CHAPTER 9: STORMWATER MANAGEMENT FACILITY

9. STORMWATER MANAGEMENT FACILITY

9.1 SELECTING A POND SITE

Selecting the most appropriate pond site for a stormwater management facility requires the work of many different offices and professionals within the Department. You, as a drainage designer, will provide critical information, but because of the many factors to consider, a team approach is essential.

There are numerous design features (depth, size, shape, treatment method, landscaping, etc.) that you can modify to accommodate a pond site. However, hydraulic constraints may preclude the use of some sites. Alternate sites and their different design features usually will result in different costs and impacts. As a result, an evaluation of alternates must be made to select the most appropriate pond site. The purpose of the evaluation is twofold: (1) it will show that alternate sites were considered and that the selected site was the most appropriate, and (2) when you combine the evaluation with the final design details, they become the documentation that justifies the need to acquire property rights.

In the case where one person owns all the property in the area and that person is agreeable to any pond location proposed by the Department, evaluating alternates may not seem necessary. In these situations, the evaluation will not be as extensive as in other situations; nevertheless, you should perform some amount of evaluation to show that the site selected results in the lowest total cost.

The evaluation should weigh and balance numerous factors, such as cost; maintainability; constructability; public opinion; aesthetics; and environmental, social, and cultural impacts. The costs associated with right of way, environmental impacts, construction, and long-term maintenance usually are the easiest factors to estimate and compare. Other factors are more subjective and qualitative.

Because the evaluation involves a broad range of subjects, you should put together a multi-functional team to select the most appropriate pond site. Teams should have representatives from right of way, design, drainage, landscape architecture, environmental management, maintenance, construction, and eminent domain. At times, other units may provide critical information to the evaluation process. Although all of the team members may not participate in the entire process, they will likely provide critical information at some stage. The project manager, with support from the Drainage and Right-of-Way offices, will be responsible for coordinating the team effort and ensuring that the appropriate personnel participate.

Consider the value of existing vegetation during site selection and pond siting. In some cases, the need to preserve existing vegetation for aesthetic purposes may justify additional project expenses (retaining walls, acquisition of additional right of way, etc.).

Perform pond site evaluations during the Project Development Phase. Often, you will re-evaluate pond sites during the Design Phase. Before doing a design reevaluation, check what commitments have been made and what work has been done during the Project Development Phase.

CONSIDERATIONS WHEN SELECTING A POND SITE	
1. Use existing FDOT properties or other state-owned property, if feasible. 2. Minimize the number of parcels required. For example, avoid using parts of two parcels when the pond will fit within one parcel. 3. Generally, property owners prefer to place ponds toward the rear of their property. For parcels that abut the roadway right-of-way, the portion of the parcel next to the road usually is the most expensive. 4. Avoid splitting a parcel, thus creating two independent parcel remainders. 5. Consider the parcels identified by the right-of-way office. Even if a parcel is not large enough to provide all the stormwater management, it may be large enough to provide the treatment for stormwater quality. Or it could replace treatment and attenuation for parcels adjacent to the road that will have their ponds removed because of the road improvements.	6. Avoid wetlands. 7. Avoid archaeological sites and historic structures listed on or eligible for listing on the National Register of Historic Places. 8. Consider a joint-use facility (on the Department and another entity share) as an alternate, if one is feasible. 9. Generally, do not consider an option that requires water quality monitoring. Historically, this has been very expensive. 10. Stormwater treatment systems must be at least 30 meters (100 feet) from any public water supply well. (Chapter 62-555, F.A.C.). 11. Locations with billboards usually are expensive. 12. Locations with mature, attractive trees that will fit into the pond design increase the aesthetic value of the pond site.

9.1.1 Estimating Right-of-Way Requirements

The right of way required for a pond site varies with the amount of additional impervious area and associated additional runoff, the ground line and groundwater elevations at the pond, the proposed road elevations, the existing on-site natural features, and sometimes the soil conditions and other factors. During the pond site evaluation stage, the accuracy to which you estimate these items and the resulting pond size varies with several factors.

Sometimes the acquisition schedule dictates that results of the pond site evaluation form the basis for the final pond site right-of-way requirements. For these projects, you should determine the pond size as accurately as if doing the final detailed design.

There are other projects where the determination of the final right-of-way requirements occurs shortly after the pond site evaluation. After establishing the final right-of-way requirements, the acquisition process starts. For these projects, you would perform a pond site evaluation only to compare alternate sites or drainage schemes. Make your size estimates accurate enough to minimize changes to the right-of-way requirements during the final design.

In a third category of projects the right-of-way acquisition is scheduled for several years after the pond site evaluation, or the acquisition is not even funded in the Department’s work program. For these projects, changes in pond size and location from that established in the original evaluation will not affect production schedules or the right-of-way acquisition process substantially. Therefore, your pond size estimates need not be very accurate. For these projects, you typically would perform a pond site re-evaluation shortly before right-of-way acquisition.

Other factors that affect the level of accuracy for pond size estimates are property costs and the existing and anticipated development of the project area. In a rural area with relatively large tracts of land, changes to pond size and location will have less impact to property owners and the Department than in an expensive urban area that is rapidly developing and has relatively small parcels. As a result, the pond size estimates you use for these evaluations in rural areas do not need to be as accurate as in urban, rapidly developing areas.

9.1.1.1 Typical Factors Controlling Surface Area Requirements

Typically, the need to fit storage volumes within upper and lower constraints dictates the amount of surface area required for a pond. The following items could control the surface area requirements for a pond:

- The ground line at the pond (or the berm elevation) minus the freeboard dictates the top of the treatment and attenuation volume.

- For urban projects, the low point in the gutter minus the hydraulic gradient clearance dictates the hydraulic gradient of the storm drain. This constraint often is critical in flat terrain but not in steep terrain.

- High groundwater elevations or sometimes discharge tailwater elevations can constrain storage volumes. The groundwater elevation constraint will vary with the method of treatment used and the requirements of the regulatory agency.

- Retention ponds must recover a certain volume in a certain time. The size of the pond bottom area sometimes controls the recovery or drawdown time. This may be particularly critical for ponds discharging to closed basins.

- For wet detention facilities, most regulatory agencies limit the treatment volume depth to 18 inches and you must provide the required permanent pool volume.

- To contain a substantial portion of the pond volume in rolling or steep terrain, you would berm the low side of the pond site. The horizontal distance of the embankment from the berm top to natural ground dictates how much right of way you will need in this direction. The embankment slope must be flat enough to be stable. For example, a 1 (vertical) to 2 (horizontal) slope in sandy soil with seepage may not be stable. In this case, it would be appropriate for you to conduct a slope stability analysis. Discuss these situations with the geotechnical engineer to establish an acceptable slope and thus a reasonable estimate of the surface area requirements.

- You might adjust the shape of the pond—and, therefore, the surface area—due to existing on-site natural features (mature vegetation, a significant stand of vegetation on a slope, a visual landscape barrier, etc).

Example 9.1-1. Estimating Pond Right-of-Way Requirements

Given:

- Flat terrain, approximately 1-percent slope
- Proposed pond discharges to open basin
- Proposed curb and gutter section with gutter elevation at the low point in profile = 59.9 ft
- Ground elevation at pond site = approx. 59 ft
- Estimated seasonal high water table (SHWT) = 2.5 ft – 3.6 ft below ground (based on NRCS soil survey)
- Treatment volume = 10,950 ft^3
- Estimated peak attenuation volume = 19,567 ft^3 (from Example 9.4-1)
- Estimated 3-year attenuation volume = 10,243 ft^3 (storm drain design frequency)

Find: Estimated surface area requirements for a pond

1. Since the SHWT is so close to the surface, you choose a wet detention pond.

> For these conditions, one of two requirements typically control the surface area. Both involve spreading the treatment and attenuation volumes over a large enough area to keep the height of the volume within limits. The height (H) of the treatment and peak attenuation volume is constrained on the top, by the ground elevation minus the freeboard, and on the bottom, by the controlling groundwater elevation. Although some Water Management Districts (WMDs) allow treatment below the SHWT, this example will assume that treatment is above the SHWT. First, determine the surface area necessary to meet these constraints. The other requirement that may control the surface area is discussed after Step 5.

2. Conservatively, assume the SHWT is 2.5 feet below ground. The standard freeboard is given in Section 5.4.4.2 of the *Drainage Manual*. The treatment and peak attenuation volume are constrained to the following height (H).

   H = Depth to SHWT – Freeboard
   H = 2.5 – 1.0
   H = 1.5 ft.

3. The total peak storage volume required is:

   $Volume_{PEAK}$ = Treatment Volume + Est. Peak Attenuation Volume
   $Volume_{PEAK}$ = 10,950 + 19,957 = 30,907 ft^3

> You will need to make assumptions about the pond configuration.
>
> Shape: Assume the shape of the pond will be rectangular. Irregular shapes usually can be approximated by a rectangular shape so this is a reasonable assumption and it greatly simplifies estimating the surface area.
>
> Length to Width Ratio (L/W): The property lines may suggest a preferred ratio to make best use of a parcel. Without other guidance, assume L/W = 2.
>
> Side Slopes: Assume flat slopes, such as 1 (vertical) to 5 or 6 (horizontal) for sites required to be aesthetically pleasing. Assume 1 (vertical) to 4 (horizontal) for most other conditions.

4. Use the formula for a rectangular box to determine the water surface area of a pond with vertical sides.

Volume = L_{RECT} W_{RECT} H

where:

H = Height (m) = 1.5 ft for the above condition
L_{RECT} = Length (ft) of vertical-sided pond
W_{RECT} = width (ft) of vertical-sided pond

Assume for this example that L/W = 2, then

30,907 ft^3 = L_{RECT} x (0.5 L_{RECT}) x 1.5 ft, then
L_{RECT} = 203 ft
W_{RECT} = 101.5 ft

5. Increase these dimensions to account for sloped sides by adding: 2 x (0.5 x H x Side Slope).

For this example, assume side slope = 5, thus adding 7.5 ft to each dimension.

Length @ top of slope = 210.5 ft
Width @ top of slope = 109 ft

Then,
Water Surface at Peak Design Stage = 210.5 x 109 = 22,944.5 ft^2 = 0.53 ac

The other requirement that may control the surface area in flat terrain is the requirement to maintain the clearance between the low point in the gutter and the hydraulic gradient in the storm drain system. On the top, the low point in the gutter minus both the hydraulic gradient clearance and the energy losses in the storm drain system constrain the treatment volume and three-year attenuation volume. On the bottom, the groundwater elevations (SHWT for this example) constrains these volumes. The standard hydraulic gradient clearance is given in the *Drainage Manual.*

You can estimate the energy losses in the storm drain system in two ways. Assume a hydraulic gradient slope. Slopes of 0.05 percent to 0.1 percent are common in flat terrain. Multiply the length between the pond and the low point by the assumed slope to obtain the losses. Another approach is to assume a fixed energy loss, ignoring the length between pond and low point. In flat terrain, a reasonable value for this purpose is two feet.

6. The SHWT elevation is 56.5 feet (59 – 2.5). For this example, you can assume the energy loss in the storm drain to be 0.7 ft. Then, the treatment and three-year attenuation volume are constrained to the following height (H):

   H = Low Point in Gutter – Clearance – Estimated Energy Losses – SHWT Elevation
   H = 59.9 – 1.0 – 0.7 – 56.5
   H = 1.7 ft

   This is greater than the height (1.5 feet) available to "stack" the peak attenuation volume (Step 2). Since the three-year attenuation volume is less than the peak attenuation volume, this constraint will not control the water surface area. If the height was less than determined in Step 2, you would estimate the water surface area as done in Step 4 except using different values for H and the total volume.

   The water surface area dimensions determined in Step 4 apply.

7. Add the maintenance berms to the water surface dimensions. The standard maintenance berm width is given in Section 5.4.4.2 of the *Drainage Manual*.

   Length = L_{TOP} + 2 (berm width) = L_{TOP} + 2(20) = 210.5 + 40 = 250.5 ft
   Width = W_{TOP} + 2 (berm width) = W_{TOP} + 2(20) = 109 + 40 = 149 ft
   Area = 250.5 x 149 = 37,324.5 ft^2 = 0.86 acre

8. Increase the value by 10 percent to 20 percent to account for the preceding information being preliminary. For this example, we will increase it by 10 percent.

   Area = 0.86 x 1.1 = 0.95 ac

> Realize that this is only the pond size estimate. You also must make estimates for access and conveyance, as discussed in the next section.

9.1.2 Access and Conveyance

The right of way required to convey the project's runoff to and from a pond and to provide access can affect which alternate pond site is the most appropriate. Determine these requirements for each alternate and include the costs and impacts in the evaluation.

Sites placed far from the project will require more right of way to get stormwater to the pond than sites adjacent to the project. Similarly, different pond sites can have different right-of-way requirements for the outfall (discharge) from the pond. Guidelines for establishing the width or "footprint" of the right-of-way requirements for conveyance are provided in Section 9.2 of this document.

The Department often provides access through the same property obtained for conveying the project's runoff. For pond sites placed far from the project, providing access from a local road closer to the pond is sometimes more reasonable.

Usually, the Department will obtain the right of way required for access and conveyance as a perpetual easement. Fee simple right of way may be appropriate sometimes. The opinion of the District Maintenance Office, balanced with property owner preference and right-of-way costs, is the primary factor for determining which type of right of way is appropriate.

Refer to Appendix B of the Drainage Manual and the FDM113. Both contain additional information about acquisition of property rights.

9.1.3 Joint-Use (Regional) Facilities

Sometimes the Department and other entities can share a stormwater management facility. Both the Department and the other entities receive the stormwater management benefits of the facility and share in its construction, operation, or both. The Department and the other entities create a written agreement describing the responsibilities of each party. Typically, these agreements are made with local governments, but sometimes private entities enter joint-use agreements. For example, the Department shares several facilities with golf course owners.

Advantages of a joint-use or regional facility are that: (1) the Department often can relieve itself of the maintenance requirements, (2) water quality improves downstream, and (3) stormwater re-use is incentivized when a larger volume of water is available. A joint-use facility can have disadvantages, such as affecting production schedules, a more complex permitting process, and resolving any non-complying discharges, if they occur.

When developing a joint-use agreement, avoid commitments that hold the Department to completing construction of the site by a certain date because there often are unforeseen delays in permitting and funding. Developing an acceptable joint-use agreement often requires an extensive coordination effort involving the project manager and representatives from numerous other offices. Discuss this option with the project manager or District Drainage Engineer.

9.1.4 Facilities on Forest Lands

Occasionally, projects are bounded by state and/or national forest lands and ponds must be located within these public preserves. In such cases, advanced coordination with the owning agency and the WMD can result in cost-effective designs that will not degrade the public purpose of the forest lands. This cooperative process can sometimes take longer to complete and should, therefore, be started early in the PD&E phase.

9.1.5 Coordination with Property Owners

Often, contacting the property owner to get his or her preference regarding the shape and location of the pond and location of the access road is beneficial from a right of way standpoint. This coordination is especially important where the Department needs only part of a parcel for a pond.

Consider contacting the owner during the evaluation of alternate sites. A situation where contacting the owner during the evaluation may be appropriate is where one person owns all the property in the area. If a contact is not made during the evaluation process, it is recommended that a contact be made shortly afterward and before starting final design. For example, perhaps the property owner may prefer a shallower pond although it would require more right of way, or the owner may be interested in re-acquiring and maintaining the pond. A certain pond shape could give the owner better use of the remainder of the parcel. This is important information to know before starting final design. In some instances, contacting homeowner's associations or abutting property owners may be beneficial to find out if a negative perception of the proposed pond exists.

Sometimes, contacting the owner may not be appropriate. Where the Department needs an entire parcel, there is no need to obtain the owner's preference about pond location.

The project manager, with participation from the right-of-way office, should decide whether to contact the property owner based on individual circumstances.

The Department's project manager or a right-of-way specialist or both could make the contact. As a drainage designer, you are the best source to answer technical questions and will likely be asked to be present when the contact is made. You cannot provide specifics early in the design process, but you can speak about general principles of stormwater management facilities.

When obtained in writing, you should accommodate the property owner's preference to the greatest degree possible. The Department may not be able to accommodate all of the owner's preferences in the design of the pond due to hydraulic constraints or other limitations. However, after weighing and balancing the owner's requests with other factors, it is likely that some aspects of the owner's preference can be satisfied, thus improving relations during the right-of-way acquisition process.

If a commitment is made to a property owner, follow through or notify the owner that the Department cannot meet the commitment. Usually, you will not have enough information to commit to anything during the first contact with the owner. Remember that the purpose of the initial contact is to learn the owner's preference regarding the shape and location of the pond and location of the access road. The most that you can commit to is to try to accommodate the owner's requests. If, during any discussion, the property owner is told about the operation, shape, or location of the pond, this is a commitment. If you subsequently design the pond differently, you should notify the property owner. If the owner is not notified, the right-of-way specialist is placed in the difficult situation of approaching the owner with a proposed pond configuration that is different than what was discussed previously.

This holds true for changes that occur through the detailed design phase. The owner must be notified if the shape, size, and location of the pond are going to be different than what was discussed previously.

9.1.6 A Suggested Evaluation Process

An outline for evaluating alternate sites follows, and a flow chart is provided in Figure 9.1-5. The process is divided into seven main steps of work, as follows:

Step 1 Coordinate with the right-of-way office

Step 2 Identify alternate drainage schemes

Step 3 Estimate the right of way required for each alternate

Step 4 Get team buy-in on the proposed alternates

Step 5 Estimate costs and assess impacts

Step 6 Summarize findings

Step 7 Select site

The steps listed below are directed toward you, the drainage designer, but there also is information about activities that team members from other offices should perform. Normally, you should proceed through the steps in order, but, often, doing certain steps earlier in the process or doing several steps concurrently will be reasonable and prudent. The most important issue is to maintain the coordination necessary to ensure that pond sites are selected using a multi-functional team.

The degree of detail will vary with individual projects and between FDOT districts. It is essential that you discuss this with the project manager or the District Drainage Engineer before starting the evaluation.

Step One Coordinate with the Right-of-Way Office

The purpose of this coordination is to provide a preliminary pond size and a general location to the right-of-way office and to ask the right-of-way office to identify potential sites.

Shortly after the roadway typical section is set, provide the right-of-way office with a preliminary estimate of the size and a general location of the pond. Use aerial contour maps, old construction plans, available surveys, and other data to identify the primary basins and the general outfall locations (discharge points). Identifying the high points along the project usually separates the primary basins. At this stage, assume that the pond site will be near the lows in the terrain and will be close to the existing outfalls. As a preliminary size estimate, use 20 percent of the roadway right of way draining to the outfall. The area identified for the general location should be large enough to allow for several alternates to be developed. Refer to Figure 9.1-1. The project manager should relay this information to the right-of-way office so it can include the preliminary costs for pond sites in the cost estimates.

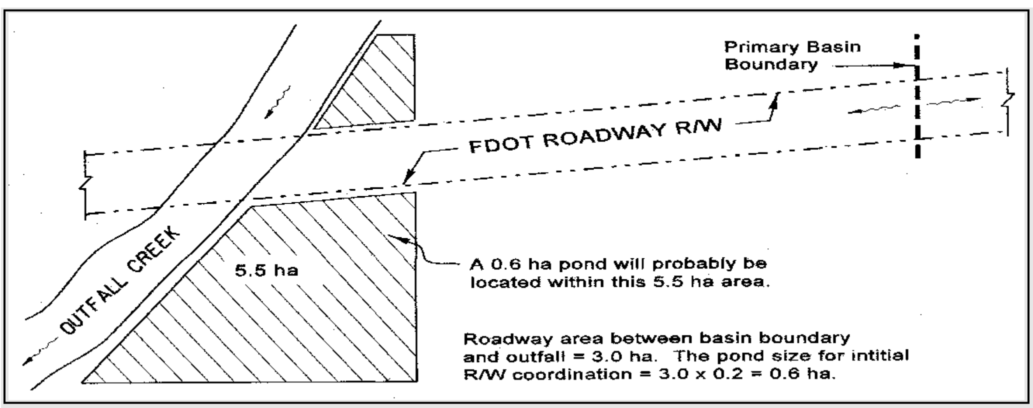


Figure 9.1-1: Size and Location for Initial Coordination with the Right-of-Way Office

When the corridor and alignment (left, right, or center) are set, the project manager should request the right-of-way office to identify parcels along the roadway that could be economical for a pond, due to the impacts of the roadway footprint. The right-of-way office also should identify existing excess property in the area.

At this stage, impacts of the roadway footprint at intersections and interchanges may be uncertain still simply because the geometry has not been set. These areas may warrant discussions with the right-of-way office at a later time.

When the right-of-way office completes this task, the project manager should arrange a meeting with the team to discuss all potential pond sites, aesthetic concerns, and possible contacts with property owners. Representatives from right of way, drainage, landscape, and environmental management should attend.

Refer to tax maps while discussing potential pond sites. The project manager should have these; if not, the local government should.

Step Two Identify alternate drainage schemes

Before developing the alternates, familiarize yourself with soils and groundwater conditions in the area and with the various stormwater quality treatment methods. Use the Natural Resources Conservation Service (NRCS) (formerly Soil Conservation Service) soil surveys to obtain the soil information. The treatment methods are discussed in Section 9.3, below.

It may be reasonable to start this step by qualitatively eliminating areas that are not hydraulically feasible. For example, some areas may be too high in elevation, or may be at the beginning of the drainage system rather than at the end.

For projects in developing areas, consider contacting the Planning (or Development) Department of the local government to find out the zoning for the area, the planned land use, and if proposed developments exist. Although this information should not automatically eliminate a site from being evaluated, it may help you to identify viable alternatives, such as a joint pond use with future land developers.

Identify two or three alternate drainage schemes for each primary basin. If two or three vacant sites are not available, then consider developed sites. Familiarize yourself with the list of considerations in Section 9.1 when identifying your drainage schemes. Also consider the sites identified by the right-of-way office in Step One. This is not to say that these sites need to be evaluated as alternates, but all of the alternates evaluated must be viable. You should consider these sites during the evaluation.

The alternates may be as simple as two different locations for a wet detention pond, or a wet detention pond compared with a dry pond with underdrain at the same location. A system using two ponds, one for off-line quality treatment and one for attenuation, could be compared with a single pond designed for both quality treatment and attenuation. In areas with expensive right of way, identifying an alternate that uses a non-standard approach—such as sand box filters or pumping stations—may be prudent. Check with the District Drainage Engineer before doing so. See Figure 9.1-2.

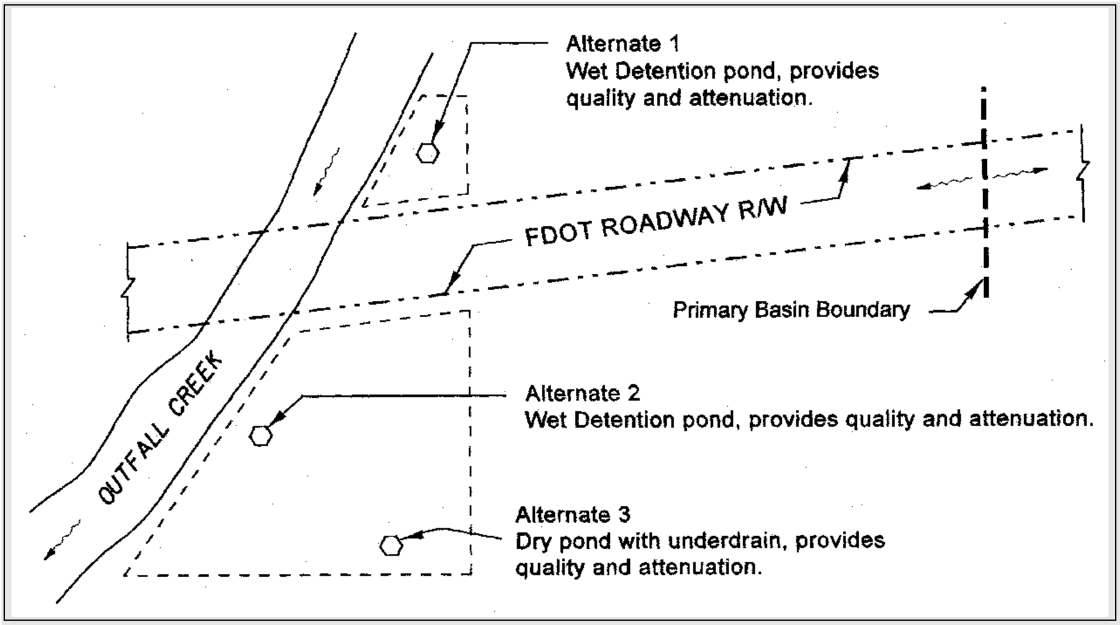


Figure 9.1-2: Alternate Drainage Schemes

Step Three Estimate the right of way required for each alternate

A. Consider the need for additional soils and groundwater information. Most of the Department's districts accept the NRCS soil surveys for pond site evaluations. For alternates using retention or exfiltration in areas where there are poor soils and for projects discharging to a closed basin, site-specific data may be appropriate. If you feel that additional information is warranted, discuss this with the District Drainage Engineer.

Steps B through G apply to ponds discharging to open basins. Ponds discharging to closed basins have the additional complication of assuring that the drawdown requirements are met (see Section 9.4).

B. Determine the required treatment (quality) volume. See the discussion of treatment volumes in Section 9.3. Refer to the appropriate regulatory agency's rules or meet with the agency at this time.

C. Estimate the required attenuation volume. See the discussion of Estimating Attenuation Volume in Section 9.4.2.

D. Coordinate with the Landscape Architect to perform a preliminary identification of existing landscape, natural and aesthetic features, and opportunities and constraints that could impact the placement and design of the pond.

E. Estimate the low point in the proposed roadway. Discuss the grade with the roadway designer as necessary.

F. Obtain ground elevations around each alternate site. Using a contour map with one-foot intervals usually is sufficient. In flat terrain where one-foot contour maps are not available, obtaining a survey of the ground elevations around each alternate site may be appropriate.

G. Determine the pond surface area necessary to satisfy all applicable criteria. Refer to the typical controlling factors in Section 9.1.1.1. If you know of aesthetic preferences that will affect the surface area, such as shape, side slopes, landscaping, or preserving existing vegetation, account for them in the surface area determination. Example 9.1-1 goes through this and the following two steps.

H. Add the maintenance berms to the above area.

I. Increase this area by 10 percent to 20 percent to account for the preceding information being preliminary.

J. Place these surface area requirements within parcel boundaries in a way that minimizes the number of parcels required. For example, avoid using part of two parcels when the pond will fit within one.

K. Determine the right-of-way requirements for access to the pond and for conveyance to and from the pond.

L. Sketch each alternate site and its requirements for conveyance and access on the tax maps (preferably on aerial background). Refer to Figure 9.1-3.

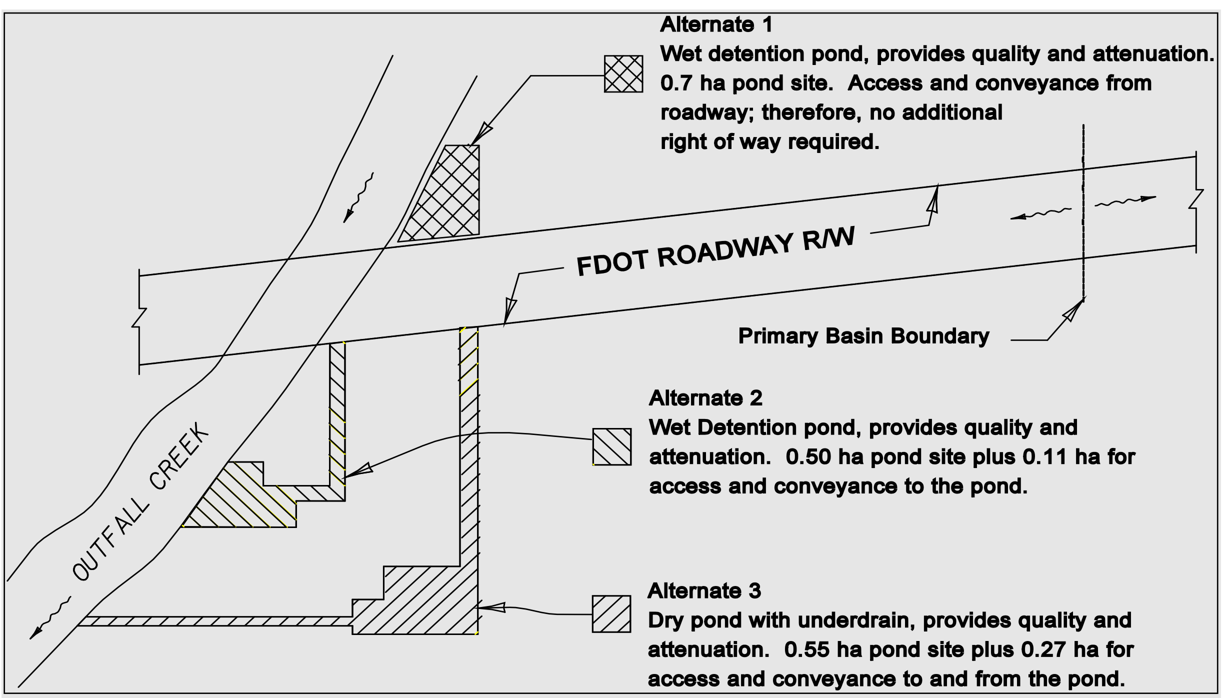


Figure 9.1-3: Sketch of Each Alternate's Estimate Requirements

Check with the project manager or District Drainage Engineer to see if they want to review the above work before proceeding to the next step.

Step Four Get team buy-in on the proposed alternates

The project manager should arrange a meeting with the team to discuss the alternates. The meeting has several purposes: (1) discuss how the right-of-way requirements fit within parcel boundaries, (2) confirm that alternates being considered are viable, (3) consider the need to contact property owners to obtain their preference of pond shape and location, (4) confirm that the access and conveyance requirements are reasonable, and (5) discuss social, cultural, and environmental impacts, including the existing landscape, natural and aesthetic features, and opportunities and constraints of each alternate.

If the property owners are contacted, their preferences should be discussed among the appropriate team members, and the sites appropriately adjusted before proceeding to the next step.

Step Five Estimate costs and assess impacts

When the team agrees on the alternate drainage schemes, the project manager should request environmental assessments, right-of-way cost estimates, and utility impact assessments for each alternate site. The purpose of the environmental assessments is to determine potential hazardous material contamination and potential impacts to environmental resources such as threatened, endangered or significant species and cultural resources. Environmental specialists from the Environmental Management Office usually do the assessments, which should include cost estimates associated with any mitigation and environmental cleanup.

The purpose of the utility assessment is to determine the existence of utility corridors through each alternate site.

You, as the drainage designer, should estimate the construction cost of each alternate, including the conveyance requirements to and from the pond. Usually, the largest costs are associated with earthwork, pond liner (when required), and pipe. Statewide average unit prices for the standard pay items are provided in the publication *Item Average Report*, which is available for download at: https://fdotwp1.dot.state.fl.us/wTWebgateReports/login.aspx (note: the user must have a login and password for a specific project). For alternates that are similar, estimating construction cost differences rather than total construction costs may be reasonable. If different alternates are expected to have substantially different maintenance costs, estimate these as well. Since maintenance costs will be spread over time, it will be necessary to equate these to initial costs using a life cycle analysis. Contact the District Maintenance Office to obtain the latest unit prices for routine maintenance activities.

Each alternate should have, at a minimum, cost estimates for right of way. When the estimates and assessments are complete, the various offices should furnish their findings to you via the project manager.

Step Six Summarize findings

For each basin, combine the findings of the other offices with your construction cost estimates. Use a summary table similar to Figure 9.1-4 to compare the alternates. The *Drainage Manual* lists the minimum documentation requirements.

Check with the project manager to see if the district staff wants to review the summary before proceeding to the next step.

Step Seven Select site

The team should meet to discuss all alternates and select the most appropriate site. Cost, maintainability, constructability, public opinion, aesthetics, and environmental (social, cultural, natural, and physical) impacts will affect the selection of a pond site. The team should weigh and balance all factors in their decision. Include documentation of the decision with the summarized findings of the previous step.

9.1.6.1 Start Final Design

For most projects, the actual right-of-way requirements will be determined during the final design of the pond. The acquisition of the pond site occurs during the process of acquiring any additional right of way for the roadway corridor. You should revisit the site evaluation process if the final right-of-way requirements are substantially different from those originally estimated. Pond locations frequently change as the final design progresses. Sometimes additional sites are evaluated, and occasionally the originally selected site is not used. Any additional evaluations of pond sites should be documented as required by the District Drainage Engineer. All changes in right-of-way requirements must be coordinated with the right-of-way office.

Alternate Pond Site Evaluation

Project Decription ______________________ Project Number __________

Basin Description ______________________

	Alternate 1		Alternate 2		Alternate 3	
Brief Description of Alternate ►						
	Comments	Costs	Comments	Costs	Comments	Costs
Right of Way						
Construction						
Hazardous Materials						
Utilities						
TES[1] Species						
Maintenance						
Cultural Resources						
Public Opinion						
Aesthetics						
Other						
Total Costs						
Comments, Advantages, Disadvantages, etc.						

1. Threatened, Endangered, or Significant

Figure 9.1-4: Summary Table to Compare Alternates

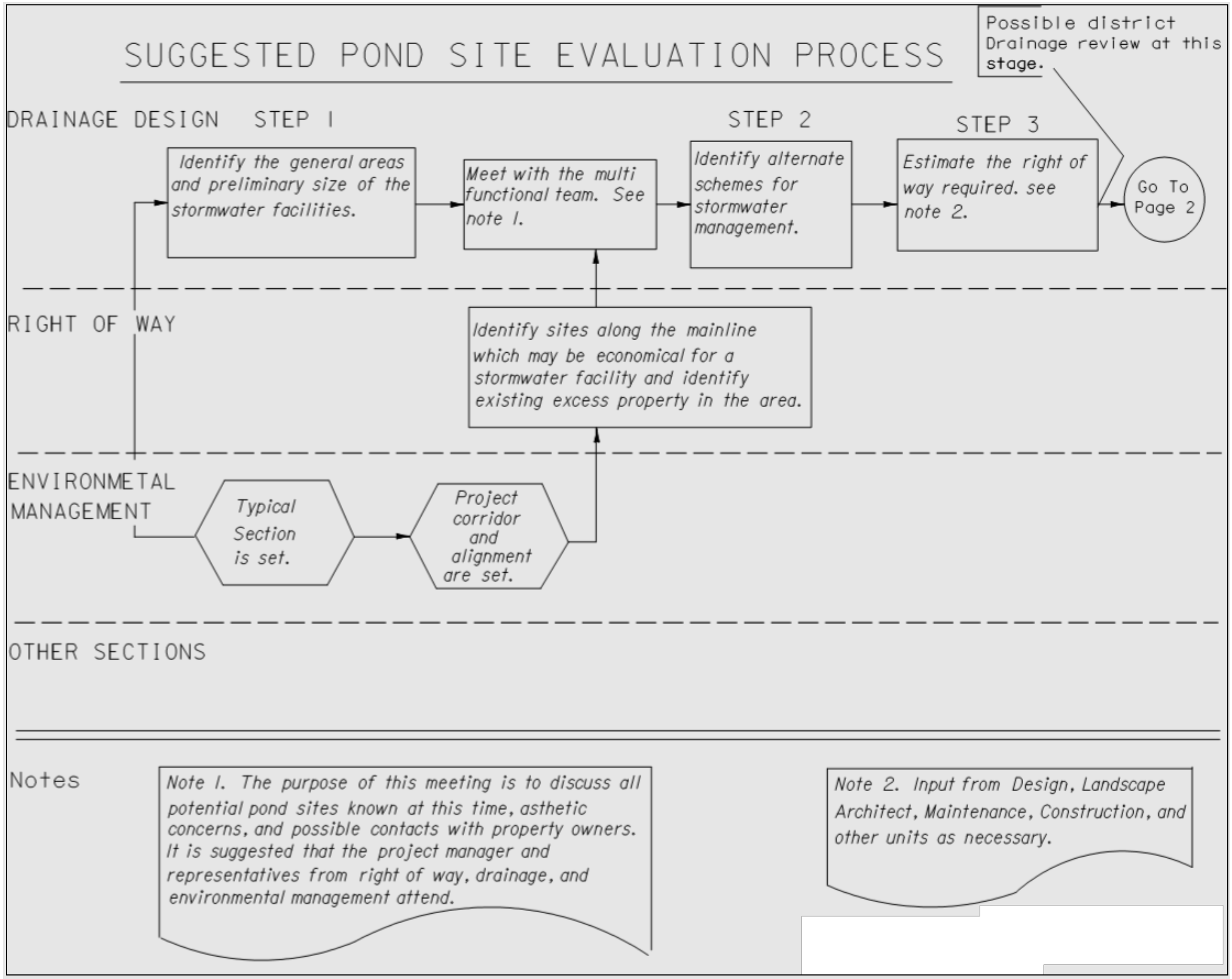


Figure 9.1-5: Pond Site Evaluation Process

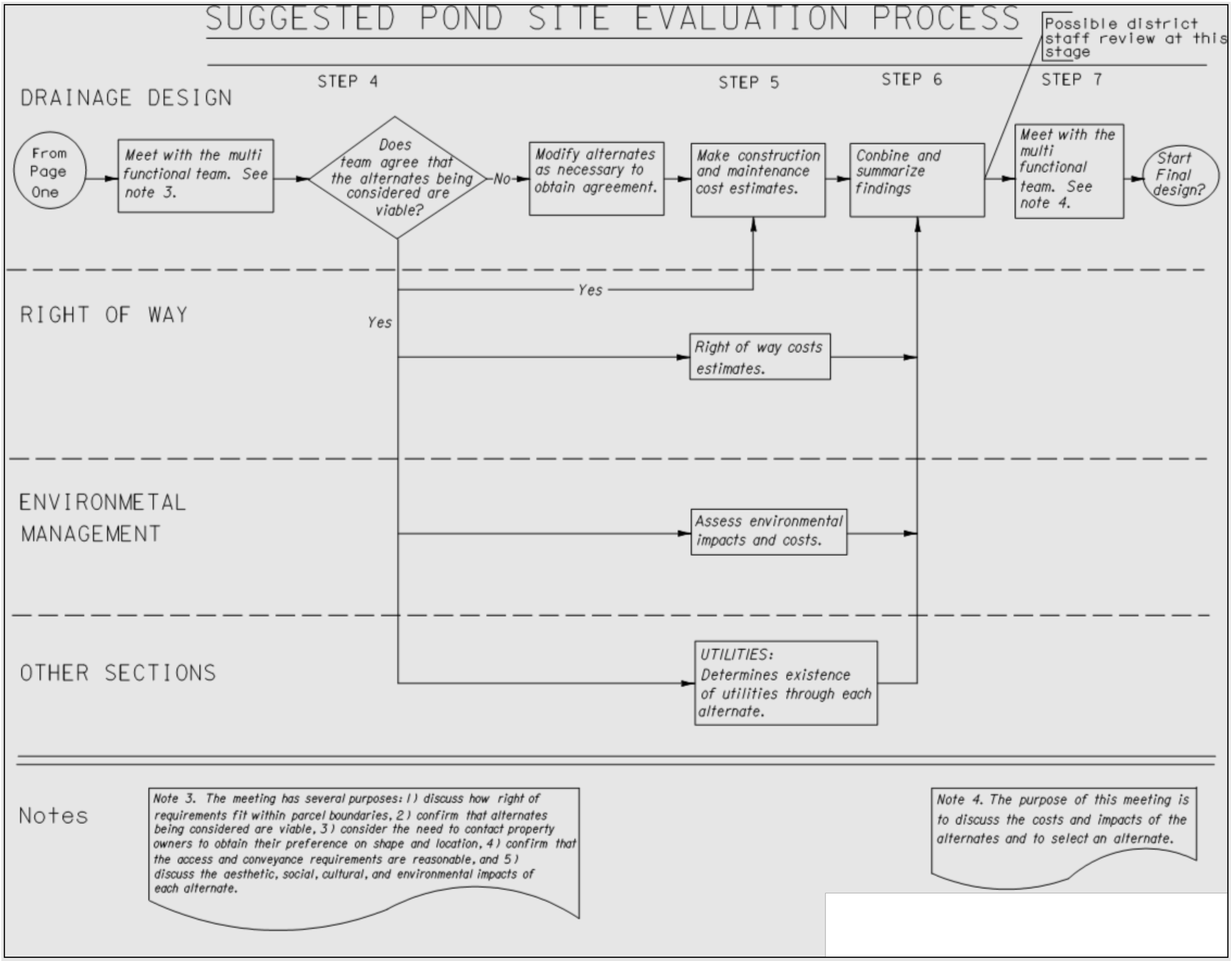


Figure 9.1-5 (continued)

9.2 MAINTENANCE, CONSTRUCTION, AESTHETICS, AND OTHER CONCERNS

9.2.1 Maintenance

Maintenance must be a consideration throughout the process of designing a stormwater facility. Long-term maintenance costs are inevitable, but they can be minimized by appropriate consideration during the design of a facility. The difference between a maintainable design and a design that is difficult and expensive to maintain often will be the difference between an attractive operating facility and a neglected, non-functioning facility generating frequent public complaints.

9.2.1.1 Pond Configurations

Side slopes:

Use a slope of 1 (vertical) to 4 (horizontal) or flatter. Steep slopes are harder to mow and are more susceptible to erosion than flat slopes. Slopes steeper than 1:3 must be mowed with special equipment. This is generally more expensive than using regular mowers.

Where possible, conserve established slope vegetation to increase stability and add an aesthetic feature to the pond.

Maintenance berms:

The *Drainage Manual* gives the minimum widths and slopes. These are acceptable for most situations, but discuss site-specific concerns with the local maintenance staff.

For ponds that will maintain a permanent or normal pool, keep the lowest point of the maintenance berm at least one foot above the top of the treatment volume. This will minimize saturation of the maintenance berm.

Corners:

Use a radius of 30 feet or larger for the inside edge of the maintenance berm. This is based on the largest piece of normal maintenance equipment. Several maintenance vehicles were modeled using the AUTOTURN program (Transoft Solution, Inc.). The GRADALL 880 requires the largest turning radius and gate opening.

Benchmark:

Have a benchmark constructed in or near all ponds. It will be used to check critical elevations of the pond and outlet control structure. Avoid installing benchmarks in areas subject to settlement such as high fill sections and areas subject to vehicle loads. An outside corner of the maintenance berm in a minimal fill section would be an appropriate location.

Sediment buildup:

Design the pond with a three-foot deep sediment sump near the inlet. In retention ponds (described in Section 9.3.4.2) where the groundwater is close to the pond bottom, the depth of the sump may need to be reduced to avoid exposing the groundwater. The area of the sump should be approximately 20 percent of the pond bottom area.

In retention ponds, the sediment is visible, but often it accumulates so slowly that it is difficult to see how much exists. A staff gage placed near the inlet allows the buildup to be measured.

Permanent (Normal) Pool Depth:

The main body (not the littoral shelf) of the permanent or normal pool should be deep enough to minimize aquatic growth, but shallow enough to maintain an aerobic environment throughout the water column. The regulatory agencies usually will specify the maximum depth for water quality credit, but this depth may be exceeded for harvesting fill needed for the project or to preclude future maintenance cleaning; in such cases, the extra pond depth will not be credited toward the regulatory permanent pool requirement. If the minimum depth is not specified, use five feet to minimize aquatic growth.

Side Bank Underdrain Filters:

Do not construct these around the entire pond. Design the pond to have at least 20 feet of the side slope without underdrains so that maintenance vehicles can get to the pond bottom without running over the underdrain.

9.2.1.2 Diversion Structures

Diversion structures of off-line systems must have a manhole for access on each side of the weir (refer to Section 3.10 of the *Drainage Manual*). Furthermore, the manholes should be located out of the roadway pavement to allow access without blocking traffic. Off-line systems are discussed in Section 9.3.

9.2.1.3 Conveyance to and from the Pond

The right of way obtained for conveyance to and from the pond must be sufficient to maintain the conveyance. This is true for either piped or open-ditch conveyance systems. Figure 9.2-2 provides typical sections for establishing the width of the right-of-way requirements.

Where the pond discharges to something other than an existing storm drain system, obtain right of way from the pond to a receiving surface water body (lake, wetland, ditch, canal, etc.) even if there are no physical changes proposed to the conveyance path. This assures that the Department will have the right to maintain the flow path.

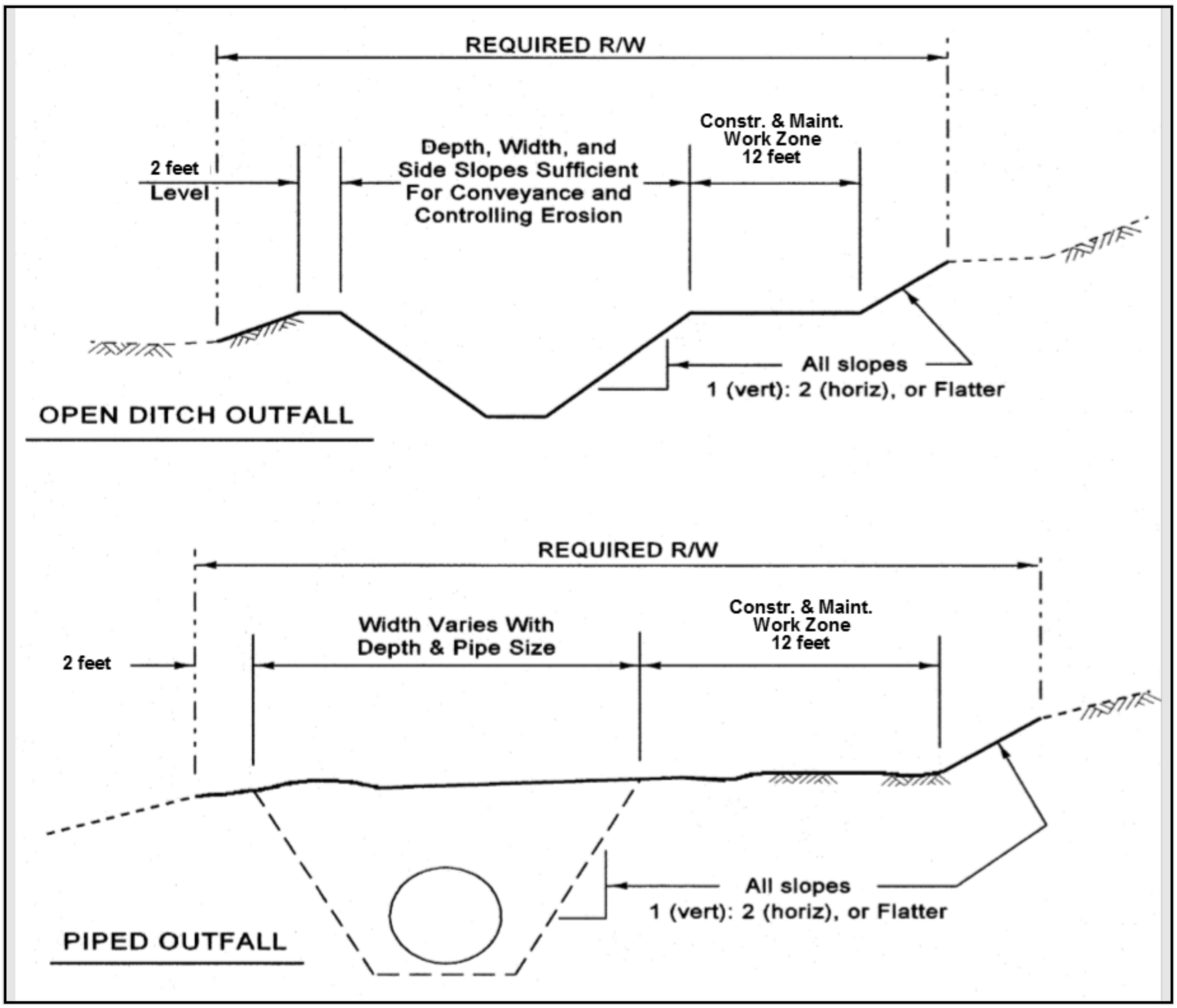


Figure 9.2-2: Required Right of Way Widths

9.2.1.4 Vehicle Access

Roads:

Sometimes, you can use the right of way available for conveyance to provide maintenance access to the pond. For pond sites located far from the project, it may be more reasonable to reach the pond from a local road. In flat terrain, an ideal width of right of way for access only (not including conveyance) is 15 feet. Larger widths may be necessary for turns. In irregular terrain, consider the distance to tie into natural ground. Concentrated flows crossing the access road may require a culvert crossing. If the vertical clearance is restricted, discuss it with maintenance personnel.

The roadway designer should design and incorporate curb cuts and driveways in the plans where the access road joins the public road.

Gates:

If you plan to fence the pond, use a 24-foot or two 12-foot sliding cantilever gates (Standard Plans, Index 550-003). This will allow the largest piece of normal maintenance equipment to enter and exit without having to back out along the access road. If you must use a swinging gate, pave the area under the arc of the gate swing. Show the gate type, location, and size in the plans.

9.2.1.5 NPDES Permits

Active National Pollutant Discharge Elimination System (NPDES) permits may cover the limits of proposed construction. The District NPDES Coordinator needs to review the proposed project to ensure compliance with any active permits.

9.2.2 Construction

Consider the right of way needed to construct the facility. The right of way needed to maintain the facility, i.e., the permanent right of way, may be, but is not always, sufficient to construct the facility. If the construction area is outside the permanent right of way, you should use temporary construction easement documents to obtain sufficient area for the contractor to construct the facility.

Some water management districts require a professional land surveyor to lay out final placement of drainage structures. Some of the Department’s districts are directing the contractor to do this. Discuss this with the project manager or district construction personnel. If they want to have the contractor survey the final placement, include the requirement in the contract documents, as directed by the district.

Often, the regulatory agencies place special requirements on the Department's projects as "conditions of the permit." Requirements that will affect the contractor's work must be incorporated into the plans or specifications for bidding and payment purposes. It is not sufficient that the permits will become part of the contract documents.

9.2.2.1 Structure Tolerances

Unless otherwise dictated, the tolerance for drainage structures is controlled by Section 5-3 of the Standard Specifications, which reads: "reasonably close conformity with the lines, grades, . . . specified in the contract documents." The tolerance is particularly important for weirs, orifices, and other flow control openings of outlet control structures. You can calculate weir dimensions quite precisely, but it is not reasonable to construct concrete structures to that same precision. Complicating this in the past, the regulatory agencies' inspectors sometimes have expected the dimensions to be exactly as shown in the plans.

During design, if you realize that the designed discharge is sensitive to small changes in weir dimensions, you should conservatively account for the tolerance in the calculations. For example, to maintain the discharge rate at or below the allowable rate, specify a weir width that is 0.05 feet smaller than the width required to discharge at the allowable rate. And include the tolerance mentioned above. If the contractor constructs the weir 0.05 feet wider than specified, it will match the designed width. If the weir is constructed 0.05 feet narrower than specified, the discharge rate still will be less than the 0.05 feet maximum allowed. In the last condition, you should check that stage has not increased to a point where the pond is now discharging through the overflow point.

Although not often used, another option is to use "bolt on weir plates" with slotted bolt holes. The plate elevation then can be adjusted to exact elevations after the structure is set.

9.2.2.2 Earthwork Tolerances

By Standard Specifications, the tolerance for earthwork within a stormwater management facility is 0.3 feet above or below plan cross section (Section 120-12). For some retention ponds, having a bottom 0.3 feet higher than anticipated may substantially reduce the treatment volume and somewhat affect the attenuation capacity. Conversely, having a bottom 0.3 feet lower than anticipated may substantially increase the retention (or treated) volume and affect the recovery time. This tolerance will not affect wet detention facilities.

Do not specify a tolerance that may conflict with the Standard Specifications. If the standard tolerance will substantially reduce the retention or treatment volume—as in a shallow retention pond—design the pond to allow for the bottom being 0.3 feet higher or lower than shown in the plans. In other words, specify a pond bottom that is 0.3 feet lower than necessary to retain the minimum volume. For example, the pond bottom may need to be 0.7 feet below the weir to provide the treatment volume. Specify the bottom to be 1.0 foot below the weir to allow for the earthwork tolerance. Determine the recovery time assuming that the pond bottom is 1.25 feet below the weir, i.e., 0.3 feet below the specified bottom elevation.

You should reserve this extra effort for facilities where the earthwork tolerance could substantially reduce the retention or treatment volume.

9.2.3 Aesthetics

The Florida Department of Transportation has adopted a Highway Beautification Policy to include aesthetic considerations in the design aspects of highways. Chapter 5 of the *Project Development & Environmental Manual* summarizes the requirements and provides direction in applying them to Department projects. Aesthetic considerations are cited in Section 5.4.4.2 of the *Drainage Manual* as an integral part of sound pond design. Often, programmatic or aesthetic commitments are made during the project development phase. If so, the environmental document will contain a discussion of visual impacts and aesthetic requirements for stormwater ponds. Discuss this with the Landscape Architect and Environmental Management Office project manager.

The location, size, shape, side slopes, fencing, and landscaping all affect the aesthetic quality of a pond. In general, irregular shapes, gradual slopes, and no fence are more aesthetically pleasing and have less visual impact than rectangular shapes and steep slopes with a chain link fence. For this reason, the *Drainage Manual* mandates that the default pond design should not include fencing, and that fencing must be justified within the design documentation. You can use irregular side slopes for permanently wet ponds to create an undulating water edge even when the perimeter of the site is rectangular. Preservation of existing vegetation and inclusion of native and wetland vegetation can greatly improve the visual appearance of a pond. Typically, this will require that you design and construct physical barriers to protect the existing vegetation from construction equipment.

In urban areas, ponds designed with a park-like appearance will encourage the local government to undertake the maintenance. If you design a pond site to be landscaped, a memorandum of agreement (MOA) for maintenance may be executed with the local government. In the absence of an MOA, the Department may undertake the landscape maintenance of a pond. The District Landscape Architect is familiar with the MOA procedure. Any landscape projects should be coordinated by the project manager with support from the District Landscape Architect.

The shape, depth, and side slopes will affect how much right of way is required for a pond. Therefore, you must evaluate and weigh aesthetics among the other factors during the site selection process (see Section 9.1). The Department has determined that pond aesthetics is an acceptable design objective that would justify acquisition of additional right of way, including eminent domain acquisition, when appropriate. Seek out the District Landscape Architect to coordinate and develop appropriate aesthetic features. Your responsibility is to ensure that the design constraints (volumes, depths, littoral shelves) are met while accommodating the aesthetic features. Coordinate with the District Landscape Architect to establish the quantity of right of way needed to meet aesthetic and design constraints.

9.2.3.1 Fence

The *Drainage Manual* mandates that the default pond design does not include fencing and that use of fencing must be justified within the design documentation. Design stormwater ponds to avoid the need for fence, if feasible. Typically, the flow velocities within a stormwater pond are low and, therefore, the velocities do not create a hazard. Unexpected deep standing water—such as an immediate 1:2 drop off at the water's edge—should be avoided or fenced. Under the Statewide Environmental Resource Permit (ERP Ch. 62-330) Rule, the *Drainage Manual,* Florida Department of Environmental Protection (FDEP), and all the water management districts allow for unfenced facilities if the slopes are 1 (vertical) to 4 (horizontal) or flatter. Refer to Section 2.6.1 of the *Drainage Manual* for further discussion of protective treatment.

When it is necessary to provide a fence, one that fits the surrounding community is ideal. The style (wood, block, chain link, wrought iron, etc.) will vary from community to community. Pay item 0550-10 series covers special fencing; however, special details and specifications will need to be included in the contract documents. Because of the extra work, special fencing has not been commonly used. Another complication with special fencing is that the Department's maintenance units do not normally have the materials to repair them; therefore, confer with the local maintenance engineer anytime you are considering special fencing.

If it is not feasible to provide a special fence, the next option is to use standard FDOT fence. In rural areas, the Type A fence, Standard Plans, Index 550-001 usually is appropriate. In urban areas, Type B fence (chain link), Standard Plans, Index 550-002 usually is appropriate.

Fence Color:

One of the simplest things you can do to reduce the visual impact of chain link fence is to specify that it be color coated. Standard Plans, Index 550-002 offers an option for PVC (vinyl) coated fence fabric that is a soft gray color; however, you can specify the color to be medium green, dark green, or black as allowed by AASHTO M 181. The posts, rails, and fittings also can be color coated. To specify color-coated fence, use a pay item footnote (0550-102x2 thru 0550-102x2, as applicable to the fence height required) similar to the following:

> *Color coat the fence fabric, posts, rails, and fittings around the stormwater facilities with xxx (state the desired color) PVC. Apply the PVC coating of the posts and rails in addition to the standard metallic coating and ensure that it meets the requirements of ASTM F 1043. The PVC coating of the fittings must meet the requirements of ASTM F 626. Include the cost of the coating in the cost of these items.*

Fence Height and Barbed Wire Attachments:

The Department has no requirement for the height of the fence surrounding a stormwater facility, nor does it require the use of barbed wire attachments on a fence surrounding a stormwater facility. Other regulatory agencies may have applicable requirements regarding fence height and barbed wire attachments.

9.2.3.2 Debris Collection

Discuss with maintenance personnel and the District Landscape Architect the need to collect debris near the inflow pipe to the pond to prevent the debris from spreading. If it is possible to collect the debris, direct it to one location where maintenance personnel can easily remove it. Figure 9.2-3 shows some possible configurations.

Do not locate inflows and outlets near preserved existing vegetation or planted landscape areas that have the potential to shed leaves, limbs, etc., that may clog pipes and structures.

9.2.4 Aviation Safety Requirements

Per the *Drainage Manual,* when a prospective pond—wet or dry—is located within five miles of an airport, the drainage designer must contact the District Aviation Coordinator to ascertain any relevant airport design restrictions. The FAA requirements are targeted to minimize the potential for bird strikes and are specific to the types of aircraft using the airport and to the layout of the airport’s runways. The district aviation coordinators are familiar with these requirements and will provide guidance to the drainage designer.

The best choice, in responding to FAA requirements, is to move the proposed pond outside the glide paths of the air traffic. If this is imprudent, dry ponds are less attractive to birds than wet ponds. Additionally, several design approaches are routinely used in wet ponds to minimize attracting birds:

- Use steep, rocked slopes, typically 1:2, without littoral zones, to discourage the presence of food sources for birds.
- Suspend nets over the surface of the pond to make the area less hospitable for ducks, geese, and other waterfowl.
- Other options may be available through consultation with the airport manager and Qualified Airport Wildlife Biologist.
- Ask districts for other techniques

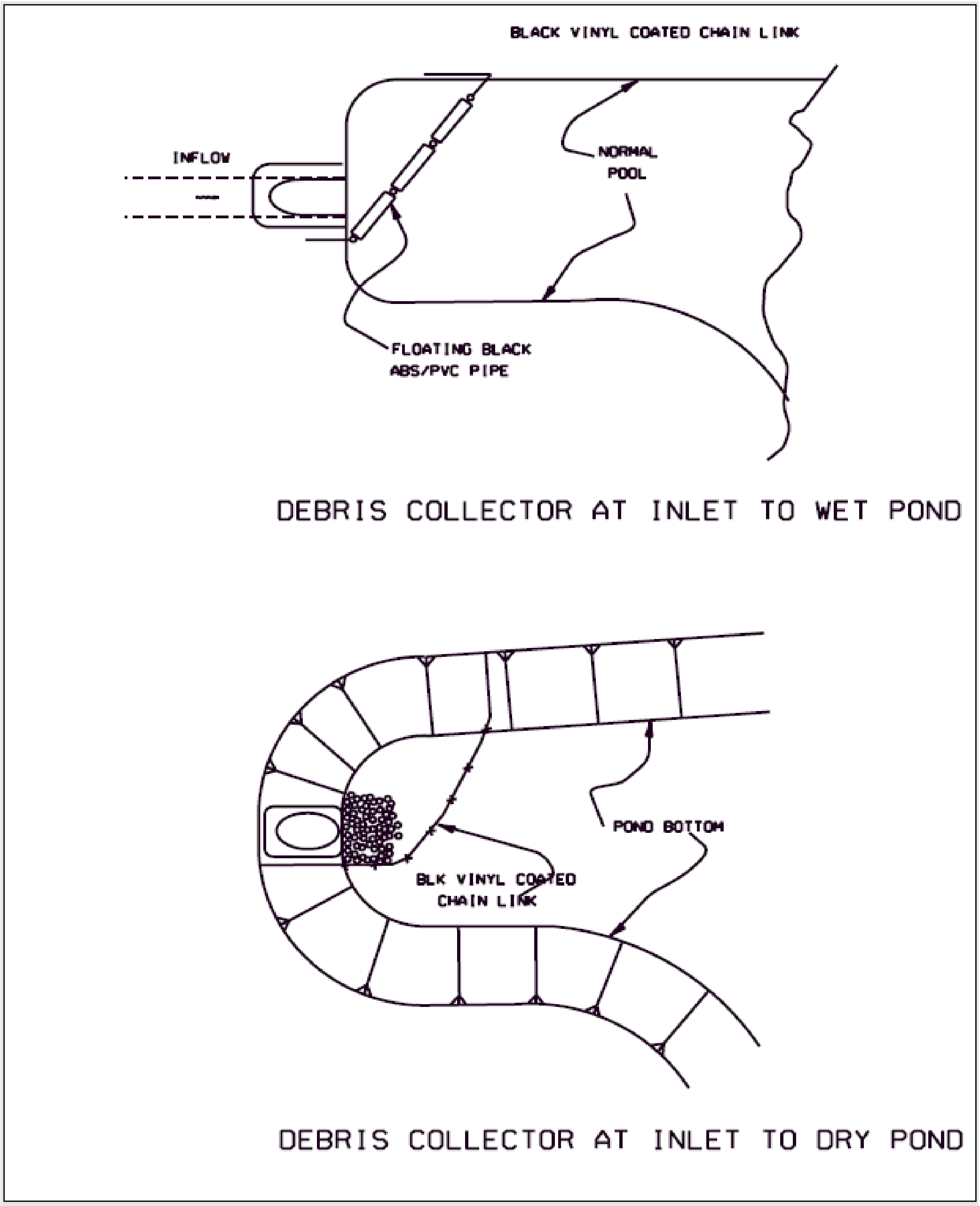


Figure 9.2-3: Possible Configurations to Collect Debris

9.3 STORMWATER QUALITY

9.3.1 Design Criteria

FDEP, the WMDs, and the delegated local governments have established design criteria for the operations of stormwater management facilities. There are two main categories of criteria: (1) water quality, and (2) water quantity (see Section 9.4). The criteria related to water quality are based on research of rainfall and runoff in Florida and were established to meet state water quality standards. See Appendix N for a discussion of the development of the typical criteria.

Although the criteria are similar around the state, there is some variation. It is essential that you become familiar with the applicable agency's criteria. Read their manuals and coordinate as necessary. Arrange a pre-application meeting to review the status of applicable rules and to identify potential problems and concerns to be addressed during design. Agencies usually have checklists and standard forms to be completed for a stormwater permit. Review these forms and address the items relating to stormwater management.

9.3.1.1 Treatment Volumes

Pollutants in stormwater runoff from urbanized areas generally exhibit a "first flush" effect. This is a phenomenon where the concentrations of pollutants in stormwater runoff are highest during the early part of the storm with concentrations declining as the runoff continues. Substantial reductions in pollutant loads to the state's waters will occur when this first flush is captured and treated. Therefore, each method of treatment requires that a volume of runoff be captured and treated before discharging to surface or groundwater. This volume is called the treatment volume.

In general, the treatment volume will vary depending on the classification of the receiving water body and whether the volume is captured on-line or off-line. Sensitive water bodies such as shellfish harvesting waters (Class II) and Outstanding Florida Waters require a larger treatment volume. The classification of the receiving water body should be identified in the Project Development phase as a part of the water quality impact evaluation. FDEP includes a list of sensitive water bodies in Rule 62-302, F.A.C.

9.3.1.2 Special Conditions

Some of the Department's districts have agreements with regulatory agencies regarding treatment requirements for certain types of highway improvements, such as bridge widening and intersection improvements. Check with the District Drainage Engineer to see if your project is covered by an agreement.

Compensatory treatment may be an option when trying to meet water quality regulations. Sometimes, limited or very expensive right of way creates hardship conditions in which it is unrealistic to provide the standard treatment. Sometimes, the Department can arrange to provide compensatory treatment for an area that currently does not receive any treatment. Providing this treatment compensates for not providing the standard treatment in the area where the hardship condition exists. Treating a larger volume of runoff at another location (drainage area) on the project usually is not considered replacement treatment.

Nutrient-Impaired Basins

When designing stormwater systems that discharge to basins verified for nutrient impairment, state law requires the applicant to demonstrate that there will be no net increase of the pollutant of concern. To satisfy this requirement, all WMDs require a pre- vs. post-development comparison of annual nutrient loading, using the Harper (2007) Methodology, to demonstrate that the post-development annual loading is not greater than the pre-development loading for the pollutant of concern. Guidance on this analysis is contained in Section 9.3.4.6 of this document.

The BMPTRAINS software, developed by the UCF Stormwater Management Academy (https://stormwater.ucf.edu/), can be used to analyze best management practice (BMP) nutrient removal from different land uses. See Section 4.5.1 for an example. Software results are readily accepted by permitting agencies around the state.

9.3.2 Concerns of Off-Line Systems

Although off-line treatment systems are preferred from a water quality standpoint and sometimes require less treatment volume, they can complicate the design. You would design off-line systems to bypass essentially all additional stormwater runoff volumes greater than the treatment volume to the receiving water or an attenuation basin. The bypass flow must pass over the weir of the diversion structure. This can present design problems in that the weir may need to be very long to keep the hydraulic gradient at an acceptable level. Skimmers need to be constructed in front of these weirs, further complicating the practicality of long weirs.

Another concern is that there will be some additional attenuation storage in the off-line basin associated with the hydraulic gradient of the peak flow passing over the weir. When there is significant attenuation storage above the treatment volume, there is a concern that the system will function more as an on-line system than as an off-line system due to mixing. You could use metal or rubberized flap gates to address this concern, but they can be a maintenance problem and a noncompliance issue if not carefully designed.

The outlet control structures of off-line systems are difficult to maintain simply because they normally are placed in junction boxes. They are neither seen nor reached as easily as the outlet control structures of on-line systems.

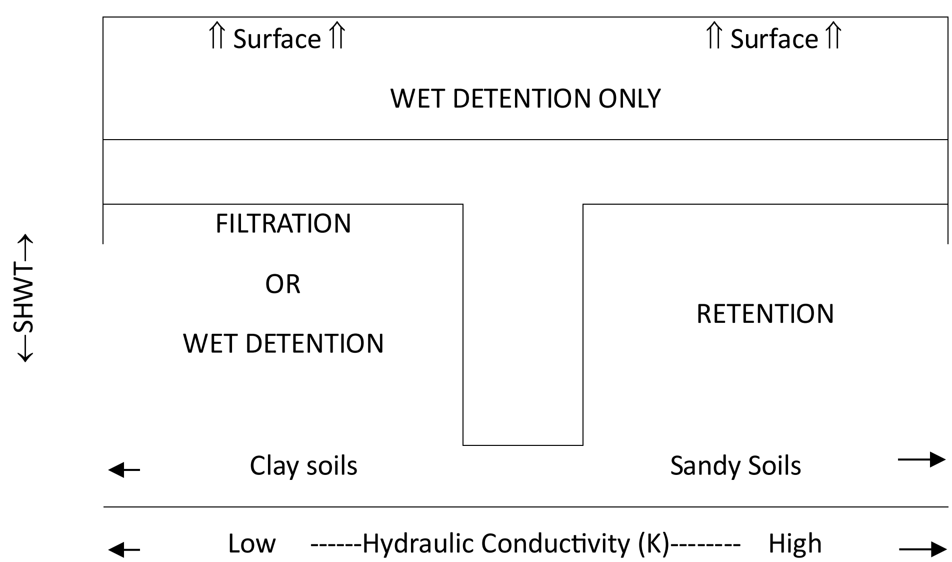
9.3.3 Seasonal High Water Table

Frequently, the first parameter considered in the design of a retention or detention BMP is the location of the water table. Define the depth to the normal high water table and the seasonal high water table (SHWT) to establish the appropriate type of BMP and the needed treatment volume. The SHWT is critical to the operation of all of the treatment methods described below. The control (or normal) water elevation of wet detention systems is related to, and sometimes set at, the SHWT. The SHWT is a critical factor in calculating the recovery time of the treatment volume in a retention system. For filtration systems, the lowest point of the underdrain pipe should be at least one foot above the SHWT.

Use the NRCS soil surveys, project-specific soil investigations, and field observations (vegetative indicators, observation wells, etc.) to estimate the SHWT. Recognize, however, that soil staining may denote a relic or historic water table that has since been lowered by other drainage features in the region.

9.3.4 Treatment Methods

The treatment methods most commonly used by the Department are wet detention, retention, filtration, and exfiltration. Refer to Chapter 7 of this document for exfiltration system BMPs. The type of soil and the SHWT control the selection of the treatment method. The following figure provides qualitative guidance for the selection.



As shown, wet detention is the only option in areas where the SHWT is near the surface. However, wet detention also may be appropriate in areas where the SHWT is far from the surface and clay soils exist. The use of retention requires that the SHWT be far from the surface and that sandy soils exist. Filtration requires that the SHWT be far from the surface unless impermeable liners are used.

You cannot apply specific values to this figure because site-specific factors—such as pond shape, groundwater boundary conditions, and drainage basin characteristics—need to be considered. Situations exist where both filtration and wet detention are suitable. In these cases, the Department should weigh and balance other factors—such as right-of-way costs, property owner preference, and long-term maintenance costs—to select the most appropriate treatment method.

9.3.4.1 Wet Detention Systems

These systems are permanently wet ponds that are designed to slowly release the treatment volume through the outlet control structure. The pollutants are removed by physical, biological, and chemical assimilation. Specifically, pollutant removal processes that occur within the permanent pool include uptake of nutrients by algae and wetland vegetation, adsorption of nutrients and heavy metals onto bottom sediments, biological oxidation of organic materials, and sedimentation.

Advantages	**Disadvantages**
1. Very effective at removing dissolved and suspended pollutants.	1. Treatment requirements are typically double the requirements for retention and filtration.
2. High probability to function as designed.	2. Depth of the treatment volume is sometimes limited to 1.5 feet.
3. Recovery of treatment volume is easily predicted.	3. Because of the above items, right-of-way requirements are greater than other methods.
4. Easy and low-cost long-term maintenance.	4. Sometimes requires planting of the littoral zone.
5. Produces on-site fill material for project needs	5. Creates a potential mosquito habitat.

Despite the disadvantages, the Department encourages the use of wet detention.

The average length-to-width ratio of the pond should be at least 2:1. Maximize the flow path of water from the inlet to the outlet to promote good mixing and avoid “dead” storage areas. If you cannot avoid short flow paths, use the littoral shelf to increase the effective flow path, provided this is acceptable to the regulatory agency. Figure 9.3-1 shows examples of pond configurations.

Per the regulatory agency requirements, you may need to plant the littoral shelf. If so, consult with the District Landscape Architect.

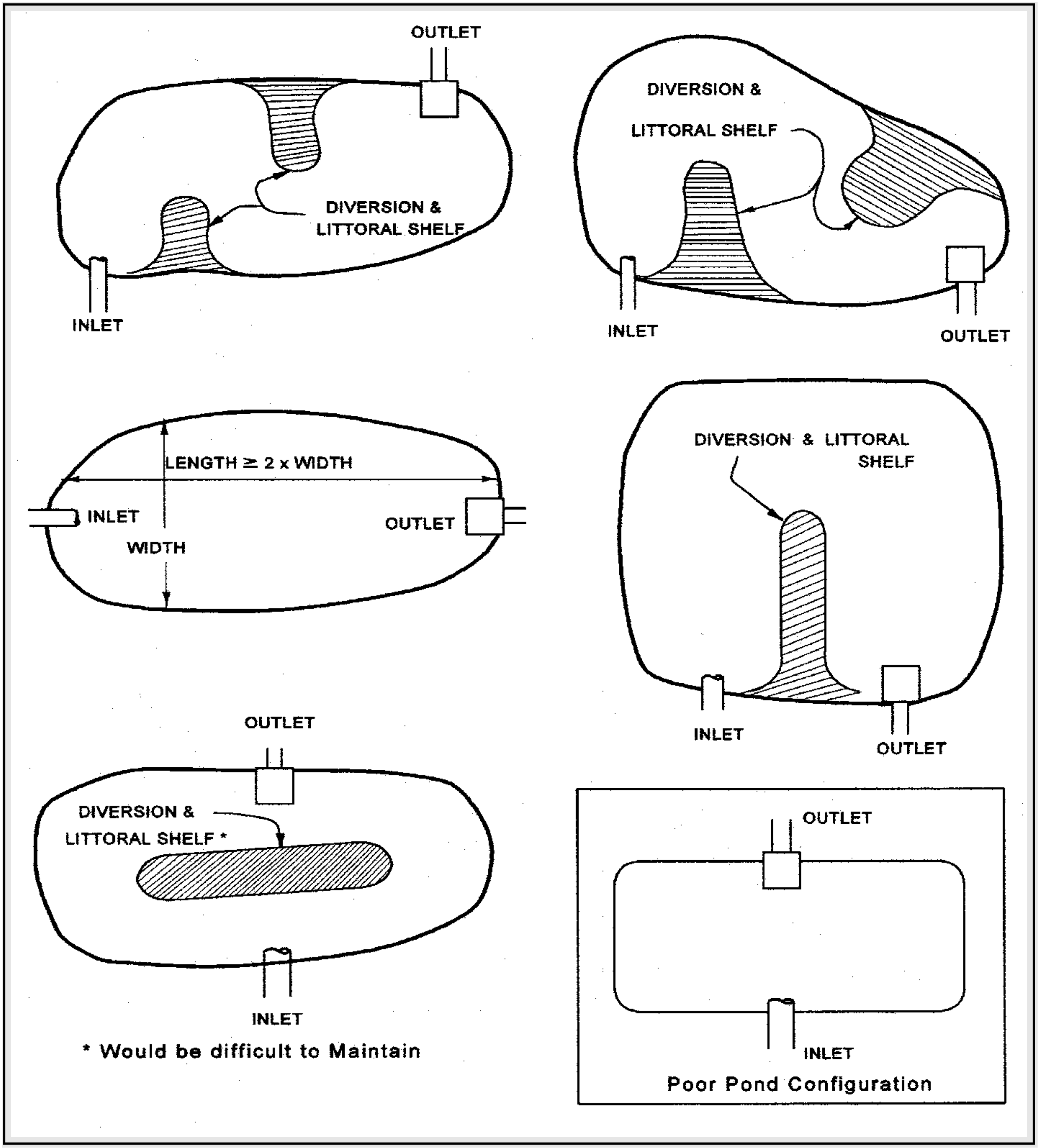


Figure 9.3-1: Wet Detention Configurations

9.3.4.2 Pond Liners

Pond Liner Applications

Due to the challenges around pond liners, there is an increased risk of failure. Using a pond liner should be thoroughly vetted and the design should be well supported by the unique environmental conditions, survey and geotechnical data. Some typical causes of failure can be inadequate cover, insufficient de-watering activities and lack of groundwater modeling.

Consult the District or State Drainage Office for Technical Specifications. While the Department does not encourage the use of liners, the following are design scenarios where the consideration of a pond liner is appropriate:

1. The stormwater facility is located within a Sensitive Karst Area Basin or the surrounding geography is susceptible to sinkholes due to excessive stormwater runoff.
2. If the stormwater facility is in proximity to hazardous environmental conditions, and water seeping from the pond risks mobilizing existing contaminants in the soil or groundwater.
3. When there is a need to preserve groundwater flows into the facility from adjacent wetlands.

Design Considerations

When designing an impermeable pond liner, a groundwater model is recommended to ensure the proposed liner does not negatively impact subsurface groundwater flows (i.e. a 20-acre wetland adjacent to a 1-acre pond or a 5-acre pond adjacent to a 1-acre wetland, will have differing impacts to pond water levels.) Factor in the Seasonal High Groundwater Elevations of existing wetland / depressional areas adjacent to the pond. If elevations in such areas are higher than the proposed top of impermeable liner, seepage into the new facility could occur. Additionally, temporary barriers may be required to protect wetland / depressional areas from being impacted by the de-watering activities necessary to construct the liner.

Installation of pond liners can be challenging and require an experienced contractor. Pond liners in facilities adjacent to right-of-way limits may require sheet piles or other temporary structural measures to stabilize the trench during the liner installation. Construction sequencing details may be needed to address the construction activities required to accomplish the grading and pond liner construction. This could include sheet piles, dewatering, excavation, seam welding, battens, perimeter anchoring and liner penetration methods.

Before a pond liner may be implemented on a project, perform a cost benefit analysis to compare the costs related to pond liner construction versus an expanded pond footprint. Examine the material within the pond footprint to see if it's worth-while fill (embankment) material. If the material can be used on the project, this could justify the cost associated with excavation and de-watering needed to complete the installation.

Materials Information

Impermeable pond liners come in many different materials: High Density Polyethylene (HDPE), Polyvinyl Chloride (PVC), Reinforced Polyethylene and clay. Below are material minimums for using a pond liner, however, always consult your District Drainage Engineer and District Geotechnical Engineer for project specific considerations.

1. PVC, use a minimum of 30 mil
2. Reinforced Polyethylene Geomembrane, use a minimum of 30 mil
3. HDPE, use a minimum of 60 mil
4. Clay liners, use a minimum thickness of two feet or an in-place hydraulic conductivity of 1 x 10-7 centimeters per second (cm/s) or less

9.3.4.3 Retention Systems

A retention system is designed to store the treatment volume, allowing it to infiltrate into the soil. Soil permeability, water table conditions, and the depth to any confining layer must be such that the retention system can infiltrate the treatment volume within a specified time following a storm event. After the pond completes the drawdown, the basin does not hold water; thus, the system is normally "dry." Unlike wet detention systems and filtration systems, the retention system will discharge the treatment volume into the ground, not to surface waters.

Most regulatory agencies require that the treatment volume be available within 72 hours after a storm. See Section 9.4.6.1 on the subject of groundwater flow from retention systems and a recommended approach to modeling recovery of the treatment volume.

9.3.4.4 Filtration/Underdrain Systems

A filtration system is designed to treat the water quality volume, allowing it to pass through a sand filter. It differs from a retention system in that the treatment volume is not infiltrated into the soil, but instead discharges to surface water. After passing through the sand filter, the water collects in perforated pipes that discharge to surface water. The Department's standard underdrain is shown in Standard Plans, Index 440-001.

Compared with the previous two treatment methods discussed, underdrains are the least reliable. They are subject to clogging during and after construction and are difficult to maintain. Vehicle loads can crush the underdrain pipes. Filtration systems also do not remove dissolved constituents, such as nitrogen and phosphorus, and therefore do not count toward load reduction credit in impaired basins. The Department realizes that using underdrains is sometimes necessary due to clayey soils but encourages a thorough evaluation of other treatment methods first.

Configuration:

When you use side bank underdrain (Standard Plans, Index 440-001, Type Va), slope the pond bottom up from the underdrain. This will reduce the saturated soil condition and localized ponding associated with a flat pond bottom. It also increases the chances of sustaining a stand of grass on the bottom. See Figure 9.3-2.

If feasible, construct underdrains out of the primary flow path to avoid directing debris and sediments there.

To account for construction tolerances, the underdrain pipe should be placed on a slope. Specify flow lines for the pipe at the beginning, at bends, and at the end of the underdrain. In all but very short runs of underdrain, the flow line should drop six inches or more to account for construction tolerances.

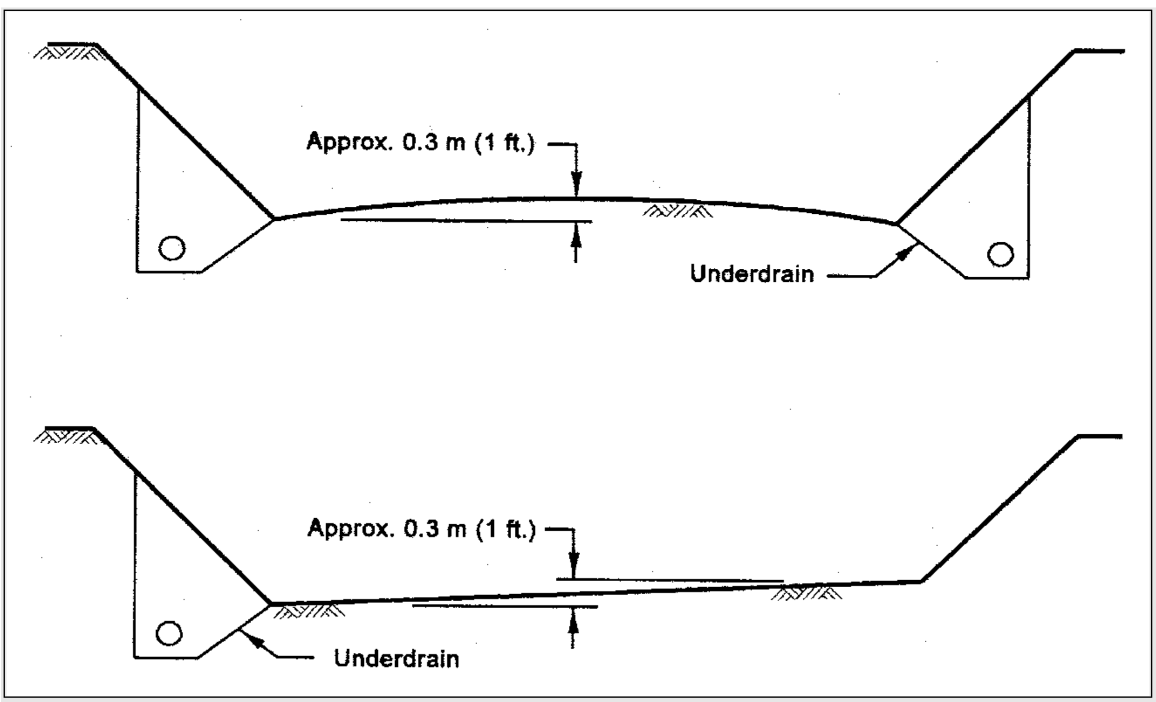


Figure 9.3-2: Bottom Configurations with Side Bank Underdrains

Design Technique

Hydraulic Conductivity of the Fine Aggregate Media:

For design purposes, use K = 0.5 ft/hr as the hydraulic conductivity of the fine aggregate media. This does not include the factor of safety of two required by the regulatory agencies. You do not have to apply that factor of safety to the hydraulic conductivity. It is sometimes applied to the length of the underdrain or to the time to draw down the treatment volume. You could refine the above value by experience from permeability testing of locally available fine aggregate media meeting the requirements of the standard specifications for underdrain filter material.

Determining the length of underdrain required is a trial-and-error process and can be accomplished by using the following procedure with Table 9.3-2.

1. Develop incremental storage volumes from the maximum elevation of retention storage (i.e., lowest elevation of the outlet control structure) down to the pond bottom. Record these in Column 1 through Column 3 of Table 9.3-2.

2. Determine the effective head (H_E), the average flow length (L_{AVG}), and the average width (W_{AVG}) for flow paths through the underdrain. Determine these for each water surface elevation considered in Step 1. See the discussion following Step 10 for a suggested approach to determining these values. Record these in Column 4 through Column 6.

3. Calculate the hydraulic gradient (i) for each water surface elevation considered in Step 1 using the values determined in Step 3, and record the results in Column 7. Hydraulic gradient (i) = H_E/L_{AVG}.

4. Assume an underdrain pipe length (L) and calculate the area of filter (A) for each water surface elevation considered in Step 1. Record results in Column 8.

5. Calculate the Darcy flow (Q) using the hydraulic conductivity (K), the hydraulic gradient, and the filter area for each water surface elevation considered in Step 1. Record results in Column 9.

6. Calculate the average flow rate for each depth interval and record results in Column 10.

7. Divide the incremental storage volume (ΔV) from Column 3 by the average flow rate from Column 10 to obtain the incremental time (ΔT) to draw down that storage volume. Record results in Column 11.

8. Sum the incremental drawdown times recorded in Column 11 to obtain the drawdown time (ΣT). Record results in Column 12.

9. If the total computed drawdown is longer than required, increase the underdrain length and return to Step 5.

10. Size the underdrain pipe to handle the design flow rate.

Determining the Effective Head, Average Flow Length, and Average Width:

Bottom Underdrain (Type Vb):

To determine the effective head (H_E) at a given water surface, use the vertical distance from the water surface to the bottom of the fine aggregate material. For the average flow length (L_{AVG}) through the filter, use the depth of fine aggregate, 2.0 feet. For the average width (W_{AVG}) of filter normal to flow, use the standard width of 1.5 feet unless you use non-standard geometry.

Side Bank Underdrains (Type Va):

The standard plans index shows the upper and lower limit to side bank underdrain. Try to avoid using the upper limit configuration because of its limited flow capacity in low head conditions. There is very little head and the length of the filter material through which the water must pass is long, resulting in a very small hydraulic gradient.

1. Make a scaled drawing of the average cross section geometry. One is shown in Figure 9.3-3. The average should represent the midpoint between the high end and low end of the underdrain.

2. For the effective head (H_E) at a given water surface elevation, use the vertical distance from the water surface to the pipe centerline. At high heads, this is non-conservative because the free draining effect of the course aggregate reduces the head. At low heads, this is a reasonable assumption.

> The combined effect of using H_E & L_{AVG} as described here should result in conservative flow rates in low head conditions and reasonable rates in high head conditions. At high heads, the non-conservatism of using the effective head (H_E) to the center line of the pipe is offset by using an average length (L_{AVG}) that is longer than the actual distance through the fine aggregate. At low heads, the conservatism of using a longer-than-actual average length (L_{AVG}) is justified because this zone of the filter is most likely to receive sediment and clog.

3. For the average flow length (L_{AVG}) through the filter at a given water surface, use the average of several straight-line distances from the outside of the pipe to the top of the fine aggregate. This is conservative because it ignores the course aggregate, which is relatively free draining. Refer to Figure 9.3-3 and Table 9.3-1 for an example.

4. For the average width (W_{AVG}) of filter normal to flow, use the average of the saturated fine aggregate area. Due to the complex transition between vertical and horizontal flow, this is best determined by “visually” estimating the average width based on your scaled drawing. Refer to Figure 9.3-3 and Table 9.3-1 for an example.

Table 9.3-1: Average Flow Length and Average Width through Side Bank Underdrain

Water Surface Elevation	L AVG Avg. Flow Length through Filter	W AVG Avg. Width of Filter Normal to Flow
WSE-5 or above	(L5 + L4 + L3 + L2 + L1 + L0) / 6	W to W5
WSE-4	(L4 + L3 + L2 + L1 + L0) / 5	W to W4
WSE-3	(L3 + L2 + L1 + L0) / 4	W to W3
WSE-2	(L2 +L1 + L0) / 3	W2A to W2B
WSE-1	(L1 + L0) / 2	W1A to W1B
Refer to Figure 9.3-3.		

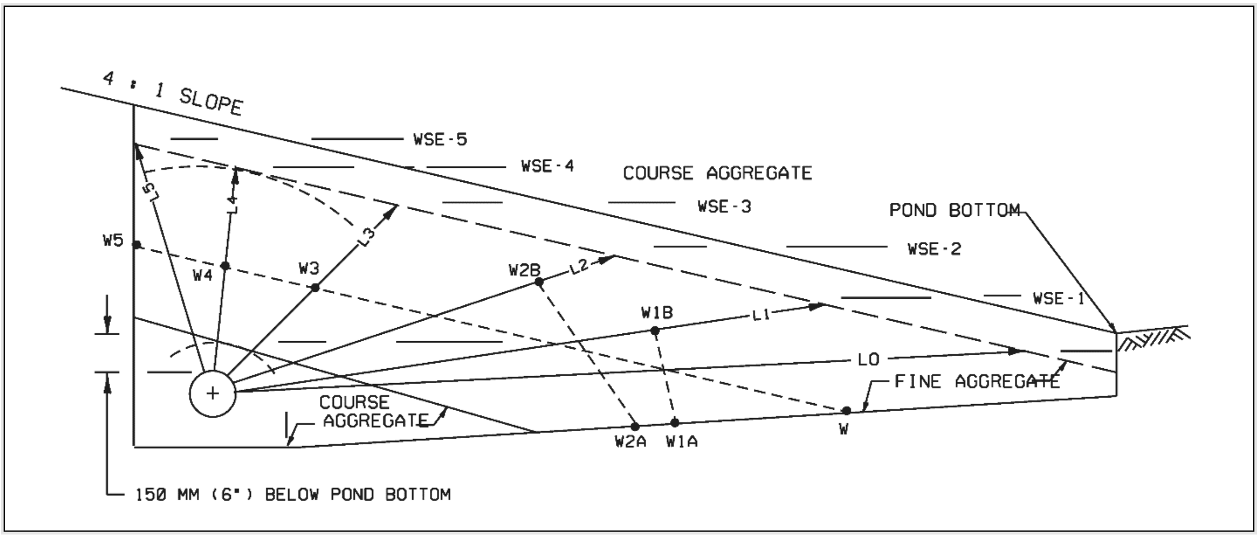


Figure 9.3-3: Side Bank Underdrain (Shown 6" Below Upper Limit)

Table 9.3-2: Drawdown Worksheet for Underdrain

DRAWDOWN WORKSHEET FOR UNDERDRAIN

(Using Darcy's Law for Flow-Through Porous Material)

1	2	3	4	5	6	7	8	9	10	11	12
Water Surface Elevation (NGVD)	V Total Volume (ft^3)	ΔV Incr. Volume (ft^3)	H_E Effective Head (ft)	L_{AVG} Avg. Flow Length Through Filter (ft)	W_{AVG} Avg. Width of Filter Normal to Flow (ft)	Hydraulic Gradient $i = H_E / L_{AVG}$ (ft/ft)	Area of Filter $A = L \cdot W_{AVG}$ (ft^2)	Darcy Flow $Q = KiA$ (ft^3/hr)	Average Flow $\frac{Q_1 + Q_2}{2}$ (ft^3/hr)	ΔT Incr. Time (hr)	ΣT Total Time (hr)

Assumed Underdrain Length L = ______________ ft

Hydraulic Conductivity K = ______________ ft/hr (fine aggregate)

9.3.4.5 Stormwater Re-Use Systems

These systems represent wet ponds that provide water for re-use—either for irrigation, alternative water supply, or supplemental water for reclaimed wastewater lines. A wet detention pond can be converted to a re-use pond simply by plugging the bleeder and pumping the treatment volume to its designated re-use. Since there is less discharge volume compared to a standard wet detention pond, there is less mass of pollutant being released from the stormwater re-use pond.

9.3.4.6 Regional Stormwater Pond Systems

Regional stormwater ponds, by definition, provide water quality treatment for a significant portion of the upstream basin, not just the onsite FDOT project. These ponds often are located downstream of the FDOT project, avoiding the more expensive land adjacent to the state highway. Typically, this approach includes the cooperation of a local government that assumes ownership and perpetual maintenance of the pond. FDOT holds a storage easement, prescribing a needed storage volume below a certain design elevation. Multiple gains result from this cooperative approach:

1. FDOT is relieved of ongoing property liability and maintenance responsibility.
2. The downstream waterway enjoys improved water quality.
3. Property adjacent to the state highway, previously targeted for usage as a pond, is available for development.
4. Oftentimes, stormwater re-use is facilitated by a single, larger stormwater pond.

Regional treatment facilities can be difficult to permit because of the Class III treatment requirements of conveyance facilities between the FDOT site and the location of the regional pond. These intermediate waterway requirements sometimes can be eliminated by classifying the manmade, intermediate conveyance waterways as part of the stormwater system, thereby severing jurisdiction. The cooperation of the WMD will be essential in such efforts.

9.3.4.7 Harper (2007) Methodology for Nutrient Loadings Computation

The 2007 Harper Methodology was the computational foundation for the 2009 Statewide Stormwater Rule. The rule was not implemented, but the Harper Methodology has been accepted by the WMDs and FDOT as a best practice for estimating annual nutrient loadings. Details of the methodology are outlined in the March 2010 draft of the Stormwater Quality Applicant's Handbook posted on the state drainage website, under Design Aids: http://www.fdot.gov/roadway/Drainage/ManualsandHandbooks.shtm

The above draft publication is referenced ONLY for its helpful outline of the background rationale and computational steps involved in the Harper Methodology, NOT for regulatory requirements.

Since 2010, event mean concentrations (EMCs) for different land uses have changed as additional data have become available. Current general EMCs are tabulated below:

Table 9.3-3: Example EMC Values for Different Land Uses

LAND USE CATEGORY*	Event Mean Concentration (EMC) mg/L	
	Total Nitrogen	Total Phosphorus
Single-Family	2.07	0.327
Multi-Family	2.32	0.520
High-Intensity Commercial	2.40	0.345
Light Industrial	1.20	0.260
Highway	1.64	0.220
Agricultural—Citrus	2.24	0.183
Agricultural—Row Crops	2.65	0.593
Agricultural—General Agriculture	2.79	0.431
Undeveloped	1.15	0.055

*Numbers may vary as more information becomes available or for specific locations.

The BMPTRAINS computer program was developed to employ the latest policy and methodology for assessing nutrient loadings and BMP performance related to nutrient removal. The program is available on-line at the UCF Stormwater Management Academy website: http://stormwater.ucf.edu/

The program includes helpful tutorials and a user's manual.

Additional helpful tools sponsored by the Academy are available under the title Stormwater Management and Design Aids (http://stormwater.ucf.edu/). A partial list of programs in the SMADA online package is below:

1. BMPTRAINS "Light"—Used to select one BMP with an estimate of nutrient pollutant removal and in the selection of BMPs for net improvement or pre/post analysis.
2. BMP performance evaluation
3. Rainfall distributions and IDF curves

4. Statistical analyses such as regression and frequency distributions
5. Time of concentration
6. Hydrograph generation
7. Unit hydrograph generation
8. Transport pipe and channel flow and sizing
9. Pollutant load calculations
10. Storm sewer design and analysis

9.3.4.8 Protection of Springsheds from Nitrates

The Harper Methodology targets annual loadings of nutrients to surface waters, making the assumption that nutrients infiltrated into the ground via retention systems are "removed." For springsheds, nitrates infiltrated into the ground are the critical transport mechanism for springshed impairment. Nitrate-removing retention BMPs currently are under development using bio-activated media (BAM). Until design methodology is released, contact your local District Drainage Engineer for guidance when designing retention ponds within Karst springshed geology.

Examples Illustrating the Use of BMPTRAINS for Nutrient Loading Analysis

FDOT has extracted relevant design criteria and combined them into one reference publication and computer program, named BMPTRAINS. The design engineer should verify the design criteria at a pre-meeting with the WMD or FDEP, since newer regulations may exist. The BMPTRAINS model provides the option to over-ride existing criteria and assumptions. An example of an assumption is the event mean concentration (EMC) data.

Example Problem 9.3-1: Wet detention, net improvement

A wet detention pond serves a section of a two-lane highway that is about 1,100 feet long and the right-of-way width is about 200 feet. The catchment area is five acres and is part of a larger watershed that may impact the design. The existing portion of highway was not served by any treatment system. The existing and proposed portion of the highway will be treated in the post-development condition. The site is located in West Palm Beach, Florida, on Hydrologic Soil Group D. The existing land use condition is assumed to be a highway with a non-DCIA Curve Number of 80 and 40 percent DCIA. The post-development land use condition is assumed to be a highway with a non-DCIA Curve Number of 80 and 85 percent DCIA. The area needs net improvement using a wet detention pond with a littoral zone (assumed 10 percent removal efficiency credit for the littoral zone) in the design. The area and depth for the pond allowed an average annual pond residence time of 50 days.

First, identify the meteorological zone, which is Zone 5, and the mean annual rainfall, which is 61 inches, as shown below.

General Site Information for Example Problem 9.3-1

GENERAL SITE INFORMATION:	GO TO INTRODUCTION PAGE	
STEP 1: Select the appropriate Meteorological Zone, input the appropriate Mean Annual Rainfall amount and select the type of analysis		
	CLICK ON CELL BELOW TO SELECT	
Meteorological Zone (Please use zone map):	Zone 5	
Mean Annual Rainfall (Please use rainfall map):	61.00	Inches
	CLICK ON CELL BELOW TO SELECT	
Type of analysis:	Net improvement	
Treatment efficiency (leave empty if net improvement analysis is used):		%

Next, the catchment site information data are summarized below.

Watershed Characteristics for Example Problem 9.3-1

WATERSHED CHARACTERISTICS	STORMWATER TREATEMENT ANALYSIS		
CATCHMENT NO.1 CHARACTERISTICS:			
	CLICK ON CELL BELOW TO SELECT		
Pre-development land use:	Highway		
	CLICK ON CELL BELOW TO SELECT		
Post-development land use:	Highway		
Total pre-development catchment area:	5.00	AC	
Total post-development catchment area:	5.00	AC	
Pre-development Non DCIA CN:	80.00		Pre-development Annua
Pre-development DCIA percentage:	40.00	%	Pre-development Annua
Post-development Non DCIA CN:	80.00		Post-development Annu
Post-development DCIA percentage:	85.00	%	Post-development Annu

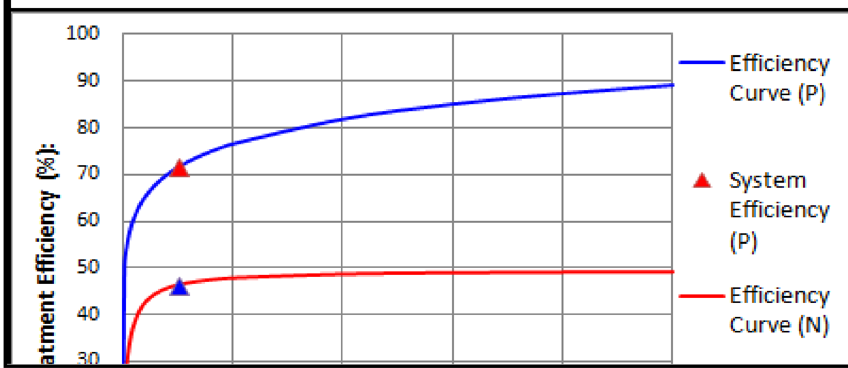
Using the BMPTRAINS program, the net improvement expected with the wet detention design is 71.5 percent removal of total phosphorus and 46.2 percent removal of total nitrogen, as shown in the BMPTRAINS program screenshot below. Note that, for the wet detention option, a residence time greater than 50 days will only marginally improve removal. Thus, a design criterion of 50 days annual residence time is above the minimum required by the water management district (21 days x 1.5 = 31.5 days) and can fit within the existing right of way.

Wet Detention Pond Effectiveness for Example Problem 9.3-1

Total pre-development watershed area:	5.000	ac
Total post-development watershed area:	5.000	ac
Average annual residence time (between 1 and 500 days):	50.00	days
Littoral Zone used in the design:	YES	
Littoral Zone efficiency credit (default credit at 10%):	10.00	%
Total **Nitrogen** removal required:	41.268	%
Total **Phosphorus** removal required:	41.268	%
Total **Nitrogen** removal efficiency provided:	46.204	%
Total **Phosphorous** removal efficiency provided:	71.485	%
Is the wet detention sufficient:	YES	

Wet Detention Pond Characteristics:

Anoxic Depth:	10.741	ft
Minimum Pond Permanent Pool:	2.472	ac-ft



NOTE FOR TREATEMENT EFFICIENCY GRAPH:

The purpose of the treatment efficiency graphs is to help illustrate the treatment efficiency of the wet detention system as the function of average annual residence time (and permanent pool volume). The graph illustrates that there is a point of diminished return as the

Example Problem 9.3-2: A highway receiving runoff from an industrial park that will have retention systems in a series—a pre- vs. post-development loading analysis is required

The water table conditions in this area are suitable for retention systems. An exfiltration trench in series with a retention basin can serve a five-acre light-intensity commercial site. The catchment also can contain 10 tree wells along the road. The tree wells are designed to be three feet deep with a six-inch depth above soil column. The length and width of the tree wells are designed to be four feet each. Use a 0.2 sustainable water storage capacity of the soil. Treat the tree wells as retention systems. The site is located in Orlando, Florida, on Hydrologic Soil Group C. The existing land use condition is assumed to be undeveloped-dry prairie with a non-DCIA Curve Number of 79 and 0.0 percent DCIA. The post-development land use condition is a low-intensity commercial area with a non-DCIA Curve Number of 85 and 65 percent DCIA. The combination of treatment systems is to provide treatment sufficient to match the post-development annual nutrient loads with the pre-development annual nutrient loads.

First identify the meteorological zone and annual rainfall for the project.

General Site Information for Example Problem 9.3-2

GENERAL SITE INFORMATION:	GO TO INTRODUCTION PAGE	
STEP 1: Select the appropriate Meteorological Zone, input the appropriate Mean Annual Rainfall amount and select the type of analysis		
	CLICK ON CELL BELOW TO SELECT	
Meteorological Zone (Please use zone map):	Zone 2	
Mean Annual Rainfall (Please use rainfall map):	51.00	Inches
	CLICK ON CELL BELOW TO SELECT	
Type of analysis:	Net improvement	
Treatment efficiency (leave empty if net improvement analysis is used):		%

A summary of the catchment characteristic data is shown below. Note that the catchment is highly developed, leaving no feasible space for a retention basin or other land-intensive BMP. You need to consider BMPs that are more useful for ultra-urban environments.

To meet pre/post conditions, the required phosphorus removal is calculated as 70.3 percent and the required nitrogen removal is 89.3 percent. These are calculated knowing the Event Mean Concentrations and the pre- and post-runoff volumes.

Catchment Characteristics Data for Example Problem 9.3-2

WATERSHED CHARACTERISTICS			STORMWATER TREATEMENT ANALYSIS
CATCHMENT NO.1 CHARACTERISTICS:			
	CLICK ON CELL BELOW TO SELECT		
Pre-development land use:	Undeveloped - Dry Prairie		
	CLICK ON CELL BELOW TO SELECT		
Post-development land use:	Low-Intensity Commercial		
Total pre-development catchment area:	5.00	AC	
Total post-development catchment area:	5.00	AC	
Pre-development Non DCIA CN:	79.00		Pre-development Annua
Pre-development DCIA percentage:	0.00	%	Pre-development Annua
Post-development Non DCIA CN:	85.00		Post-development Annu
Post-development DCIA percentage:	65.00	%	Post-development Annu

The tree wells receive runoff water first and, thus, the effectiveness associated with a design is examined first. The capture effectiveness is low (1.3 percent) because of the number and size of the catchment. The results are shown below.

Effectiveness of 10 Tree Wells in the Catchment or Watershed of Example Problem 9.3-2

Note: As the BMPTRAINS model is improved, output screens may change.

VEGETATED AREAS (Example Tree Wells):					
Vegetated Areas (tree wells or similar) for:	Facility handbook example 2				
	Catchment 1	Catchment 2	Catchment3	Catchment 4	
Contributing catchment area:	5.000	0.000	0.000	0.000	ac
Required treatment efficiency (**Nitrogen**):	70.300				%
Required treatment efficiency (**Phosphorus**):	89.257				%
Vegetated Area (Tree Well) depth	3.00				ft
Vegetated Area (Tree Well) water depth above soil column:	0.50				ft
Vegetated Area (Tree Well) length:	4.00				ft
Vegetated Area (Tree Well) width:	4.00				ft
Sustainable water storage capacity of the soil:	0.20				
Number of similar Areas within watershed:	10.00				
Retention depth for provided hydraulic capture efficiency:	0.010	0.000	0.000	0.000	in
Is this a retention or detention system?	Retention				
Type of soil augmentation: View Media Mixes					
Provided treatment efficiency (**Nitrogen**):	1.307	0.000	0.000	0.000	%
Provided treatment efficiency (**Phosphorus**):	1.307	0.000	0.000	0.000	%
Is/are the vegetated areas sufficient?	NO				

Next, you can use an exfiltration system to collect some of the runoff water. Using the geometric design for the exfiltration, the retention storage volume is calculated as 0.55 inches. The effectiveness of using the exfiltration design is shown below.

Exfiltration Trench Design and Effectiveness for Example Problem 9.3-2

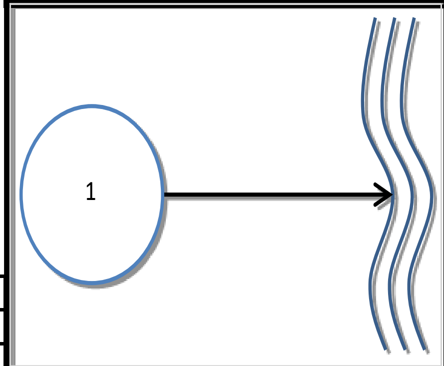
EXFILTRATION TRENCH:		
EXFILTRATION TRENCH SERVING ENTIRE CONTRIBUTING WATERSHED:		
Contributing watershed area:	5.000	ac
Required Treatment Eff (**Nitrogen**):	70.300	%
Required Treatment Eff (**Phosphorus**):	89.257	%
Required retention for the entire watershed to meet required efficiency:	1.756	in
Required water quality retention volume:	0.732	ac-ft
EXFILTRATION TRENCH FOR MULTIPLE TREATMENT SYSTEMS (use only if other BMP method is oversized or undersized) :		
Provided retention depth:	0.550	in
Provided treatment efficiency (**Nitrogen**):	56.620	%
Provided treatment efficiency (**Phosphorus**):	56.620	%
Remaining treatment efficiency needed (**Nitrogen)**:	31.535	%
Remaining treatment efficiency needed (**Phosphorus)**:	31.535	%
Remaining retention depth needed if retention:	1.206	in

Finally, you can use a retention basin at the discharge from the watershed. The land for the retention basin within the watershed is part of the industrial park but did not provide sufficient removal by itself to meet post-equal-pre average annual mass loading. The retention basin can hold 0.50 inches of runoff and, thus, was limited to the removal effectiveness associated with that volume of storage. Using a combination of retention basin, tree wells, and an exfiltration trench provided by the roadway was sufficient to meet the post-equal-pre requirements.

The retention options are in a series. The first flush of water is captured by the tree wells and what is not captured is routed to the inlets for the exfiltration trench. The exfiltration trench can handle a fraction of that runoff, so the bypassed water is routed to the retention basin. All of these retention BMPs are designed to be off-line BMPs. Summary results of the tree well, exfiltration, and retention basin designs with the overall effectiveness removal are shown below.

Summary Loadings and Removal Effectiveness for Example 9.3-2

Summary Performance			
Catchment Configuration	A - Single Catchment		3/25/2014
Nitrogen Pre Load (kg/yr)	5.35		BMPTRAINS MODEL
Phosphorus Pre Load (kg/yr)	0.29		
Nitrogen Post Load (kg/yr)	18.00		
Phosphorus Post Load (kg/yr)	2.73		
Target Load Reduction (N) %	70		
Target Load Reduction (P) %	89		
Target Discharge Load, N (kg/yr)	5.35		
Target Discharge Load, P (kg/yr)	0.29		
Provided Overall Efficiency, N (%):	57		
Provided Overall Efficiency, P (%):	57		
Discharged Load, N (kg/yr & lb/yr):	7.72	17.00	
Discharged Load, P (kg/yr & lb/yr):	1.17	2.58	
Load Removed, N (kg/yr & lb/yr):	10.28	22.65	
Load Removed, P (kg/yr & lb/yr):	1.56	3.44	



For additional example problems, see the User's Manual to be used with the BMPTRAINS model (www.stormwater.ucf.edu) located at:

University of Central Florida Stormwater Management Academy "Managed Stormwater is Good Water"

What's New

BMPTRAINS Stormwater Best Management Practices Analysis Model (Latest Version; https://stars.library.ucf.edu/bmptrains/**)** and User's Manual (https://stars.library.ucf.edu/bmptrains/25/).

9.4 STORMWATER QUANTITY CONTROL

9.4.1 The Department's Design Storms

A problem with developing a design storm distribution is that actual storms have an unlimited combination of durations and intensity patterns. What should the duration of the design storm be? Should the peak rainfall occur near the beginning, in the middle, or near the end of the storm? Should there be multiple peaks?

Most of the current widely used rainfall distributions address this by nesting short-duration, high-intensity storms in the middle of a long duration storm, although very intense peaks do not usually occur in long storms. You usually would place the largest intensity value in the middle of the storm pattern, then place the remaining values alternately before and after this point, in order of decreasing intensity. The various NRCS distributions, the South Florida Water Management District (SFWMD) three-day distributions, and the St. Johns River Water Management District (SJRWMD) four-day distributions are examples of design storm distributions created using this approach. These "nested" distributions are not indicative of actual rainfall patterns and subsequently may produce inaccurate representations of actual runoff characteristics.

You may have used these distributions in the past for the design of conveyance systems because they give conservatively high runoff estimates. But, when you use these distributions to determine the pre-developed discharge, they can overestimate it. In the developed condition, the outlet control structure would be designed to pass the "overestimated pre-developed discharge," thereby discharging more in the post-developed condition.

Another problem with these distributions is that different drainage areas will react differently to the same rainfall pattern. Small basins with short times of concentration and little storage will have higher runoff rates from short, intense storms than from long-duration, low-intensity storms. Long-duration, low-intensity storms usually do not generate peak discharges from small basins. The opposite is true for large basins. Very large basins with large amounts of storage will have less runoff from short, intense storms than from long-duration, low-intensity storms. Large river systems and static water bodies such as lakes reach peak stages when extreme antecedent conditions exist and variations in intensity usually do not affect their stages.

To overcome the concerns of a single design storm distribution, the Suwannee River Water Management District (SRWMD) developed a series of design distributions to better reflect actual rainfall patterns. They developed distributions for 1-, 2-, 4-, and 8-hour storms and for 1-, 3-, 7-, and 10-day storms using National Oceanic and Atmospheric Administration (NOAA) hourly and sub-hourly data. SRWMD requires the use of these distributions for projects within the district.

Chapter 14-86, Florida Administrative Code

In 1986, the Department established Chapter 14-86 of the F.A.C., requiring adjacent developments to maintain discharges at or below pre-developed discharges using a multiple storm approach. In the Department's Drainage Connection Handbook (February 1987), the SRWMD design distributions mentioned above were accepted as appropriate for the entire state. These distributions can be found at the Department's website, listed below:

https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/roadway/drainage/files/fdotrainfalldistributions.pdf.

In a July 1988 memorandum, the State Roadway Design Engineer directed the districts to design the Department's stormwater management systems to Chapter 14-86. In October 1992, the *Drainage Manual* was revised to require the design of the Department's stormwater management systems to comply with Chapter 14-86. In 2013, the *Drainage Manual* was amended to require the application of Chapter 14-86 on FDOT stormwater designs only for closed basins and areas where downstream historical flooding is documented.

9.4.1.1 Critical Duration

Since the time the Department developed Chapter 14-86, there have been two interpretations of the critical duration and how to apply the multiple storm concept. The definition of critical duration (shown below), as defined in Chapter 14-86, lends itself to two interpretations.

> "Critical Duration" means the duration of a specific storm event (i.e., 100-year storm) that creates the largest volume or highest rate of net stormwater runoff (post-development runoff less pre-development runoff) for typical durations up through and including the 10-day duration event. The critical duration is determined by comparing various durations of the specified storm and calculating the peak rate and volume of runoff from each. The duration resulting in the highest peak rate or largest total volume is the "critical duration" storm.

(A) Peak Discharge Approach

This interpretation of critical duration and the multiple storm concept allows a post-developed runoff rate, for a given frequency, that is equal to or less than the highest pre-developed runoff rate of any duration. For example, given the pre-developed runoff rates shown in the table below, the allowable runoff rate would be 70, regardless of the duration associated with the peak post-developed runoff rate. The post-developed runoff rates shown are acceptable because none are greater than 70. You need only run enough durations in the post-developed condition to be assured that runoff rates of the other durations do not exceed the allowable.

Duration	Pre-Dev Runoff XX Year Event	Acceptable Post-Dev Runoff XX Year Event
1-hour	65	
2-hour	70	60
4-hour	66	70
8-hour	60	65
24-hour	30	35
3-day	25	
7-day	24	
10-day	21	

This approach is consistent with the last sentence of the definition of critical duration. *"The duration resulting in the highest peak rate . . . is the critical duration."* With this approach, the pre-developed critical duration can be different from the post-developed critical duration, as shown in the values above. Also, the pre-developed runoff rate could be calculated with the rational method (Q = CIA) for small basins; therefore, it would not be directly associated with any of the eight durations. The examples in the *Drainage Connection Handbook* follow this interpretation.

The above discussion pertains to discharges to open basins only with historical flooding documented. For discharges to closed basins, a similar approach is used with an additional constraint on the runoff volume. For a given frequency, the allowable post-developed runoff volume is the largest pre-developed runoff volume of any duration. When using the NRCS technique for computing runoff, the 10-day duration event will always produce the largest runoff volume and, therefore, be the critical duration. But, for other more-refined approaches to modeling infiltration, the critical duration could be something other than the 10-day duration.

(B) Storm for Storm Approach (Preferred)

This interpretation of critical duration and the multiple storm concept requires, for a given frequency, that the post-developed runoff rate for each duration be less than or equal to the pre-developed runoff rate of corresponding duration. For example, in the table below, the allowable runoff rate for each duration is the pre-developed runoff rate. The post-developed runoff rates shown are acceptable because they are all less than or equal to the pre-developed runoff rate of corresponding duration. The 4-hour duration is critical because it most closely matches the pre-developed runoff rate.

Duration	Pre-Dev Runoff XX Year Event	Acceptable Post-Dev Runoff XX Year Event
1-hour	65	60
2-hour	70	68
4-hour	66	66
8-hour	60	57
24-hour	30	26
3-day	25	23
7-day	24	22
10-day	21	20

This approach is consistent with the first sentence of the definition of critical duration. *“Critical Duration means the duration . . . that creates the . . . highest rate of net stormwater runoff (post-development runoff less pre-development runoff). . . .”* In the example above, when you subtract the pre-development runoff rate from the corresponding post-development runoff rate, all the “net stormwater runoff” values are negative except the 4-hour duration, which has zero “net stormwater runoff.” So, the 4-hour duration has the highest rate of net stormwater runoff; therefore, it is the critical duration. This approach is better than the peak discharge approach, where the release timing of the facility is critical. FHWA’s Hydraulic Engineering Circular No. 22 (HEC 22) contains a discussion of the concern for release timing.

The above discussion pertains to discharges to open basins only with historical flooding documented. For discharges to closed basins, a similar approach is used with an additional constraint on the runoff volume. For a given frequency, the post-developed runoff volumes for each duration cannot exceed the pre-developed runoff volumes of corresponding duration.

Although both the Peak Discharge Approach and the Storm for Storm Approach have been applied to FDOT projects in the past, the Department prefers that you use the Storm for Storm Approach on its projects. The examples in Section 9.4 are based on the Storm for Storm Approach.

9.4.1.2 Storm Frequencies

The previous sections primarily discuss durations and the multiple storm concept. Chapter 14-86 [14-86.003 (3)(c) 2 & 3] requires that we consider various rainfall event frequencies up to and including the 100-year event. The rule does not say that all frequencies must be evaluated.

The more frequent FDOT design storms (2-year to 50-year) do not usually control the size of the pond because the runoff from these storms is less than the runoff for the 100-year storm. The purpose of evaluating the less frequent storms is to ensure that the pre-developed discharges are not exceeded. And so it becomes a check of the operation of the outlet control structure under various rainfall event frequencies.

Where the discharge is controlled by a simple rectangular weir (one with a constant width), it may be reasonable to run only the 2-year, 25-year, and 100-year events. Where the discharge is controlled by a complex weir (width varies with elevation), an orifice, a pipe, tailwater conditions, or any combination of these, evaluate all frequencies (2-year, 5-year, 10-year, 25-year, 50-year, and 100-year). Some software programs can run all the frequencies at once. If these programs are available to you, run all the frequencies, regardless of the outlet control structure configuration.

9.4.2 Estimating Attenuation Volume

A first step in estimating attenuation volume is identifying outfalls and their associated drainage basins. At this stage, consider if it will be necessary to divert runoff from one basin to another. Although the Department does not encourage diverting runoff, doing so sometimes allows the Department to provide stormwater management (treatment and attenuation) in more economical locations. For example, an economical parcel for a pond site may be available in one drainage basin while the parcels in an adjacent basin are very expensive. Diverting some roadway runoff to the economical parcel basin from the expensive parcel basin may be more economical even when other costs, such as construction and maintenance, are considered. Before you propose diverting runoff, be sure it is acceptable to the regulatory agency.

When diverting runoff, be careful how you calculate the allowable discharge. Base your allowable (pre-developed) discharge calculations on the pre-developed drainage area that discharges to the proposed outfall. If an area does not drain to the proposed outfall in the pre-developed condition, do not include that area in the allowable (pre-developed) discharge calculations. Therefore, in a basin you divert runoff to, the pre-developed drainage area is smaller than the post-developed drainage area. Conversely, in a basin you divert runoff from, the pre-developed drainage area is larger than the post-developed drainage area.

The actual attenuation volume cannot be determined until you “route” the design storms and design the pond. There are several methods for estimating the attenuation volume. The methods more commonly used on the Department’s projects are discussed below.

9.4.2.1 Pre Versus Post Runoff Volume

A common technique for estimating attenuation volume is to calculate the difference in runoff volume between the post-developed conditions and the pre-developed conditions using the NRCS equation for runoff.

$$Q_R = \frac{(P - 0.2S)^2}{P + 0.8S}$$

As written, this assumes the initial abstraction (I_a) = 0.2S & S =(1000/CN) – 10

where:

Q_R = Runoff depth (in inches)

P = Rainfall depth (in inches); Use the 100-year, 24-hour depth for evaluating alternate drainage schemes or pond sites

S = Maximum retention or soil storage (in inches)

CN = Watershed curve number

The runoff volume is determined from: VOL = (Q_R) (Drainage Area)

A similar approach can be taken using the Rational Equation Method.

VOL = (C_{POST} – C_{PRE}) (P) (Drainage Area)

An advantage of this technique is that it does not involve any design storm distributions. So there is no concern for which storm duration is critical. On the other hand, this technique ignores the timing differences between the pre-developed and post-developed hydrographs. As a result, it may underestimate the attenuation volume.

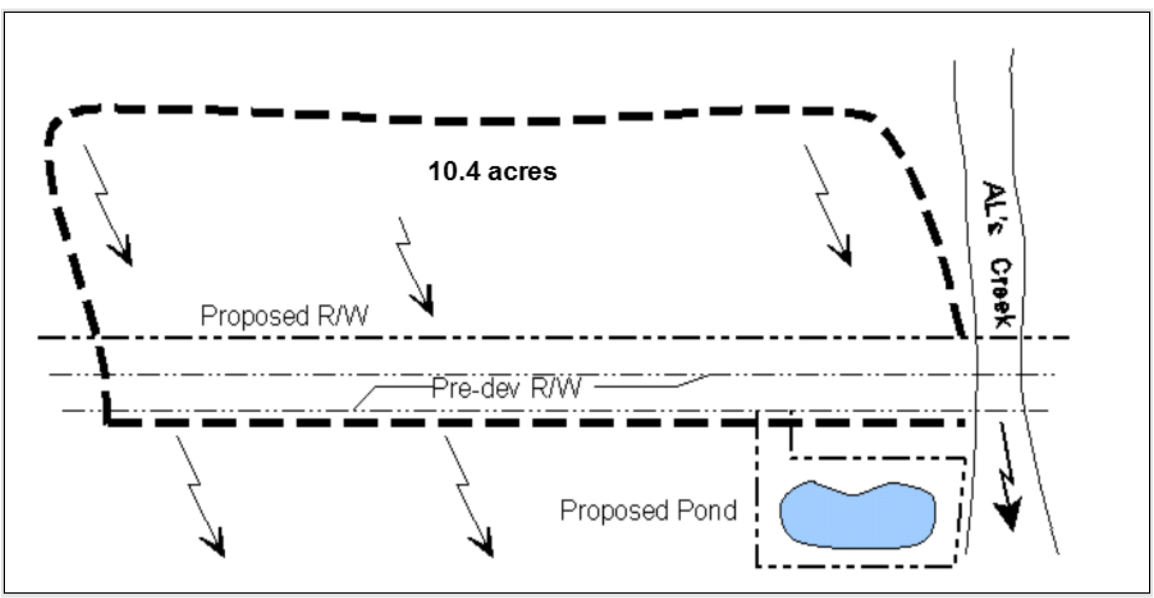
Example 9.4-1: Estimating attenuation volume using differences in runoff volume

Given:

- Pre-developed roadway pavement = 2 10-foot lanes
- Drainage area: includes roadway right of way & off-site drainage to the roadway = 10.4 ac

For preliminary pond sizing, use the information from the old drainage map unless you have reason not to.

- Offsite land use = Residential lots averaging 1/2 ac
- Proposed typical section = 5-lane urban section; Combined roadway, curb, and sidewalk width = 83 ft
- Proposed right-of-way width = 100 ft
- Length of roadway within drainage area = 1,706 ft
- Offsite runoff draining to the project will be taken through the pond, not bypassed around.
- Project located in Somewhere City, Florida, flat terrain <1 percent grade, Hydrologic Soil Group B/D, project drains to open basin.



Example 9.4-1

Find: The estimated attenuation volume.

1. Pre-developed area & curve number:

Roadway pavement:	= 0.79 ac @ CN = 98	(20 ft x 1,706 ft)
Pervious area	= 9.61 ac @ CN = 85	(10.4 ac – 0.79 ac)
Proposed pond area	= 0.77 ac @ CN = 85	
Total	= 11.2 ac @ CN = 85.9	

Assume the pond area is 20 percent of the roadway right of way (0.2 x 1,706 ft x 98 ft = 0.77 ac). For this example, the proposed pond is located outside the area draining to the roadway; thus, the pond must be added to the other areas.

For this example, the roadway right of way to be acquired is within the area draining to the roadway. For your project, the acquired right of way may be outside the area draining to the road, thereby requiring that the additional right of way be added to the other areas.

2. Post-developed area and curve number:

Roadway, curb, and sidewalk:	= 3.24 ac @ CN = 98	(82.7 ft x 1,706 ft)
Pervious area	= 7.17 ac @ CN = 85	(10.4 ac – 3.24 ac)
Pond area	= 0.77 ac @ CN = 98	
Total	= 11.2 ac @ CN = 89.7	

3. Calculate the difference in runoff volume between the pre-developed conditions and post-developed conditions for the 100-year, 24-hour storm using the NRCS equation for runoff.

Refer to the NOAA website link in Section 1.4 of the *Drainage Manual* to obtain location-specific precipitation data for the 100-year, 24-hour volume. For this example, the 100-year, 24-hour volume for Somewhere City, Florida, is 10.7 inches.

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad \text{where: } S = (1{,}000 / CN) - 10$$

	Pre	Post
Potential abstraction (S) =	1.64	1.15
Runoff depth (Q) in inches	8.95	9.44
Runoff volume (ac-ft) =	8.36	8.81

Volume difference = 0.45 ac-ft

The estimated attenuation volume is this volume difference of 0.45 ac-ft.

9.4.2.2 Simple Pond Model Procedure

Another technique for estimating attenuation volume is to route a design storm through a simple pond model. It works best with a routing program that allows a rating curve for the stage-discharge relationship and a stage-storage (not area) relationship for the pond configuration. The model should be set up as follows:

- Arbitrarily select pond bottom and top elevations.
- Use two points for the stage-discharge relationship:
  (1) Zero discharge @ pond bottom, and (2) Allowable discharge @ pond top
- Use two points for the stage-storage relationship:
  (1) Zero storage @ pond bottom, and (2) Estimated storage @ pond top

As with any routing, this is an iterative process. During each iteration, the estimated storage volume is changed to bring the routed peak stage close to the top of the pond. The storage volume that causes the peak pond stage to match the top of the pond is the estimated attenuation storage.

This approach is useful when the discharge rate is limited to something other than the pre-developed rate. It is complicated when working with the Department's multiple design storms. Which design storm do you route? The following suggestions will help to simplify working with the multiple design storms:

- Determine the pre-developed discharges for the 100-year, 1-hour design storm through the 100-year, 8-hour design storm. Use the smallest of these calculations as the allowable discharge rate. For the Storm for Storm Approach to critical duration, the post-developed discharge rate will be limited to all of the corresponding pre-developed rates, so using the rate for estimating purposes is reasonable. The basis for running only the 1-hour through the 8-hour design storm is that one of these design storms usually is critical to sizing ponds discharging to open basins.
- Route the post-developed conditions using a "nested" design storm such as the NRCS Type 2 Florida modified or the applicable WMD design storm. These distributions often are as severe as or more severe than the Department's distributions.

Example 9.4-2: Estimating attenuation volume using a simple pond model

Given:

- The same conditions as in Example 9.4-1
- Pre-developed time of concentration = 29 min.
- Post-developed time of concentration = 21 min.

Find: The estimated attenuation volume

1. Pre-developed runoff:

   Determine the pre-developed discharge rates for the 100-year FDOT 1-hour and 8-hour design storms. Using a typical program based on the NRCS unit hydrograph approach, you should obtain values similar to these when using a peak shape factor of 256. The rainfall volumes for Somewhere City, Florida, are tabulated in Step 1 of Example 9.4-3.

Pre-Developed Peak Runoff Rates (cfs)			
1-hour, 100-yr	2-hour, 100-yr	4-hour, 100-yr	8-hour, 100-yr
33.2	30.1	25.5	27.8

   The discharge associated with the 4-hour, 100-year design storm is the smallest and will be used as the allowable discharge.

2. Develop a simplified pond model as follows.

	Elevation	Discharge (cfs)	Storage
Pond Bottom	0	0	0
Top of Pond	10	25.5	Trial and Error

3. Route a nested design storm through the pond using post-developed conditions. For this example, we will route the 25-year, SFWMD 72-hour storm. Adjust the storage as necessary to have the routed peak stage match the top of pond. After numerous iterations, a storage value of 1.3 ac-ft was found acceptable, so:

   The estimated attenuation volume is 1.3 ac-ft.

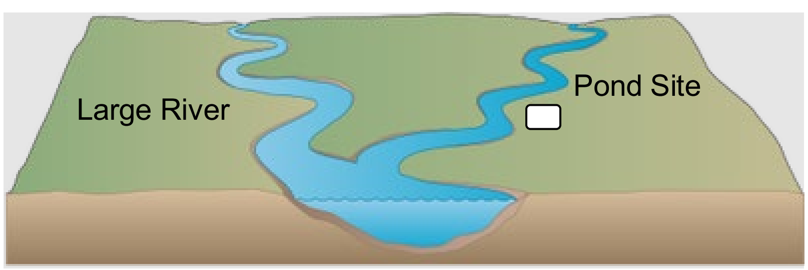
9.4.2.3 Other Techniques

FHWA's HEC 22 provides several methods to estimate attenuation volume, including examples and comparisons. Although most of these techniques are reasonably accurate, they—like the previous method—are complicated when working with the Department's multiple design storms.

9.4.3 Tailwater Conditions

Tailwater conditions can affect the design of the outfall structure, the size of the pond, and even the evaluation of alternate pond sites. The pond must meet the attenuation requirements during the tailwater conditions expected to occur coincident with the design storms. Predicting the tailwater condition sometimes can be difficult. Our facilities usually discharge to points associated with watersheds that are much larger than the drainage area of our facility. It may be appropriate to model the larger watershed and apply design storms to both the road project and the larger watershed simultaneously. This method will help to address any timing-related effects.

Tailwater conditions can become more challenging when discharging at or close to the confluence of two streams, as shown in the figure below. Depending on the relative size of each basin, it may be overly conservative to use the combined (or coincident) 100-year probability for each stream. National Cooperative Highway Research Program (NCHRP) conducted Project 15-36: *Estimating Joint Probabilities of Design Coincident Flows at Stream Confluences* (http://www.trb.org/Publications/Blurbs/169456.aspx) to develop practical procedures for estimating joint probabilities of coincident flows at stream confluences. This paper focuses on two practical design methods and provides a step-by-step application guide for designers in Appendix H of the document.



A simpler approach is to estimate the worst-case tailwater condition and see if it submerges the control point of the outlet control structure. If it does not, the tailwater condition can be ignored in the design of the weir/orifice of the outlet control structure.

Placing a pond in a 100-year riverine floodplain can complicate the design due to high tailwater conditions that may be coincident with the design storm. Other complications such as flood plain compensation and changes to floodway conveyances may exist as well. Chapter 5 (Bridge Hydraulics) addresses impacts to floodway conveyances.

9.4.4 Routing Calculations

Most engineers currently use computer programs to route hydrographs through stormwater facilities. The majority of computer programs use the storage indication method for this process. HEC 22 contains a discussion of the storage indication method, with an example. The *Drainage Connection Handbook* also discusses this method.

Although the computer reduces the effort, it does not eliminate the iterative process of modifying the pond and outlet control structure after each run. To design an acceptable pond and outlet control structure, you usually will run numerous iterations. You can adjust six items to meet the discharge requirements: (1) weir width (or orifice size), (2) weir crest (or orifice invert) elevation, (3) pond surface area, (4) pond depth, (5) pond length to width ratio, and (6) outlet pipe size. Although some of these items may be constrained by regulatory requirements, the following provides general guidance for making adjustments during the iterative process.

If the only change made is:	The results are:
Increasing weir width (or orifice size)	Increases discharge and lowers stage.
Lowering weir crest (or orifice invert)[1]	Increases discharge (volume more than rate) and lowers peak stage.
Increasing pond surface area (increases storage above and below weir crest)	Decreases discharge and lowers peak stage. For retention systems, increases infiltration and shortens recovery time.
Lowering pond depth[1] (increases storage below the weir only)	Decreases discharge and lowers peak stage. For retention systems, decreases infiltration and lengthens recovery time when saturated groundwater flow conditions exist.
Increasing length to width ratio	Increases discharge and raises peak stage, due to slight reduction in storage area for the same surface area. For retention systems, increases infiltration and shortens recovery time when saturated groundwater flow conditions exist.
Decreasing outlet pipe size	Increases discharge and raises peak stage due to additional friction losses in the pipe. Increases outlet velocity in discharge pipe.
1. Normally applicable to only retention systems.	

9.4.5 Discharges to Watersheds with Positive Outlet (Open Basins) Using Chapter 14-86

Using the Storm for Storm Approach, the Department's criterion for discharges to open basins requires that—for a given frequency—the post-developed discharge rate for each duration must be less than or equal to the pre-developed discharge rate of corresponding duration. Most of the regulatory agencies also have requirements for post-developed discharge rates. You must meet these requirements and the Department's criterion.

Example 9.4-3: Discharge to watershed with positive outlet (open basin)

This example uses information developed in Examples 9.1-1, 9.4-1, and 9.4-2.

Given:
The following information has been verified since the time of the pond site evaluation.

- SHWT elevation at pond site: = 56.1 ft — Agreed to by regulatory agency
- Lowest ground elev. around pond site = 59.1 ft — From design survey

Find:
The required pond configuration to meet the FDOT criterion. For this example, the pond also will be designed to meet SWFWMD and SFWMD criteria.

1. Determine the location-specific rainfall volumes using the NOAA website link in Section 1.4 of the *Drainage Manual*.

Rainfall Volumes: Somewhere City, Florida						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
1-hr	2.4	2.95	3.25	3.75	4.1	4.5
2-hr	2.8	3.5	3.9	4.5	5.0	5.5
4-hr	3.3	4.0	4.6	5.4	6.0	6.6
8-hr	3.8	4.9	5.6	6.5	7.3	8.0
1-day	4.8	6.3	7.7	8.7	9.7	10.7
3-day	6.1	7.9	9.1	10.8	12.2	14.1
7-day	7.5	9.4	11.5	13	14.8	16.8
10-day	8.5	11	13	15	17	19

First Round of Iterations

2. Determine the pre-developed runoff rates: This will establish the allowable discharge rates.

Time of concentration = 29 min (from Ex. 9.4-2)
Pre-developed CN:
Roadway pavement: = 0.79 ac @ CN = 98 (from Ex. 9.4-1)
Pervious area: = 9.61 ac @ CN = 85 (from Ex. 9.4-1)
Proposed pond area: = 0.94 ac @ CN = 85
Total: = 11.3 ac @ CN = 85.9

The proposed pond size is from Example 9.1-1, Pond Siting Stage. This is a reasonable assumption for the first iteration.

To simplify this problem, we have used the time of concentration, roadway pavement area, and offsite land use from prior examples. Actually, you should use the latest information from the design surveys and field reviews of the proposed project to establish the pre-developed conditions. Using a typical program, which uses the NRCS unit hydrograph approach, you should obtain values similar to these when using a peak shape factor of 256. This peak shape factor is used throughout this example.

> For the first round of iterations for a pond discharging to an open basin (with documented flooding history), it is usually sufficient to run the 100-year FDOT 1-hour to 8-hour duration design storms and the regulatory agency design storm.

Pre-Developed Runoff (cfs)					
DOT 1-hr, 100-yr	DOT 2-hr, 100-yr	DOT 4-hr, 100-yr	DOT 8-hr, 100-yr	FLT2M, 25-yr	SF72, 25-yr
33.6	30.4	25.8	28.1	30.6	36.3

3. Post-developed runoff:

In urban sections, the time of concentration is best determined from the storm sewer design tabulations. For this example, assume the storm sewer tabs have a Tc = 21 min.

Time of concentration: = 21 min
Post-developed area & CN:
Roadway, curb, and sidewalk: = 3.24 ac @ CN = 98 (from Ex. 9.4.1)
Pervious area: = 7.17 ac @ CN = 85 (from Ex. 9.4.1)

Pond:	= 0.94 ac @ CN = 98
Total:	= 11.3 ac @ CN = 89.7

4. Develop a stage-storage relation (pond configuration) for the first round of iterations.

   Dimensions at peak stage = 210.5 ft by 109 ft (from Ex. 9.1-1)

> For the first iteration, use the configuration estimated in the pond siting evaluation unless you have reasons not to.

   Peak stage = 58.1 ft to maintain freeboard between ground line of 59 ft.

Although some WMDs allow treatment below SHWT, this example assumes that treatment is above SHWT. Then, the pond length and width at SHWT elevation (for routing purposes, the SHWT elevation is considered pond bottom) are:

Bottom length = Top length – 2 [side slope (peak stage – $elev_{SHWT}$)]
= 210.5 m – 2 [5 (58.1 ft – 56.1 ft)] (1:5 side slopes)
= 191 ft

Similarly, Bottom width = 90 ft

Using these dimensions and side slopes, develop a stage-storage relationship. The values below were obtained using the equation for the volume of a frustum of a pyramid.

Stage (ft)	Storage (ac-ft)
56.10	0.00
56.50	0.16
56.90	0.34
57.30	0.52
57.70	0.72
58.10	0.92

5. Develop an outfall structure for the first round of iterations. Do so using the maximum allowable stage and discharge. For this example, the maximum allowable stage is the ground elevation minus the freeboard [59.1 ft – 1.0 ft = 58.1 feet]. The maximum allowable discharge is the largest pre-developed

discharge, which, for this example, is the SFWMD 72-hour, 25-year design storm (see Step 2).

Weir crest elevation = 56.7 ft — The treatment volume (10,950 ft^3, given in Ex. 9.1-1) stacks 0.59 ft high.

Weir width (L) $= Q / (C \times H^{1.5})$ — from $Q = C \times L \times H^{1.5}$

$= 36.3 \text{ cfs} / (3.1 \times 1.37^{1.5})$

The max head = 58.1 ft – 56.7 ft = 1.37 ft = 7.3 ft

For this example, we have assumed no tailwater effects. For your projects, you will need to consider the effects of the tailwater conditions on the outfall control structure.

During this round of iterations, ignore the effects of the water quality bleed down orifice and start the routings at the top of the treatment volume.

6. Route the selected design storms. Using a typical routing program, you should obtain values similar to the following.

Table 9.4-1

<table>
<tr><th></th><th>Design Storm</th><th>Discharge (cfs)</th><th>Peak Pond Stage (ft)</th></tr>
<tr><td rowspan="6">Pond configuration:
Pond dimensions at SHWT = 191 ft x 90 ft
SHWT elev. = 56.1 ft
Avg side slope = 1:5
Weir crest elev. = 56.7 ft
Weir width = 7.3 ft
Starting WS = 56.7 ft

Allowable Stage = 58.1ft</td><td>FDOT 1-hr, 100-yr
Pre
Post</td><td>
33.6
38.2</td><td>

58.1</td></tr>
<tr><td>FDOT 2-hr, 100-yr
Pre
Post</td><td>
30.4
35.0</td><td>

58.1</td></tr>
<tr><td>FDOT 4-hr, 100-yr
Pre
Post</td><td>
25.8
28.8</td><td>

57.9</td></tr>
<tr><td>FDOT 8-hr, 100-yr
Pre
Post</td><td>
28.1
30.6</td><td>

57.9</td></tr>
<tr><td>SCS-T2FLM, 25-yr
Pre
Post</td><td>
30.6
33.0</td><td>

58.0</td></tr>
<tr><td>SFWMD 72-hr, 25-yr
Pre
Post</td><td>
36.3
38.0</td><td>

58.1</td></tr>
</table>

From this table, it appears that the 100-year, 1-hour or 2-hour may be critical because they exceed the pre-developed discharge more than the others. Overall, the configuration used in the first iteration is close to meeting the requirements. Shorten the weir length to decrease the peak discharge. Doing so will cause the stage of the 1-hour, the 2-hour, and the SFWMD design storm to exceed the allowable stage, so the pond size needs to be increased also.

After making several runs, the stage-storage relationship shown below and a weir width of 6.0 ft is close to meeting the requirements of the design storms modeled. The second row in the table is the weir crest elevation sufficient to store the treatment volume.

Stage (ft)	Storage (ac-ft)
56.10	0.00
56.40	0.27
56.50	0.36
56.90	0.73
57.30	1.11
57.70	1.52
58.10	1.94

Using this configuration, you should obtain values as shown below.

Table 9.4-2

<table>
<tr><th></th><th>Design Storm</th><th>Discharge
cfs</th><th>Peak Pond
Stage
ft</th></tr>
<tr><td rowspan="6">Pond configuration:
Pond dimensions at SHWT = 288.7 ft x 131.2 ft
SHWT elev. = 56.1 ft
Average side slope = 1:5
Weir crest elev. = 56.4 ft
Weir width = 6.0 ft
Starting WS = 56.4 ft

Allowable Stage = 58.1</td><td>FDOT 1-hr, 100-yr
Pre
Post</td><td>
33.6
27.1</td><td>

57.7</td></tr>
<tr><td>FDOT 2-hr, 100-yr
Pre
Post</td><td>
30.4
27.3</td><td>

57.7</td></tr>
<tr><td>FDOT 4-hr, 100-yr
Pre
Post</td><td>
25.8
26.4</td><td>

57.7</td></tr>
<tr><td>FDOT 8-hr, 100-yr
Pre
Post</td><td>
28.1
27.4</td><td>

57.7</td></tr>
<tr><td>SCS-T2FLM, 25-yr
Pre
Post</td><td>
30.6
27.5</td><td>

57.7</td></tr>
<tr><td>SFWMD, 72-hr, 25-yr
Pre
Post</td><td>
36.3
30.7</td><td>

57.8</td></tr>
</table>

From this table, it appears that the FDOT 4-hour is critical since it is the only duration for which the post-developed discharge is not less than the pre-developed discharge. The SFWMD design storm creates the highest stage of the storms modeled.

Second Round of Iterations

7. Adjust the drainage basin characteristics due to the pond size being increased in the previous step. Remember that, for this example, the pond is outside the area draining to the pond so increasing the pond size also increases the total area. See Example 9.4-1. During the first iteration, we assumed the entire pond area had a CN = 98. A more refined estimate of the pond area curve number can be made at this time.

   Pond Area:

Water surf dims at peak stage	= 308 ft. x 150 ft.
Water surface area at peak stage	= 1.06 ac @ CN = 98
Total pond area (incl maint berms)	= 1.53 ac
Grassed area within total pond area	= 0.47 ac @ CN = 85

   Total Project Area and Curve Number:

   Pre-developed CN:

Roadway pavement:	= 0.79 ac @ CN = 98 (same as Step 2)
Pervious area:	= 9.61 ac @ CN = 85 (same as Step 2)
Proposed pond area:	= 1.53 ac @ CN = 85
Total:	= 11.9 ac @ CN = 85.9

   Post-developed CN:

Roadway, curb, and sidewalk:	= 3.24 ac @ CN = 98 (same as Step 3)
Pervious area:	= 7.64 ac @ CN = 85 [7.17 ac (Step 3) + 0.47 ac]
Pond:	= 1.06 ac @ CN = 100
Total	= 11.9 ac @ CN = 89.9

8. Calculate the pre-developed runoff and then route the design storms. For this example, we will add the FDOT 24-hour, 100-year design storm at this time. The results are shown in the following table.

Table 9.4-3

	Design Storm		Discharge (cfs)	Peak Pond Stage (ft)
Pond configuration: (Same as previous table) Pond Dimensions at SHWT = 288.7 ft x 131.2 ft SHWT El. =56.1ft Avg Side Slope = 1: 5 Weir Crest El. = 56.40 ft Weir Width = 6.0 ft Starting WS = 56.4 ft Allowable Stage = 58.1ft	FDOT 1-hr, 100-yr			
		Pre	35.2	
		Post	28.7	57.8
	FDOT 2-hr, 100-yr			
		Pre	31.9	
		Post	28.9	57.8
	FDOT 4-hr, 100-yr			
		Pre	27.0	
		Post	27.9	57.7
	FDOT 8-hr, 100-yr			
		Pre	29.5	
		Post	28.9	57.8
	FDOT 24-hr, 100-yr			
		Pre	11.2	
		Post	11.1	57.2
	SCS-T2FLM, 250-yr			
		Pre	32.1	
		Post	29.0	57.8
	SFWMD, 72-hr, 25-yr			
		Pre	38.1	
		Post	32.5	57.9

From this table, we can see the discharge for the 4-hour needs to be reduced and the stage of the SFWMD storm can still be increased, so the weir width can be reduced. After several iterations, a weir 4.5 ft wide works. The results are as follows.

Table 9.4-4

	Design Storm		Discharge (cfs)	Peak Pond Stage (ft)
Pond configuration: Pond dimensions at SHWT = 288.7 ft x 131.2 ft SHWT elev. = 56.1 ft Avg. side slope = 1:5 Weir crest elev. = 56.4 ft Weir width = 4.5 ft Starting WS = 56.4 ft Allowable stage = 58.1	FDOT 1-hr, 100-yr			
		Pre	35.2	
		Post	25.8	57.9
	FDOT 2-hr, 100-yr			
		Pre	31.9	
		Post	26.7	57.9
	FDOT 4-hr, 100-yr			
		Pre	27.0	
		Post	26.8	57.9
	FDOT 8-hr, 100-yr			
		Pre	29.5	
		Post	27.6	58.0
	FDOT 24-hr, 100-yr			
		Pre	11.2	
		Post	10.9	57.3
	SCS-T2FLM, 25-yr			
		Pre	32.1	
		Post	27.5	58.0
	SFWMD, 72-hr, 25-yr			
		Pre	38.1	
		Post	30.3	58.0

Since this configuration meets the requirements for these design storms, the pond size is probably adequate. We need to make sure that the discharges are not exceeded for the less frequent (2-year through 50-year) DOT design storms. We also will check the longer-duration storms, though it appears that the long duration storms (24-hour to 240-hour) will not control the size of the pond, since the stages and discharges of the 24-hour are much less than the 1-hour through 8-hour duration storms.

9. Check the size of the bleed down orifice. For this example, you will need a 1.5-inch diameter orifice or less to meet the typical wet detention criteria [discharge no more than half of the treatment volume in 60 hours and discharge the total treatment volume in no less than 120 hours]. At maximum pond stage, the discharge through this orifice is less than 0.1 cfs. This is insignificant for this problem. The orifice flow will be ignored and the routing calculations will be started at the weir crest, as done in previous iterations.

If the discharge through the bleed down orifice at peak stage is small, ignore it. If not, model the orifice in the routing. If the orifice is modeled, the starting water surface should reflect some amount of drawdown. The average inter-event period between storms is 72 hours. Most wet detention systems hold at least half of the treatment volume for 60 hours. Therefore, for most wet ponds, starting the water surface at an elevation associated with half of the treatment volume would be reasonable. If the regulatory requirements allow for a quicker drawdown, it may be reasonable to start the water surface at the bleed down orifice.

10. Run the other design storms. The other design storms were routed through the above pond configuration and all the post-developed rates were less than the pre-developed rates, except one. A summary of these is shown below.

Table 9.4-5 (Example 9.4-3)

Pond Config. as in Table 5.3-4	100-yr Discharge (cfs)	50-yr Discharge (cfs)	25-yr Discharge (cfs)	10-yr Discharge (cfs)	5-yr Discharge (cfs)	2-yr Discharge (cfs)
1-hr						
Pre	35.2	31.0	27.4	22.3	19.3	14.0
Post	25.8	22.3	19.4	15.3	13.0	9.1
2-hr						
Pre	31.9	28.0	24.2	19.8	16.9	11.9
Post	26.7	23.3	20.0	16.1	13.7	9.6
4-hr						
Pre	27.0	24.0	21.1	17.1	14.2	10.8
Post	26.8	23.7	20.7	16.8	13.8	10.5
8-hr						
Pre	29.5	26.5	23.0	19.0	16.0	11.3
Post	27.6	24.5	21.1	17.3	14.3	9.9
24-hr						
Pre	11.2	10.0	8.9	7.7	6.0	4.3
Post	10.9	9.7	8.6	7.4	5.8	4.1
3-day						
Pre	8.2	7.1	6.2	5.2	4.5	3.4
Post	8.2	7.1	6.2	5.2	4.4	3.3
7-day						
Pre	5.9	5.2	4.5	4.0	3.2	2.5
Post	5.9	5.2	4.5	4.0	3.2	2.6
10-day						
Pre	7.8	6.9	6.1	5.3	4.4	3.4
Post	7.8	6.9	6.1	5.3	4.4	3.4

The 7-day, 2-year post-developed discharge rate is greater than the pre-developed rate. If carried to three significant digits, the increase is 0.02 cfs (2.56-2.54). This is within the accuracy of these calculations and would be acceptable for most projects. If you or your project reviewers are concerned about an increase like this, you could modify the weir configuration slightly, as is done in Step 8 of Example 9.4-4.

11. Fine tune pond dimensions.

    The stage-storage values used in this example are based on length and width dimensions applied to a frustum of a pyramid. When you apply the radii to the corners, you would reduce the storage using the same pond dimensions, so use an equivalent stage-area relationship when working with the contours within the CADD file. Doing so also will allow you to configure the pond for aesthetic purposes while maintaining the necessary stage-storage relationship.

9.4.6 Discharges to Watersheds without Positive Outlet (Closed Basins) Using Chapter 14-86

Using the Storm for Storm Approach, the Department's criterion for projects discharging to a closed basin is that—for a given frequency—the post-developed discharge (rate and volume) for each duration must be less than or equal to the pre-developed discharge (rate and volume) of corresponding duration.

Ensure the retention volume is large enough that the post-developed discharge volumes do not exceed the pre-developed discharge volumes. The retention volume is the volume between the pond bottom and lowest discharge elevation of outlet control structure.

When using the NRCS runoff methodology, you can conservatively calculate the retention volume as the difference between the pre-developed and post-developed discharge volume for the 100-year, 10-day event. Some of this volume is infiltrated into the soil during the storm, so the actual retention volume is sometimes less than this. During long-duration design storms, such as the 3-day through 10-day durations, the volume infiltrated during the storm can be substantial. You can account for this by using a program that models the infiltration while routing the storm hydrograph. When you do this, you will not know the required retention volume until you have routed the storms and know how much volume infiltrates during the storm event.

The retention volume must recover at a rate such that half of the volume is available in seven days and the total volume is available in 30 days. When measuring the recovered volume, the pond is instantly (or over a very short time) filled with a runoff volume equal to the retention volume. Then, the water can infiltrate with no inflow to the pond.

9.4.6.1 Retention System Groundwater Flow Analysis

The approach described below is based on the current approach to modeling the recovery of the treatment volume in retention systems. You can apply the same techniques to the infiltration of retention systems discharging to closed basins.

The next several pages summarize the critical information contained in the following documents. We suggest that you read these documents before designing a retention system.

a) *Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers.* Prepared by Jammal and Associates, Inc., 1989 (Revised 1991), for the SWFWMD, Brooksville, Florida. See web link below:

http://publicfiles.dep.state.fl.us/dwrm/stormwater/stormwater_rule_development/docs/retpond_infil_analys.pdf

b) *Full-Scale Hydrologic Monitoring of Stormwater Retention Ponds and Recommended Hydro-Geotechnical Design Methodologies.* Prepared by PSI, Jammal and Associates Division, for the SJRWMD, August 1993, Special Publication SJ93-SP10. See web link below:

http://static.sjrwmd.com/sjrwmd/secure/technicalreports/SP/SJ93-SP10.pdf

During a storm event, runoff from the drainage basin enters the pond and infiltrates the pond bottom. At the beginning of a storm, the groundwater beneath the pond moves primarily vertically downward through unsaturated soil. If runoff to the pond exceeds the infiltration, the water depth in the pond increases as the wetting front continues to move down. Although the soil between the wetting front and the pond bottom is wet, it is not totally saturated due to entrapped air. After the wetting front reaches the water table, the vertical infiltration adds water to the water table aquifer. At this time, the groundwater moves primarily horizontally within the saturated aquifer while the water table begins to mound and saturate the soil beneath the pond. If infiltration continues, the mound rises to and above the pond bottom. When the mound reaches the pond bottom, the area beneath the bottom is fully saturated and flow moves primarily horizontally. See Figure 9.4-1.

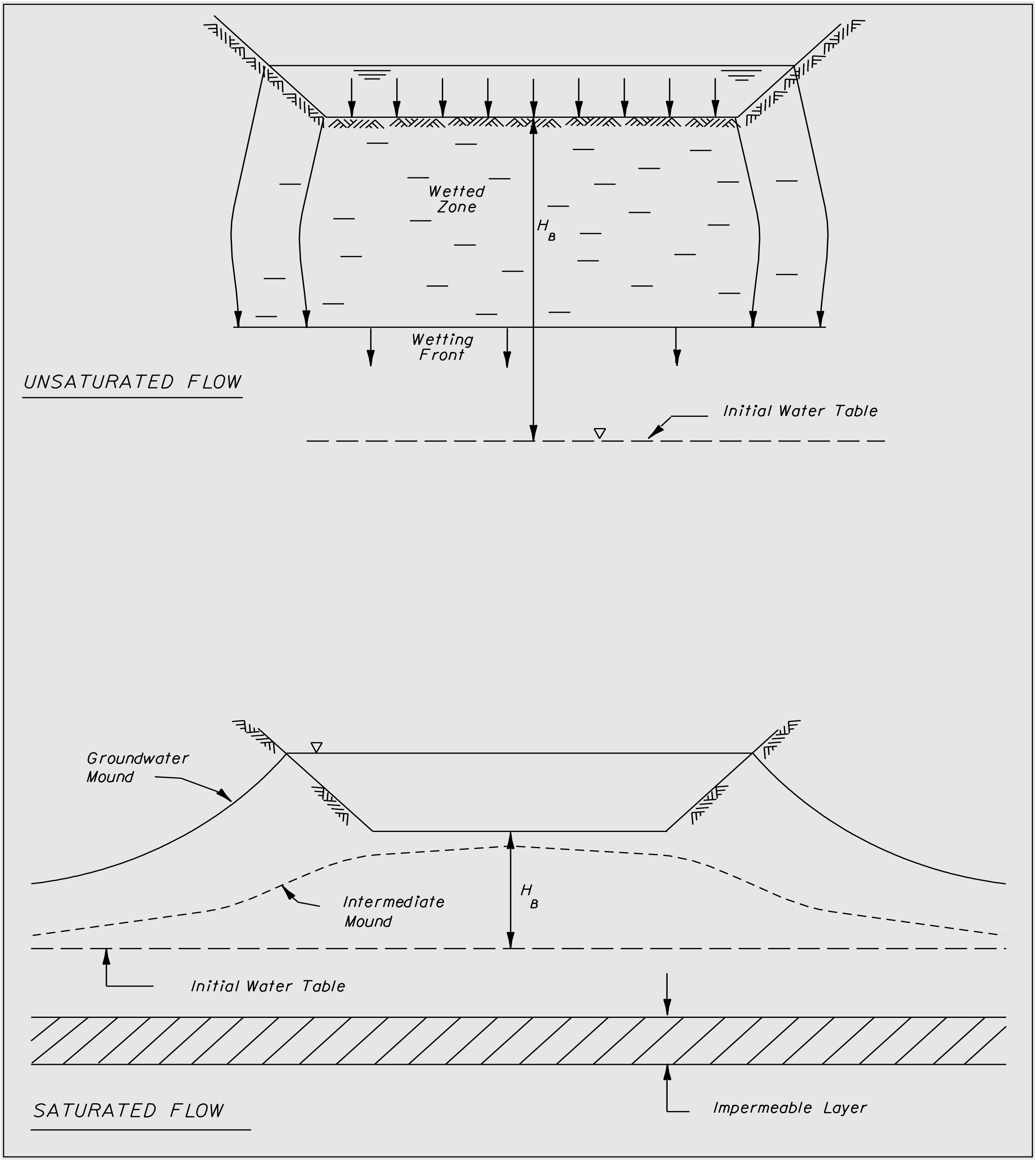


Figure 9.4-1: Groundwater Flow Characteristics during Infiltration

Determining the drawdown characteristics and the recovery time may involve modeling the downward vertical flow through unsaturated soil, or the horizontal saturated flow of the groundwater mound, or both.

(C) Unsaturated Flow

The design infiltration rate is: $I_D = \dfrac{K_{VU}}{FS}$

The time necessary to saturate the soil below the pond is: $T = \dfrac{f\, H_B}{I_D}$

The source for the above equations is the modified Green and Ampt infiltration equation. Their derivation is presented in *Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers* (Jammal and Associates, 1991).

The total volume of water required to saturate the voids in the soil below the pond bottom is: $VOL_{VOIDS} = (A_{PB})\,(H_B)\,(f)$

where:

H_B =	Height of pond bottom above groundwater (see Figure 9.4-1)
I_D =	Design infiltration rate
K_{VU} =	Unsaturated vertical hydraulic conductivity You can obtain this typically from a Double Ring Infiltration (DRI) test. Although infiltration is occurring during the test, the soil is not fully saturated due to entrapped air. The unsaturated K is less than the saturated K. Unsaturated K ranges from one-half to two-thirds saturated K (Bouwer 1978, ASTM D 5126, & Jammal and Assoc., 1991).
f =	Fillable porosity. See description in following pages.
A_{PB} =	Area of pond bottom
FS =	Factor of safety, usually 2.0.

You can use this factor of safety to account for the variability of the measurements and for the sediment that inevitably will enter the pond and clog the bottom surface.

(D) Saturated Flow

In most areas of the state, except the high sandy ridges, the groundwater mound likely will rise to the pond bottom, forcing the groundwater into a saturated horizontal flow. The most common approach to analyzing saturated flow conditions is to assume flow to be purely horizontal and uniformly distributed across the thickness of the receiving aquifer.

Model the aquifer as having a single homogeneous layer of uniform thickness and a horizontal initial water table.

Several computer models are available to analyze saturated flow. Most use a form of the USGS program MODFLOW. A simplified approach was developed by Jammal and is discussed in the SJRWMD's, *"Applicant's Handbook for Regulation of Stormwater Management Systems."* Regardless of which program or technique is used, four parameters are needed to model saturated flow: (1) the thickness of the aquifer, (2) the groundwater table elevation, (3) the fillable porosity, and (4) the horizontal saturated hydraulic conductivity of the aquifer.

- Thickness (or Elevation of the Base) of Mobilized Aquifer:

  This is the thickness of soil through which the horizontal flow will occur. You usually will measure this depth to the top of a confining or very dense layer, such as hardpan, that will restrict the downward vertical movement of groundwater. Use the lesser of the depth of the soil boring or the width of the pond as the maximum value in the analysis. (The maximum depth of the mobilized aquifer is about equal to the width of the pond. [Bouwer, 1978]).

- Groundwater Elevation:

  For modeling the recovery of the treatment volume, you usually will use the SHWT elevation. For modeling the infiltration of a pond discharging to a closed basin, this groundwater elevation should represent the groundwater elevation during an extreme event like the 100-year, 10-day design storm. Currently, there is no standard procedure for determining this elevation; nevertheless, it could be substantially higher than the SHWT. For example, where the pond is located near the low in the watershed (lake or flood plain at the low), it may be reasonable to use the 100-year lake or floodplain elevation as the extreme event groundwater elevation. Where the pond is located higher in the watershed, the extreme event groundwater elevation may be closer to the SHWT. Use your judgment and handle these situations on a case-by-case basis.

- Fillable Porosity:

  This is sometimes called specific yield, storage coefficient, effective storage coefficient, or effective porosity. It is the difference between volumetric water content of soil before and after wetting. The total porosity of a soil is the percentage of the total volume of the material occupied by pores or interstices. The fillable porosity is less than the total porosity because some water exists in soils above the water table; therefore, not all of the unsaturated void space is available for filling. In the zone immediately above the groundwater, capillary rise causes the voids to be substantially filled with water. In fine sands, the distance saturated due to capillary rise is roughly six inches. Therefore, the fillable porosity varies with the depth to the water table.

Specific field or laboratory testing usually is not required for determining the fillable porosity. For most calculations associated with fine sands, the fillable porosity will vary from 0.1 to 0.3 (10 percent to 30 percent). The SFWMD has produced soil storage curves that you can use to estimate the fillable porosity. For fine sand aquifers, the SJRWMD recommends using a fillable porosity in the range of 20 percent to 30 percent in infiltration calculations. The higher values of fillable porosity will apply to the well-to-excessively-drained, hydrologic group “A” fine sands, which generally are deep and contain less than 5 percent by weight passing the No. 200 sieve.

With all other dimensional and aquifer factors being the same, the predicted recovery time decreases as the assumed value of fillable porosity increases. Increasing the fillable porosity from 0.2 (20 percent) to 0.3 (30 percent) decreases the recovery time by 15 percent to 30 percent.

- Horizontal Saturated Hydraulic Conductivity of the Aquifer:

  Since you assume horizontal flow for the saturated analysis, the hydraulic conductivity should represent that direction. This should represent the weighted value of the soil above the confining layer. There are numerous techniques for measuring this value, and they are briefly described below. The Department recommends applying a safety factor of 1.5 to 2.0 to the measured values. You can apply this factor of safety to account for the variability in the elevation of the impermeable layer, measurement of the conductivity, and the estimate of fillable porosity.

  Cased hole tests:

  Generally, measure horizontal hydraulic conductivity if the casing bottom is below the water table during the test. Generally, measure vertical hydraulic conductivity if the casing bottom is above the water table during the test. Use the results with caution if the bottom of the casing is near an impermeable or confining layer.

  Uncased hole tests:

  This also applies to cased holes that use screen, perforated pipe, or rock bottom to maintain borehole shape. These generally measure horizontal hydraulic conductivity K.

Double Ring Infiltration (DRI) test:

Generally, the DRI measures the vertical unsaturated hydraulic conductivity. Although the Department does not encourage the use of the DRI to obtain the weighted horizontal saturated hydraulic conductivity, if it is the only test information you have, the saturated horizontal hydraulic conductivity could be estimated by applying two adjustment factors as follows:

KVS = 1.5 KVU

KHS = 1.5 KVS (conservative SWFWMD guideline)

or

KHS = 1.5 x 1.5 KVU

Pumping tests:

These tests generally are expensive and should be reserved for highly sensitive projects. They can overestimate hydraulic conductivity if the bore holes extend into a highly permeable layer which is below a confining layer and the proposed pond bottom is above the confining layer.

(E) Special Saturated Analysis

If you cannot model the aquifer as having the characteristics discussed above, you may need to use a more complicated, fully three-dimensional model with multiple layers, such as MODFLOW.

(F) Coordination with the Geotechnical Engineer

When requesting the soils investigation, provide the Geotechnical Engineer with the following information:

- Pond location
- Approximate pond shape (length, width, plan area configuration)
- Estimated pond bottom elevation
- Your estimate of SHWT
- The ideal functional characteristics of the pond, such as: “This pond will be designed to retain a volume of stormwater for flood control purposes. It should

infiltrate half the retention volume in no more than seven days and all of the volume in no more than 30 days."

- The anticipated groundwater flow conditions/analysis you expect to model based on your preliminary review of the soil and groundwater conditions.

The Geotechnical Engineer needs to know the information listed above because the soils investigation can vary depending on the groundwater flow condition anticipated during your design conditions. Refer to Table 9.4-6

Table 9.4-6: Typical Soil Investigations

Anticipated Groundwater Flow Conditions/Analysis	Soil Investigation[1]
Saturated	1) Thickness of mobilized aquifer. 2) Determine SHWT elevation. 3) Determine weighted saturated horizontal hydraulic conductivity of mobilized aquifer.
Unsaturated (Probably limited to high, sandy ridges)	1) Obtain unsaturated vertical hydraulic conductivity at or near pond bottom. 2) Determine SHWT or confirm that SHWT is at least as low as drainage engineer estimated. 3) Confirm that no confining layer exists between pond bottom and SHWT.
Karst Areas	See discussion in this section.
1 Preliminary results of the soil investigation may dictate that a different soil investigation is necessary. For example, you may have estimated sandy conditions down to a deep water table, planned on doing an unsaturated analysis, and requested appropriate soil information. Then the initial soil borings could indicate a confining layer close enough to the pond bottom to warrant a saturated analysis.	

If the groundwater elevation is within two feet of the pond bottom, you can assume that horizontal saturated flow will occur. If the groundwater is farther from the pond bottom, you should compare the volume of the voids under the pond to the volume of runoff that must be infiltrated.

For estimating the groundwater flow conditions, the volume to be infiltrated should be the treatment volume for retention systems discharging to open basins with known historical flooding. It should be the difference between the 100-year, 10-day runoff volume for ponds discharging to closed basins. If the volume to be infiltrated is larger than the volume of the voids under the pond, the groundwater mound will rise to the pond bottom, thus forcing saturated horizontal flow.

Karst Areas:

The WMDs and FDEP have identified known Karst areas and usually have special requirements for stormwater facilities in these areas to assure that water quality of the aquifer is maintained. Sink holes can present problems during or after construction, so it is important that you are aware of potential sink hole locations.

Some sink holes can be only a meter or two in diameter, thus making it difficult to identify their potential as a hazard. Sometimes potential sink holes can be identified in the field by localized depressions in the ground surface. You may find it useful to try ground penetrating radar in some situations, but this tool has a disadvantage in that it does not penetrate clay layers. Work closely with the Geotechnical Engineer to identify potential sink holes.

As a preventive measure, you could place a permeable geotextile strong enough to span a small opening several feet below the pond bottom. This would allow small sink holes to develop without requiring any maintenance work. Doing this will add substantial costs and may not be warranted for all facilities in Karst areas. You and the Geotechnical Engineer should make a joint decision to follow this approach.

Example 9.4-4: Discharge to watershed without positive outlet (closed basin)

Given:

- Pre-developed roadway pavement = 2 10-foot lanes
- Drainage area: includes roadway right of way and offsite draining to the roadway: =13.0 ac
- Offsite land use = Residential lots averaging 1/2 ac
- Proposed typical section = 4-lane urban section
- Combined roadway, curb, and sidewalk width = 73 ft
- Length of roadway within drainage area = 2,313 ft
- Treatment volume = 17,600 ft^3
- Maximum allowable pond stage = 104 ft
- Offsite runoff draining to the project will be taken through the pond, not bypassed around.

- Project located near Somewhere City, Florida; rolling terrain, approx. 2 percent grades, Hydrologic Soil Group B
- A confining or impermeable layer exists at approximately elevation 92 ft
- The saturated horizontal hydraulic conductivity was estimated to be 8 ft/day
- The SHWT was estimated at approximately elevation 93 ft

Find: Pond size and outlet control structure configuration.

1. Pre-developed runoff:

Time of concentration	= 21 min (given)
Area & curve number:	
Roadway pavement:	= 1.06 ac @ CN = 98 (20 ft x 2,313 ft)
Pervious area:	= 11.94 ac @ CN = 70 (13 ac – 1.06 ac)
Proposed pond area:	= 1.50 ac @ CN = 70 (preliminary size)
Total:	= 14.5 ac @ CN = 72.1

As in Example 9.4-1, the proposed pond is outside the area draining to the roadway; thus, the pond area must be added to the other areas.

Also, as in Example 9.4-1, the roadway right of way to be acquired is within the area draining to the roadway. For your project, the acquired right of way may be outside the area draining to the road, thereby requiring that the additional right of way be added to the other areas.

2. Post-developed runoff:

Time of concentration	= 16 min. (given)	
Area and curve number:		
Roadway, curb, and sidewalk:	= 3.88 ac @ CN = 98	(72.8 ft x 2,313 ft)
Pervious area:	= 9.12 ac @ CN = 70	(13 ac – 3.88 ac)
Pond:	= 1.50 ac @ CN = 98	
Total:	= 14.5 ac @ CN = 80.4	

3. Determine the location-specific rainfall volumes using the NOAA website link in Section 1.4 of the *Drainage Manual*.

Rainfall Volumes (inches): Somewhere City, FL						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
1-hr	2.2	2.7	3.0	3.5	3.8	4.1
2-hr	2.7	3.3	3.7	4.2	4.6	5.1
4-hr	3.1	3.9	4.4	5.0	5.6	6.1
8-hr	3.6	4.6	5.1	5.9	6.6	7.3
1-day	4.4	5.8	6.8	7.8	8.7	9.6
3-day	5.6	7.2	8.3	9.9	11	12.4
7-day	7.0	8.9	10	12	13.4	15
10-day	7.6	9.5	11.2	13.7	15.2	16

For this example, we will use peak shape factor = 323 for all NRCS hydrograph runs.

4. Assumptions:

a) Unsaturated vertical hydraulic conductivity: A DRI could not be performed because of the depth of the pond bottom. The unsaturated vertical hydraulic conductivity was estimated from the saturated horizontal conductivity (K_{HS} = 8 ft/day)

8 ft/day □ (1.5 – 1.5) = 3.6 ft/day (see discussion of DRI)

A factor of safety of 2 was applied to both values; thus, the modeled values are K_{HS} = 4 ft/day, and K_{VU} = 1.8 ft/day

b) Groundwater elevation: The extreme event groundwater elevation is assumed to be 3 feet above the SHWT. Then, extreme event groundwater elevation = 96.0 feet.

c) Fillable porosity is assumed = 0.1 (10 percent), worst case for fine sands

First Round of Iterations

5. Develop a starting-size pond.

You can take any approach to develop the starting trial size for the pond. Perhaps you found a preliminary estimate in the pond siting stage, or you can make an educated guess or a guess based on experience from a similar project. The following approach could be used:

- Assume the retention volume will be the difference in runoff volume for the 100-year, 10-day design storm. Using the approach of Example 9.4-1, the volume difference is 66,588 ft^3 for this example.

- Assume a height of the peak stage over the weir crest. For this example, we will use 1 foot. With a peak pond stage of 104 feet, this puts the weir crest at approximately 103 feet.

- Assume a pond bottom elevation, staying several feet above the estimated extreme event groundwater elevation. For this example, we will start 4 feet above the groundwater elevation with a pond bottom of 100 feet maintaining 4 feet between estimated peak groundwater and pond bottom.

- Determine a pond size and shape that will fit the retention volume between the pond bottom and the weir crest. For this example, a pond with a 200 foot x 100 foot bottom and 1:4 side slopes meets these constraints and will be used as a starting size.

6. Calculate the pre-developed discharge rates and volumes, and route the post-developed runoff through the pond. The weir width was arbitrarily selected for this iteration. Using a typical routing program that models infiltration during the storm, you should obtain values similar to the following.

For the first round of iterations for a pond discharging to a closed basin, it is usually sufficient to run the 100-year, FDOT 8-hour through 10-day duration storms.

Table 9.4-7

<table>
<tr><td rowspan="9">Pond Configuration:
Pond bottom dimensions = 200 ft x 100 ft
Pond bottom elevation = 100 ft
Avg side slope = 1: 4
Weir crest elevation = 102.9 ft
Weir width = 5 ft
Volume below weir crest = 68,585 ft^3
Allowable stage = 104 ft

Modeled Soil Conditions:
Aquifer base elevation = 92 ft
Saturated horizontal condition (K_{HS}) = 4 ft/day
Water table elevation = 96 ft
Fillable porosity = 0.1(10%)
Unsat Vert Cond. (K_{VU}) = 1.8 ft/day</td><td colspan="2">Design Storm</td><td>Disch. Volume (ft^3 x 10^3)</td><td>Disch. Rate (cfs)</td><td>Peak Pond Stage (ft)</td></tr>
<tr><td>FDOT 8-hr, 100-year</td><td>Pre
Post</td><td>216
182</td><td>29.1
27.3</td><td>104.2</td></tr>
<tr><td>FDOT 24-hr, 100-year</td><td>Pre
Post</td><td>323
289</td><td>10.4
10.9</td><td>103.7</td></tr>
<tr><td>FDOT 3-day, 100-year</td><td>Pre
Post</td><td>459
422</td><td>8.2
8.4</td><td>103.6</td></tr>
<tr><td>FDOT 7-day, 100-year</td><td>Pre
Post</td><td>589
543</td><td>6.1
6.2</td><td>103.5</td></tr>
<tr><td>FDOT 10-day, 100-year</td><td>Pre
Post</td><td>639
589</td><td>7.5
7.7</td><td>103.5</td></tr>
<tr><td colspan="5">Quantity Control Retention Volume</td></tr>
<tr><td colspan="3">Total recovered in 28 days</td><td>7-Day
(ft^3)</td><td>30-Day
(ft^3)</td></tr>
<tr><td colspan="3">Vol. Reqd. to be Recovered =
Vol. Recovered (infiltrated) =</td><td>33,300
30,600</td><td>66,590
53,400</td></tr>
</table>

All the post-developed discharge volumes are substantially less than the pre-developed discharge volumes of corresponding duration, so the pond retains more volume than needed. That is, the post-developed discharge volumes could be increased. This is done by lowering the weir. Although most of the post-developed discharge rates exceed the pre-developed rates, they are close to the pre-developed rates. To maintain similar post-developed rates, we will need to reduce the weir width as it is lowered. After making several iterations of weir adjustments, the following configuration produces the results in the following table.

For this example, we will add the 1-hour, 2-hour, and 4-hour duration storms.

Table 9.4-8

<table>
<tr><td rowspan="12"><u>Pond Configuration</u>:
Pond bottom dimension = 200 ft x 100 ft
Pond bottom elevation = 100 ft
Avg side slope = 1:4
Weir crest elevation = 101.5 ft
Weir width = 1.5 ft
Volume below weir crest = 32,768 ft^3
Allowable stage = 104 ft

<u>Modeled Soil Conditions:</u>
Aquifer base elevation = 92 ft
Saturated horizontal cond.(K_{HS}) = 4 ft/day
Water table elevation = 96 ft
Fillable porosity = 0.1(10%)
Unsat vertical cond. (K_{VU}) = 1.8 ft/day</td><td colspan="2">Design Storm</td><td>Disch. Volume (ft^3 x 10^3)</td><td>Disch. Rate (cfs)</td><td>Peak Pond Stage (ft)</td></tr>
<tr><td>FDOT 1-hr, 100-year</td><td>Pre
Post</td><td>83.3
64.3</td><td>33.8
16.8</td><td>103.5</td></tr>
<tr><td>FDOT 2-hr, 100-year</td><td>Pre
Post</td><td>122
109</td><td>31.1
19.9</td><td>103.7</td></tr>
<tr><td>FDOT 4-hr, 100-year</td><td>Pre
Post</td><td>164
155</td><td>25.4
22.6</td><td>103.9</td></tr>
<tr><td>FDOT 8-hr, 100-year</td><td>Pre
Post</td><td>217
212</td><td>29.1
24.0</td><td>104.0</td></tr>
<tr><td>FDOT 24-hr, 100-year</td><td>Pre
Post</td><td>323
322</td><td>10.5
10.2</td><td>103.0</td></tr>
<tr><td>FDOT 3-day, 100-year</td><td>Pre
Post</td><td>459
458</td><td>8.2
8.3</td><td>102.8</td></tr>
<tr><td>FDOT 7-day, 100-year</td><td>Pre
Post</td><td>588
581</td><td>6.1
6.2</td><td>102.6</td></tr>
<tr><td>FDOT 10-day, 100-year</td><td>Pre
Post</td><td>638
631</td><td>7.6
7.7</td><td>102.8</td></tr>
<tr><td colspan="5">Retention Volume</td></tr>
<tr><td colspan="2">Total recovered in 17 days</td><td colspan="2">7-Day (ft^3)</td><td>30-Day (ft^3)</td></tr>
<tr><td colspan="2">Vol. Reqd. to be Recovered =
Vol. Recovered (infiltrated) =</td><td colspan="2">16,390
24,200</td><td>32,770
32,770</td></tr>
</table>

This pond configuration meets the drawdown and discharge volume requirements. The rate requirements are close to being met as the 3-day through 10-day storms at only 0.1 cfs above the pre-developed discharge rates.

Second Round of Iterations

7. Adjust the drainage basin characteristics due to the pond size being smaller than estimated in Step 1. Remember that, for this example, the pond is located outside the area draining to the road, so changing the pond size also changes the total area. In Step 2, we assumed the entire pond area had a CN = 98. A more-refined estimate of the pond area curve number can be made at this time.

   Pond Area:

Water surface dimension at peak stage	= 232 ft x 132 ft
Water surface area at peak stage	= 0.70 ac
Total pond area (including maintenance berms & slopes)	= 1.1 ac
Grassed area within total pond area	= 0.40 ac

Total Project Area and CN:

Pre-developed area & curve number:
Roadway pavement: = 1.06 ac @ CN = 98 (from Step 1)
Pervious area: = 11.94 ac @ CN = 70 (from Step 1)
Proposed pond area: = 1.11 ac @ CN = 70
Total: = 14.1 ac @ CN = 72.1

Post-developed area and curve number:
Roadway, curb, and sidewalk: = 3.88 ac @ CN = 98 (from Step 2)
Pervious area: = 9.54 ac @ CN = 70 (9.12 ac (from Step 2) + 0.42 ac)
Pond: = 0.70 ac @ CN = 100
Total: = 14.1 ac @ CN = 79.2

8. Calculate the pre-developed discharge rates and volumes and route the post-developed runoff through the pond. Using the same pond/weir configuration as in the previous table produces the following results.

Table 9.4-9

<table>
<tr><td rowspan="12"><u>Pond Configuration</u>:
Pond bottom dimensions = 200 ft x 100 ft
Pond bottom elevation = 100 ft
Avg side slope = 1: 4
Weir crest elevation = 101.5 ft
Weir width = 1.5 ft
Volume below weir crest = 32,768 ft^3

Allowable stage = 104 ft

<u>Modeled Soil Conditions:</u>
Aquifer base elevation = 92 ft
Saturated horizontal cond.(K_{HS}) = 4 ft/day
Water table elevation = 96 ft
Fillable porosity = 0.1(10%)
Unsaturated vertical cond. (K_{VU}) = 0.8 ft/day</td><td colspan="2">Design Storm</td><td>Disch. Volume (ft^3 x 10^3)</td><td>Disch. Rate cfs</td><td>Peak Pond Stage ft</td></tr>
<tr><td>FDOT 1-hr, 100-year</td><td>Pre
Post</td><td>81.2
56.0</td><td>32.9
14.5</td><td>103.3</td></tr>
<tr><td>FDOT 2-hr, 100-year</td><td>Pre
Post</td><td>119
98.7</td><td>30.3
17.9</td><td>103.6</td></tr>
<tr><td>FDOT 4-hr, 100-year</td><td>Pre
Post</td><td>160
142</td><td>24.7
21.1</td><td>103.8</td></tr>
<tr><td>FDOT 8-hr, 100-year</td><td>Pre
Post</td><td>211
197</td><td>28.4
22.1</td><td>103.9</td></tr>
<tr><td>FDOT 24-hr, 100-year</td><td>Pre
Post</td><td>315
303</td><td>10.1
9.7</td><td>103.0</td></tr>
<tr><td>FDOT 3-day, 100-year</td><td>Pre
Post</td><td>447
435</td><td>7.9
8.0</td><td>102.8</td></tr>
<tr><td>FDOT 7-day, 100-year</td><td>Pre
Post</td><td>572
554</td><td>5.9
6.0</td><td>102.6</td></tr>
<tr><td>FDOT 10-day, 100-year</td><td>Pre
Post</td><td>621
602</td><td>7.3
7.4</td><td>102.8</td></tr>
<tr><td colspan="5">Retention Volume</td></tr>
<tr><td colspan="2">Total recovered in 17 days</td><td colspan="2">7-Day (ft^3)</td><td>30-Day (ft^3)</td></tr>
<tr><td colspan="2">Vol. Reqd. to be Recovered =
Vol. Recovered (infiltrated) =</td><td colspan="2">16,390
24,,200</td><td>32,770
32,770</td></tr>
</table>

This essentially meets all the requirements. The 24-hour and 3-day are critical durations for discharge volume. The 8-hour duration creates the highest stage. The 3-day through 7-day are critical durations for discharge rate and they exceed the pre-developed discharge rates by less than 2 percent. This may be acceptable. For this example, several more iterations could be made to bring these rates down without increasing the pond size.

Notice that the retention volume recovered in 7 days was more than necessary and the total volume was recovered in only 17 days. This indicates that we can lower the pond bottom. We can lower the weir crest the same amount that the pond bottom is lowered and maintain similar discharge volumes, which we need to do. As we lower the weir crest, we can reduce the weir width to reduce the discharge rate, which is the primary intent. So after several iterations, the following configuration using two weirs seems to do the trick. Notice it involves a compound weir.

Table 9.4-10

<table>
<tr><td rowspan="12"><u>Pond Configuration:</u>
Pond bottom dimension: = 192 ft x 92 ft

Pond bottom elevation = 99 ft
Avg side slope = 1:4
#1 weir crest elevation = 100.5 ft
#1 weir width = 0.5 ft
Volume below #1 weir crest = 29,120 ft^3
#2 weir crest elevation = 103.3 ft
#2 weir width = 12 ftAllowable stage = 104 ft

<u>Modeled Soil Conditions:</u>
Aquifer base elevation = 92 ft
Saturated horizontal cond. (K_{HS}) = 4 ft/day
Water table elevation = 96 ft
Fillable porosity = 0.1(10%)
Unsat vertical cond. (K_{VU}) = 1.8 ft/day</td><td colspan="2">Design Storm</td><td>Disch. Volume (ft^3 x 10^3)</td><td>Disch. Rate (cfs)</td><td>Peak Pond Stage (ft)</td></tr>
<tr><td>FDOT 1-hr, 100-year</td><td>Pre
Post</td><td>81.2
38.9</td><td>32.9
8.2</td><td>103.0</td></tr>
<tr><td>FDOT 2-hr, 100-year</td><td>Pre
Post</td><td>119
78.5</td><td>30.3
13.8</td><td>103.5</td></tr>
<tr><td>FDOT 4-hr, 100-year</td><td>Pre
Post</td><td>160
124</td><td>24.7
21.1</td><td>103.7</td></tr>
<tr><td>FDOT 8-hr, 100-year</td><td>Pre
Post</td><td>211
185</td><td>28.4
21.7</td><td>103.7</td></tr>
<tr><td>FDOT 24-hr, 100-year</td><td>Pre
Post</td><td>315
299</td><td>10.1
8.1</td><td>103.0</td></tr>
<tr><td>FDOT 3-day, 100-year</td><td>Pre
Post</td><td>447
436</td><td>7.9
7.4</td><td>102.9</td></tr>
<tr><td>FDOT 7-day, 100-year</td><td>Pre
Post</td><td>572
556</td><td>5.9
5.9</td><td>102.6</td></tr>
<tr><td>FDOT 10-day, 100-year</td><td>Pre
Post</td><td>621
610</td><td>7.3
7.3</td><td>102.9</td></tr>
<tr><td colspan="5">Quantity Control Retention Volume</td></tr>
<tr><td colspan="2">Total recovered in 28 days</td><td colspan="2">7-Day (ft^3)</td><td>30-Day (ft^3)</td></tr>
<tr><td colspan="2">Vol. Reqd. to be Recovered =
Vol. Recovered (infiltrated) =</td><td colspan="2">14,560
17,590</td><td>29,120
29,120</td></tr>
</table>

This configuration meets all the requirements for the storms modeled. The 7-day and 10-day durations are critical for discharge rate. The 3-day and 10-day durations are critical for discharge volume, and the 4-hour and 8-hour durations create the highest stage. The total retention volume is recovered in 28 days, just under the 30-day requirement. Although it appears that the pond size could be reduced slightly, remember that the earthwork tolerance will slightly affect characteristics of this pond. A slightly lower pond bottom will reduce the aquifer thickness, thus reducing the recovery time. A slightly higher pond bottom will reduce the retention volume and increase the discharge. So, when considering the construction tolerance, this configuration looks good.

9. Run the other design storms.

   The other storm frequencies should be calculated to check that the pre-developed discharges are not exceeded. The results are in Table 9.4-1.

10. The stage-storage values used in this example have been based on length and width dimensions applied to a frustum of a pyramid. When you apply the radii to the corners, you would reduce the storage using the same pond dimensions, so use an equivalent stage-area relationship when working with the contours within a CADD file. Doing so will allow you to configure the pond for aesthetic purposes while maintaining the necessary stage-storage relationship.

Table 9.4-11: Example 9.4-4, Closed Basin

Same Config. as in Table 9.4-9	100-year		50-year		25-year	
	Disch. Vol. (cfs x $10^{3)}$	Disch. Rate (cfs)	Disch. Vol. (cfs) x 10^3	Disch. Rate (cfs)	Disch. Vol. (cfs) x 10^3	Disch. Rate (cfs)
1-hour						
Pre	81.2	32.9	70.6	28.7	60.4	24.7
Post	38.9	8.2	31.5	6.7	24.5	5.2
2-hour						
Pre	119	30.3	100	25.2	84.8	21.3
Post	78.5	13.8	60.7	9.1	48.3	7.3
4-hour						
Pre	160	24.7	139	21.7	115	18.1
Post	124	21.1	103	16.4	78.9	9.9
8-hour						
Pre	211	28.4	181	24.4	151	20.4
Post	185	21.7	155	14.9	125	9.8
24-hour						
Pre	315	10.1	273	8.8	233	7.5
Post	299	8.1	258	7.0	217	5.8
3-day						
Pre	447	7.9	380	6.9	329	6.1
Post	436	7.4	369	6.4	317	5.6
7-day						
Pre	572	5.9	495	5.2	427	
Post	556	5.9	478	5.2	411	4.64.6
10-day						
Pre	621	7.3	582	6.9	509	6.2
Post	610	7.3	570	6.9	497	6.2
	10-year		5-year		2-year	
1-hour						
Pre	44.6	18.3	35.8	14.8	22.8	9.4
Post	14.1	3.0	8.9	1.9	2.4	0.6
2-hour						
Pre	67.2	16.6	53.9	13.1	35.8	8.4
Post	33.9	5.2	23.5	3.7	10.2	1.7
4-hour						
Pre	92.1	14.6	74.1	11.9	47.6	7.7
Post	58.3	7.3	42.3	5.4	19.8	2.6
8-hour						
Pre	119	16.1	100	13.4	63.8	8.5
Post	93.0	7.1	73.9	5.5	38.6	2.7
24-hour						
Pre	189	6.1	147	4.7	92.2	2.9
Post	173	4.6	130	3.5	72.9	2.0
3-day						
Pre	255	4.9	207	4.1	139	2.9
Post	242	4.5	193	3.7	123	2.6
7-day						
Pre	333	3.8	283	3.3	198	2.4
Post	316	3.8	265	3.3	177	2.4
10-day						
Pre	389	4.9	310	4.0	224	3.0
Post	376	4.9	296	4.0	208	3.0

9.4.7 Off-Site Inflows

In 2013, House Bill 599 (2012), enacted as Chapter 2012 174, Laws of Florida, amended Chapter 373, F.S. to create provision Section 373.413(6). This provision states that "FDOT is responsible for treating stormwater generated from state transportation projects but is not responsible for the abatement of pollutants and flows entering its stormwater management systems from off-site sources; however, this subsection does not prohibit the Department of Transportation from receiving and managing such pollutants and flows when cost effective and prudent. Further, in association with right-of-way acquisition for state transportation projects, the Department of Transportation is responsible for providing stormwater treatment and attenuation for the acquired right-of-way but is not responsible for modifying permits for adjacent lands affected by right-of-way acquisition when it is not the permittee."

FDOT generally has four options when dealing with offsite flows that would be intercepted by a linear transportation project:

1) Bypass offsite flows around the project's treatment system
2) Accept offsite flows and direct them to a treatment system that is designed to treat the transportation project and the offsite flow
3) Accept offsite flows and direct them to a treatment system that is designed to treat only the project
4) Accept offsite flows and direct them to a treatment system that is designed to treat the project and partially treat the off-site property

Empirical nutrient loading model results (Harper Methodology) show that—in all cases involving wet detention treatment, even when the treatment facility is designed for only the project area—there is an overall environmental benefit achieved by commingling (i.e., the net pollutant reduction is greater).

The same modeling shows that—for retention-type treatment systems, when the offsite lands provide equal or greater nutrient loading when compared to the FDOT project—there is also an overall environmental benefit achieved by commingling, even when the treatment facility is designed for only the project area. Thus, in these cases, the water quality at downstream points of discharge from the commingled system will be equal to or better than those systems that bypass offsite flows. Based on these results, FDEP and the WMDs support allowing commingling in these cases without requiring further analysis as long as the proposed treatment pond meets the ERP design requirements for the runoff from the project area and results in an overall environmental benefit.

The same empirical nutrient loading model results (Harper Methodology) show that—where undeveloped or unimproved offsite lands flow into onsite FDOT dry retention ponds—the water quality at downstream points of discharge from the commingled system may, in some cases, be worse than those systems that bypass offsite flows. As such, these designs should be evaluated on a case-by-case basis to ensure that environmental protection is not diminished.

In summary:

- For wet detention:
    - ✓ Commingle offsite inflows unless cost or hydraulic issues lead to bypassing
- For dry retention:
    - ✓ Commingle developed offsite inflows unless cost or hydraulic issues lead to bypassing
    - ✓ For inflows from lower EMC areas, consult the District Drainage Engineer
        - Calculate change in nutrient removal
        - If reduction in treatment, evaluate B/C

9.4.8 Commingling of Untreated Onsite Runoff

When you are adding new lanes to an existing roadway that has no formal water quality treatment, if you leave the drainage system for the existing roadway untouched, water quality treatment does not need to be provided for the existing unchanged lanes. Regardless, as a matter of good environmental stewardship, attempt, if economically prudent, to bring the runoff from the existing roadway into the treatment system for the new lanes. Just as in the section above for offsite inflows, commingling of existing onsite runoff will always result in improved downstream water quality, even if the stormwater management system is sized only for the new lanes. If economically prudent, consider increasing the pond sizes to treat the old system, even though not required.

9.5 OUTLET CONTROL STRUCTURES

9.5.1 Weirs

The most common form of flow control is a weir notched into the side of a concrete structure. To maximize the predictability of the flow, the weir should be smaller than the distance between the inside edges of the walls. This smaller size will allow air to get under the nappe. Using a weir size equal to the inside edges of the walls would create an unstable condition when the flow is attempting to spring free from the leading edge of the weir.

Sometimes outlet control structures contain multiple (or staged) weirs, such as a small weir at a low elevation with a larger weir at a higher elevation. These compound weirs can be handled in one of two ways. SWFWMD recommends treating the lower slot as an orifice, with head (H) measured to the centroid when the opening is submerged. Then you can model the upper portion with standard weir formulas and the two flows are added. Alternatively, you can extend the lower slot computations to the water surface. Then you model the flows from the sides of the upper slot as a separate weir and add the flows. In either case, a totally smooth transition in the performance curve at the stage of the upper weir crest cannot be expected. Some amount of manipulation of the curve should be made to smooth it at the transition.

9.5.2 Discharge Coefficients

The following coefficients are recommended for the typical concrete box outlet control structure. You will find these values documented in a report titled “Performance and Design Standards for Control Weirs, An Investigation of Discharge Through Slotted Weirs,” based on a study by the University of South Florida, March 1993; WPI nos. 0510610, & 0510522. Contact the FDOT Research Center at (850) 414-4615 to obtain a copy.

The first two tables apply to control devices formed into the wall of the outlet control structure. As a result, the thickness of the structure wall will affect the discharge coefficient. The discharge coefficient first rises with increasing head and then remains constant. This behavior is observed for both orifices and weirs and is caused by attachment of the flow at the sides of the opening. The wall thickness of the typical FDOT structure can vary depending on whether the structure is precast or “cast in place.” Unless you specify “cast in place,” assume that the structure will be precast. The Roadway and Traffic Design Standards specify the wall thickness.

Table 9.5-1: Orifice Discharge Coefficients

ORIFICE	Discharge Coefficient, C_D	
Condition of upstream edge	H/b<0.6	H/b>0.7
Concrete edge[1]	0.276 (H/b) + 0.491	0.709
90° elbow fitting	0.620 (H/b) + 0.284	0.645
[1] These values account for edge imperfections, chipping, wear, and some amount of bevel.		
C_D is dimensionless, to be used with the equation: $Q = C_D A_O (2gH)^{1/2}$ A_O = area of opening H = distance of water surface above orifice center b = thickness of the structure wall		

Table 9.5-2: Weir Discharge Coefficients

RECTANGULAR WEIR	Weir Coefficient, C_W			
Condition of upstream edge	0.25<H/b<2.0[1]		H/b>2.0[1]	
Concrete edge[2]		0.468(H/b) +2.45		3.45
[1] A typographical error exists in the original report, which shows this value to be 2.5 instead of 2.0. [2] These values account for edge imperfections, chipping, wear, and some amount of bevel.				
C_W is dimensional and calculated from $C_W = (2g)^{½} C_D$ C_W is to be used in the equation: $Q = C_W L H^{1.5}$ L = width of weir H = distance of water surface above the weir crest b = thickness of the structure wall				

Thin plate weirs fabricated from metal and bolted over a larger opening in the wall provide a more-uniform, predictable performance. Install the metal weir plate over an opening of sufficient size to ensure that the flow passing over the weir encounters no interference from the headwall. The plate's thickness should be 0.25 inch or less to approximate a sharp edge. If you construct it as discussed here, the weir coefficient is as follows and is independent of height.

	Metric	US Customary
Weir Coefficient C_W for Thin Plates	1.73	3.13

9.5.2.1 Submerged Control Devices

For weirs, use the Villemonte relationship to compute the ratio of flow under submerged conditions to flow under free discharge.

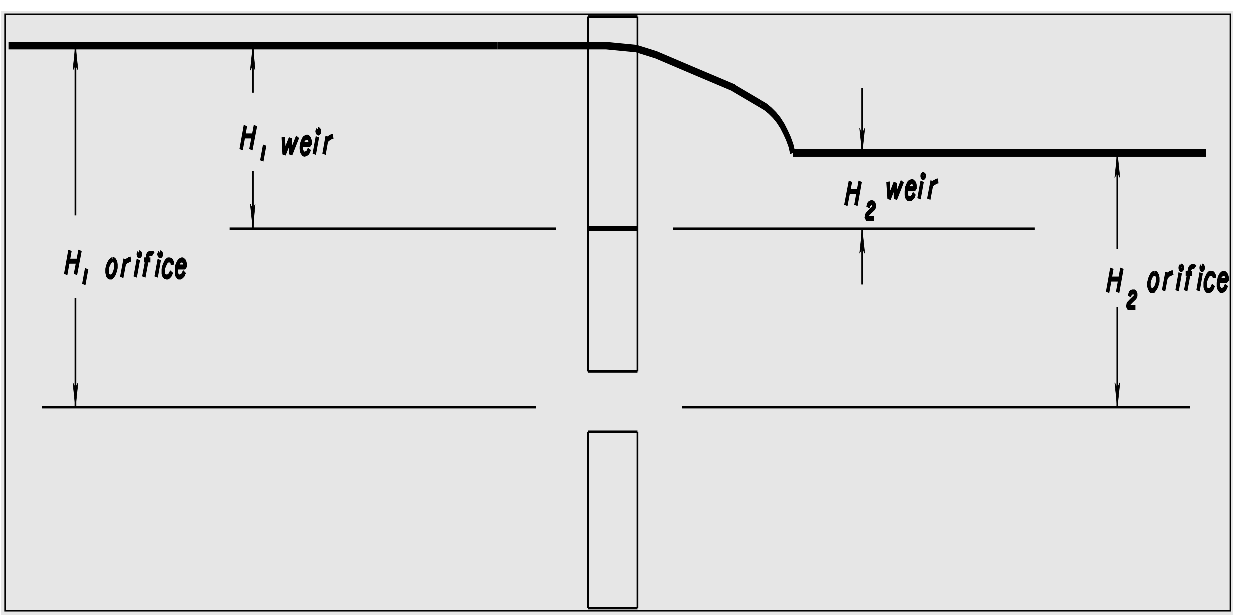
$$\frac{Q_S}{Q_F} = (1 - S^n)^{0.385}$$

where:

Q_S = Flow under submerged conditions
Q_F = Flow under free discharge
S = H_2/H_1 = Submergence ratio
H_1 = Upstream headwater
H_2 = Downstream headwater
n = 1.5 for rectangular weirs, & 2.5 for triangular weirs

Use the following similar relationship for orifices.

$$\frac{Q_S}{Q_F} = (1 - S)^{0.5}$$



9.5.3 Skimmers

Regulatory agencies commonly require skimmers to prevent oil and grease from leaving the pond. The head loss due to skimmers is minimized if the flow area under the skimmer is three times larger than the flow area of the weir. If this area is provided, you need not calculate the head loss associated with the skimmer.

If it is impossible to provide the flow area mentioned above, the head loss across the skimmer can be calculated using this formula:

$$H_L = k\, V^2/2g$$

where:
k = Loss coefficient
V = Velocity under the skimmer

A loss coefficient, k, of 0.2 is recommended based on a May 25, 1988, SWFWMD Technical Memorandum by R.E. Benson Jr., P.E., Ph.D.

9.5.4 Miscellaneous

To minimize plant growth, construct a concrete apron around the outlet control structure. You should extend it five feet from the structure.

In wet detention facilities, the outlet control structure generally includes a drawdown device, such as an orifice or a v-notch weir, to establish the normal water level and to slowly release the treatment volume. If the drawdown device is smaller than three inches wide or less than 20 degrees for v-notches, include a device to eliminate clogging. Examples of such devices include baffles, grates, screens, and pipe elbows.

It is not necessary to use the ditch bottom inlet type grates on outlet control structures unless needed for safety. If the structure is accessible to the public or maintenance vehicles will traverse it, grates are recommended.

Always consider the effects of storms that are more severe than what was designed for. Sometimes an overflow spillway can be built into the berm. Or additional flow can sometimes pass through the top of the outlet control structure while using the freeboard to store more volume and create additional head.