when last increment of water enters filter ( $h_{bot}$ )
6. Side slope of pond (S).
7. Dimensions of sand filter  $X_o$  and  $L_o$  as shown on Figure 7-1.
8. Percent of volume between  $A_{top}$  and  $A_{bot}$  to be filtered.
9. Background seepage & other baseflows into pond, if any. This is estimated from the "Background seepage" module.

Note that the filter detail shown on Figure 7-1 utilizes a gravel envelope around the drain pipe since it is hydraulically much more efficient than completely surrounding the pipe with the sand filter media.

7.3 EXAMPLE #7 - FILTRATION SYSTEM (WITH BASEFLOWS)

(contained in data file EXAMPLE7.DAT)

Assume that the pond in example #4 uses a side-bank filtration system for treatment of the water quality volume. The roadway underdrain system of example #6 is also assumed to be connected to the pond. The objective of this example is to determine the length of filter required to recover the pollution abatement volume and background seepage components given the following pond data:

Stage-Storage Data

Surface area of pool at top of volume to be filtered ( $A_{top}$ ) =	100,000 ft <sup>2</sup>
Surface area of pool at lowest level where water can enter the filter ( $A_{bot}$ ) =	78,624 ft <sup>2</sup>
Head difference between pool stage at $A_{top}$ and drain pipe centerline ( $h_{top}$ ) =	4.9 ft
Head difference between pool stage at $A_{bot}$ and drain pipe centerline ( $h_{bot}$ ) =	0.9 ft
Volume between $A_{top}$ and $A_{bot}$ =	357,248 ft <sup>3</sup> (8.20 acre-ft)

Treatment Volume Data

Pollution abatement (or treatment) volume =	340,000 ft <sup>3</sup> (7.81 acre-ft)
Therefore, Percent Recovery (P) = Treatment Volume/Volume between $A_{top}$ and $A_{bot}$	95.2%
Drawdown time for pollution abatement volume (t) =	3 days

Filter Details

Side slope of pond (S) =	4 Horiz:1 Vert
Hydraulic conductivity of filter media =	60 ft/day
Thickness of filter media $X_o$ =	2 ft
Maximum effective length of filter/pool interface $L_o$ =	2.8 ft

Background Seepage

Total baseflow into pond from ground water (see results of example #4)	13.55 gpm
Total baseflow into pond from roadway underdrain (see results of example #6)	6.67 gpm
Therefore, total baseflow	20.22 gpm

Factor of Safety

Factor of safety for analysis	2.0
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The output results of the analysis are presented in Exhibit 7 and shows that the length of filter required is 1227 feet with a factor of safety of 2. This computed length includes allowances for the baseflows.

**EXHIBIT 7  
PRINTOUT OF EXAMPLE PROBLEM #7**

**MODULE #6**  
**CALCULATION OF # OF VERTICAL VOLUME**  
**RECOVERY (FILTRATION) STRUCTURES (aka VVRSs)**  
**INPUT PARAMETERS & EXAMPLE PROBLEM**

## 8. VERTICAL VOLUME RECOVERY STRUCTURES

### 8.1 GENERAL

Conceptual model ..... see Figure 8-1 below  
Description of input parameters ..... see page 8-2  
Theory ..... see Appendix F  
Example Problems ..... pp. 8-3

Figure 8-1: Conceptual Model of Vertical Hollow Cylindrical Filter

8.2 LIST OF INPUT PARAMETERS

<u>PARAMETER DESCRIPTION</u>	<u>SYMBOL</u>
<i>Pond &amp; Stormwater Volume Parameters</i>	
Treatment volume, V (cubic feet):	V
Top elevation of treatment volume, TOP (ft datum)	TOP
Bottom elevation of treatment volume, BOT (ft datum)	BOT
Tailwater or invert elevation, TWL (ft datum)	TWL
<i>VVRS Details</i>	
Inside diameter of VVRS structure, Din (ft)	Din
Outside diameter of VVRS structure, Dout (ft)	Dout
Elevation of top of concrete base, BASE (ft datum)	BASE
Hydraulic conductivity of filter media, k (ft/day)	k
<i>Design Criteria</i>	
Recovery Time, T (day):	T
Factor of Safety, FS (dimensionless):	FS
<i>Background Seepage</i>	
Ground water baseflow into pond, qb (gpm)	qb

**8.3 EXAMPLE #8 - RECOVERY ANALYSIS OF RETENTION POND WITH VVRS FILTERS**

(contained in data file EXAMPLE8.DAT)

The objective of this problem is to calculate the number of VVRS filters required for the pond described in Example #7. Recall that in Example #7 1227 lineal feet of side-bank filter was computed as the minimum requirement.

The typical VVRS for this pond will be embedded 3 feet below the pond bottom and will have a positive outfall with the same tailwater elevation as the side-bank filter (i.e., 0.9 ft below the pond bottom). Note that the bottom elevation of the treatment volume is computed from equation A.7 in Appendix B. Assume that the pond bottom is at an arbitrary elevation datum of +0.0 ft. The hydraulic conductivity of the VVRS filter media is 200 feet/day.

*Pond & Stormwater Volume Parameters*

Treatment volume, V (cubic feet):	340,000 ft <sup>3</sup>
Top elevation of treatment volume, TOP (ft datum)	4.0 ft datum
Bottom elevation of treatment volume, BOT (ft datum)	0.217 ft datum
Tailwater or invert elevation, TWL (ft datum)	-0.9 ft datum

*VVRS Details*

Inside diameter of VVRS structure, Din (ft)	2 feet
Outside diameter of VVRS structure, Dout (ft)	6 feet
Elevation of top of concrete base, BASE (ft datum)	-3.0 ft datum
Hydraulic conductivity of filter media, k (ft/day)	200 ft/day

*Design Criteria*

Recovery Time, T (day):	3 days
Factor of Safety, FS (dimensionless):	2

*Background Seepage*

Ground water baseflow into pond, qb (gpm)	20.22 gpm
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As noted on the output on Exhibit 8, the number of vertical hollow cylindrical sand filters (VVRS) recommended is 19 for this particular pond. For comparison, the same pond required 1227 lineal feet of side bank filter (Example #7) or 5118 lineal feet of lateral underdrain (Example #9).

**EXHIBIT 8  
PRINTOUT OF EXAMPLE PROBLEM #8**

**MODULE #7**  
**UNDERDRAIN DESIGN CALCULATIONS**  
**INPUT PARAMETERS & EXAMPLE PROBLEM**

## 9. UNDERDRAIN DESIGN CALCULATIONS

### 9.1 GENERAL

Conceptual model ..... see Figure 9-1 below  
Description of input parameters ..... see page 9-2  
Theory ..... see Appendix E  
Example Problems ..... pp. 9-3

Figure 9-1: Conceptual Model of Underdrain System

## 9.2 LIST OF INPUT PARAMETERS

PARAMETER DESCRIPTION	<u>SYMBOL</u>
<i>Pond &amp; Stormwater Volume Parameters</i>	
Treatment Volume, PAV (cubic feet):	PAV
Area at Top of Pond, ATOP (square feet):	ATOP
Vertical Distance From Natural Ground or Finished Grade to Pond Bottom (ft)	d
<i>Aquifer Parameters</i>	
Hydraulic Conductivity of Soil, K (ft/day):	K
Depth to Base of Aquifer From Pond Bottom, B (ft):	B
<i>Drain Parameters</i>	
Drain diameter, D (inch)	D
Thickness of gravel envelope above pipe, t (inch)	t
Depth of soil cover on underdrain, H (ft)	H
<i>Design Criteria</i>	
Recovery Time, T (day):	T
Factor of Safety, FS (dimensionless):	FS
Minimum distance between water table mound and pond bottom, R (ft)	R
<i>Background Seepage</i>	
Ground water baseflow into underdrained pond, qb (gpm)	qb
<i>Parameters to Compute Minimum Diameters of Lateral Drain and Main Drain</i>	
Manning's "n" Value For Lateral Drain Pipe (dimensionless)	n1
Slope of Lateral Drains, I1 (%)	i1
Maximum Length of Single Lateral Drain, l (ft)	l
Manning's "n" Value For Main Drain Pipe (dimensionless)	n2
Slope of Main Drain, I2 (%)	i2
Percent of Total Length of Laterals Connected To Main Drain, P (%)	P

**9.3 EXAMPLE #9 - DESIGN OF UNDERDRAIN POND**  
 (contained in data file EXAMPLE9.DAT)

Assume that the pond described in example #4 is to be designed as an underdrain system and an outfall is available to allow gravity flow of the subsurface drainage system. The following are the model input parameters.

*Pond & Stormwater Volume Parameters*

Treatment Volume, PAV (cubic feet):	340,000 ft <sup>3</sup>
Area at Top of Pond, ATOP (square feet):	100,000 ft <sup>2</sup>
Vertical Distance From Natural Ground or Finished Grade to Pond Bottom, d (ft)	5 ft

*Aquifer Parameters*

Hydraulic Conductivity of Soil, K (ft/day):	10 ft/day
Depth to Base of Aquifer From Pond Bottom, B (ft):	7 feet

*Drain Parameters*

Drain diameter, D (inch)	6 inches
Thickness of gravel envelope above pipe, t (inch)	3 inches
Depth of soil cover on underdrain, H (ft)	2 feet

*Design Criteria*

Recovery Time, T (day):	3 days
Factor of Safety, FS (dimensionless):	2
Minimum distance between water table mound and pond bottom, R (ft)	0.5 feet

*Background Seepage*

Ground water baseflows into underdrained pond, qb (gpm)	20.22 gpm
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*Parameters to Compute Minimum Diameters of Lateral Drain and Main Drain*

Manning's "n" Value For Lateral Drain Pipe (dimensionless)	0.015
Slope of Lateral Drains, I1 (%)	0.45%
Maximum Length of Single Lateral Drain, l (ft)	200 feet
Manning's "n" Value For Main Drain Pipe (dimensionless)	0.016
Slope of Main Drain, I2 (%)	0.50%
Percent of Total Length of Laterals Connected To Main Drain, P (%)	25%

As noted on the output in Exhibit 9, the required lateral drain spacing is approximately 20 feet and a total lateral drain length of 5118 feet is required. A lateral drain diameter of 6 inches is adequate, but an 8-inch diameter main drain pipe is needed. The outfall will have to accommodate at least 2.67 cfs during recovery of the treatment volume.

**EXHIBIT 9  
PRINTOUT OF EXAMPLE PROBLEM #9**

**EXHIBIT 9 (continued)  
PRINTOUT OF EXAMPLE PROBLEM #9**

**APPENDIX A**  
**METHODOLOGY FOR ESTIMATING GROUND WATER BASEFLOW INTO PONDS**

**An Application of MODFLOW To Estimate Ground Water  
Drawdown Impact Distances and Ground Water Baseflows  
Into Ponds, Ditches, Trenches, Roadway Underdrains, Borrow Pits, Etc.**

**by  
Devo Seereeram, Ph.D., P.E.**

## **ABSTRACT**

This is an explanation of a simple, rational procedure for estimating drawdown impact distances and background ground water inflow into ponds (or other systems) where the control water level is established below the normal seasonal high water table. The need for a consistent, easily understood calculation of this type is becoming much more important as the study of wetland dehydration and other ground water resource impacts are areas of increasing concern.

## **INTRODUCTION**

Except for experienced ground water modelers, the USGS MODFLOW computer code is not widely used by the engineering practitioners due to the time-consuming nature and complexity in setting up the input files for these models and interpreting their output. However, PONDS has been developed as a convenient pre-processor and post-processor for the USGS MODFLOW model which allows menu-driven setting up of a problem using easily understood inputs and viewing meaningful results in a matter of minutes. The following describes how PONDS is used for calculating simultaneously the drawdown versus distance curve and the background ground water flow rate into a pond or other dewatering system such a drainage ditch, canal, underdrain, interceptor trench, borrow pit, etc.

## **GENERAL EXPLANATION OF PHENOMENA**

Drawdown is a transient phenomena which takes place as the ground water level rises in response to rainfall recharge. During the wet season (June, July, August, and September = 120 days approx.), it rises from its normal seasonal low level to its seasonal high level. It is well known that lateral ground water seepage rates are negligible compared to vertical recharge rates in shallow aquifer regimes in Florida. Lateral ground water seepage rates are generally less than 150 feet per year.

Depending on the hydrologic soil group, slope, vegetative cover, etc., natural recharge to the uppermost aquifer (i.e., rainfall-evapotranspiration-runoff) varies. On high water table sites (such as Myakka and Smyrna fine sands), the rainfall recharge is about 10 to 12 inches during the wet season, which is consistent with a 4 feet rise in the water table. This assumes a specific yield of 25% for the uppermost aquifer which is reasonable. On deeper sands such as Candler and Astatulas, rainfall recharge may be 1.5 to 2 times as high during the wet season resulting in a water table rise of as much as 8 feet. This is consistent with the range of seasonal fluctuations typically found on the high and dry sand hills. A separate manual has been prepared by the author on estimating the seasonal high water table and its fluctuation and the interested reader may contact the author on how to obtain this manual.

It will be the geotechnical engineer's or hydrogeologists' judgement to select the applicable water table rise (i.e., rainfall recharge) in the uppermost aquifer during the wet season or some other design event such as a 25year-24 hour storm. As the water table rises during the wet season and reaches the control elevation in the pond, the dewatering effect causes a lowering of the water table adjacent to the pond (or ditch, underdrain, borrow pit, etc.), and the cone of depression spreads laterally with time and with the rate of recharge outside the pond. The most critical case is at the end of the wet season or at the end of the study period when the the water table peaks following a design storm event.

## APPROACH & METHODOLOGY

For computational purposes, the hydrostratigraphy is characterized as a 1-layered, unconfined aquifer system which is characterized based on site-specific data usually contained in the project geotechnical report.

The computer model chosen for the analysis of the dewatering due to the pond is titled, "A Modular Three-Dimensional Finite-Difference Groundwater Flow Model" by McDonald and Harbaugh (1984, U.S. Geological Survey). This model simulates ground water flow in three dimensions by writing a Taylor expansion for the system of second order partial differential equations governing steady state or transient flow through porous media. A transient simulation is appropriately utilized in the simulation of the dewatering.

## Model Grid

The areal extent of the model subdomain for this analysis (or the domain of influence) is defined based on simple theoretical considerations and experience with maximum impact distances for the real world systems being analyzed. A grid generation routine has been developed to automatically produce a rectangular grid based on pond dimensions (i.e., equivalent length and width) to keep the boundaries of the grid at conservative distances of 1000 to 2000 feet from all sides of the ponds. For a width (or length) dimension of 2 to 3 feet (such as a subsurface drain for a road), the distance to the boundary is approximately 1000 feet and increases to 2250 feet for pond widths (or lengths) up to and greater than 700 feet. The pond is idealized as an equivalent rectangle occupying the central portion of the model area.

The model domain is divided into 30 rows and 30 columns, with a grid spacing that increases with distance from the edge of the pond and adheres to the conventions on maximum ratio of the spacing of adjacent cells, etc.

## Boundary & Initial Conditions

No flow cells are set at the outermost periphery of the finite difference grid. This boundary condition allows the water table to rise uniformly in all cells outside the influence of the pond.

The starting water table is initialized at a uniform level in all cells and is set equal to the wet season elevation minus the anticipated rise during the time specified for the input water table rise (such as the duration of a storm event or the entire wet season). With typical water table gradients of .25%, the assumption that the initial water table is flat is not an oversimplification when one considers that the radius of influence is usually less than 500 feet or so.

The heads in all cells, except at the boundaries, are free to vary during the transient simulation. Within the pond, however, drain cells are used to constrain the maximum level at the specified control elevation.

## Stress Periods

The total simulation time (or time specified for the water table rise to occur) is divided into six equal stress periods for computational accuracy. For additional numerical precision, the last (sixth) stress period is divided into 10 time steps.



## Recharge & Drains

Recharge to the surficial aquifer is used to simulate the rise of the water table. The recharge rate is assumed to apply uniformly during the length of the loading period and its magnitude is selected to produce the input rise in the water table factoring in the specific yield (or fillable porosity) of the aquifer.

Within the pond, recharge is used to model a simultaneous rise of the pond water level with the aquifer water table up to the pond control level, beyond which the pond is controlled at its control level for the remainder of the simulation using the MODFLOW DRAIN module to represent the pond cells. The summation of the discharge rates from the drains for the last stress period minus the direct rate of recharge on the pond cells represents the ground water baseflow into the system.

## Hydrostratigraphic Idealization

The results of the field and laboratory investigation documented in the site-specific geotechnical report are used to input the following idealized aquifer parameters:

### Surficial unconfined aquifer (Layer #1)

- Elevation of bottom of aquifer
- Hydraulic conductivity of aquifer. Note that in the pond cells the hydraulic conductivity is set to a high value of 1000 feet/day to represent open water.
- Specific yield of aquifer
- Initial ground water table elevation = seasonal high water table minus rise of water table following a specified rainfall event such as a normal wet season, 25 yr-24hr storm, etc.

Example problems are described in the body of the USER MANUAL.

**APPENDIX B**  
**THEORY FOR ANALYSIS OF POND RECOVERY USING**  
**SIDE-BANK OR POND BOTTOM FILTERS**

**insert paper from Scott Hayes**

## **APPENDIX C**

### **THEORY FOR RECOVERY ANALYSIS OF DRY OR WET BOTTOM RETENTION PONDS USING SIMPLIFIED ANALYTICAL MODEL**



## THEORY

The Simplified Analytical Method was developed to compute the time for recovery of retention ponds or exfiltration trenches in water table aquifers. The assumptions of the theory are listed below:

- 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.
- The pond fills up suddenly with the treatment volume (i.e., a slug loading). This assumption is consistent with SJRWMD criteria which does not allow for ground water or surface water discharge during the storm event.
- 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, the volume of water which infiltrates out of the pond/trench is equal to the volume of water in soil storage in the triangular saturated prism at any instant.
- 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 Dupuit-Forchheimer theory loses accuracy if the depth of the impermeable layer increases, because of the increased importance of vertical flow. Bouwer (1969) found that the Dupuit-Forchheimer theory gave reasonable results if the distance of the impermeable layer below the pond bottom was not more than twice the width of the water level in the pond.

Therefore, twice the pond width should be greater than the pond bottom elevation minus the base of aquifer elevation for the theory to be valid.

The model predicts the time for recovery of the treatment volume and the radius of influence of the hydraulic loading in the pond as derived on page C-1.

The computational steps in using the methodology are described below, but first the notation is presented.

## NOTATION

### Pond Parameters

1. Equivalent Pond Length, L (ft)
2. Equivalent Pond Width, W (ft)
3. Pond Bottom Elevation, PB (ft datum)
4. Porosity of Material Within Pond, p (%)  
Note: if open pond, p = 100%; if exfiltration trench, p < 100%

### Aquifer Parameters

5. Base of Aquifer Elevation, B (ft datum)
6. Water Table Elevation, WT (ft datum)
7. Horizontal Saturated Hydraulic Conductivity,  $K_h$  (ft/day)
8. Fillable Porosity, n (%)
9. Vertical Unsaturated Infiltration Rate,  $I_v$  (ft/day) (OPTIONAL)

### Runoff Parameters

11. Runoff Volume, V (cubic feet)
12. Percent Recovery of Runoff Volume, PV (%)

## COMPUTATIONAL STEPS

1. Compute available volume in soil voids immediately below pond  $V_1$ :

$$V_1 = L W n (PB - WT) \quad \text{if } (PB - WT) \geq 0$$

$$V_1 = 0 \quad \text{if } (PB - WT) < 0$$

2. Compute recovery time for unsaturated vertical flow  $T_1$ :

$$T_1 = \frac{V_1}{I_v L W}$$

3. Compute volume to be recovered by saturated lateral flow  $V_2$ :

$$V_2 = (PV * V) - V_1$$

4. If  $V_2 < 0$ , then stop analysis and report  $T_1$  as recovery time with  $T_2$  (due to saturated flow) as zero. Also, if  $(PV \times V) \leq V_1$ , do not perform saturated flow analysis. If  $V_2 > 0$  and  $(PV \times V) > V_1$ , then continue with saturated analysis following the additional steps outlined below.

5. Compute  $h_{\max}$  as

$$h_{\max} = \frac{V - V_1}{p \times L \times W} + (PB - WT) \quad \text{if } (PB - WT) \geq 0$$

$$h_{\max} = \frac{V - V_1}{p \times L \times W} - WT \quad \text{if } (PB - WT) < 0$$

6. Compute  $h_{\min}$  as

$$h_{\min} = h_{\max} - \frac{V_2}{p \times L \times W}$$

7. Compute initial saturated thickness of aquifer as  $b = WT - B$ .

8. Compute recovery time for saturated flow component  $T_2$  using equation (6) on page C-1.
9. Compute total recovery time  $T = T_1 + T_2$
10. Compute radius of influence "R" substituting  $h_{min}$  for  $h$  in equation (2) on page C-1.

## **APPENDIX D**

### **MODEL FOR RECOVERY ANALYSIS OF DRY OR WET BOTTOM RETENTION PONDS DURING & FOLLOWING TRANSIENT HYDRAULIC LOADING**

**MODELING OF SIMULTANEOUS GROUND WATER AND SURFACE WATER  
DISCHARGES DURING AND FOLLOWING TRANSIENT-VARIABLE RATE  
HYDRAULIC LOADING OF STORMWATER DETENTION PONDS**

by  
Devo Seereeram, Ph.D., P.E.

## INTRODUCTION

The analytical model of retention pond recovery described in Appendix C of this manual assumes that the runoff volume instantaneously fills the pond. This assumption is appropriate for the simulation of recovery of the treatment or water quality volume, or other situations where regulations do not allow credit for ground infiltration or discharges to surface waters during storm events. However, particularly for well drained soils with deep water tables, a significant portion of the runoff volume entering a pond can infiltrate into the aquifer during the storm event. By taking credit for the infiltration which occurs during the storm event, a design engineer can optimize the detention volume required for attenuation of peak flow rates.

Such an analysis necessarily combines elements of surface water and subterranean hydrology since inflows to the pond and the ground water and surface water discharges take place simultaneously. In addition, the methodology must allow for time-dependent, variable inflow rates as generated by synthetic rainfall distributions for design storm events.

The computer code selected for this coupled ground water - surface water interaction model of stormwater ponds is the USGS MODFLOW computer program (Mc Donald and Harbaugh, 1988). This is a fully three-dimensional, finite difference numerical technique for modeling ground water flow. The original MODFLOW code was modified by the author to include modules which have the ability to simulate discharges through typical pond control structures such as weirs and orifices. The modified MODFLOW code represents an integrated approach which circumvents the difficulty and imprecision which can result from linking dedicated surface water and ground water models. It is a true ground water - surface water interaction model since the "weir" discharge module is incorporated directly into the system of finite difference equations solved in MODFLOW. This ensures that the conditions of continuity and volume balance are preserved for each time step.

Another major modification to MODFLOW is the introduction of a stage-dependent storage coefficient for the grid cells representing the pond. This feature allows the model to

simulate the true stage-volume relationship of the pond instead of treating it as a rectangular box.

Since MODFLOW does not simulate unsaturated flow, a separate routine was written by the author to calculate ground water and surface water discharges for unsaturated flow which occurs prior to saturated flow.

PONDS essentially acts as a pre- and post-processor to these saturated flow and unsaturated flow calculation routines. It allows the user to input parameters and interpret results in formats recognizable to stormwater management design engineers who do not have a working knowledge of MODFLOW. PONDS also has a hydrograph generation routine which can create hydrographs using the SCS methodology and facilitate the importation of this input data directly into the model. The program can also import hydrographs from the adICPR computer program.

## CONCEPTUAL MODEL

A conceptual model is a pictorial representation of the ground water flow system. In the practice of developing a conceptual model, it is desirable to strive for parsimony, by which it is implied that the conceptual model has been simplified as much as possible yet retains enough complexity so that it adequately reproduces system behavior. The conceptual model described hereunder can be applied to the majority of design situations in the state of Florida.

Figure D-1 illustrates the conceptual stormwater management pond and aquifer system. The key elements of this conceptual model are as follows:

- The receiving aquifer system is idealized as a laterally infinite, single-layered, homogenous, isotropic, unconfined aquifer of uniform thickness, with a horizontal pre-loading water table.
- The three dimensional shape of the pond is defined by a stage-area relationship.
- Transient analysis is discretized into an appropriate number of time steps or stress periods.
- Time-varying runoff is input in the form of a hydrograph which is a table of elapsed time (in minutes) versus inflow rate (cfs). The hydrograph data set also includes a recharge value which can simulate time-varying fluctuation of the water table adjacent to the pond during the storm event.

- **Transient, lateral saturated-flow ground water discharge is modeled using a modified version of the USGS MODFLOW numerical technique. The details of the MODFLOW model are described in a subsequent section.**
- **Transient vertical unsaturated flow is modeled using an algorithm developed by the author. The details of this algorithm are described in the next section.**
- **Surface water discharges through free (or unsubmerged) weirs, orifices, or notches can be modeled during unsaturated or saturated flow. A module to simulate these discharges have been developed by the author and incorporated into MODFLOW and the unsaturated flow algorithm.**
- **Dupuit-Forchheimer assumptions are applicable for saturated flow; i.e.,**
  1. **Flow is considered to be purely horizontal**
  2. **Flow is assumed to be uniformly distributed with depth**

**The Dupuit-Forchheimer theory loses accuracy if the depth of the impermeable layer increases, because of the increased importance of vertical flow. Bouwer (1969) found that the Dupuit-Forchheimer theory gave reasonable results if the distance of the impermeable layer below the pond bottom was not more than twice the width of the water level in the pond. Therefore, twice the pond width should be greater than the pond bottom elevation minus the base of aquifer elevation for the theory to be valid.**

Figure D-1: Conceptual Model of Refined Method of Retention Pond Recovery

### UNSATURATED FLOW ALGORITHM

Unsaturated flow is computed on an incremental basis for each time step. If there is discharge through the weir or orifice during unsaturated flow, the algorithm iteratively solves for the stage elevation which satisfies the following volumetric balance:

**Runoff Inflow = Discharge Through Weir + Unsaturated Infiltration + Increase in Storage**

Schematic D-1 shows the logic of this algorithm. This model assumes that unsaturated flow is:

- i) only taking place through the bottom area of the pond at an infiltration rate equal to  $I_v$  (ft/day) (note: the bottom area of the pond changes as the water level changes),
- ii) purely vertical,
- iii) filling the soil voids directly beneath the water surface, and
- iv) complete once the soil voids beneath the pond are filled.

If the inflow rate is faster than the rate at which water can vertically infiltrate through the effective pond bottom, water accumulates in the pond. If water accumulates in the pond above the weir discharge elevation, discharge is modeled using the weir equations.

This is a relatively simple representation but it is one that is applicable to most cases where the water table is not deep (say less than 5 feet below the pond bottom). For situations where the water table is over 5 feet below the pond bottom, lateral flow can be significant during the unsaturated flow regime but this representation will give conservative results.

The two input parameters for this module are the vertical infiltration rate (for unsaturated conditions) and the maximum area available for vertical unsaturated infiltration.

## SCHEMATIC D-1: ALGORITHM FOR UNSATURATED FLOW CALCULATIONS

Step #1

Compute inflow volume increment ( $\Delta V_i$ ) into pond during computational time step "i" of duration  $\Delta t_i$ . Add this volume increment to the volume of water (if any)  $V_i$  which was in the pond at start of time step "i" to obtain  $V$ .

Step #2

Compute maximum potential volume  $(V_{\max})_i$  that can be infiltrated during the time step "i" based on

bottom area ( $A_u$ ) and infiltration rate ( $I$ ) available

Step #3

Compute the available storage beneath the pond at the start of time step "i"  $[(V_{av})_i]$  based on the bottom area  $A_u$ , the fillable porosity of the soil, and the distance between the water table and the pond bottom.

Step #4

If  $V$  is less than  $(V_{max})_i$  and also less than  $(V_{av})_i$ , then the volume infiltrated during the time step is equal to  $V$  and the pond is dry during this time step. If  $V$  is greater than  $(V_{max})_i$ , then  $V_{excess}$  is the

Step #5

balance which goes into storage. If  $V_{excess}$  is greater than storage volume below the lowest discharge elevation, then a portion of  $V_{excess}$  is discharged. MODFLOW module for saturated flow analysis iteratively solving for the

stage height which produces a volume balance of the following equation:

$$\text{discharge volume} = \text{Vol. above lowest discharge} + V_{excess} - \text{Volume below lowest discharge}$$

**SATURATED FLOW ALGORITHM**

**Saturated flow commences after unsaturated flow is complete. It is possible to deactivate the unsaturated flow algorithm and assume saturated flow conditions at the initiation of the storm. If such an option is selected, the model assumes that the water table directly beneath the pond is at the pond bottom elevation at time  $t = 0$ .**

Methodology

The ground water flow model used in this evaluation is a modified version of the USGS MODFLOW model. A new module has been written by the author and incorporated directly into the original MODFLOW code to simulate discharge through unsubmerged weirs, orifices, and notches.

### Finite Difference Grid

Since the conceptual model is symmetrical with respect to the major axes of the equivalent rectangular pond, it is only necessary to model a symmetrical quarter of the pond. Table D-1 (on page D-13) lists the grid dimensions and spacing of the axi-symmetric quarter rectangle. As noted, the grid comprises 60 rows by 60 columns with a nodal spacing that increases with distance from the edge of the pond.

"Distant" specified head boundaries are arbitrarily located about 1000 feet from the pond to ensure that the hydraulic stresses in the pond will not reach the boundaries during the simulation. This is realistic for most practical cases. Since the problem is axisymmetric, no flow boundary cells are set along the axes.

The grid is designed so that nodes are closely spaced where steep hydraulic gradients are expected (i.e., immediately adjacent to the pond). Nodal spacings are increased by no more than 1.5 times the previous nodal spacing. Note that this nodal spacing convention does not apply within the pond itself since the water surface is "flat" within these cells (due to the high hydraulic conductivity) and the approximation to the second derivative is zero (see explanation on page 64-65 of Anderson and Woessner, 1992).

### Boundary & Initial Conditions

As mentioned previously specified head boundaries are set along Row #1 and Column #60 of the finite difference mesh. These fixed heads are set at the input value of the water table elevation. No-flow cells are set along the axi-symmetric boundaries (i.e., Row #60 and Column #1) of the grid. All other nodes within the grid are free to vary during the simulation. Within, the cells representing the pond however, the heads are also influenced by the discharge through the weirs or orifices (which are akin to MODFLOW DRAINS with a non-linear head vs. discharge relationship). The heads in the variable head cells outside the pond are initialized to the input water table elevation. On the other hand, the heads in the variable head cells within the pond are initialized to the pond bottom elevation if the ground water mound is expected to intersect the pond bottom. There is an option for setting the initial head within the pond cells to the input water table elevation if the unsaturated flow option is not activated.

### Hydrostratigraphic Idealization and Parameters

The MODFLOW model is a 1-layered system which requires specification of the following water table aquifer parameters:

- a. Horizontal saturated hydraulic conductivity
- b. Fillable porosity of aquifer
- c. Base elevation of aquifer
- d. Initial water table elevation (usually the seasonal high water table)

If the ground water mound is expected to intersect the pond bottom, the hydraulic conductivity of the cells within the pond is automatically set to a high value (1000 ft/day). However, if the user indicates that the ground water mound will not intersect the pond bottom, the program appropriately sets the hydraulic conductivity within the pond cells to the same value as the aquifer.

#### Time Discretization & Recharge

Each time increment of the input hydrograph, where saturated flow occurs, is equivalent to a MODFLOW stress period. Therefore, if the hydrograph has 600 data points there will be 599 MODFLOW stress periods. Volumetric inflow to the pond is simulated using the MODFLOW recharge package. The volume entering the pond during a time increment of the input hydrograph is converted to an equivalent uniform recharge rate over the pond cells. Similarly, time-varying recharge to the water table (in ft/day) outside the pond may also be specified on each line of the hydrograph input.

Influence of Adjacent Ditches

Adjacent ditches or canals are simulated using specified head cells in the corresponding row or column which is specified by the user by an offset distance from the edge of the pond. Since the model is axi-symmetric, if a ditch is specified parallel to the X-axis, then the model assumes that there are ditches symmetrically placed with respect to that axis. In other words, if the user specifies a ditch 40 ft east of the pond, the model assumes that there is another ditch 40 ft west of the pond (i.e., a mirror image). This limitation must be appreciated.

Simulation of Discharge Through Unsubmerged Weirs, Notches, and Orifices

The MODFLOW DRAIN module is designed to simulate the effects of features, such as agricultural drains, which remove water from the aquifer at a rate proportional to the difference between the head in the aquifer and some fixed head or elevation, so long as the head in the aquifer is above that elevation. There is no discharge if the head in the aquifer falls below the control level of the drain.

The MODFLOW DRAIN package models discharge using the following linear relationship:

$$q = C_D \langle h - d \rangle \tag{1}$$

where C = drain conductance (ft<sup>2</sup>/s)  
 q = discharge in ft<sup>3</sup>/s  
 h = head in the aquifer (ft datum)  
 d = drain invert or tailwater elevation

Note that the operator < > indicates that the value is always positive and zero if it is negative.

The general form of the equation for free discharge (i.e., unsubmerged conditions) through a weir or orifice is

$$q = C_1 \langle h - d_1 \rangle^{n_1} + C_2 \langle h - d_2 \rangle^{n_2} + C_3 \langle h - d_3 \rangle^{n_3} + \dots \tag{2}$$

where C<sub>1</sub> = weir discharge coefficient  
 n = weir exponent

$d$  = weir discharge elevation or centroid of orifice

To minimize modifications to the original MODFLOW code, the drain conductance  $C$  of equation (1) is made a function of the head difference (between the aquifer and the discharge elevation) by combining equations (1) and (2) to obtain:

$$C = C_1 \langle h - d \rangle^{n-1} \tag{3}$$

Equation (3) uses two parameters to model discharge from a DRAIN cell:  $C_1$  and  $n$ , whereas the original MODFLOW code used only the conductance parameter  $C$  [as in equation (1)].  $C_1$  is referred to herein as the Modified Drain Conductance.

The MODFLOW model uses 49 drain cells to represent the axisymmetric quarter of an equivalent rectangular pond. These 49 drain cells are used to model weir discharge as outlined below for up to 3 discharge structures.

**CASE #1: 1 DISCHARGE STRUCTURE**

	2	3	4	5	6	7	8
53	w1						
54	w1						
55	w1						
56	w1						
57	w1						
58	w1						
59	w1						

where the modified drain conductance for weir 1 is:

$$Conductance = \frac{C_1 \times L \times 86400}{4 \times 49}$$

## CASE #2: 2 DISCHARGE STRUCTURES

	2	3	4	5	6	7	8
53	w1	w2	w1	w2	w1	w2	w1
54	w2	w1	w2	w1	w2	w1	w2
55	w1	w2	w1	w2	w1	w2	w1
56	w2	w1	w2	w1	w2	w1	w2
57	w1	w2	w1	w2	w1	w2	w1
58	w2	w1	w2	w1	w2	w1	w2
59	w1	w2	w1	w2	w1	w2	w1

where the modified conductance for weir 1 is as follows:

$$Conductance = \frac{C_1 \times L_1 \times 86400}{4 \times 25}$$

and for weir 2 is

$$Conductance = \frac{C_2 \times L_2 \times 86400}{4 \times 24}$$

CASE #3: 3 DISCHARGE STRUCTURES

	2	3	4	5	6	7	8
53	w1	w2	w3	w1	w2	w3	w1
54	w2	w3	w1	w2	w3	w1	w2
55	w3	w1	w2	w3	w1	w2	w3
56	w1	w2	w3	w1	w2	w3	w1
57	w2	w3	w1	w2	w3	w1	w2
58	w3	w1	w2	w3	w1	w2	w3
59	w1	w2	w3	w1	w2	w3	w1

where modified drain conductance for weir 1 is as follows:

$$Conductance = \frac{C_1 \times L_1 \times 86400}{4 \times 17}$$

modified drain conductance for weir 2 is as follows:

$$Conductance = \frac{C_2 \times L_2 \times 86400}{4 \times 16}$$

and modified drain conductance for weir 3:

$$Conductance = \frac{C_3 \times L_{32} \times 86400}{4 \times 16}$$

This modified drain conductance equation has been verified by testing it against closed form solutions and by independent evaluators (see, for example, Seebali and Sua, 1993). A verification problem is present below.

For a circular sharp-edged orifice, the drawdown equation is as follows:

$$h_1^{1/2} - h_2^{1/2} = \frac{C \times A \times t}{2 \times L \times W}$$

- where C =  $0.6 \times \{2g\}^{1/2}$  .....weir discharge coefficient  
g = 32.16 ft/sec
- A = area of orifice in ft<sup>2</sup>
- t = drawdown time in seconds
- L = pond length in ft
- W = pond width in ft
- h<sub>1</sub> = initial head in ft
- h<sub>2</sub> = final head in ft

Example problem:

- C = 4.812 ft<sup>1/2</sup>/sec
- A = .05811 ft<sup>2</sup>
- t = 216000 sec (i.e., 60 hr)
- L = 431 ft
- W = 220 ft
- h<sub>1</sub> = 1.3 ft
- h<sub>2</sub> = ? ft

From the equation above, h<sub>2</sub> = 0.675 ft

If a low permeability is set for the aquifer (say 0.05 ft/day), PONDS reproduces this closed-form solution.

Table D-1: Grid Dimensions For Axisymmetric Representation of Rectangular Pond (L = Length, W = Width)

Offset Distance From Long Edge of Pond (ft)	Column Number	Cell Type	Column Spacing (ft)	Row Number	Row Spacing (ft)	Offset Distance From Short Edge of Pond (ft)
	1	no flow	2	60	2	
	2	pond w/weir	$((L/2)-2)/6$	59	$((W/2)-2)/6$	
	3	pond w/weir	$((L/2)-2)/6$	58	$((W/2)-2)/6$	
	4	pond w/weir	$((L/2)-2)/6$	57	$((W/2)-2)/6$	
	5	pond w/weir	$((L/2)-2)/6$	56	$((W/2)-2)/6$	
	6	pond w/weir	$((L/2)-2)/6$	55	$((W/2)-2)/6$	
	7	pond w/weir	$((L/2)-2)/6$	54	$((W/2)-2)/6$	
	8	pond w/weir	2	53	2	
1	9	variable head	2	52	2	1
3	10	variable head	2	51	2	3
5	11	variable head	2	50	2	5
7	12	variable head	2	49	2	7
9	13	variable head	2	48	2	9
11	14	variable head	2	47	2	11
13	15	variable head	2	46	2	13
15	16	variable head	2	45	2	15
17	17	variable head	2	44	2	17
19	18	variable head	2	43	2	19
21	19	variable head	2	42	2	21
23	20	variable head	2	41	2	23
25.5	21	variable head	3	40	3	25.5
28.5	22	variable head	3	39	3	28.5
31.5	23	variable head	3	38	3	31.5
35	24	variable head	4	37	4	35
39	25	variable head	4	36	4	39
43	26	variable head	4	35	4	43
47.5	27	variable head	5	34	5	47.5
52.5	28	variable head	5	33	5	52.5
57.5	29	variable head	5	32	5	57.5
64	30	variable head	8	31	8	64

Offset Distance From Long Edge of Pond (ft)	Column Number	Cell Type	Column Spacing (ft)	Row Number	Row Spacing (ft)	Offset Distance From Short Edge of Pond (ft)
CONTINUED ON NEXT PAGE						
72	31	variable head	8	30	8	72
80	32	variable head	8	29	8	80
88	33	variable head	8	28	8	88
97	34	variable head	10	27	10	97
107	35	variable head	10	26	10	107
117	36	variable head	10	25	10	117
127	37	variable head	10	24	10	127
139.5	38	variable head	15	23	15	139.5
154.5	39	variable head	15	22	15	154.5
169.5	40	variable head	15	21	15	169.5
189.5	41	variable head	25	20	25	189.5
214.5	42	variable head	25	19	25	214.5
239.5	43	variable head	25	18	25	239.5
264.5	44	variable head	25	17	25	264.5
297	45	variable head	40	16	40	297
337	46	variable head	40	15	40	337
377	47	variable head	40	14	40	377
417	48	variable head	40	13	40	417
457	49	variable head	40	12	40	457
497	50	variable head	40	11	40	497
542	51	variable head	50	10	50	542
592	52	variable head	50	9	50	592
642	53	variable head	50	8	50	642
692	54	variable head	50	7	50	692
742	55	variable head	50	6	50	742
792	56	variable head	50	5	50	792
847	57	variable head	60	4	60	847
907	58	variable head	60	3	60	907
967	59	variable head	60	2	60	967

Offset Distance From Long Edge of Pond (ft)	Column Number	Cell Type	Column Spacing (ft)	Row Number	Row Spacing (ft)	Offset Distance From Short Edge of Pond (ft)
1027	60	Constant Head	60	1	60	1027

**References**

Anderson, Mary P., and W.W. Woessner. *Applied Groundwater Modeling*. Academic Press, Inc., Harcourt Brace Jovanovich Publishers, San Diego, California. 1992.

Mc Donald, M. G., and A.W. Harbaugh. *A Modular Three-Dimensional Finite-Difference Ground-Water Flow Model*. Techniques of Water Resources Investigations of the U.S. Geological Survey. Chapter A1, Book 6., 1988.

Sebaali, S.J. and David Sua. November 30, 1993. *Review of PONDS Software*. Brooksville Permitting Department, Southwest Florida Water Management District, Brooksville, Florida.

## **APPENDIX E**

### **METHODOLOGY FOR RECOVERY ANALYSIS OF "UNDERDRAINED" RETENTION PONDS (i.e., retention ponds with subsurface drains in the natural soil)**

**DISCUSSION OF METHODOLOGY FOR RECOVERY  
ANALYSIS OF "UNDERDRAINED" RETENTION PONDS**

by  
**Devo Seereeram, Ph.D., P.E.**

## INTRODUCTION

Underdrained retention ponds is a relatively new stormwater management system permitted by St. Johns River Water Management District (effective late 1991). In the past, the term "underdrain" has been associated with systems which are formally known as filtration systems. The new underdrain system is different from typical side-bank and pond-bottom filters in many respects:

1. Generally, on-line underdrain systems require retention of 66% of the first flush retention volume required for filtration systems. As a result, underdrain systems use smaller ponds which are generally the same size as dry retention ponds. In addition, filtration systems are generally not permitted for systems which have a contributing drainage area greater than 10 acres in size. Filtration systems are sometimes discouraged where there is discharge to sensitive water bodies.
2. The underdrain consists of perforated tubing with a filter fabric sock. The tubing is buried at least 2 feet below the pond bottom in the natural soil. This subsurface drainage system is laid out like an agricultural ground water control system to suppress the water table beneath the pond and provide dissipation capacity for recovery of the treatment volume in a specified period. These subsurface drains are typically laid out in patterns of laterals connecting to a main line. Figure E-1 shows typical layouts of these drainage networks.
3. In addition to computing the total length of subsurface drains required, the drain spacing and layout is critical. The drain pipe sizes must also be checked.

## THEORY

The theory for the design of underdrain systems is based on the ellipse theory illustrated on Figure E-2. This theory is well known to agricultural drainage engineers and is described in the book "Drainage of Agricultural Land: A Practical Handbook For the Planning, Design,

**Construction, and Maintenance of Agricultural Drainage Systems" by the Soil Conservation Service of the USDA.**

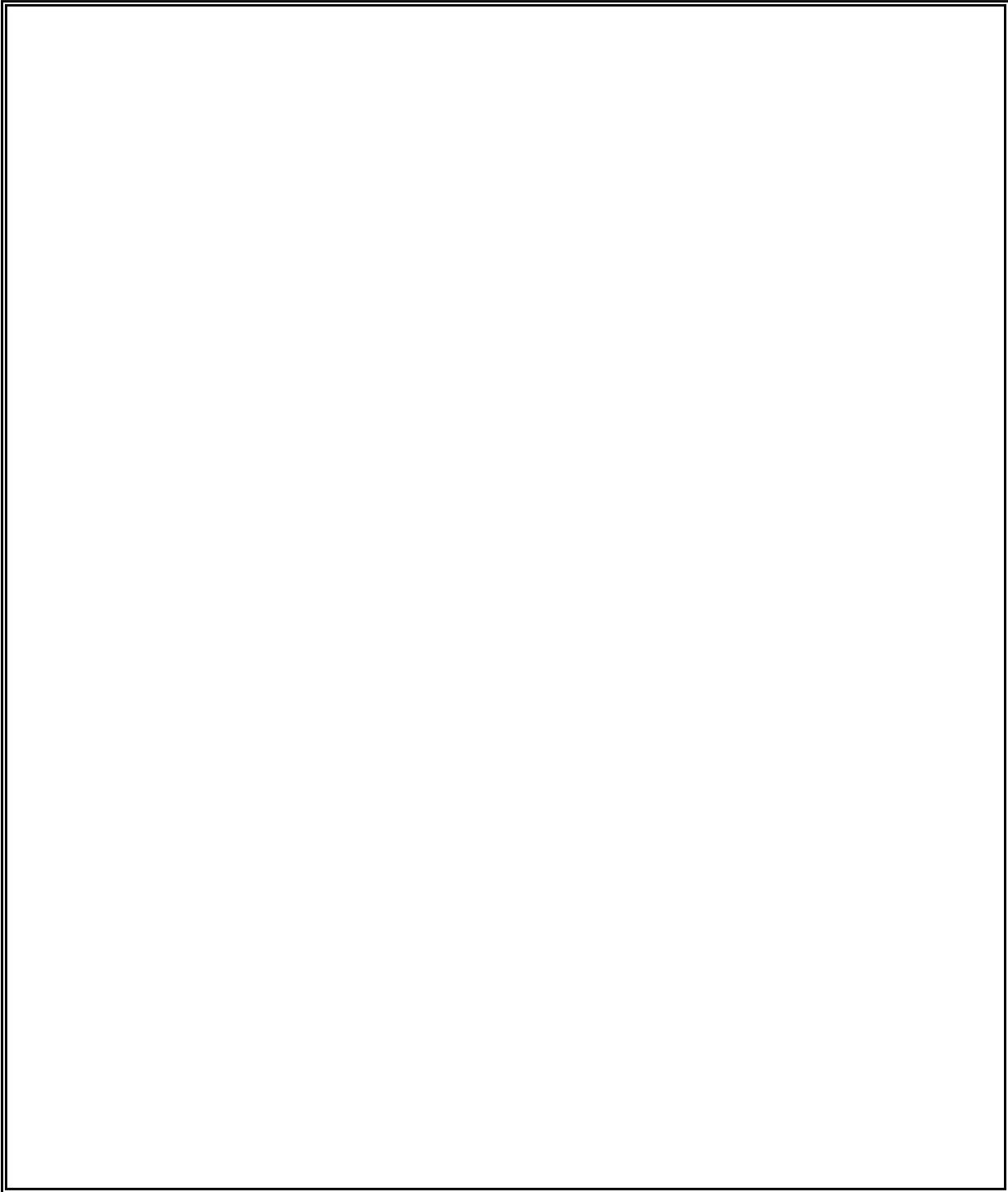


Figure E-1: Various layouts of subsurface drain pipes within pond bottom,(USDA 1973)

$$S = \sqrt{\frac{4 K (m^2 + 2 a m)}{q}}$$

**K = soil hydraulic conductivity (& has same units as recharge rate q)**

Figure E-2: Illustration of the ellipse theory for calculating drain spacing, USDA 1973)

NOTATION

(see Figure E-3 for description of parameters)

PARAMETER DESCRIPTION

SYMBOL

*Pond & Stormwater Volume Parameters*

Treatment Volume, PAV (cubic feet):	PAV
Area at Top of Pond, ATOP (square feet):	ATOP
Vertical Distance From Natural Ground or Finished Grade to Pond Bottom (ft) (Note: this parameter is not used in the calculations but is used to check the validity of the ellipse equation)	d

*Aquifer Parameters*

Hydraulic Conductivity of Soil, K (ft/day):	K
Depth to Base of Aquifer From Pond Bottom, B (ft):	B

*Drain Parameters*

Drain diameter, D (inch)	D
Thickness of gravel envelope above pipe, t (inch) (Note: set $t = 0$ if no gravel envelope)	t
Depth of soil cover on underdrain, H (ft) (Note: usually a minimum of 2 feet of cover is required)	H

*Design Criteria*

Recovery Time, T (day): (Note: usually 3 days)	T
Factor of Safety, FS (dimensionless): (Note: usually a minimum of 2 and applies to recovery time)	FS
Minimum distance between water table mound and pond bottom, R (ft) (Note: usually 0.5 ft or more)	R

*Background Seepage*

Ground water baseflow into underdrained pond, qb (gpm)	qb
--	----

Figure E-3: Cross-section of underdrain system showing interpretation of parameters

## NOTATION (continued)

<u>PARAMETER DESCRIPTION</u>	<u>SYMBOL</u>
<i>Parameters to Compute Minimum Diameters of Lateral Drain and Main Drain</i>	
Manning's "n" Value For Lateral Drain Pipe (dimensionless) (Note:        n = 0.015 for 3-6" diameter ADS corrugated polyethylene pipe) n = 0.016 for 8" diameter ADS corrugated polyethylene pipe)	n1
Slope of Lateral Drains, I1 (%) (Note:        0.3% to 2.0% recommended for 6" diameter pipe with n=0.015) 0.2% to 1.3% recommended for 8" diameter pipe with n=0.015)	i1
Maximum Length of Single Lateral Drain, l (ft)	l
Manning's "n" Value For Main Drain Pipe (dimensionless) (Note:        n = 0.015 for 3-6" diameter ADS corrugated polyethylene pipe) n = 0.016 for 8" diameter ADS corrugated polyethylene pipe)	n2
Slope of Main Drain, I2 (%) (Note:        0.3% to 2.0% recommended for 6" diameter pipe with n=0.015) 0.2% to 1.3% recommended for 8" diameter pipe with n=0.015)	i2
Percent of Total Length of Laterals Connected To Main Drain, P (%)	P

## VALIDITY CHECK

The ellipse theory is valid when  $B \leq B_{\max}$ , where

$$B_{\max} = 2 \times \left( d + H + \frac{t}{12} + \frac{D}{24} \right) - d$$

## COMPUTATIONAL STEPS

1. Compute volume of ground water baseflow during recovery period.

$$V = qb \times \frac{1440}{7.4808} \times \frac{T}{FS} \quad (\text{cubic feet})$$

Note qb is input in gpm and must therefore be converted to ft<sup>3</sup>/day.

2. Compute equivalent uniform recharge rate q [units will be ft/day]

$$q = \frac{PAV + V}{ATOP} \times \frac{FS}{T} \quad (\text{ft/day})$$

3. Compute the vertical distance between center of underdrain pipe and base of aquifer (parameter "a" in feet)

$$a = B - H - \frac{t}{12} - \frac{D}{24} \quad (\text{ft})$$

Note that the diameter of the drain pipe is input in inches and is converted to feet in the above equation as is the thickness of the gravel envelope.

4. Compute the vertical height of the water table mound above the center of the underdrain pipe (parameter "m" in feet)

$$m = H + \frac{t}{12} + \frac{D}{24} - R \quad (ft)$$

Note that the diameter of the drain pipe is input in inches and is converted to feet in the above equation as is the thickness of the gravel envelope.

5. Compute underdrain spacing (S) using the ellipse equation (units will be in feet)

$$S = \sqrt{\frac{4 K (m^2 + 2 a m)}{q}} \quad (ft)$$

6. Compute total length of laterals (L) required

$$L = \frac{ATOP}{S} \quad (ft)$$

7. Compute cumulative flow rate from the theoretical length of laterals (i.e., the flow rate through the pond outfall pipe from underdrain system)

$$Q = \frac{PAV + V}{T} \times \frac{FS}{86400} \quad (cfs)$$

8. Compute flow rate per lineal foot of lateral

$$q_l = \frac{Q}{L} \quad (cfs \text{ per lineal foot})$$

9. Compute cumulative flow rate for the longest lateral in the system (i.e., the lateral length specified on the input screen)

$$q_c = q_l \times l \quad (cfs)$$

10. Compute minimum pipe diameter from Manning's formula for the longest lateral in the system using the following equation:

$$D_l = 12 \times \sqrt[3]{\frac{4 \times 4^{2/3} \times n_l \times q_c}{1.48 \times \pi \times \sqrt{\frac{il}{100}}} \quad (inches)$$

11. Compute cumulative flow rate for the main line which is connected to P% of the total length of laterals in the system

$$q_m = Q \times \frac{P}{100} \quad (cfs)$$

12. Compute minimum pipe diameter from Manning's formula for the main line in the system using the following equation:

$$D_m = 12 \times \sqrt[3]{\frac{4 \times 4^{2/3} \times n_2 \times q_m}{1.48 \times \pi \times \sqrt{\frac{i_2}{100}}} \quad (inches)$$

## STANDARD NOTES

1. Laterals should be no farther than  $S/2$  from the top of the basin
2. A gravel envelope at least 3 inches thick is recommended around the underdrain pipes. If a gravel envelope is used, a filter fabric will be required around this envelope.
3. The underdrain pipe should have a filter fabric sock to prevent fines from moving into and clogging the perforated pipe.
4. Ensure outfall elevation for system will allow gravity flow without tailwater backpressure to the underdrains.
5. Theory is applicable where ground water flow is largely in a horizontal direction (i.e., natural gradients less than 1%).
6. Capped & sealed inspection and cleanout ports which extend to the ground surface are recommended at the following locations for each drain pipe: a) the terminus, and b) at every 400 feet or every bend of 45 or more degrees, whichever is shorter.
7. Underdrain basin should be stabilized with permanent vegetative cover.

## **APPENDIX F**

### **THEORY FOR RECOVERY ANALYSIS OF RETENTION PONDS USING VERTICAL HOLLOW CYLINDRICAL SAND FILTERS (aka VVRS)**

**RECOVERY ANALYSIS OF PONDS WITH  
VERTICAL HOLLOW CYLINDRICAL SAND FILTERS (aka VVRS)**

by  
**Devo Seereeram, Ph.D., P.E.**

**ABSTRACT**

Vertical hollow cylindrical sand filters are used to filter and then discharge stormwater retained in stormwater management ponds. They are known commercially as "Vertical Volume Recovery Structures" or VVRS, and are preferred as an alternative to conventional pond side-bank filters since they require less maintenance.

Stormwater regulations in the state of Florida specify the time period in which the treatment volume must be filtered from the pond to ensure that storage volume is available to accommodate runoff from the next storm. Typical regulations in Florida call for retaining the first 1 to 3 inches of stormwater runoff, depending on the sensitivity of the receiving water body, and filtering this retained runoff volume within 3 days.

This paper presents theoretical equations for estimating the number of vertical hollow cylindrical sand filters (or VVRS) required for recovery of the stormwater treatment volume in a prescribed time period. The derivation leads directly to a closed-form solution which is a falling head variant of the theory of horizontal flow through vertical sand drain wells presented previously by Cedergren (1977). The variables in the derived formula are:

- 1) the inside and outside diameters of the hollow cylindrical filter,
- 2) volume, and maximum & minimum stage elevations of treatment volume,
- 3) desired recovery time,
- 4) tailwater elevation of receiving water body and base elevation of VVRS,
- 5) hydraulic conductivity of filter media, and
- 6) factor of safety.

The theory also include allowances for direct and indirect ground water baseflows into the pond. Equations are also presented for computing maximum flow rates per structure to ensure outfall pipes are properly sized.

## DESCRIPTION OF VVRS

### Typical Detail

Figure F-1 shows a typical construction detail for a vertical hollow cylindrical sand filter or VVRS. The VVRS consists of two concentrically and vertically placed, perforated aluminum pipes which are set on a concrete base. Although not shown on this typical detail, it is recommended that future design details include geonets on the inside face of the outer cylinder and the outside face of the inside cylinder to ensure horizontal flow through the entire submerged area of the sand filter.

The diameters of the inside and outside pipes are selected to provide a minimum annular space of 2 feet. Inside diameters are typically about 2 feet and outside diameters about 6 feet. The annular space is backfilled with fine aggregate filter media forming a hollow cylindrical filter. As shown on Figure F-2, the gradation of the typical VVRS filter sand is just within the coarse side of the allowable gradation limits. It is specially blended by commercial aggregate suppliers (such as Florida Rock). The effective grain size ( $D_{10}$ ) is 0.50 mm and the uniformity coefficient ( $D_{60}/D_{10}$ ) is 4.0. Hydraulic conductivity values of this filter media is typically on the order of 200 feet/day.

VVRSs are located within the pond (not in the side slopes) and the base of the structure is usually established 2 to 4 feet below the pond bottom or normal water level. A wedge of ½ to 1 inch gravel backfill is placed around the portion of the structure beneath the pond bottom.

### Advantages of VVRS over Conventional Filters

VVRSs are above-ground and therefore visible to maintenance personnel who can observe their performance first-hand. The structure is designed such that the filter media and filter fabric can be periodically replaced without total replacement of the system. If minor clogging of the filter media occurs, the system can be backflushed. VVRSs are easy to install and can be cost effective in the long-term.

Figure F-1: Typical Detail for VVRS (note: geonets are recommended on inside face of outside cylinder and outside face of inside cylinder)

Figure F-2: Gradation Curve for VVRS Filter Media

## THEORY

Horizontal flow through a hollow cylindrical sand filter is similar to flow through a vertical sand drain well. The theory of flow through vertical sand drain wells is described by H.R. Cedergren in the well-known text book *Seepage, Drainage, and Flow Nets* (pp. 362-368, 2nd edition, John Wiley & Sons, 1977). Applying this theory to the representation shown on Figure F-3, the instantaneous Darcian flow rate ( $q$ ) through a VVRS at any instant of elapsed (or recovery) time "t" is as follows:

$$q = k h \left( \frac{n_f}{n_d} \right) (h + y) \quad (1)$$

where

$k$	=	hydraulic conductivity of filter media
$n_f/n_d$	=	shape factor of flow net which depends on the ratio of the inside diameter to outside diameter of the VVRS filter (see Figure F-4).
$h$	=	head at outside edge of filter at time t
$y$	=	tailwater elevation - elevation of concrete base ( $y \geq 0$ )

Consider a time increment  $\Delta t$  in which the head drops by  $\Delta h$  causing a volume increment  $\Delta V$  to discharge from the pond through the VVRS filters. Mathematically, this is equivalent to

$$\Delta V = A \Delta h = N_1 q \Delta t = N_1 k h \left( \frac{n_f}{n_d} \right) (h + y) \Delta t \quad (2)$$

where

$N_1$	=	theoretical number of identical VVRS structures required to recover treatment volume
$A$	=	area of water surface at time t

Figure F-3: Graphic illustration of geometric parameters for typical VVRS

Figure F-4: Flow net shape factors for horizontal flow within filter media surrounding vertical pipes (Cedergren, 1977)

Since the surface area of ponds increase or stay constant with increasing stage, it is conservative to assume that the area A is constant and equal to

$$A = \frac{V}{h_t - h_b} \tag{3}$$

where

V	=	treatment (or water quality) volume
h <sub>t</sub>	=	top elevation of treatment volume
h <sub>b</sub>	=	bottom elevation of treatment volume

Substituting equation (3) in (2) and introducing the factor of safety variable (FS) yields

$$\frac{V}{h_t - h_b} \Delta h = N_1 \frac{k}{FS} h \left(\frac{n_f}{n_d}\right) (h + y) \Delta t \tag{4}$$

which may be written in differential form

$$dt = \frac{V \quad FS}{(h_t - h_b) \left(\frac{n_f}{n_d}\right) N_1 k} \int_{h_{\min}}^{h_{\max}} \frac{1}{h (h + y)} dh \tag{5}$$

Integrating equation (5) gives the following equation for the theoretical number of VVRS structures required to recover the treatment volume V in time T:

$$N_1 = \frac{V \quad FS}{(h_t - h_b) \left(\frac{n_f}{n_d}\right) T k} \int_{h_{\min}}^{h_{\max}} \frac{1}{h (h + y)} dh \tag{6}$$

where

$$\int_{h_{\min}}^{h_{\max}} \frac{1}{h(h+y)} dh = \frac{\ln(h_{\max}) - \ln(h_{\max} + y)}{y} - \frac{\ln(h_{\min}) - \ln(h_{\min} + y)}{y} \quad (7)$$

The maximum and minimum flow rates ( $q_{\max}$  and  $q_{\min}$ , respectively) through a VVRS is given by the following equations:

$$q_{\max} = k h_{\max} \left(\frac{n_f}{n_d}\right) (h_{\max} + y) \quad (8)$$

$$q_{\min} = k h_{\min} \left(\frac{n_f}{n_d}\right) (h_{\min} + y) \quad (9)$$

If ground water baseflow contributions to the pond ( $q_b$ ) are positive, then the theoretical number of VVRSs required to accommodate the baseflow contributions ( $N_2$ ) is given by

$$N_2 = \frac{q_b FS}{\frac{q_{\min} + q_{\max}}{2}} \quad (10)$$

Therefore, the theoretical number of VVRSs required is  $N_1 + N_2$  and the recommended number of VVRS will be the integer value greater than or equal to  $N_1 + N_2$ .

## REFERENCES

Cedergren, H.R. 1977. *Seepage, Drainage, and Flow Nets*. (pp. 362-368) 2nd edition, John Wiley & Sons, New York, NY.

## NOTATION

<b>A</b>	=	average surface area of treatment volume in pond
<b>D<sub>in</sub></b>	=	diameter of inner cylinder
<b>D<sub>out</sub></b>	=	diameter of outer cylinder
<b>FS</b>	=	safety factor in analysis
<b>h</b>	=	head at outside edge of filter at some time t
<b>h<sub>max</sub></b>	=	maximum driving head through VVRS
<b>h<sub>min</sub></b>	=	minimum driving head through VVRS
<b>h<sub>t</sub></b>	=	elevation of top of treatment volume
<b>h<sub>b</sub></b>	=	elevation of bottom of treatment volume
<b>k</b>	=	saturated hydraulic conductivity of filter media
<b>n<sub>f</sub>/n<sub>d</sub></b>	=	flow net shape factor for horizontal flow through VVRS
<b>N</b>	=	recommended number of identical VVRS structures required to recover treatment volume and baseflow contributions
<b>N<sub>1</sub></b>	=	theoretical number of identical VVRS structures required to recover treatment volume only
<b>N<sub>2</sub></b>	=	theoretical number of identical VVRS structures required to recover baseflow contributions only
<b>q</b>	=	instantaneous flow rate through a single VVRS at some time t
<b>q<sub>b</sub></b>	=	flow rate from baseflow contributions
<b>q<sub>max</sub></b>	=	maximum flow rate through a single VVRS during recovery (excluding factor of safety)
<b>q<sub>min</sub></b>	=	minimum flow rate through a single VVRS during recovery (excluding factor of safety)
<b>T</b>	=	recovery period
<b>V</b>	=	treatment volume
<b>y</b>	=	tailwater elevation - elevation of concrete base (y ≥ 0)