ENVIRONMENTAL RESOURCE PERMIT APPLICANT'S HANDBOOK VOLUME II (Design and Performance Standards Including Basin Design and Criteria)

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION AND NORTHWEST FLORIDA WATER MANAGEMENT DISTRICT

All Appendices are Incorporated by Reference in subsection 62-330.010(4), F.A.C.

Effective June 28, 2024

FOR USE WITHIN THE GEOGRAPHIC LIMITS OF THE NORTHWEST FLORIDA WATER MANAGEMENT DISTRICT

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION

NORTHWEST FLORIDA WATER MANAGEMENT DISTRICT

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PART I — INTRODUCTION, ORGANIZATION, APPLICABILITY

1.0 Introduction

This is **Volume II** of a two-volume "**Environmental Resource Permit Applicant's Handbook**. It accompanies **Applicant's Handbook—Volume I** (General and Environmental). The Handbook Volumes have been developed to assist persons in understanding and applying the rules, procedures, standards, and criteria implementing the environmental resource permit (ERP) program under Part IV of Chapter 373 of the Florida Statutes (F.S.). The ERP program regulates all types of projects, including stormwater management systems, dams, impoundments, reservoirs, appurtenant work, or works, and dredging or filling, as those terms are defined in Sections 373.403(13) and (14), F.S., or any combination thereof. These terms are defined in Sections 373.019 and 373.403, F.S., and in **Section 2.0 of Handbook Volume I**.

1.1 Applicability

Volume I (General and Environmental) applies statewide to the Department of Environmental Protection ("Department" or "DEP") and all the water management districts ("WMDs" or "Districts").

This **Volume II** is applicable only within the geographic boundaries of the Northwest Florida Water Management District (NWFWD). It is incorporated by reference in paragraph 62-330.010(4)(b)1., F.A.C., and therefore operates as a rule of DEP and NWFWMD in accordance with Section 373.4131, F.S. Separate **Volume IIs** have been adopted by each of the other WMDs for use within the geographical boundaries of each District. Each of those other **Volume IIs** also are incorporated by reference in paragraph 62-330.010(4)(b)2. through 5., F.A.C., and therefore operate as rules of DEP and each applicable District within the geographical area of that District for activities regulated under Chapter 62-330, F.A.C.

Volume II is applicable only to those ERP activities that involve the design of a stormwater management system that requires a permit under Chapter 62-330, F.A.C. More specifically, it provides specific, detailed design and performance methodologies designed to meet the water quality and quantity requirements of stormwater management systems. It also will assist persons who are designing activities to comply with the general permit in Section 403.814(12), F.S.

This Volume also contains District-specific appendices for regionally-specific criteria applicable to such things as Sensitive Karst areas. There is a separate document titled "[References and Design Aids]" that contains example calculations and design aids for stormwater systems within the Florida Panhandle. The Design Aids Document is for reference and to provide examples, but it is not adopted by rule.

A stormwater management system is defined in Sections 373.403(10) and 403.031(16), F.S., and in Section 2.0 of Handbook **Volume I,** as a system that is designed and constructed or implemented to control discharges which are necessitated by rainfall events, incorporating methods to collect, convey, store, absorb, inhibit, treat, use, or harvested water to prevent or reduce flooding, overdrainage, environmental degradation, and water pollution or otherwise affect the quantity and quality of discharges from the system.

Volume II generally is not applicable to projects that generate no more than an incidental amount of stormwater runoff, such as:

- Dredging and filling to construct such things as most "stand-alone" seawalls, docks and "in water" types of activities, such as channel dredging. This would not include dredging and filling in wetlands or other surface waters to construct such things as bridges or culverted road crossings, parking areas, building sites, or land fill which may or may not contain structures;

- Pervious (e.g., slatted decking) piers that do not convey vehicular traffic.
- An overwater pier, dock, or a similar structure located in a deepwater port subject to subsection 373.406(12), F.S. This would not include activities landside of a wharf bulkhead at a port facility;
- Construction of an individual, single family residence, duplex, triplex, or quadruplex dwelling unit that is not part of a larger plan of development;
- "Stand-alone" dredging, including maintenance dredging; or
- Activities that do not add new impervious surfaces, such as the installation of overland and buried electric and communication transmission and distribution lines.

Only **Volume I** would apply to most of the above projects because, unless specifically exempt, the above projects are still subject to regulation under Chapter 62-330, F.A.C.

In cases where conflicting or ambiguous interpretations of the information in this Volume result in uncertainty, the final determination of appropriate procedures to be followed will be made using Chapters 120 and 373, F.S., applicable rule chapters, and best professional judgment of Agency staff. The term "Agency", where used in this Volume, shall apply to DEP, the District, or a delegated local government, as applicable, in accordance with division of responsibilities specified in the Operating Agreements incorporated by reference in Chapter 62-113, F.A.C., except where a specific Agency is otherwise identified.

1.2 District-Specific Thresholds

There are no permitting thresholds under Chapter 62-330, F.A.C., that are specific to the NWFWMD.

1.3 District-Specific Exemptions

A permit under Chapter 62-330, F.A.C., is not required for those agricultural and silvicultural activities within the Northwest Florida Water Management District that are regulated under Chapter 40A-44, F.A.C.

There are no other exemptions specific to the NWFWMD geographical area, except those established under Section 373.4145(6), F.S. All applicable exemptions are in Rules 62-330.051 and .0511, F.A.C.

PART II — GENERAL CRITERIA

2.0 General Criteria for all Stormwater Management Systems

2.0.1 General Criteria

All stormwater management systems must be designed, constructed, operated, and maintained in accordance with the stormwater quality criteria of **Part II, Part IV, and Part V ~~IV~~ of ~~this~~ Volume I as well as Part IV and Part V of Volume II**. In addition, systems that exceed the thresholds of Section 3.1 of Part III of this Volume, whether a stand-alone system or a system that is part of a larger common plan of development or ownership, must also be designed, constructed or altered, operated and maintained to comply with the stormwater quantity/flood control criteria.

Activities that require a stormwater management system under this Volume shall additionally meet all the general design and performance criteria requirements of **Part II of this Volume**.

2.0.2 Criteria for Evaluation – Reasonable Assurance

An applicant for an individual permit must provide reasonable assurance that a stormwater management system, dam, impoundment, reservoir, works, or appurtenant work will meet the criteria in Rules 62-330.301 and .302, F.A.C. This includes a determination that the activity:

(a) Will not cause adverse water quantity impacts to receiving waters and adjacent lands;

(b) Will not cause adverse flooding to on-site or off-site property;

(c) Will not cause adverse impacts to existing surface water storage and conveyance capabilities;

(d) Will not cause or contribute to a violation of the water quality standards set forth in Chapters 62-4, 62-302, 62-520, 62-522, and 62-550, F.A.C., including the provisions of Rules 62-4.243, 62-4.244, and 62-4.246, F.A.C., the antidegradation provisions of paragraphs 62-4.242(1)(a) and (b), F.A.C., subsections 62-4.242(2) and (3), F.A.C., and Rule 62-302.300, F.A.C., and any special standards for Outstanding Florida Waters (OFWs) and Outstanding National Resource Waters (ONRWs) set forth in subsections 62-4.242(2) and (3), F.A.C.;

(e) Will not cause adverse secondary impacts to the water resources, and will not otherwise adversely impact the maintenance of surface or ground water levels or surface water flows established pursuant to Section 373.042, F.S.;

(f) Will be capable, based on generally accepted engineering and scientific principles, of being performed and of functioning as proposed;

(g) Will be conducted by an entity with the financial, legal, and administrative capability of ensuring that the activity will be undertaken in accordance with the terms and conditions of the permit, if issued; and

(h) Will comply with any applicable special basin or geographic area criteria rules incorporated by reference in subparagraph 62-330.301(1)(k)1., F.A.C., including meeting any applicable Sensitive Karst Area Basin requirements in **Section 6.0 of this Volume**.

(i) Will not adversely impact the value of functions provided to fish and wildlife and listed species by wetlands and other surface waters.

Specific to a stormwater management system that is either "stand alone" and does not involve any activities in wetlands or other surface waters, or a component of a larger surface water management system that involves work in wetlands and other surface waters, a showing by the applicant that the a stormwater management system has been designed in accordance with the following provisions of the Applicant's Handbook creates a presumption that reasonable assurance has been provided that the stormwater management system component of the activity meets the following specific conditions for issuance as listed above:

Compliance with:	**Creates a presumption of compliance with:**
Part III, Volume II	Sections 2.0.2(a), (b), (c), and (e)
Part IV, Volume II	Section 2.0.2(d)
Part II, Volume I	Section 2.0.2(d)
Part V, Volume I	Section 2.0.2(f)
Part V, Volume I	Section 2.0.2(g)

A stormwater management system that complies with the above identified design and performance criteria does not necessarily provide that other components of a project associated with the stormwater management system, including any work in, on, over, or adjacent to wetlands and other surface waters, will meet the Conditions for Issuance or the Additional Conditions for Issuance in Rules 62-330.301 and .302, F.A.C. This is why compliance with those design and performance criteria does not create a presumption of compliance with **Sections 2.2(h) and (i), above**; the entire project as a whole must be evaluated for compliance with Rules 62-330.301 and .302, F.A.C.

2.0.3 Agriculture and Silviculture

Agricultural and silvicultural activities that are not exempt from permit requirements under Section 373.406(2) or (3), F.S. or Rule 62-330.0511, F.A.C., are regulated under Chapter 40A-44, F.A.C.

2.1 Definitions

(a) The definitions and terms below are used for purposes of Chapter 62-330, F.A.C., and this Volume. **Section 2.0 of Volume I** contains most of the definitions that apply to the ERP program.

1. "100-year flood/One Percent Annual Chance of Flood," means that flood which has a one percent probability of recurrence in any one year. The 100-year flood elevation is the highest elevation of flood waters during the 100-year flood and is calculated or estimated from the best available information. The 100-year flood elevation shall not include coastal storm surge elevations unless such elevations have been developed in an approved Federal Emergency Management Agency Flood Insurance Study and such approved storm surge elevations have been accepted for implementation by the appropriate unit of local or state government.

2. “Closed basin” or “closed-lake” watershed means a watershed that does not have a surface outfall for conditions up to and including the 100-year, 24-hour flood stage. Rainfall depths associated with this event are provided as Figure 3.3-2.

3. “Control elevation” means the lowest elevation at which water can be released through a control device.

4. “Floodway” means the permanent channel of a stream or other watercourse, plus any adjacent floodplain areas that must be kept free of any encroachment in order to discharge the 100 year flood without cumulatively increasing the water surface elevation more than a designated amount (not to exceed one foot except as otherwise established by the Department or District or established by a Flood Insurance Rate Study conducted by the Federal Emergency Management Agency [FEMA]). For purposes of this Handbook, this term does not have the same meaning as the term “floodway” or “regulatory floodway” as defined and implemented by FEMA in 44 C.F.R. Chapter I, Part 9.4 (October 1, 2002), 44 C.F.R. Section 59.1 available at https://www.fema.gov/library/viewRecord.do?id=3064, or 44 C.F.R. Part 60 available at: http://www.gpo.gov/fdsys/pkg/CFR-2002-title44-vol1/pdf/CFR-2002-title44-vol1-part9.pdf (October 26, 1976).

5. “Littoral zone” means that portion of a stormwater management system that is designed to contain rooted emergent plants.

6. “Off-line” means the storage of a specified portion of the stormwater such that runoff in excess of the specified volume of stormwater does not flow into the area storing the treatment volume.

7. “On-line” means the storage of a specified portion of the stormwater such that runoff in excess of the specified volume of stormwater flows into or through the area storing the treatment volume.

8. “Permanent pool” means that portion of a wet detention pond that normally holds water (e.g., between the normal water level and the pond bottom), excluding any water volume claimed as wet detention treatment volume as shown in **Figure 8.1-1 of this Volume**.

9. “Tailwater” means the receiving water elevation (or pressure) at the final discharge point of a stormwater management system.

10. “Wetlands stormwater management system” means a stormwater management system that incorporates those wetlands described in **Section 10.2 of this Volume** into the stormwater management system to provide stormwater treatment.

(b) Definitions and terms that are not defined above shall be given their ordinary and customary meaning or usage of the trade or will be defined using published, generally accepted dictionaries, together with any rules and statutes of the Agencies that have additional authority over the regulated activities.

2.2 Professional Certification

All construction plans and supporting calculations submitted to the Agency for projects that require the services of the registered professional must be signed, sealed, and dated by a registered professional.

2.3 Legal Authorization

Applicants who propose to utilize offsite areas that are not under their ownership or control must obtain sufficient legal authorization prior to permit issuance to use the area in order to satisfy the requirements for issuance listed in Rules 62-330.301 and 302, F.A.C., and **Section 2.0.2 of this Volume.** For example, an applicant who proposes to locate the outfall pipe from the stormwater pond on an adjacent property owner's land must obtain a recorded drainage easement or other appropriate legal authorization from the adjacent owner. Other appropriate legal authorization must include a binding reservation on the land, that is recorded such that the provisions “run with the land” and are not subject to change if the property is sold. Further, any alteration to stormwater discharges to adjacent private properties resulting from permitted activities such as increase of flow, concentration of flow, or change of discharge location also requires appropriate legal authorization from adjacent owners receiving the discharge. A copy of the legal authorization must be submitted with the permit application.

Legal authorization generally is not required for systems that discharge to public rights-of-way; waters of the state such as a natural lake, creek, or wetland, except if located on state-owned submerged lands; or large multiple-owned systems; provided there is capacity to accept flows without causing harm to the water resources, or adverse impacts to property owners. However, any such discharge must also have appropriate down-gradient energy dissipaters and erosion protection. Such discharges may also require the written permission of the receiving system owner in the case of a department of transportation, county, or city conveyance system.

2.4 Public Safety

2.4.1 Side Slopes

Detention, retention, and normally dry ponds that are capable of impounding more than two feet of water, must meet one of the following:

(a) Side slopes that are no steeper than 4H:1V (horizontal to vertical) extending to a depth of 2 feet below the control elevation.

(b) The pond be fenced or otherwise restricted from public access if the slopes must be deeper due to space limitations or other constraints.

2.4.2 Pond Side Slope Stabilization

All stormwater pond side slopes shall be stabilized by either vegetation or other means or materials to minimize erosion of the pond due to flow velocity and runoff from the banks. Good engineering practices shall be employed, taking into consideration soil, flow, and drainage characteristics. The retardation of overland runoff and soil stabilization using naturally occurring vegetation coverage shall be considered before paving, riprap, lining, energy dissipation and other structural measures are employed. Guidance on erosion and sedimentation best management practices during the construction phase is contained in **Part IV of Applicant's Handbook Volume I**.

2.4.3 Control Structures

Control structures that are designed to contain more than two feet of water within the structure under the design storm and have openings of greater than one-foot minimum dimension must be restricted from public access.

2.5 Tailwater Considerations

Tailwater is an important component of the design and operation of nearly all stormwater management systems and can affect any of the following management objectives of the system:

(a) Peak discharge from the stormwater management system;

(b) Peak stage in the stormwater management system;

(c) Level of flood protection in the project;

(d) Recovery of peak attenuation and stormwater treatment volumes; and

(e) Control elevations, normal water elevation regulation schedules, and ground water management.

2.5.1 Tailwater for Water Quality Design

Stormwater management systems designed in accordance with **Part II of Volume I and Part IV of this Volume,** must provide a gravity or pumped discharge that effectively operates (i.e., meets applicable rule criteria) under tailwater conditions. Acceptable criteria for demonstrating effective tailwater conditions include criteria such as:

(a) Maximum stage in the receiving water resulting from the two-year, 24-hour storm. This rainfall depth is shown on the isopluvial map in **Figure 2.5-1**. Generally, applicants utilizing this option would model the receiving waters utilizing standard hydrologic and hydraulic methods for the two-year, 24-hour storm to determine peak stages at various points of interest. Lower stages may be utilized if the applicant demonstrates that flow from the project will reach the receiving water prior to the time of maximum stage in the receiving water.

(b) Mean annual high tide for tidal areas. This elevation is the average of all the high tides for each year. This elevation may be determined from tide charts or other similar information.

(c) Mean annual wet-season high water elevation. This elevation may be determined by water lines on vegetation or structures, historical data, adventitious roots or other hydrological or biological indicators, design of man-made systems, or estimated by a registered professional using standard hydrological methods based on the site and receiving water characteristics.

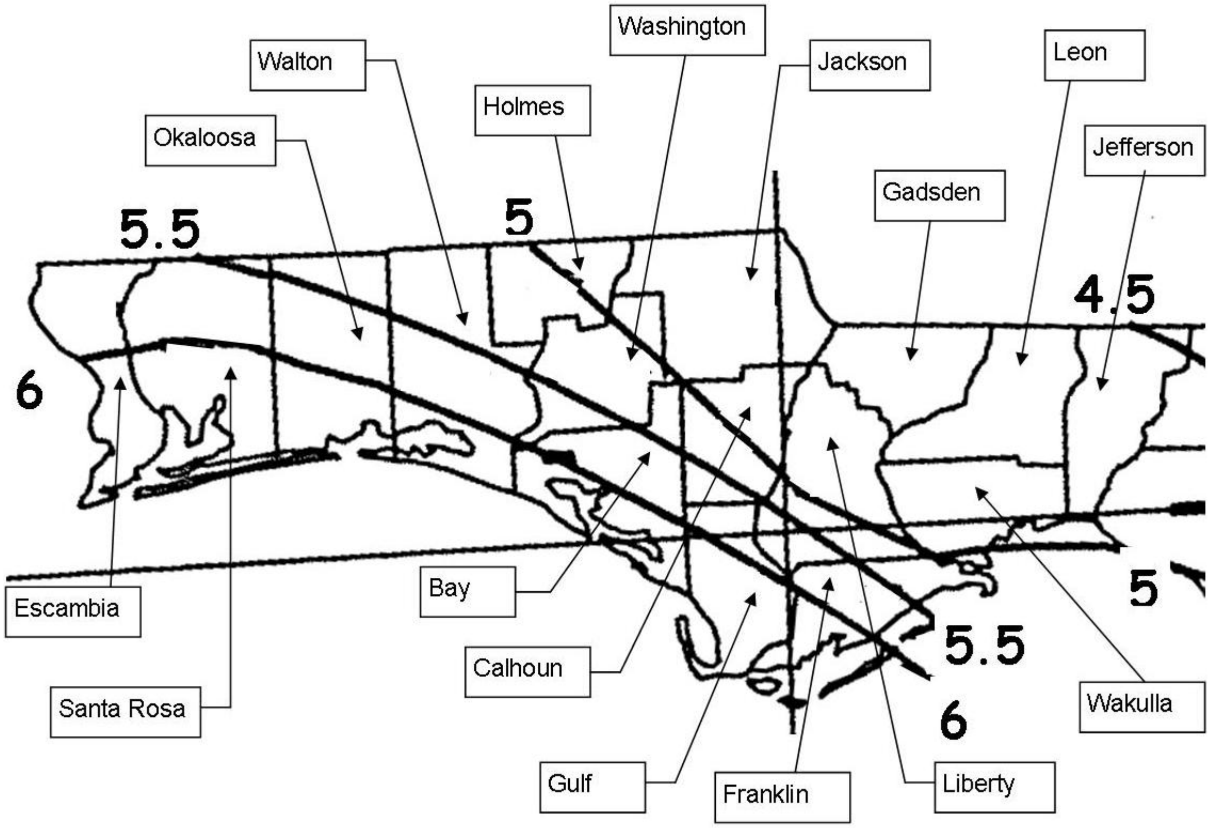

Figure 2.5-1 2-year, 24-hour Maximum Rainfall depth.

(d) The applicant may use applicable criteria established by a local government, state agency, or stormwater utility with jurisdiction over the project. The Agency will approve the use of such alternative criteria if the Agency determines that the alternative criteria will provide equivalent or greater reasonable assurance as the applicable criteria of this Volume.

In this case, the applicant is encouraged to consult with Agency staff prior to submitting an application.

2.5.2 Tailwater for Water Quantity Design

Stormwater management systems designed in accordance with **Part III of this Volume** must consider tailwater conditions. Receiving water stage can affect the amount of flow that will discharge from the project to the receiving water. This stage may be such that tailwater exists in portions of the project system, reducing the effective flow or storage area. Typical examples of this are illustrated in **Figures 2.5-2** (gravity) and **2.5-3** (pumped).

The stage in the receiving water shall be considered to be the maximum stage which would exist in the receiving water from a storm equal to the project design storm. Lower stages may be used if the applicant can show that the flow from his project will reach the receiving water prior to the time of maximum stage in the receiving water.

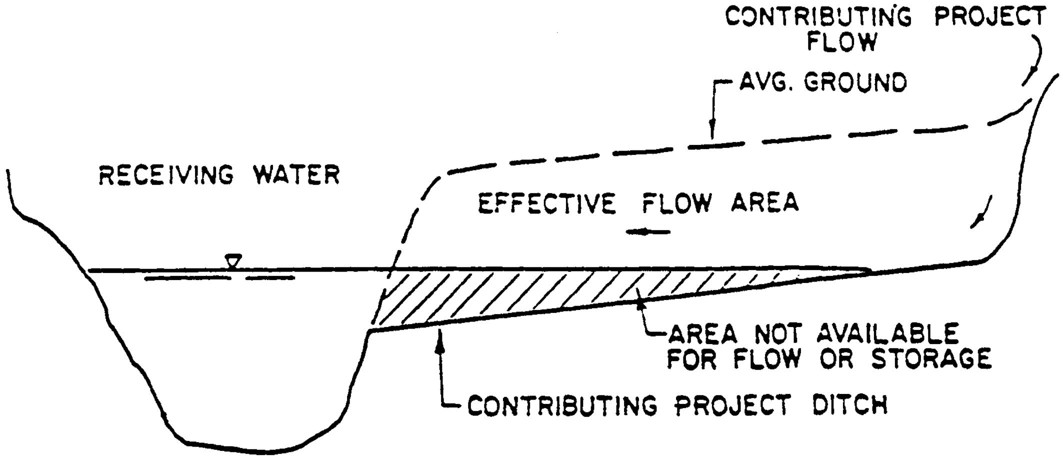

Figure 2.5-2 Gravity tailwater example.

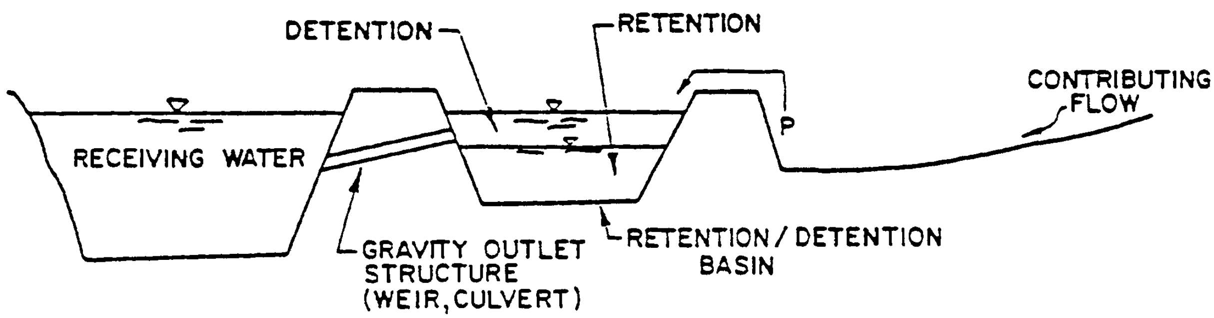

Figure 2.5-3 Pumped flow tailwater example.

2.6 Retrofits of Existing Stormwater Management Systems

A stormwater retrofit project is typically proposed by a county, municipality, state agency, or water management district to provide new or additional treatment or attenuation capacity, or improved flood control to an existing stormwater management system or systems. Stormwater retrofit projects shall not be proposed or implemented for the purpose of providing the water quality treatment or flood control needed to serve new development or redevelopment.

Example components of stormwater retrofit projects are:

1. Construction or alteration that will add additional treatment or attenuation capacity and capability to an existing stormwater management system;
2. Modification, reconstruction, or relocation of an existing stormwater management system or stormwater discharge facility;

3. Stabilization of eroding banks through measures such as adding attenuation capacity to reduce flow velocities, planting of sod or other vegetation, and installation of rip rap boulders;

4. Excavation or dredging of sediments or other pollutants that have accumulated as a result of stormwater runoff and stormwater discharges.

The applicant for any stormwater retrofit project must design, construct, operate, and maintain the project so that it:

1. Will not cause or contribute to a water quality violation;

2. Does not reduce stormwater treatment capacity or increase discharges of untreated stormwater. Where existing ambient water quality does not meet water quality standards the applicant must demonstrate that the proposed activities will not cause or contribute to a water quality violation. If the proposed activities will contribute to the existing violation, measures shall be proposed that will provide a net improvement of the water quality in the receiving waters for those parameters that do not meet standards.

3. Does not cause any adverse water quality impacts in receiving waters; or

4. Will not cause or contribute to increased flooding of adjacent lands or cause new adverse water quantity impacts to receiving waters;

2.6.1 Types of Stormwater Retrofits

(a) Stormwater Quality Retrofits

1. The applicant for a stormwater quality retrofit project must provide reasonable assurance that the retrofit project itself will, at a minimum provide additional water quality treatment such that there is a net reduction of the stormwater pollutant loading into receiving waters. Examples are:

    a. Addition of treatment capacity to an existing stormwater management system such that it reduces loadings of stormwater pollutants of concern to receiving waters;

    b. Adding treatment or attenuation capability to an existing developed area when either the existing stormwater management system or the developed area has substandard stormwater treatment and attenuation capabilities, compared to what would be required for a new system requiring a permit under Part IV of Chapter 373, F.S.; or

    c. Removing pollutants generated by, or resulting from, previous stormwater discharges.

2. The pollutants of concern are based upon the existing water quality data within the area subject to the retrofit, and the degree of impairment or water quality violations in the receiving waters. If no water quality data exists and there are no listed impairments or water quality violations in the receiving waters, the applicant shall demonstrate such a net improvement whereby the pollutant loads discharged from

the system shall be less than those discharged based on the project's existing condition for total nitrogen and total phosphorus.

(b) Stormwater Quantity (Flood Control) Retrofits

The applicant for a stormwater quantity retrofit project must provide reasonable assurance that the retrofit project will reduce existing flooding problems in such a way that it does not cause any of the following:

1. A net reduction in water quality treatment provided by the existing stormwater management system or systems;

2. Increased discharges of untreated stormwater entering adjacent or receiving waters;

If the applicant has conducted, and the Agency has approved, an analysis that provides reasonable assurance that the stormwater quantity retrofit project will comply with the above, the project will be presumed to comply with the requirements in **Sections 3.1 through 3.3 of this Volume**.

2.7 Flexibility for State Transportation Projects and Facilities

With regard to state linear transportation projects and facilities the Agencies shall be governed by Section 373.413(6), F.S.

2.8 Dam Safety

As part of the determination as to whether a dam meets the criteria in Rule 62-330.301, F.A.C., a dam over five feet in height (as measured from the crest of the dam to the lowest elevation on the downstream toe) with the potential to store 50 acre feet or more of water, and any dam that is 10 feet or more in height must be designed, constructed, operated, and maintained consistent with generally accepted engineering practices as applied to local conditions, considering such factors as: the type of materials used to construct the dam, the type of soils and degree of compaction, hydrologic capacity, construction techniques, and downstream hazard potential. (referenced in Section 8.4.5 and Appendix L of Volume I). An additional document that provides useful information for this purpose is *Design of Small Dams*, U.S. Department of Interior, Bureau of Reclamation, Third Edition, 2006.

Dams shall be designed with spillway capacities adequate to safely conduct the runoff through the impoundment based on the appropriate SCS rainfall distribution, in accordance with the following minimum storm routing requirements:

Minimum Storm Routing Requirements for Dams

Downstream Hazard Potential ~~Rating~~	**Principal spillway**	**Combination of spillways**
Low	25-year, 24-hour	25-year, 24-hour
Significant	25-year, 24-hour	100-year, 24-hour
High	100-year, 24-hour	Probable Maximum Flood

PART III — STORMWATER QUANTITY/FLOOD CONTROL

3.0 General Flood Control Requirements

3.1 Stormwater Management Systems That Must Meet Water Quantity Criteria

Stormwater management systems that meet any of the following thresholds must be designed, constructed, operated, and maintained in accordance with this Part:

(a) Systems that serve projects of 40 or more acres of total land area;

(b) Systems that provide for the placement of 12 or more acres of impervious surface, which also constitutes more than 40 percent of the total project area; or

(c) Systems that are at any time capable of impounding a volume of water exceeding 40 acre-feet, as measured at the top of the berm.

3.2 Standards that Apply and Relationship to Part IV

In addition to the criteria in this Part, all activities that require a stormwater management system (in accordance with **Section 2.1 of this Volume**) must also comply with the water quality criteria in **Part II of Volume I and Part IV of this Volume**.

As an example, a system that has 14 acres of impervious surface that comprises 54 percent of the total project area of 26 acres would have to meet the stormwater quantity/flood control criteria of this Part, because such a system exceeds the 12-acre/40 percent threshold. Because the project exceeds thresholds for stormwater management systems, the criteria in Part IV also apply. Additionally, because the project involves greater than 50 percent impervious area, the project must also be designed according to the streambank protection discharge criteria as required in **Section 4.5.1 of this Volume**. However, a system that consists of 13 acres of impervious surface within a 39 acre project area would not have to meet the stormwater quantity/flood control criteria of this Part (assuming the system does not impound more than 40 acre-feet of stormwater), because even though such a system exceeds the 12-acre threshold in **3.1(b), above**, it constitutes an impervious surface of only 33 percent, and therefore does not exceed the second part of **3.1(b), above**, or the criteria in **3.1(a)**. Such a system also would not have to be designed to meet the streambank protection discharge criteria. As another example, a system that consists of 2 acres of impervious surface within a 3-acre project area also would not have to meet the stormwater/flood control criteria of this Part because it does not exceed the 12-acre threshold. However, such a system exceeds the 50% impervious threshold (67% impervious) and therefore is required to comply with the streambank protection provisions.

3.3 Stormwater Quantity: Rate and Volume Controls

Except as provided in **Section 3.3(c), below**, the post-development stormwater discharge rate and volume must be controlled as follows.

(a) Rate Control

Any project involving construction that exceeds 50 percent impervious surface must provide rate control in accordance with **Section 4.5.1 of this Volume**.

If the project is located totally within a stream or open-lake watershed, detention systems must be installed such that the peak rate of post-development runoff will not exceed the pre-development peak-rate of runoff for the 25-year, 24-hour design storm event, utilizing a Natural Resources Conservation Service (NRCS) type III rainfall distribution, with an antecedent moisture condition II. Rainfall associated with the 25-year, 24-hour event is provided in **Figure 3.3-1**. Outlet controls shall be designed such that required detention volumes are available within 14 days following the design storm event.

(b) Volume Control

Flood elevation shall be determined using the most accurate information available, such as:

1. Actual data, including water level, stream flow and rainfall records;

2. Hydrologic/hydraulic modeling;

3. Federal Flood Insurance Rate Maps and supporting flood study data; or

4. Floodplain analysis studies approved by the Agency.

For systems discharging within a closed basin or closed-lake watershed, the post-development volume of runoff discharged offsite must not exceed the pre-development volume of runoff discharged offsite resulting from a 25-year, 96-hour design storm. Retention of the post-development increase in volume can be recovered by infiltration, or, if soil conditions are not sufficient for infiltration, then detention must be provided for a duration sufficient to prevent adverse impacts on flood stages. Rainfall depths associated with the 25-year, 96-hour design storm are provided in **Figure 3.3-3**. The applicant may provide a time-dependent model utilizing a 25-year, 96-hour hyetograph in conjunction with a rating curve (or equivalent) to estimate the rate of infiltration, from the system during the storm.

For systems discharging to closed basins or closed-lake watersheds that are wholly-owned, the applicant is not required to demonstrate compliance with **Section 3.3(a) or (b) of this Volume**. However, the flood damage requirements of **Section 3.6 of this Volume**

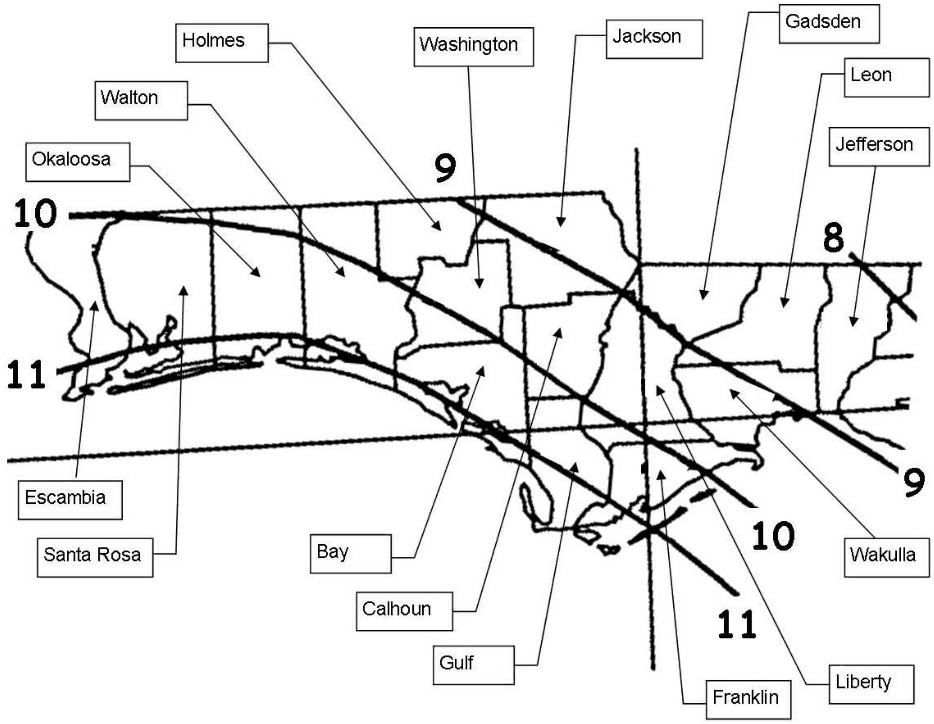

Figure 3.3-1 Rainfall Depths Associated with the 25-year, 24-hour Storm Event

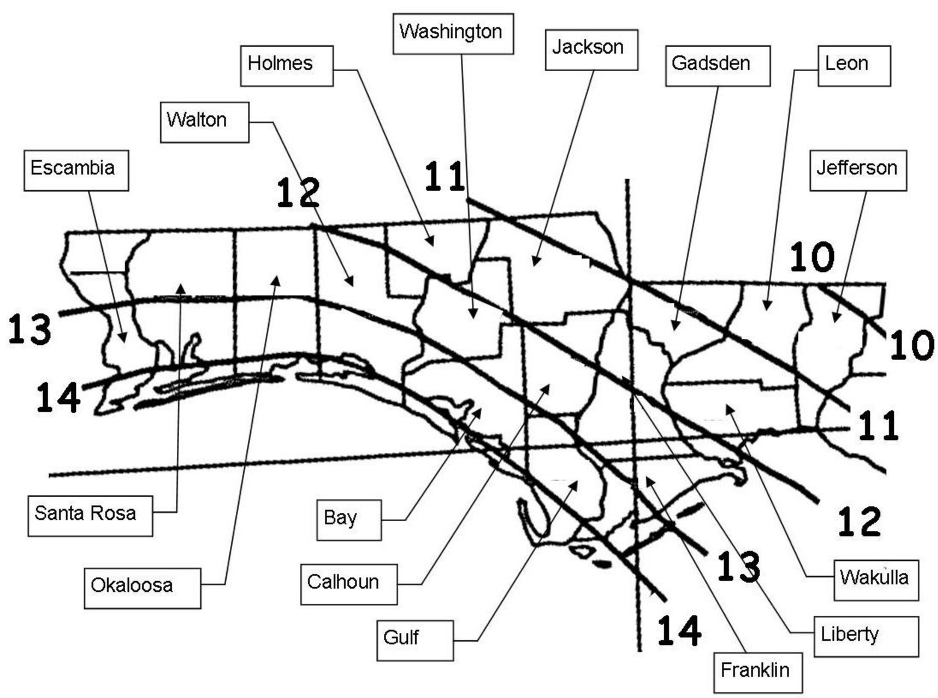

Figure 3.3-2 Rainfall Depths Associated with the 100-year, 24-hour Storm Event

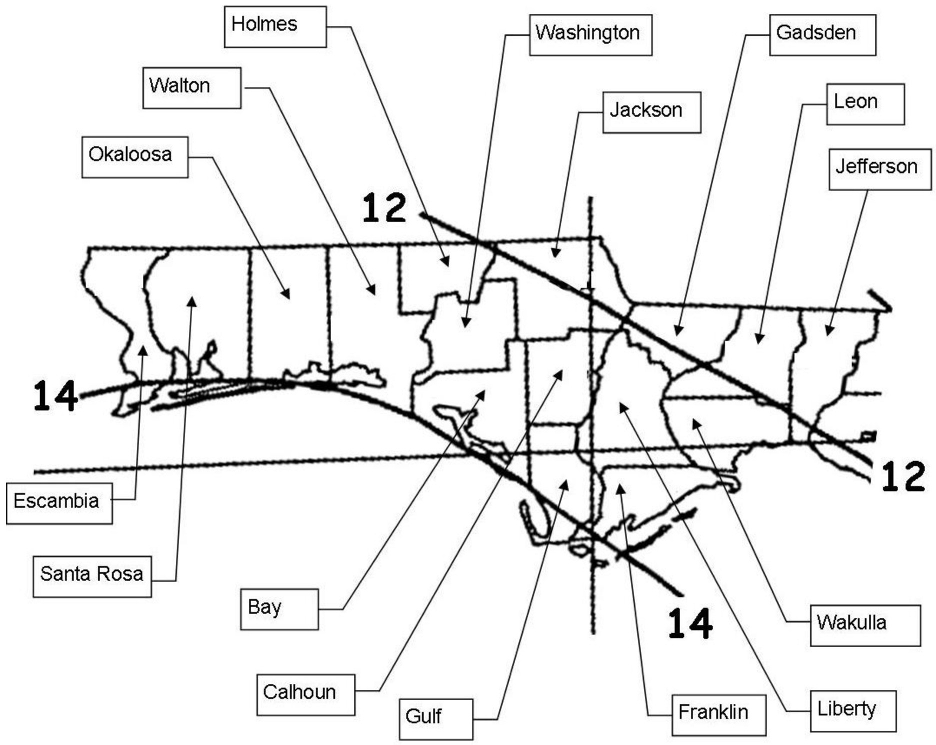

Figure 3.3-3 Rainfall Depths Associated with the 25-year, 96-hour Storm Event

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must still be met. Additionally, for the purposes of this paragraph, minimum finished floor elevations must be located above the post-development design storm elevation associated with the 25-year, 96-hour storm event.

Post-development volume controls must be provided in accordance with this section, unless the applicant can demonstrate that cumulative increases in runoff volume from potential development will not cause an adverse impact on the frequency, duration or extent of off-site flood stages resulting from the 25-year, 96-hour design storm.

(c) Discharges to Tidally-influenced waters

The peak discharge requirements of this section are not required for systems that discharge directly into tidally-influenced waterways. For the purposes of this section, "tidally-influenced waterways" includes surface waters that are characterized by a repeatable monthly average tide range of more than 0.1 feet.

3.3.1 Alternative Peak Rate Discharge Criteria

As an alternative to the peak discharge attenuation criteria in **Section 3.3 above**, applicants may propose to utilize applicable storm event, duration, or criteria specified by a local government, state agency (including FDOT), or stormwater utility with jurisdiction over the project. The Agency will approve the use of the alternative criteria if the Agency determines that the alternative criteria will provide equivalent or greater reasonable assurance as the criteria of **Section 3.3 above**.

Applicants proposing to use alternative criteria are encouraged to have a pre-application conference with Agency staff.

3.3.2 Methodologies

A peak discharge analysis typically consists of generating pre-development and post-development runoff hydrographs, routing the post-development hydrograph through a detention system, and sizing an overflow structure to control post-development discharges at or below pre-development rates.

Peak discharge computations shall consider the duration, frequency, and intensity of rainfall, the antecedent moisture conditions, upper soil zone and surface storage, time of concentration, tailwater conditions, changes in land use or land cover, and any other changes in topographic and hydrologic characteristics. Large systems should be divided into sub-basins according to artificial or natural drainage divides to allow for more accurate hydrologic simulations. Examples of accepted methodologies for computing runoff are:

(a) Soil Conservation Service Method [see U.S. Department of Agriculture, Soil Conservation Service "National Engineering Handbook, Section 4, Hydrology," TR-55 ("Urban Hydrology for Small Watershed") or TR-20 User's Manuals].

(b) Santa Barbara Urban Hydrograph Method.

(c) U.S. Army Corps of Engineers HEC-HMS Computer Programs.

(d) Storm Water Management Model (SWMM) 5 or higher

(e) Interconnected Channel and Pond Routing Model (ICPR)

(f) PONDS

Other hydrograph and routing methods may be proposed and will be approved by the Agency if the applicant provides reasonable assurance that the alternative method has comparable accuracy and reliability as the above methods.

Peak discharge calculations must make proper use of the SCS Peak Rate Factor or K'. The Peak Rate Factor reflects the effect of watershed storage on the hydrograph shape and directly and significantly impacts the peak discharge value. As such, K' must be based on the true watershed storage of runoff, and not on the slope of the landscape which is more accurately accounted for in the time of concentration. However, the average slope of natural watersheds is highly interrelated with the surface storage potential. Land development will generally result in a reduction of natural storage. As a result, the K' value should either increase or remain constant, but never decrease. In most cases, post-development conditions will include detention storage areas; this storage should be accounted for by routing the hydrograph based on a defined stage-storage-discharge relationship and should therefore not be considered in determining K'. The most conservative approach is to use a K' = 484 for post-development. However, in some cases where surface storage is maintained, K' may be reduced to same value used in the pre-development condition.

Recommended K' values for various site conditions are provided below:

K'=484:

Standard peak rate factor developed for watersheds with little or no storage. Represents watersheds with moderate to steep slopes and/or significant drainage works. Typical ecological communities include long leaf pine, and turkey oak hills.

K'=323-384:
Intermediate peak rate factor representing watersheds with moderate surface storage in some locations due to depressional areas, mild slopes and/or lack of existing drainage features. Typical ecological communities include oak hammock, upland hardwood hammock, mixed hardwoods and pine.

K'=256-284:
Represents watersheds with very mild slopes, recommended for watersheds with average slopes of 0.5% or less. Significant surface storage throughout the watershed. Limited on-site drainage ditches. Typical ecological communities include North Florida flatwoods, freshwater marsh and ponds, swamp hardwoods, cabbage palm flatlands, and cypress swamp.

3.3.3 Aggregate Discharge

Where multiple off-site discharges are designed to occur, if the combined discharges meet all other requirements of Chapter 62-330, F.A.C., and discharge to the same receiving water body, the Agency will allow the total post-development peak discharge for the combined discharges to be used rather than each individual discharge.

3.3.4 Rainfall Intensity and Volume

In determining peak discharge rates, intensity of rainfall values shall be obtained through a statistical analysis of historical long-term rainfall data or from sources or methods generally accepted as good engineering practice.

(a) Examples of acceptable sources include:

1. USDA Soil Conservation Service, "Rainfall Frequency Atlas of Alabama, Florida, Georgia, and South Carolina for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years" January 1978; Gainesville, Florida.

2. U.S. Weather Bureau Technical Paper No. 49.

3. U.S. Weather Bureau Technical Paper No. 40.

4. U.S. Department of Interior, Bureau of Reclamation, "Design of Small Dams." ~~2nd~~ 3rd Edition.

(b) For a drainage basin greater than 10 square miles, the areal rainfall can be calculated from point rainfall data using a method that has been well documented. The converting factor as described in U.S. Weather Bureau Technical Paper No. 49 can be used.

3.3.5 Upper Soil Zone Storage and Surface Storage

In most instances, the upper soil zone storage and surface storage capacities will have an effect on the pre-development and post-development peak discharges and should be considered in these computations. Any generally accepted and well-documented method may be used to develop the upper soil zone storage and surface storage values.

(a) The soil zone storage at the beginning of a storm shall be estimated by using reasonable and appropriate parameters consistent with generally accepted engineering and scientific principles to reflect drainage practices, average wet season water table elevation, the antecedent moisture condition (generally AMC II) and any underlying soil characteristics that would limit or prevent infiltration of storm water into the entire soil column. The soil storage used in the computation shall not exceed the difference between the maximum soil water capacity and the field capacity (for example, gravitational water) for the soil columns above any impervious layer or seasonal ground water table.

(b) Surface storage, including that available in wetlands and low-lying areas, shall be considered as depression storage. Depression storage shall be analyzed for its effect on peak discharge and the time of concentration. Depression storage can also be considered in post-development storage routing which would require development of stage-storage relationships; if depression storage is considered, then both pre-development and post-development storage routing must be considered.

3.4 Storage and Conveyance

Floodways and floodplains, and levels of flood flows or velocities of adjacent streams, impoundments or other water courses must not be altered so as to adversely impact the off-site storage and conveyance capabilities of the water resource. Projects that alter existing conveyance systems (such as by rerouting an existing ditch) must not adversely affect existing conveyance capabilities. Also, the applicant shall provide reasonable assurance that proposed velocities are non-erosive or that erosion control measures (such as riprap and concrete lined channels) are sufficient to safely convey the flow.

(a) A system shall not cause a net reduction in flood storage within a 10-year floodplain except for structures elevated on pilings or traversing works.

(b) A system shall not cause a reduction in the flood conveyance capabilities provided by a floodway except for structures elevated on pilings or traversing works. Such works or other structures shall cause no more than a one-foot increase in the 100-year flood elevation immediately upstream and no more than one tenth of a foot increase in the 100-year flood elevation 500 feet upstream.

(c) An applicant will not have to meet the requirements of (a) or (b) above if reasonable assurance is provided that the singular and cumulative impacts of not meeting those criteria will not contravene subsections 62-330.301(1) and (2), F.A.C., considering all other persons who could impact the surface water of any impoundment, stream, or other watercourse by floodplain encroachment to the same degree as proposed by the applicant.

(d) As an alternative, the applicant may propose to utilize applicable criteria established by a local government, another state agency, or a stormwater utility with jurisdiction over the project. DEP will approve the use of such alternative criteria if the alternative criteria provide reasonable assurance that the proposed project will not adversely affect existing conveyance capabilities.

3.5 Low Flow and Base Flow Maintenance

3.5.1 Low Flow

(a) Systems with both of the following conditions must meet the low flow performance criteria in **Sections 3.5.1(b) and (c), below**.

1. Systems that impound water for purposes in addition to temporary detention storage. Water impounded longer than a 14-day bleed down period is considered conservation storage for benefits other than detention storage (for example, recreation and irrigation).

2. Systems that impound a stream or other watercourse which, under pre-development conditions, discharged surface water off-site to receiving water during 5-year, 30-day drought frequency conditions.

(b) Any system meeting the conditions of **Section 3.5.1(a), above**, shall be designed with an outlet structure to maintain a low flow discharge of available conservation storage. When the conservation storage is at the average dry season design stage, the low flow discharge shall equal the average pre-development surface water discharge which occurred from the project site to receiving waters during the 5-year, 30-day drought.

(c) The system shall be operated to provide a low flow discharge whenever water is impounded. The actual discharge will vary according to the water stage in the impoundment. When conservation storage is at the average dry season design stage, the discharge will be the 5-year, 30-day average low flow. When storage is below the average dry season design stage, the discharge may be less than the 5-year, 30- day average low flow.

3.5.2 Base Flow

Design and performance criteria for maintaining acceptable base flow conditions include:

(a) Storage volumes in detention or retention systems shall be calculated so as not to include volumes below the seasonal high-water table for the project area;

(b) Underdrain systems shall be allowed provided that lowering of the groundwater table is restricted to the immediate vicinity of the treatment system; and

(c) Water tables shall not be lowered to a level that would decrease the flows or levels of surface water bodies below any minimum level or flow established by a water management district Governing Board pursuant to Section 373.042, F.S.

3.6 Flood Damage

For the purposes of this section, the design storm for determining the 100-year flood elevations shall be the 100-year, 24-hour event. In evaluating the potential for flood damages to residences, public buildings, the following criteria will be utilized:

(a) Residential buildings shall have the lowest floor elevated above the post-development 100-year flood elevation for that site.

(b) Industrial, commercial, and other non-residential buildings susceptible to flood damage must have the lowest floor elevated above the 100-year flood elevation or be designed and constructed so that below the 100-year flood elevation, the structure and attendant utility facilities are watertight and capable of resisting the effects of the regulatory flood. The design should take into account flood velocities, duration, rate of rise, hydrostatic and hydrodynamic forces, the effect of buoyancy and impacts from debris. Flood proofing measures must be operable without human intervention and without an outside source of electricity.

(c) Accessory buildings may be constructed below the 100-year flood elevation provided there is minimal potential for significant damage by flooding. An accessory building is a structure on the same parcel of property as a principal structure and the use of which is incidental to the use of the principal structure and not for human habitation. For example, a residential structure may have a detached garage, a carport, or storage shed for garden tools as accessory structures. Other examples of accessory structures include gazebos, picnic pavilions, boathouses, pole barns, storage sheds, and similar buildings.

PART IV — ADDITIONAL STORMWATER QUALITY STANDARDS AND REQUIREMENTS

4.0 Purpose

All stormwater management systems that require an individual permit under Chapter 62-330, F.A.C., must also be designed, constructed, operated, and maintained in conformance with the criteria in **Part II, Part IV, and Part V of Volume I** and in this Part. In addition, those systems that exceed the thresholds in **Section 3.1 of this Volume** must also be designed, constructed, operated, and maintained in accordance with **Part III of this Volume**.

4.1 Criterion

Florida's stormwater quality regulations are "technology-based" not "water quality effluent-based." Collectively, the design criteria in **Part II, Part IV, and Part V of this** Volume in addition with **Part II of Volume I** are presumed to meet the minimum levels of stormwater treatment established in Chapter 62-40, F.A.C., the Water Resource Implementation Rule.

4.2 Integration with the Water Resource Implementation Rule

Subsection 62-40.432(2), F.A.C. (Water Resource Implementation Rule), provides minimum stormwater treatment performance standards. These standards, in part, provide that when a stormwater management system complies with rules establishing the design and performance criteria for such systems, there shall be a rebuttable presumption that the discharge from such systems will comply with state water quality standards.

Systems meeting the design and performance criteria of Part II of Volume I as well as this Part are presumed to meet the Water Resource Implementation Rule performance standards stated above. However, as new research on the design and effectiveness of stormwater treatment systems becomes available, the design and performance criteria of Part II of Volume I and this Volume will be revised as appropriate through future rulemaking.

4.3 State Water Quality Standards

4.3.1 Surface Water Quality Standards

State surface water quality standards are set forth in Chapters 62-4 and 62-302, F.A.C., including the antidegradation provisions of paragraphs 62-4.242(1)(a) and (b), and subsections 62-4.242(2) and (3), F.A.C., Rule 62-302.300, F.A.C., and the special standards for OFWs set forth in subsections 62-4.242(2) and (3), F.A.C. Furthermore, the Agency cannot authorize permits that modify the quantity of water discharged offsite if such discharge will cause adverse environmental or water quality impacts.

4.3.2 Ground Water Quality Standards

State water quality standards for ground water are set forth in Chapter 62-520, F.A.C. In addition to the minimum criteria, Class G-I and G-II ground water must meet primary and secondary drinking water quality standards for public water systems established pursuant to the Florida Safe Drinking Water Act, which are listed in Rules 62-550.310 and 62-550.320, F.A.C.

Only the minimum criteria apply within a zone of discharge. A zone of discharge is defined as a volume underlying or surrounding the site and extending to the base of a specifically designated aquifer or aquifers, within which an opportunity for the treatment, mixture or dispersion of wastes into receiving ground water is afforded. Generally, stormwater systems have a zone of discharge 100 feet from the system boundary or to the project's property boundary, whichever is less.

Stormwater retention and detention systems are classified as moderate sanitary hazards with respect to public and private drinking water wells. Stormwater treatment facilities shall not be constructed within 100 feet of an existing public drinking water ~~well, and~~ well and shall not be constructed within 75 feet of an existing private drinking water well.

4.3.3 How Standards are Applied

The quality of stormwater discharged to receiving waters is presumed to meet the surface water standards in Chapters 62-4, and 62-302, F.A.C., and the ground water standards in Chapters 62-520 and 62-550, F.A.C., if the system is permitted, constructed, operated, and maintained in accordance with Chapter 62-330, F.A.C., **Part II, Part IV, and Part V of Volume I** and **Parts II, Part IV, and Part V of this Volume**. All stormwater treatment systems shall provide a level of treatment sufficient to accomplish the nutrient load reduction criteria listed in Section 8.3 of Volume I. The nutrient load reduction is demonstrated by the modeling or calculations of the type of treatment system, i.e. retention, wet detention, etc. If off-site runoff is not prevented from combining with on-site runoff prior to treatment, then treatment must be provided for the combined off-site/project runoff.

4.4 Reasonable Assurance

As part of providing reasonable assurance that a system meets the general criteria for issuance described in **Section 2.0.2 of this Volume**, a stormwater management system must meet the design and performance standards described in the applicable **Parts III, IV, and V of this Volume** in addition to the treatment requirements described in **Part II of Volume I**.

4.5 Criteria to Protect Streambanks

Urbanization increases total runoff volume, peak discharge rates, and the magnitude and frequency of flood events. With an increase in the number of flood events a stream is subjected to, the potential for accelerated erosion of both the stream banks and channel bottom is enhanced, resulting in degradation of surface waters. Proper design of detention systems to limit post-development peak discharge rates to predevelopment rates can minimize some of the stormwater effects of urbanization.

Proper selection of the design storm for peak discharge control is crucial to determining the effectiveness of the detention system. Historically, stormwater programs only regulated the peak discharge from large storm events (for example, a 25-year, 24-hour storm). Unfortunately, that approach suffers from the following drawbacks:

(a) If a detention pond is only designed to reduce the peak of the 25-year storm, the discharge rates from lesser events such as the 2-, 5-, and 10-year flood events may not be controlled. The ineffectiveness of controlling small flood events may appear to be unimportant with respect to flood damages. However, these more frequent events do cause localized flood damage and are of prime importance as a cause of channel and streambank erosion.

(b) Cumulative water quantity impacts may occur from several projects that are below the thresholds for quantity control located within the same watershed.

To address these concerns, peak discharge rate must be controlled for the 2-year, 24-hour storm event and potentially for a larger storm event. The 2-year, 24-hour storm was selected as the design event for this rule because the shape and form of natural channels is controlled by approximately the 2-year return frequency storm. The rainfall depth for the 2-year, 24-hour storm for the Florida panhandle is shown in **Figure 2.7-1**. The rainfall depth at a particular location may be established by interpolating between the nearest isopluvial lines.

The 2-year event may be accommodated along with the larger flood control storm event (when required) by designing a multi-staged outlet structure to attenuate both the flood control and 2-year, 24-hour storm events, such as through the use of two-staged weirs, risers with multiple orifice controls, and combinations of weir and orifice controls. See **Figure 4.5-1** for a conceptual design of a multi-staged outlet structure.

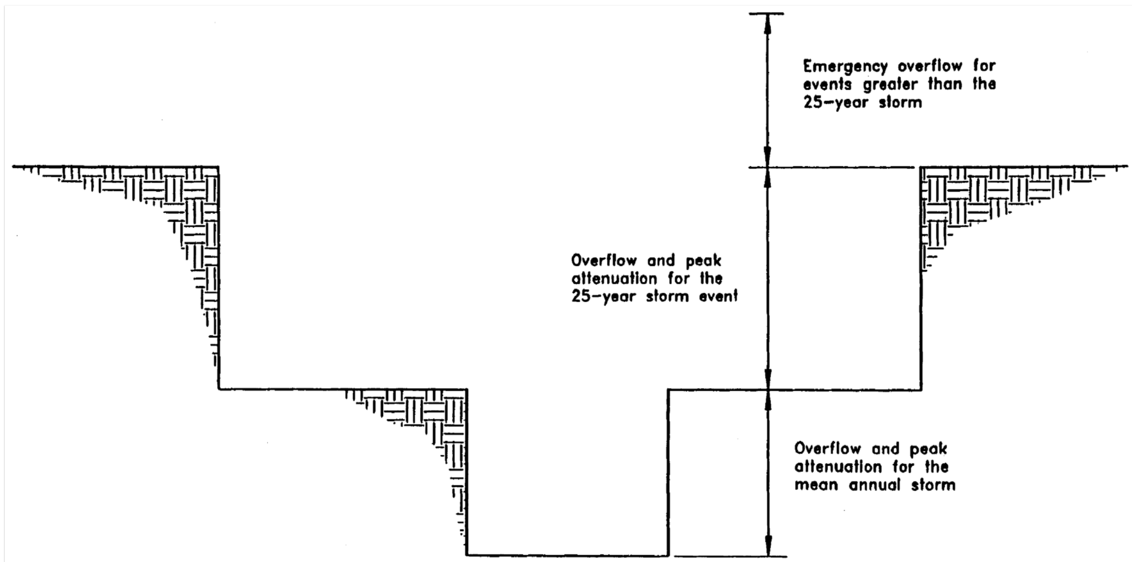

Figure 4.5-1 Conceptual design of a multi-stage outlet structure.

4.5.1 Peak Discharge Attenuation Criteria to Protect Streambanks

For systems serving new construction that is greater than 50 percent impervious (excluding water bodies and the area providing stormwater treatment) over the project area, the post-development peak discharge rate must not exceed the pre-development peak discharge rate for the 2-year, 24-hour design storm event, utilizing a Natural Resources Conservation Service (NRCS) type III rainfall distribution with an antecedent moisture condition II. Outlet controls shall be designed so that required detention volumes are fully bled-down at sufficient rates that result in non-erosive velocities Projects that modify existing systems, including adding or removing impervious surfaces, are not exempt from this criterion and are required to demonstrate that the modification will not cause significant adverse impacts to water resources using the criteria in Rule 62-330.301, F.A.C. Projects that discharge to tidally-influenced waters tide in accordance with **Section 3.3(c) of this Volume** are exempt from this criterion.

Pervious concrete and turf blocks are not considered impervious surface for this purpose. However, pervious asphalt, compacted soils, limerock, or gravel surfaces, are considered impervious for the purpose of determining the percentage of impervious surface.

The streambank protection criteria must be met concurrent with applicable flood control requirements under **Part III of this Volume**, including any project that also requires peak discharge attenuation of the 25-year, 24-hour storm event.

4.5.2 Modified Rational Hydrograph Method for Streambank Protection Calculations

The rational method is a popular method for estimating peak runoff rates for small urban areas. Specifically, the rational method generates peak discharge rates rather than a runoff hydrograph. However, the rational method can be modified to generate a runoff hydrograph by utilizing the rainfall intensity for various increments of a design storm.

The rate of discharge at any point in time during a storm can be calculated by combining the rainfall intensity for that time increment with the traditional rational formula. The modified rational hydrograph equation is as follows:

$$Q = C\,(I/P_{Total})\,(P_{Total})\,A$$

where:
Q = Discharge for a given time increment (cfs)
C = Runoff coefficient
I/P_{Total} = Intensity for a given time increment (in/hr-in)
P_{Total} = Total rainfall depth (in)
A = Drainage area (acres)

Calculating the peak discharge in 15-minute increments over a 24-hour period generates a synthetic hydrograph. Intensities are typically derived from intensity-duration-frequency (IDF) curves such as those published by the FDOT. The maximum allowable drainage area (A) is 600 acres.

Similar to the rational method for peak discharge, the modified rational method must be limited to small drainage basins with short times of concentration. The use of the modified rational method for generating a runoff hydrograph is limited to systems meeting the following conditions:

(a) The drainage area is less than 40 acres,

(b) The pre-development time of concentration for the system is less than 60 minutes, and

(c) The post-development time of concentration for the system is less than 30 minutes.

The Agency does not accept the modified rational hydrograph method for use in generating hydrographs for the 25-year, 24-hour storm event for use in complying with peak discharge requirements in **Section 3.3 of this Volume**. For projects requiring peak discharge evaluation in accordance with **Section 3.3 of this Volume**, the modified rational hydrograph method is acceptable only for evaluation of the 2-year, 24-hour storm, and not for other events (e.g., the 25-year, 24-hour storm).

4.6 On-Line and Off-line Stormwater Systems

There are two basic types of configurations for capturing the stormwater runoff volume: On-line and Off-line systems.

On-line systems (**Figure 4.6-1**) consist of a storage area which provides storage of the required treatment volume for smaller storm events and, if required, temporary detention storage for peak discharge control during larger storm events. Runoff volumes in excess of the required treatment volume mix together in the basin and potentially transport a portion of the pollutant mass load through the basin control structure.

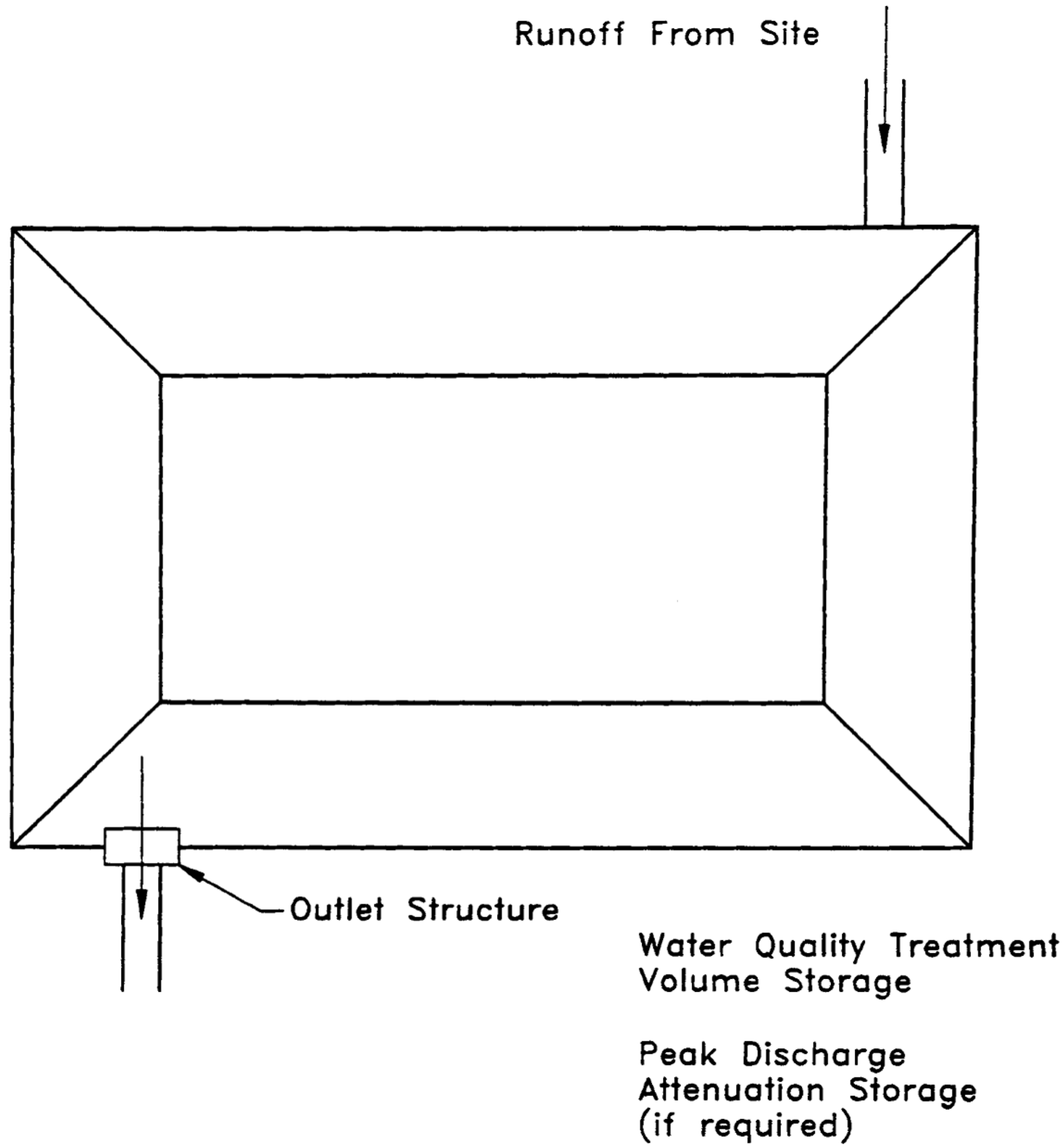

Figure 4.6-1 On-line treatment system.

Off-line treatment systems (**Figure 4.6-2**) divert the treatment volume into an off-line system that is designed for storage of the required treatment volume. Runoff volumes in excess of the required treatment volume by-pass the off-line system and are discharged to either the receiving water or routed to a detention system that if peak discharge attenuation is required. A diversion box (**Figure 4.6-3**) typically is used to divert the required treatment volume to the off-line system and route excess flows away from it.

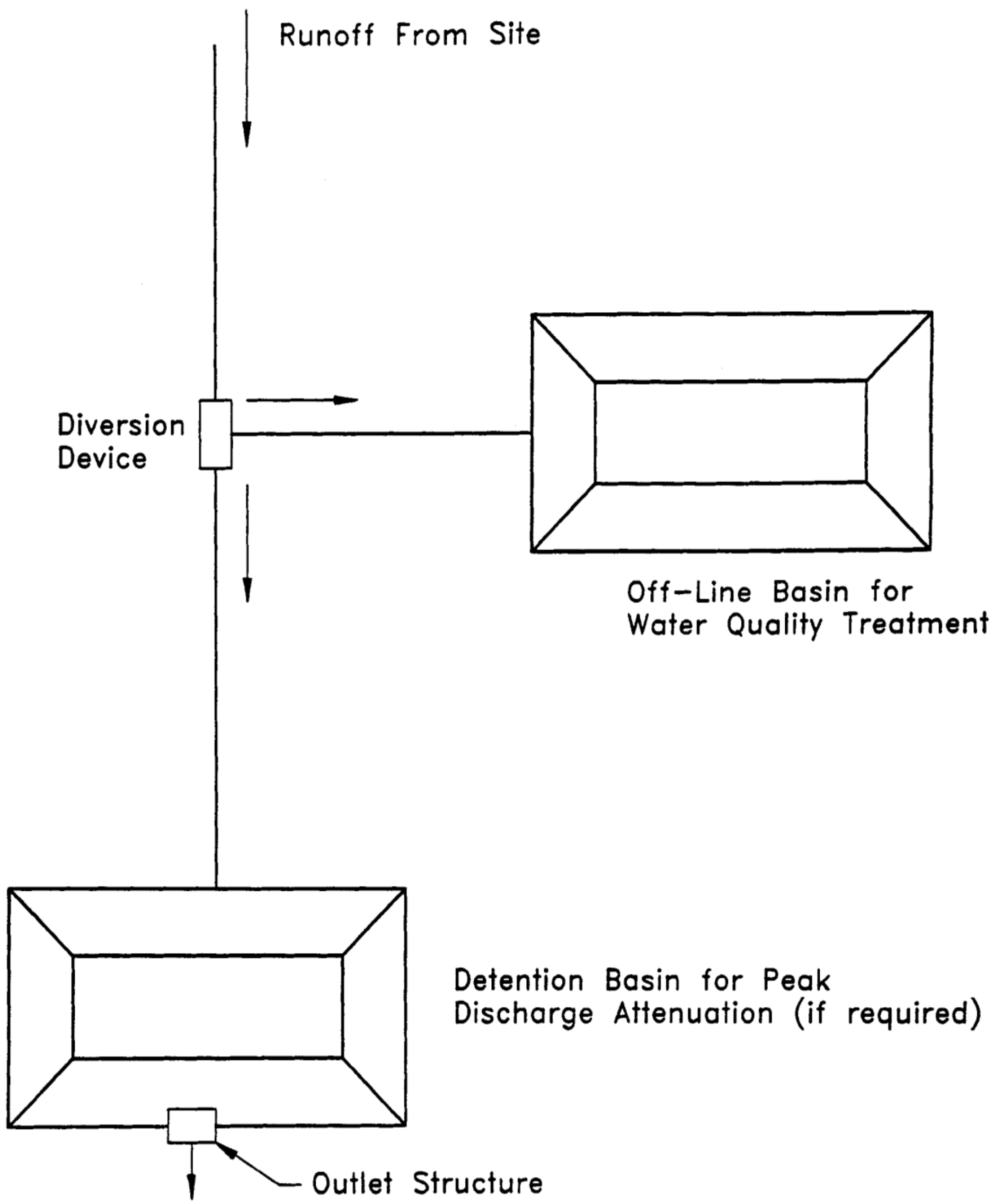

Figure 4.6-2 Off-line treatment system.

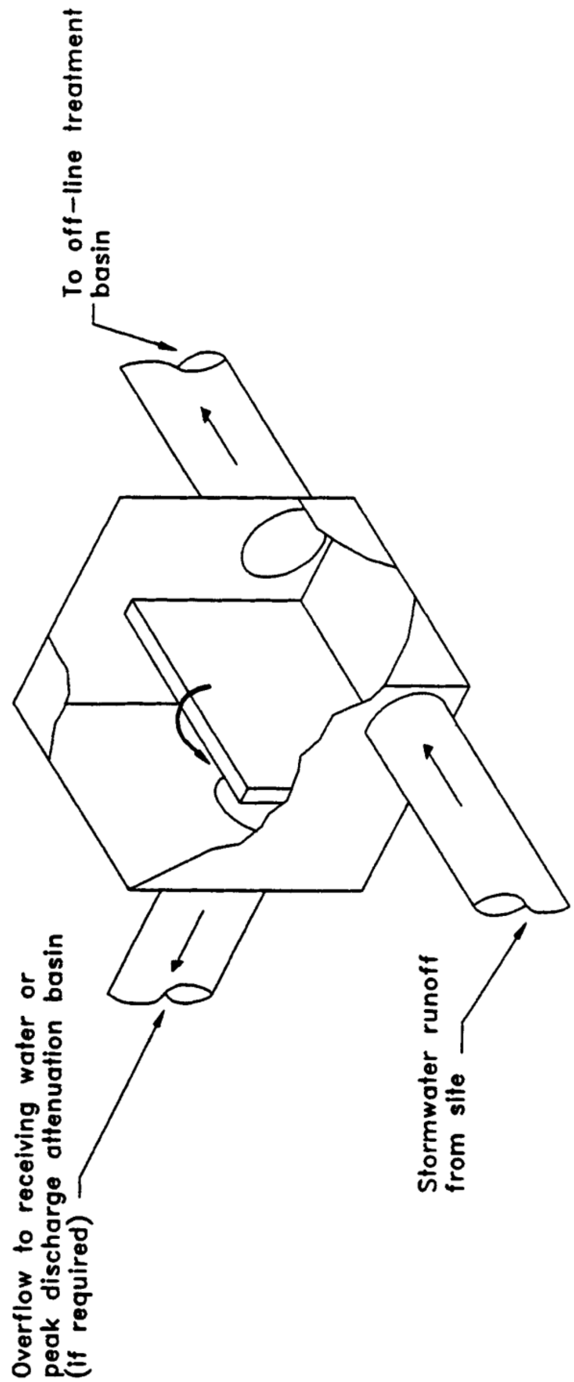

Figure 4.6-3 Diversion box (N.T.S.).

Off-line systems shall be designed to bypass essentially all additional stormwater runoff volumes greater than the required treatment volume to a discharge point or other detention storage area. There will be some incremental additional storage in the off-line system associated with the hydraulic grade line at the weir structure in the typical diversion structure. This will depend on the size of the weir, but the weir shall be sized to pass the design or excess flow with minimal headwater.

Proposed off-line systems that will also serve to provide significant detention storage above the required off-line treatment volume will be considered to function as on-line systems. These systems shall either be designed to meet on-line treatment volume requirements, or the registered professional must discuss the merits of the particular system (in terms of potential of flushing accumulated pollutants) with Agency staff in a pre-application conference.

4.7 Runoff Coefficient and Curve Number for Stormwater Management Ponds

Stormwater management ponds, including dry retention ponds, detention ponds with filtration, dry retention ponds with underdrains, and wet detention ponds, shall be considered as impervious area for calculating composite runoff coefficients (C), and composite curve numbers. This area is measured at the elevation of the required treatment volume.

4.8 Rural Subdivisions

Systems serving subdivisions with no more than five percent impervious area are considered a rural subdivision provided that:

(a) No drainage system shall act in a manner that would divert and channelize large areas of overland sheet flow, thereby creating point source discharges that will adversely affect wetlands, or beyond the applicant's perpetual control; and

(b) The applicant's demonstration of compliance with this subsection shall include provision of a typical lot layout showing proposed driveways, buildings, and other impervious areas and the anticipated percentage of impervious surfaces resulting from projected construction on individual residential lots.

Drainage areas from individual lots in rural subdivision are not required to provide treatment or attenuation of stormwater provided they are designed, constructed, and maintained in accordance with this Section. However, portions of individual lots that drain to a system that serves other activities such as roads, clubhouses, etc., must be included in the treatment and attenuation calculations for that system.

PART V — BEST MANAGEMENT PRACTICES

5.0 Design Criteria and Guidelines for Retention Systems

5.0.1 Description

The term “retention system” is defined as a storage area designed to store a defined quantity of runoff, allowing it to infiltrate through permeable soils into the shallow ground water aquifer. Stormwater retention works best using a variety of retention systems throughout the project site. Examples of retention systems include:

- Man-made or natural depressional areas where the pond bottom is graded as flat as possible and turf is established to promote infiltration and stabilize the pond slopes (see **Figure 5.0.1-1**);
- Shallow landscaped areas designed to store stormwater;
- Vegetated swales with swale blocks or raised inlets;
- Exfiltration systems or underground vaults; and
- Pervious concrete or pavement with continuous curb.

Soil permeability and water table conditions must be such that the retention system can infiltrate the desired runoff volume within a specified time following a storm event. After drawdown has been completed, the pond does not hold any water, thus the system is normally “dry.” Unlike detention systems, the treatment volume for retention systems is not discharged to surface waters.

Retention systems provide excellent removal of stormwater pollutants. Substantial amounts of suspended solids, oxygen demanding materials, heavy metals, bacteria, some varieties of pesticides and nutrients such as phosphorus are removed as runoff to infiltrates through the vegetation and soil profile.

Besides pollution control, retention systems can be utilized to promote the recharge of ground water to prevent saltwater intrusion in coastal areas or to maintain groundwater levels in aquifer recharge areas. Retention systems can also be used to help meet the runoff volume criteria for systems that discharge to closed basins or land-locked lakes (see **Section 3.3(b) of this Volume**).

There are several design and performance criteria specific to retention systems that are described below.

5.0.2 Treatment Volume

The required treatment volume necessary to achieve the treatment efficiency shall be routed to the retention pond and infiltrated into the ground. The required nutrient load reduction for the retention pond and, if necessary, associated BMPs in the BMP treatment train will be determined by the applicable performance standard as set forth in Section 8.3. of Volume I and methodology described in Section 9 of Volume I. Treatment volume shall be determined by the treatment efficiency.

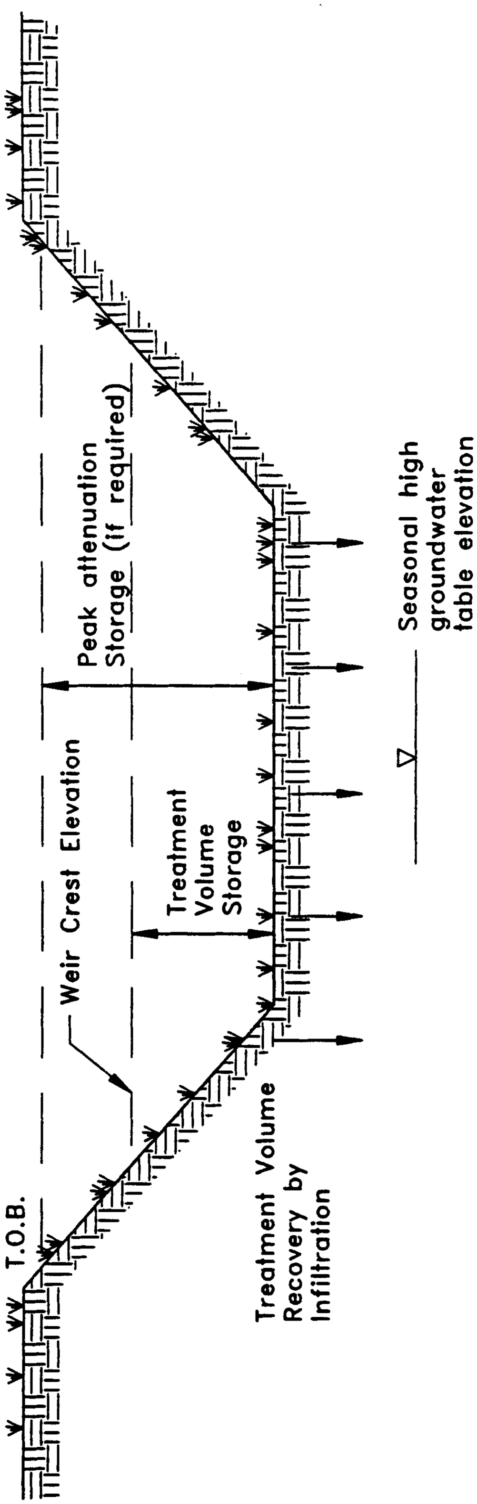

Figure 5.1-1 Typical retention system (N.T.S.).

5.0.3 Recovery Time

The retention system must provide the capacity for the appropriate treatment volume of stormwater specified in **Section 5.2 of this Volume** within 72 hours following a storm event assuming average antecedent moisture conditions. In retention systems, the stormwater is drawn down by natural soil infiltration and dissipation into the ground water table, evaporation, or evapotranspiration, as opposed to underdrain systems which rely on artificial methods like drainage pipes.

Antecedent moisture condition (AMC) refers to the amount of moisture and storage in the soil profile prior to a storm event. Antecedent soil moisture is an indicator of wetness and availability of soil to infiltrate water. The AMC can vary from dry to saturated depending on the amount of rainfall received prior to a given point in time. Therefore, "average AMC" means the soil is neither dry nor saturated, but at an average moisture condition at the beginning of a storm event when calculating recovery time for retention systems.

The antecedent condition has a significant effect on runoff rate, runoff volume, infiltration rate, and infiltration volume. The infiltration volume is also known as the upper soil zone storage. Both the infiltration rate and upper soil zone storage are used to calculate the recovery time of retention systems and shall be estimated using any generally accepted and well documented method with appropriate parameters consistent with such generally accepted and well documented method to reflect drainage practices, seasonal high water table elevation, consideration of groundwater mounding, the AMC, and any underlying soil characteristics which would limit or prevent infiltration of storm water into the soil column. **Section 1.3 of the Design Aids for Volume II** provides an accepted methodology for calculating the recovery time.

5.0.4 Pond Stabilization

The retention pond shall be stabilized with pervious material or permanent vegetative cover. To provide proper treatment of the runoff in very permeable soils, permanent vegetative cover must be utilized when U.S. Department of Agriculture Natural Resources Conservation Service (NRCS, SCS) hydrologic group "A" soils underlie the retention, except for pervious pavement systems.

5.0.5 Retention Construction

Retention pond construction procedures and the overall sequence of site construction are two key factors that can contribute to the effectiveness of retention ponds. Sub-standard construction methods or improper construction sequence can render the pond inoperable prior to completion of site development.

Since stormwater management systems typically are required to be constructed during the initial phases of site development, retention ponds are often exposed to poor quality surface runoff. Stormwater runoff during construction contains considerable amounts of suspended solids, organics, clays, silts, trash and other undesirable materials. For example, the subgrade stabilization material utilized during construction of roadways and pavement areas typically consist of clayey sand or soil cement. If a storm occurs when these materials are exposed (prior to placement of the roadway wearing surface), considerable amounts of these materials end up in the retention pond. Another source of fine material generated during construction is disturbed surface soil that can release large quantities of organics and other fine particles. Fine particles of clay, silt, and organics at the bottom of a retention pond create a poor infiltrating surface.

The following construction procedures are required to avoid degradation of retention pond infiltration capacity due to construction practices:

(a) Initially construct the retention pond to rough grade by under-excavating the pond bottom and sides by approximately 12 inches.

(b) After the drainage area contributing to the pond has been fully stabilized, the interior side slopes and pond bottom shall be excavated to final design specifications. The excess soil and undesirable material must be carefully excavated and removed from the pond so that all accumulated silts, clays, organics, and other fine sediment material has been removed from the pond area. The excavated material shall be disposed of in a manner so as to not cause or contribute to violations of water quality standards.

(c) Once the pond has been excavated to final grade, the entire pond bottom must be deep raked and loosened for optimal infiltration.

(d) Finally, the pond must be stabilized according the **Section 5.0.4 of this Volume**.

5.1 Exfiltration Trench Design and Performance Criteria

5.1.1 Description

An exfiltration trench is a subsurface system consisting of a conduit such as perforated pipe surrounded by natural or artificial aggregate which temporarily stores and infiltrates stormwater runoff (**Figure 5.1.1-1**). Stormwater passes through the perforated pipe and infiltrates through the trench walls and bottom into the shallow groundwater aquifer. The perforated pipe increases the storage available in the trench and helps promote infiltration by making delivery of the runoff more effective and evenly distributed over the length of the system. Generally, exfiltration trench systems are utilized where space is limited and/or land costs are high (i.e., downtown urban areas).

Soil permeability and water table conditions must be such that the trench system can infiltrate the required stormwater runoff treatment volume within a specified time following a storm event. The trench system is returned to a normally "dry" condition when drawdown of the treatment volume is completed. Like retention ponds, the treatment volume in exfiltration trench systems is not discharged to surface waters. Thus, exfiltration is considered a type of retention system.

Like other types of retention systems, exfiltration trench systems provide excellent removal of stormwater pollutants. Substantial amounts of suspended solids, oxygen demanding materials, heavy metals, bacteria, some varieties of pesticides and nutrients such as phosphorus are removed as runoff infiltrates through the soil profile. Exfiltration trench systems should not be located in close proximity to drinking water supply wells (see **Section 4.3.2 of this volume**).

Besides pollution control, exfiltration trench systems can be utilized to promote the recharge of ground water and to prevent saltwater intrusion in coastal areas, or to maintain groundwater levels in aquifer recharge areas. Exfiltration trench systems can also be used to help meet the runoff volume criteria for projects which discharge to land-locked lakes (see **Section 3.3(b) of this Volume**).

The operational life of an exfiltration trench is short (possibly 5 to 10 years) for most exfiltration systems. Sediment accumulation and clogging by fines can reduce the life of an exfiltration trench. Total replacement of the trench may be the only possible means of restoring the treatment capacity and recovery of the system. Periodic replacement of the trench should be considered routine operational maintenance when selecting this management practice.

There are several design and performance criteria which must be met in order for an exfiltration trench system to meet the rule requirements. A description of each criterion is presented below.

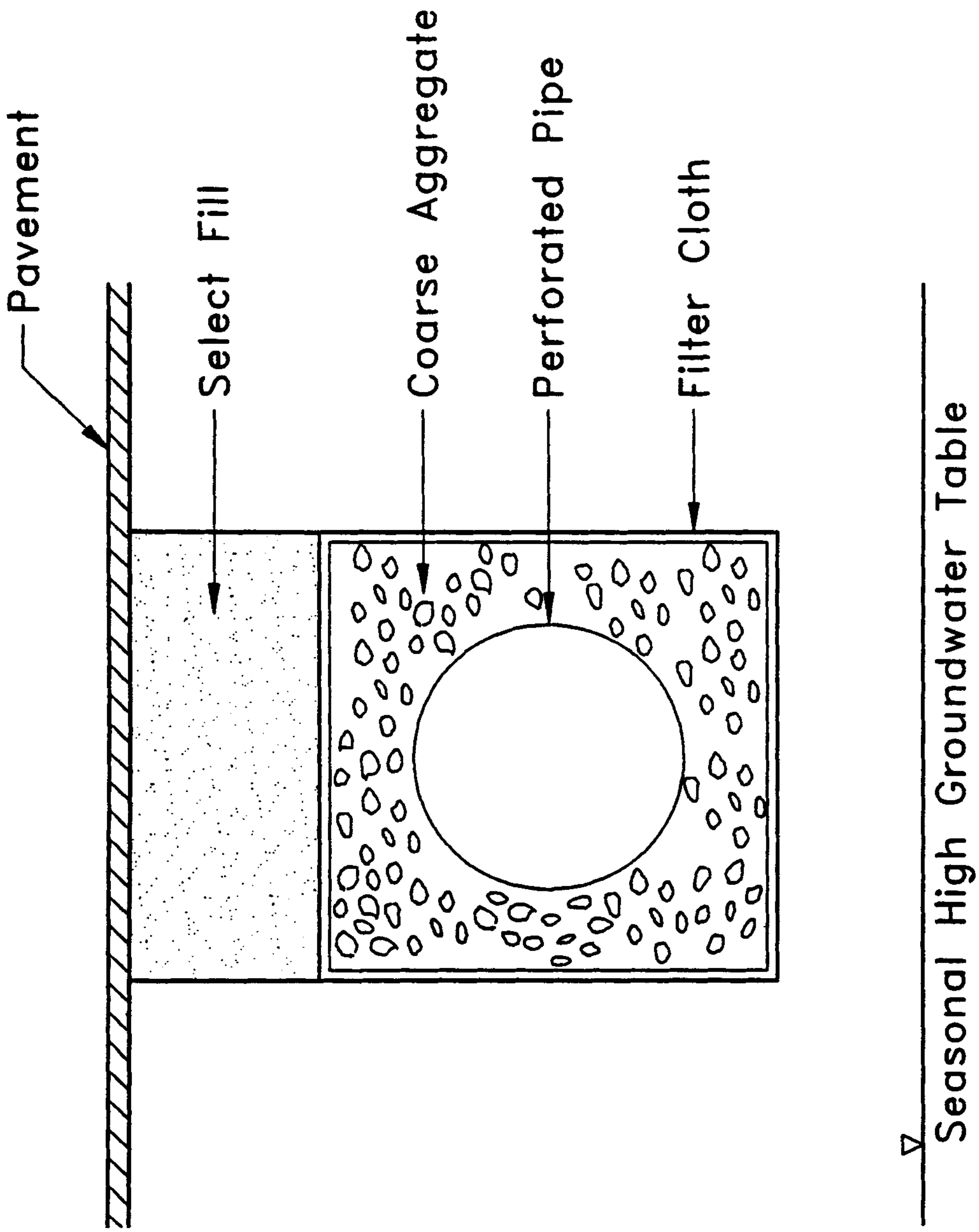

Figure 5.1.1-1 Cross-section of typical underground exfiltration trench (N.T.S.).

5.1.2 Treatment Volume

The required treatment volume necessary to achieve the treatment efficiency shall be routed to the exfiltration trench and infiltrate into the ground. The required nutrient load reduction for the exfiltration trench and, if necessary, associated BMPs in the BMP treatment train will be determined by the applicable performance standard as set forth in Section 8.3. of Volume I and methodology described in Section 9 of Volume I. Treatment volume shall be determined by the treatment efficiency.

5.1.3 Recovery Time

The system shall be designed to recover the required treatment volume of stormwater runoff within 72 hours following a storm event assuming average antecedent moisture conditions. The stormwater is drawn down by infiltration into the soil.

Antecedent moisture condition (AMC) refers to the amount of moisture and storage in the soil profile prior to a storm event. Antecedent soil moisture is an indicator of wetness and availability of soil to infiltrate water. The AMC can vary from dry to saturated depending on the amount of rainfall received prior to a given point in time. Therefore, “average AMC” means the soil is neither dry nor saturated, but at an average moisture condition at the beginning of a storm event when calculating recovery time for exfiltration systems.

The antecedent condition has a significant effect on runoff rate, runoff volume, infiltration rate, and infiltration volume. The infiltration volume is also known as the upper soil zone storage. Both the infiltration rate and upper soil zone storage are used to calculate the recovery time of retention systems and must be estimated using any generally accepted and well documented method with appropriate parameters consistent with such generally accepted and well documented method to reflect drainage practices, seasonal high water table elevation, the AMC, and any underlying soil characteristics which would limit or prevent infiltration of storm water into the soil column.

5.1.4 Safety Factor

The exfiltration trench system must be designed with a safety factor of at least two unless the applicant affirmatively demonstrates based on plans, test results, calculations or other information that a lower safety factor is appropriate for the specific site conditions. For example, two possible ways to apply this factor are:

(a) Reducing the design infiltration rate by half; and

(b) Designing for the required drawdown within 36 hours instead of 72 hours.

5.1.5 Minimum Dimensions

The perforated pipe shall be designed with a 2-inch minimum inside pipe diameter or hydraulic equivalent, and a 3-foot minimum trench width. The perforated pipe shall be located within the trench Section to minimize the accumulation of sediment in the aggregate void storage and maximize the preservation of this storage for stormwater treatment. To meet this goal, it is recommended that the perforated pipe be located at or within 6 inches of the trench bottom.

5.1.6 Filter Fabric

Exfiltration trench systems shall be designed so that aggregate in the trench is enclosed in filter fabric. This serves to prevent migration of fine materials from the surrounding soil that could result in clogging of the trench.

Alternatively, filter fabric may also be utilized directly surrounding the perforated pipe. In this instance, sedimentation of particulates will occur in the perforated pipe. Consequently, the pipe is more prone to clogging and reductions in capacity may occur more often than usual. However, the pipe may be cleaned relatively easy using high pressure hoses, vacuum systems, etc. On the other hand, designs without the fabric directly surrounding the perforated pipe requires complete replacement when clogging occurs.

5.1.7 Inspection and Cleanout Structures

Inspection and cleanout structures that extend exfiltration pipe to the surface of the ground shall be provided, at a minimum, at the inlet and terminus of each exfiltration pipe. Inlet structures shall include sediment sumps. These inspection and cleanout structures provide four primary functions:

(a) Observation of how quickly the trench recovers following a storm;

(b) Observation of how quickly the trench fills with sediment;

(c) Maintenance access to the perforated pipe; and

(d) Sediment control (sumps).

Standard precast concrete inlets and manholes are widely used to furnish the inspection and cleanout access.

5.1.8 Ground Water Table

The exfiltration trench system shall be designed so that the invert elevation of the trench is at least two feet above the seasonal high ground water table elevation unless the applicant affirmatively demonstrates based on plans, test results, calculations or other information that an alternative design is appropriate for the specific site conditions.

5.1.9 Construction

During construction, every effort should be made to limit the parent soil and debris from entering the trench. Any method used to reduce the amount of fines entering the exfiltration trench during construction will extend the life of the system. The use of an aggregate with minimal fines is also recommended.

5.2 Wet Detention Design and Performance Criteria

5.2.1 Description

Wet detention systems are permanently wet ponds which are designed to slowly release collected stormwater runoff through an outlet structure. A schematic of a typical wet detention system is shown in **Figure 5.2.1-1**.

Wet detention systems provide significant removal of both dissolved and suspended pollutants by taking advantage of physical, chemical, and biological processes within the pond. Wet detention systems offer an effective alternative for the long term control of water levels in the pond, provide a predictable recovery of storage volumes within the pond, and are easily maintained by the maintenance entity.

In addition to providing good removal of pollutants from runoff, wet detention systems also provide other benefits such as flood detention, passive recreation activities adjacent to ponds, and aesthetic amenities. As stormwater treatment systems, these ponds should not be designed to promote in-water recreation (i.e., swimming, fishing, and boating). To exclude such uses, measures such as fencing, signage, and other methods designed to prevent unauthorized pedestrian, vehicle, and boat access to the system shall be used.

There are several components in a wet detention system which must be properly designed to achieve the level of stormwater treatment required by Chapter 62-330, F.A.C. A description of each design feature and its importance to the treatment process is presented below. The design and performance criteria for wet detention systems are discussed below. A methodology for the design of wet detention systems is provided in **Section 3 of the References and Design Aids**.

5.2.2 Treatment Volume

The required nutrient load reduction for the wet pond and, if necessary, associated BMPs in the BMP treatment train will be determined by the applicable performance standard as set forth in Section 8.3. of Volume I and methodology described in Section 9 of Volume I. Treatment volume shall be determined by the treatment efficiency.

5.2.3 Recovery Time

The outfall structure shall be designed to drawdown one-half the required treatment volume between 48 and 60 hours.

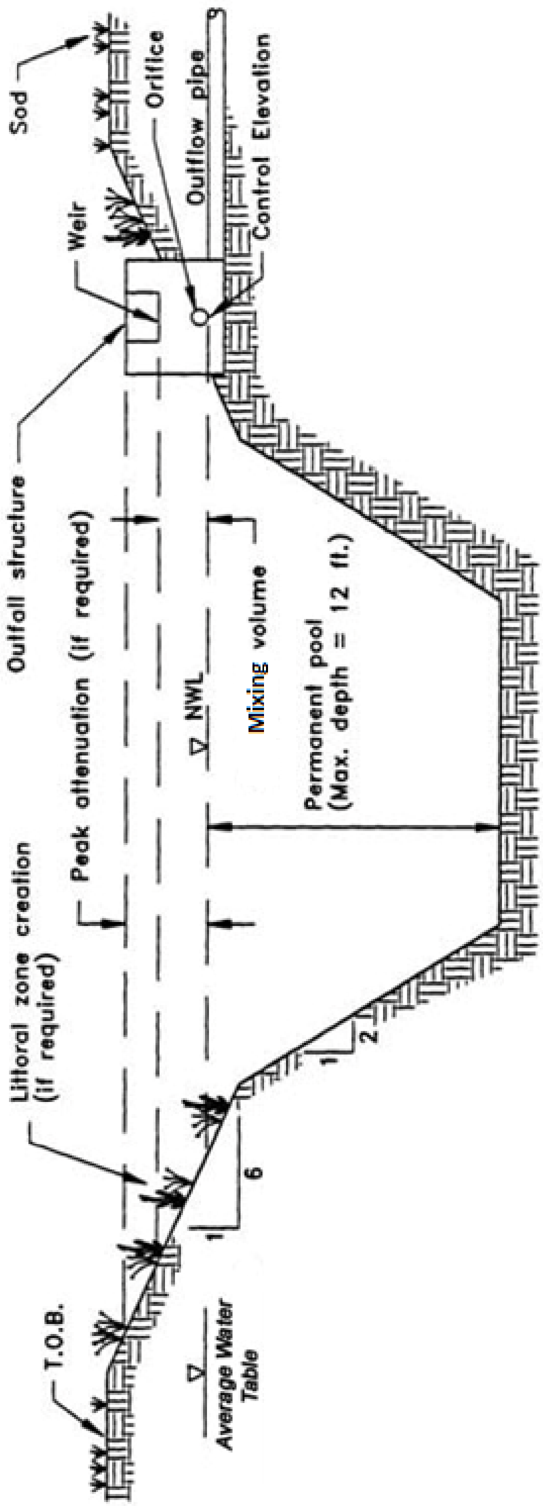

Figure 5.2.1-1 Typical wet detention system (N.T.S.).

5.2.4 Outlet Structure

The outlet structure generally includes a drawdown device (such as an orifice or "V" or square notch weir) set to establish a normal water control elevation and slowly release the treatment volume (see **Figures 5.2.4-1 and 5.2.4-2**). The design of the outfall structure must also accommodate the passage of flows from upstream stormwater management systems (see **Figure 5.2.4-3**).

The control elevation shall be set at or above the design tailwater elevation so the pond can effectively recover the treatment storage. Also, drawdown devices smaller than 3 inches minimum width or less than 20 degrees for "V" notches shall include a device to eliminate clogging. Examples of such devices include baffles, grates, screens, and pipe elbows.

5.2.5 Permanent Pool

A significant component and design criterion for the wet detention system is the storage capacity of the permanent pool (i.e., the **Section** of the pond that holds water at all times). The permanent pool shall be sized to provide at least a 21-day residence time based upon average wet season rainfall (rainfall occurring over the wettest four months of an average year; for Northwest Florida, these are June through September).

Important pollutant removal processes that occur within the permanent pool include: uptake of nutrients by algae, adsorption of nutrients and heavy metals onto bottom sediments, biological oxidation of organic materials, and sedimentation. Uptake by algae is probably the most important process for the removal of nutrients. Sedimentation and adsorption onto bottom sediments is likely the primary means of removing heavy metals.

The storage capacity of the permanent pool must be large enough to detain the untreated runoff long enough for the treatment processes described above to take place. Since one of the major biological mechanisms for pollutant removal in a wet detention pond is phytoplankton growth, the average hydraulic residence time of the pond must be long enough to ensure adequate algal growth.

5.2.6 Littoral Zone

The littoral zone is that portion of a wet detention pond which is designed to contain rooted aquatic plants. The littoral area is usually provided by extending and gently sloping the sides of the pond down to a depth of 2 to 3 feet below the normal water level or control elevation. Also, the littoral zone can be provided in other areas of the pond that have suitable depths (i.e., a shallow shelf in the middle of the lake). Littoral Zones are not required but can be used to increase the treatment efficiency of the wet pond system.

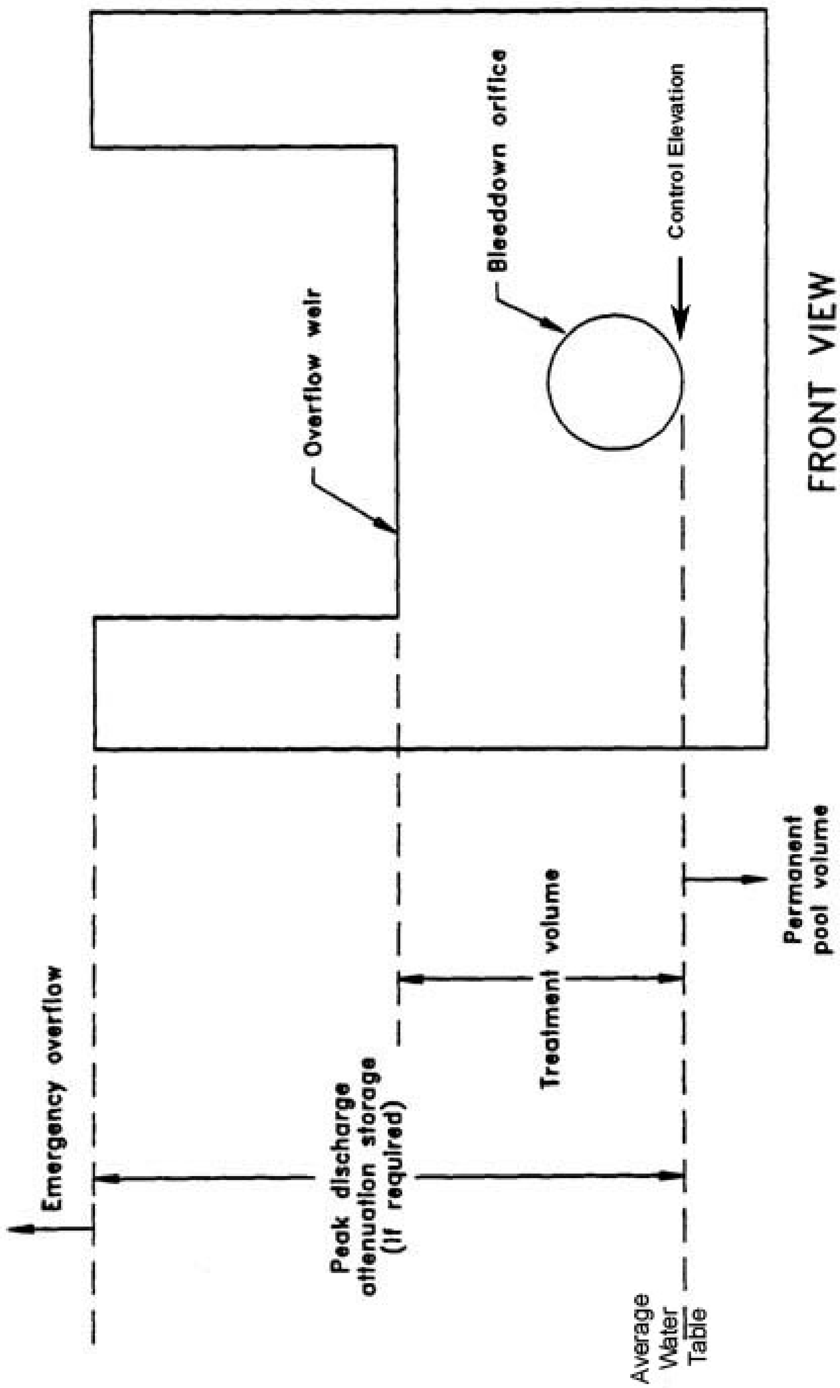

Figure 5.2.4-1 Typical wet detention outfall structure (N.T.S.).

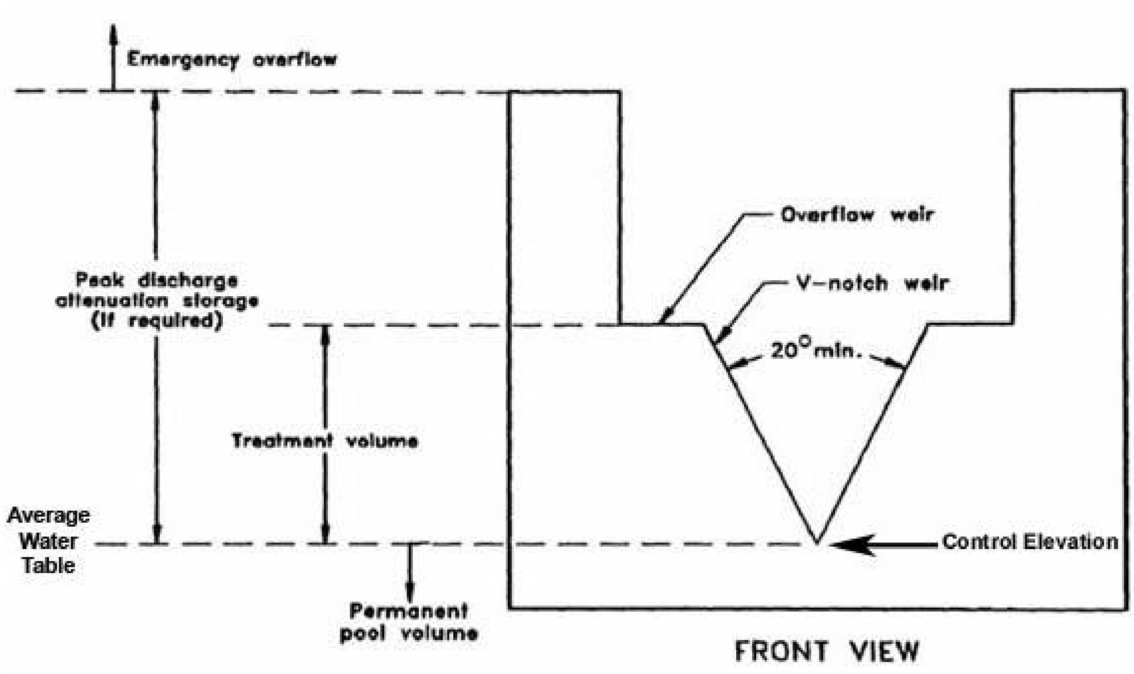

Figure 5.2.4-2 Typical wet detention outfall structure with "V"-notch weir (N.T.S.).

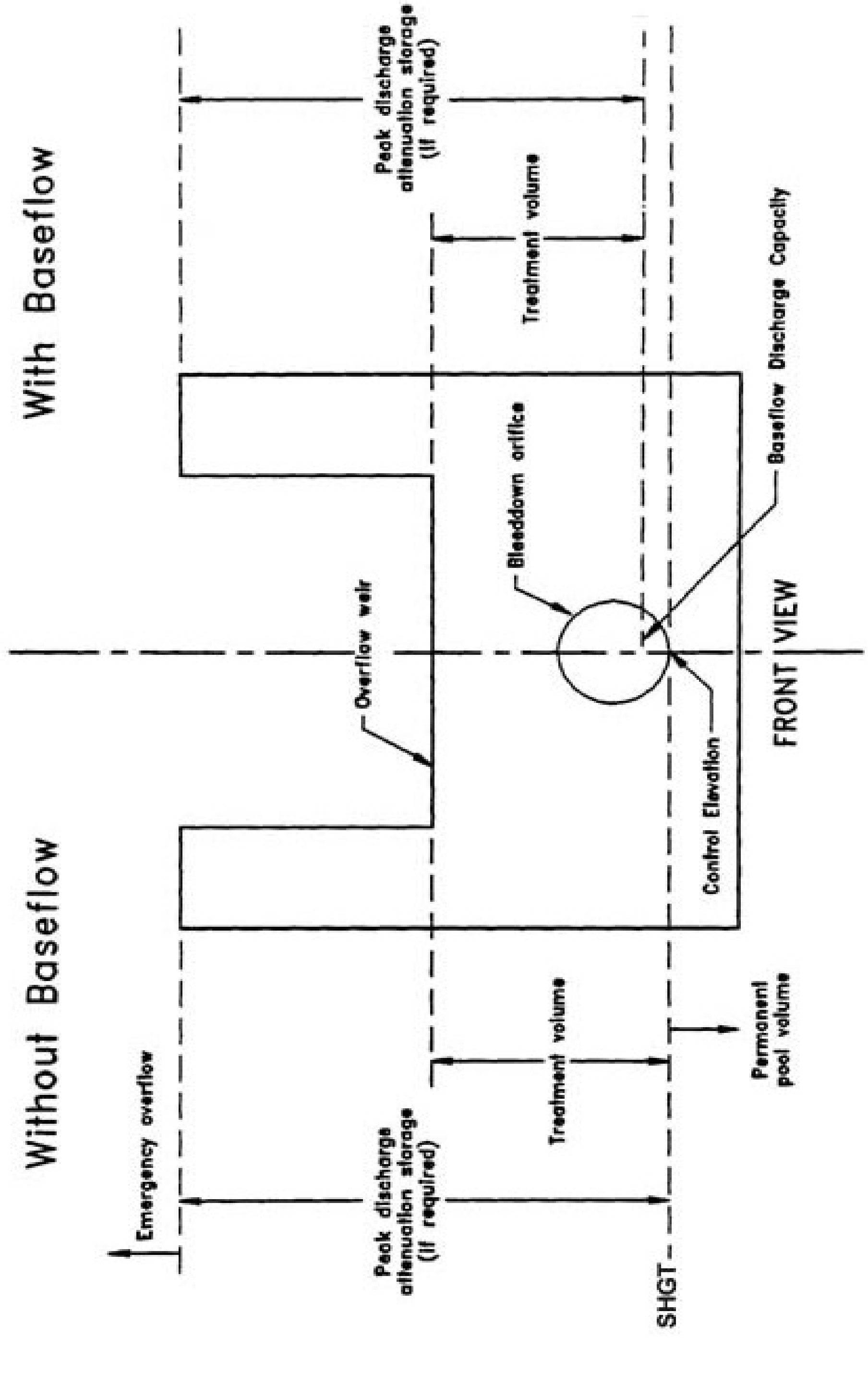

Figure 5.2.4-3 Typical wet detention outfall structure with and without baseflow conditions (N.T.S.).

The littoral zone is established with native aquatic plants by planting and/or the placement of wetland soils containing seeds of native aquatic plants. A specific vegetation establishment plan must be prepared for the littoral zone. The plan must consider the hydroperiod of the pond and the type of plants to be established. Additional information for a list of recommended native plant species is included in the **References and Design Aids for Volume II**. In addition, a layer of muck can be incorporated into the littoral area to promote the establishment of the wetland vegetation. When placing muck, special precautions must be taken to prevent erosion and turbidity problems in the pond and at its discharge point while vegetation is becoming established in the littoral zone.

The following is a list of the design criteria for wet detention littoral zones:

(a) The littoral zone shall be gently sloped (6:1 Horizontal:Vertical or flatter), and 30 to 40 percent of the wet detention pond surface area shall consist of a littoral zone. The percentage of littoral zone is based on the ratio of vegetated littoral zone to surface area of the pond at the control elevation.

(b) The treatment volume shall not cause the pond level to rise more than 18 inches above the control elevation unless the applicant affirmatively demonstrates that the littoral zone vegetation can survive at greater depths.

(c) Within 24 months of completion of the system, 80 percent coverage of the littoral zone by suitable aquatic plants is required.

(d) Planting of the littoral zone is recommended to meet the 80% coverage requirement. As an alternative to planting, portions of the littoral zone may be established by placement of wetland top soils (at least a four inch depth) containing a seed source of desirable native plants. When utilizing this alternative, the littoral zone must be stabilized by mulching or other means and at least the portion of the littoral zone within 25 feet of the inlet and outlet structures must be planted.

Routine custodial maintenance must be performed to remove nuisance or exotic plant species such as cattails (*Typha* spp.).

5.2.7 Pond Depth

A maximum pond depth shall not exceed 12 feet. Deeper ponds are allowable, provided the registered professional affirmatively demonstrates that any design for deeper pond depths will not cause stratification within the water column and will prevent resultant anoxic bottom waters and sediments. Many of the nutrients and metals removed from the water column accumulate in the top few inches of the pond bottom sediments. If a pond is deep enough, it will have a tendency to stratify, creating the potential for anoxic conditions developing at the bottom of the pond. An aerobic environment should be maintained throughout the water column in wet detention ponds in order to minimize the release of nutrients and metals from the bottom sediments. The maximum depth criteria minimizes the potential for significant thermal stratification which will help maintain aerobic conditions in the water column that should maximize sediment uptake and minimize sediment release of pollutants.

On the other hand, the minimum mean depth criteria minimizes aquatic plant growth which may be excessive if the pond is too shallow.

5.2.8 Pond Configuration

The average length to width ratio of the pond must be at least 2:1. It is important to maximize the flow path of water from the inlets to the outlet of the pond to promote good mixing (i.e., no dead spots). Under these design conditions, short circuiting is minimized and pollutant removal efficiency and mixing is maximized.

If short flow paths are unavoidable, the effective flow path can be increased by adding diversion barriers such as islands, peninsulas, or baffles to the pond. Inlet structures shall be designed to dissipate the energy of water entering the pond. Examples of good and poor pond configurations are given in **Figure 5.2.8-1**.

5.2.9 Ground Water Table

To minimize ground water contributions which may lower treatment efficiencies, the control elevation shall be set at or above the normal on-site ground water table elevation. This elevation may be determined by calculating the average of the seasonal high and seasonal low ground water table elevations. In areas where the seasonal low water table is not determinable, the applicant may propose using the seasonal high water table elevation minus one foot. The decision to use this alternative should be made by a professional with significant experience with and knowledge of the historic weather patterns and groundwater conditions of the local area. Regardless of which method is used, the system cannot cause adverse secondary impacts to adjacent wetlands or other surface waters, such as dewatering.

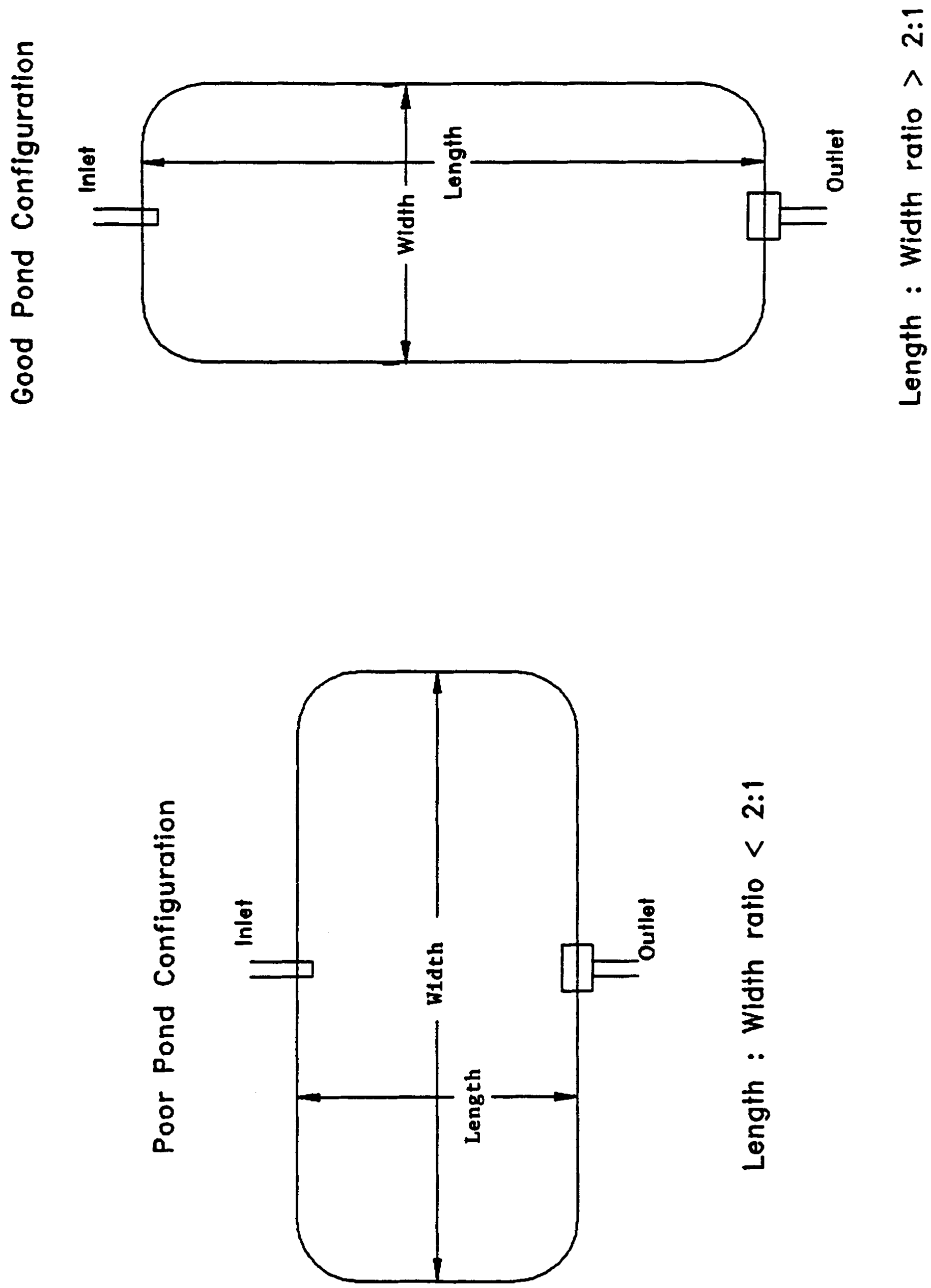

Figure 5.2.8-1 Examples of good and poor wet detention pond configurations (N.T.S.).

5.2.10 Treatment Train Nutrient Reduction

BMPs can be implemented in combination or in conjunction with one another in a series called a "BMP Treatment Train." If used, BMP Treatment Train efficiencies must account for the reduced loading transferred to subsequent downstream treatment devices. As stormwater pollutant concentrations are reduced in each BMP in the treatment train, the ability of a BMP Treatment Train to further reduce stormwater pollutant concentrations and loads is diminished. This is shown in Equation 9-5. This equation assumes each BMP acts independently of upstream BMPs and that upstream BMPs do not impact performance of downstream BMPs. If the BMP acts in combination with the upstream BMP, the designer will consider the use of another methodology to determine the resultant efficiency of the BMP Treatment Train.

Equation 9-5: Overall Treatment Train Efficiency for systems in series

$$\begin{aligned} &Overall\ Treatment\ Train\ Efficiency \\ &= Eff1 + [(1 - Eff1) x\ Eff2] + [(1 - (Eff1 + Eff2))\ x\ Eff3\ ] \end{aligned}$$

Eff1 = efficiency of initial treatment system
Eff2 = efficiency of second treatment system
Eff3 = efficiency of third treatment system

For developments where the appearance of the lake is important, pre-treatment a series of BMPs can reduce the chances of algal blooms and slow the eutrophication process. Some types of Green Stormwater Infrastructure or Low Impact Development practices include utilizing vegetative swales for conveyance instead of curb and gutter, perimeter swales or berms around the lake, oil and grease skimmers on inlet structures, retention storage in swales with raised inlets, or shallow landscaped retention areas (when soils and water table conditions will allow for adequate percolation).

5.2.10 Pond Side Slopes

The pond must be designed so that the pond side slope measured between the control elevation and two feet below the control elevation is no steeper than 4H:1V (horizontal: vertical). Because the pond sediments are an important component in the wet detention treatment processes, this criterion will ensure sufficient pond bottom/side slope area for the appropriate processes to occur. Littoral zone areas must be 6H:1V or flatter as described in **Section 5.2.6 of this Volume**.

5.3 Design Criteria for Swale Systems

When a stormwater management system relies in part on a swale to meet the conditions for issuance of Rule 62-330.301, F.A.C., and of this Volume, the following design criteria for swale systems apply.

5.3.1 Description

Swales are a man-made or natural system shaped or graded to required dimensions and designed for the conveyance and rapid infiltration of stormwater runoff. Swales are designed to infiltrate a defined quantity of runoff through the permeable soils of the swale floor and side slopes into the shallow ground water aquifer (**Figure 5.3.1-1**). Suitable vegetation is established to promote infiltration and stabilize the side slopes. Soil permeability and water table conditions must be such that the swale can infiltrate the desired runoff volume from the 3-year, 1-hour storm event. The swale holds water only during and immediately after a storm event, thus the system is normally "dry." Unlike retention ponds, swales are "open" conveyance systems. This means there are no physical barriers such as berms or check-dams to impound the runoff in the swale prior to discharge to the receiving water.

Swales provide excellent removal of stormwater pollutants. Substantial amounts of suspended solids, oxygen demanding materials, heavy metals, bacteria, some varieties of pesticides and nutrients such as phosphorus are removed as runoff infiltrates through the vegetation and soil profile.

Besides pollution control, swale systems can be utilized to promote the recharge of groundwater to prevent saltwater intrusion in coastal areas, and to maintain ground water levels in aquifer recharge areas. Swales can be incorporated into the design of a stormwater management system to help meet the runoff volume criteria for projects requiring permits under Chapter 62-330, F.A.C., which discharge to land-locked lakes (see **Section 3.3(b) of this Volume)**.

Swales can also be utilized as part of a treatment train to provide treatment of runoff prior to its release to another treatment BMP such as wet detention (see **Section 5.2.10 of this Volume**). Incorporating swales as part of a treatment train reduces the pollutant loading to the downstream treatment system, increases the pollutant efficiency of the overall stormwater management system, and reduces maintenance. In the case of wet detention systems, swales may be used to meet the performance standards set forth in Section 8.3 of Volume I. For developments where the appearance of the downstream system (i.e., wet detention lake) is important, swales can reduce the probability of algal blooms occurring and slows the eutrophication process.

The design and performance criteria specific to swale systems are described below.

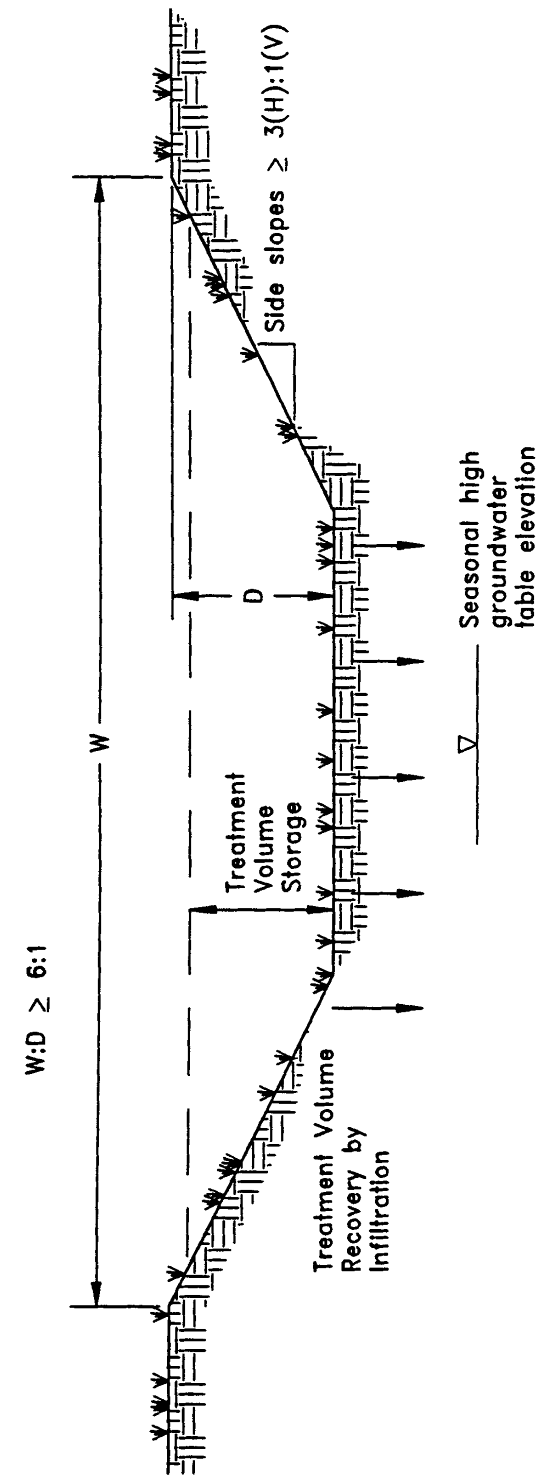

Figure 5.3.1-1 Cross-section of typical swale system (N.T.S.)

5.3.2 Treatment Volume

The required nutrient load reduction will be determined by the type of water body to which the swale and associated BMP treatment train discharges and the associated performance standard as set forth in Section 8.3 of Volume I. The treatment volume necessary to achieve the desired treatment efficiency shall be routed to the swale and associated BMP treatment train before discharge.

5.3.3 Soils Requirements

Swale systems must be constructed on Hydrologic Soils Group (HSG) A or B soils and swale system design shall consider antecedent moisture conditions.

Antecedent moisture condition (AMC) refers to the amount of moisture and storage in the soil profile prior to a storm event. Antecedent soil moisture is an indicator of wetness and availability of soil to infiltrate water. The AMC can vary from dry to saturated depending on the amount of rainfall received prior to a given point in time. Therefore, "average AMC" means the soil is neither dry nor saturated, but at an average moisture condition at the beginning of a storm event when calculating recovery time for swale systems.

The antecedent condition has a significant effect on runoff rate, runoff volume, infiltration rate, and infiltration volume. The infiltration volume is also known as the upper soil zone storage. Both the infiltration rate and upper soil zone storage are used to calculate the recovery time of retention systems and should be estimated using any generally accepted and well documented method with appropriate parameters to reflect drainage practices, seasonal high water table elevation, consideration of ground water mounding, the AMC, and any underlying soil characteristics which would limit or prevent infiltration of storm water into the soil column.

5.3.4 Dimensional Requirements

Swales must have a top width to depth ratio of the cross-section equal to or greater than 6:1 or side slopes equal to 3:1 (horizontal to vertical) or flatter.

5.3.5 Construction and Stabilization

Construction of swale systems must be in conformance with procedures that avoid degradation of swale infiltration capacity due to compaction and construction sedimentation. Construction of swale systems must conform to the construction practices in **Section 5.0.5 of this Volume** for dry retention ponds.

Swales shall be stabilized with vegetative cover suitable for soil stabilization, stormwater treatment, and nutrient uptake. Also, the swale shall be designed to take into account the soil erodibility, soil infiltration, slope, slope length, and drainage area so as to prevent erosion and reduce pollutant concentrations.

5.4 Design Criteria for Wetlands Stormwater Management Systems

5.4.1 Description

Wetlands are an essential part of nature's stormwater management system. Important wetland functions include the conveyance and storage of stormwater. These function to dampen flooding impacts; reduce flood flows and velocity of stormwater which in turn reduces erosion, increases sedimentation, and helps the assimilation of pollutants typically carried in stormwater. Accordingly, there is interest in the incorporation of natural wetlands into stormwater management systems, especially wetlands which have been previously drained. This concept provides an opportunity to use wetlands to help meet the requirements of this subsection. In addition, by using wetlands for stormwater management, drained wetlands can be revitalized and landowners and developers have greater incentive to preserve or restore wetlands.

For wetlands stormwater management systems the Agency must ensure that a proposed wetlands stormwater management system is compatible with the existing ecological characteristics of the wetlands proposed to be utilized for stormwater treatment. The Agency must also ensure that water quality standards will not be violated by discharges from wetlands stormwater management system. To achieve these goals, specific performance criteria are set forth herein and are described below for systems which incorporate wetlands for stormwater treatment.

5.4.2 Types of Wetlands that may be Utilized for Stormwater Treatment

The only wetlands which may be considered for use to provide stormwater treatment are those which:

(a) Are isolated and wholly-owned by one individual; or

(b) Are connected to other waters solely by artificial watercourses.

5.4.3 Treatment Volume

The required treatment volume necessary to achieve the treatment efficiency shall be routed to the wetland and infiltrate into the ground. The required nutrient load reduction for the wetland and, if necessary, associated BMPs in the BMP treatment train will be determined by the applicable performance standard as set forth in Section 8.3. of Volume I and methodology described in Section 9 of Volume I. Treatment volume shall be determined by the treatment efficiency.

5.4.4 Recovery Time

The system shall be designed to bleed down one-half the treatment volume specified above between 60 and 72 hours following a storm event, with the remainder bled down within 120 hours.

5.4.5 Inlet Structures

Inlet structures shall be designed to dissipate the energy of runoff entering the wetland and minimize the channelized flow of stormwater. Methods include design features such as sprinklers, pipe energy dissipators, overland flow, or spreader swales. Alternative designs may be proposed if they provide comparable reasonable assurance.

5.4.6 Wetland Function

Provisions must be made to remove sediment, oils and greases from runoff entering the wetland. This can be accomplished through incorporation of adjacent sediment sumps, forebays, baffles and dry vegetated swales or a combination thereof. Normally, a dry vegetated swale system designed for detention of the first one-fourth inch of runoff with an overall depth of no more than 4 inches will satisfy the requirement for removal of sediment, oils and greases. Additional BMP's can be utilized as part of a treatment train to attenuate stormwater volumes and peak discharge rates so that the wetland's hydroperiod is not adversely altered.

5.4.7 Residence Time

The design features of the system shall maximize residence time of the stormwater within the wetland to enhance the opportunity for the stormwater to come into contact with the wetland sediment, vegetation, and micro-organisms. This can be accomplished by several means. The inlets and outlets should be located to maximize the flow path through the wetland. Energy dissipators and spreader swales can promote overland flow and reduce the possibility of channelized flow occurring. In some instances, berms in wetlands can act as baffles to increase the flow path of surface flow through the wetland.

5.5 Design Criteria for Vegetated Natural Buffers

5.5.1 Description

Vegetated natural buffers (VNB) are defined as naturally vegetated areas that are set aside between developed areas and a receiving water or wetland for stormwater treatment purposes. Under certain conditions, VNBs are an effective best management practice for the control of nonpoint source pollutants in overland flow by providing opportunities for filtration, deposition, infiltration, absorption, adsorption, decomposition, and volatilization.

VNBs are most commonly used as an alternative to swales or berms installed between back-lots and the receiving water. Buffers are intended for use to avoid the difficulties associated with the construction and maintenance of backyard swales controlled by individual homeowners. Potential impacts to adjacent wetlands and upland natural areas are reduced because fill is not required to establish grades that direct stormwater flow from the back of the lot towards the front for collection in the primary stormwater management system. In addition, impacts are potentially reduced since buffer strips can serve as wildlife corridors, reduce noise, and reduce the potential for siltation into receiving waters.

Vegetative natural buffers are not to be the primary stormwater management system for residential developments. They are most commonly used only to treat those rear-lot portions of the development that cannot be feasibly routed to the system serving the roads and fronts of lots. A schematic of a typical VNB and its contributing area is presented in **Figure 5.5-1**.

The design and maintenance criteria for VNBs and their contributing areas are described in ~~**sections**~~ **Sections 5.5.2 through 5.5.9 of this Volume**.

5.5.2 Contributing Area

The contributing area is defined as the area that drains to the VNB.

Rear-lots of residential areas are allowed to contribute runoff to a VNB only if routing the runoff from such areas to the primary stormwater management system serving the development is not practical. The use of a VNB for other types of development shall only be allowed if the applicant demonstrates that there are no practical alternatives for those portions of the project, and only if the VNB and contributing areas meet all of the criteria of **Sections 5.5.2 through 5.5.9 of this Volume**.

To promote overland flow, the maximum width (dimension parallel to the flow direction) of the contributing area is 300 feet. The contributing area must be stabilized with permanent vegetative cover that is consistent with the Florida Yards and Neighborhood program. No fertilizer shall be applied to the contributing area.

Erosion control measures such as those described in **Part IV of Applicant's Handbook Volume I** must be utilized during development of the contributing area so as to prevent siltation of the buffer area.

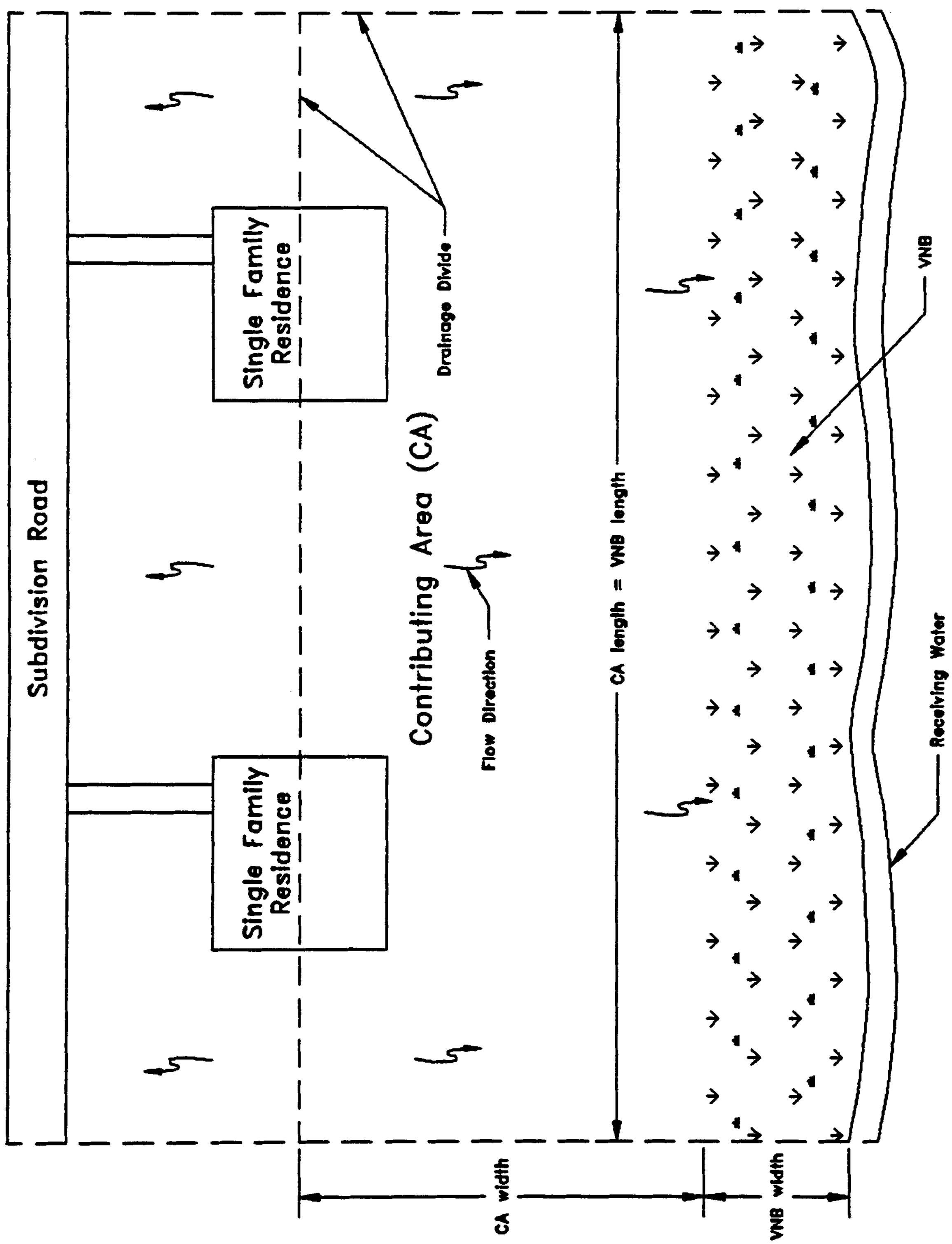

Figure 5.5-1 Plan View Schematic of Typical Vegetative Natural Buffer

5.5.3 Buffer Area Vegetation

The VNB area is an existing undeveloped area which contains naturally occurring native vegetation. The existing vegetation must not be disturbed during the development of the project.

5.5.4 Buffer Width

In all cases, a minimum buffer width of 25 feet is required to ensure the integrity of the treatment system. Factors affecting the minimum width (measured parallel to the direction of runoff flow) of VNBs include ground slope, rainfall, cover and soil characteristics, depth to water table and overland flow length. Infiltration is the primary means of treatment when soil characteristics and depth of ground water table promote infiltration. For sites with poor infiltration potential (i.e. hydrologic soil group C or D soils), pollutant removal occurs due to travel time across the buffer and is primarily a result of filtration and assimilation rather than infiltration. For design purposes, buffer widths shall be based upon the more conservative approach that utilizes a minimum travel time for overland flow.

Vegetated Natural Buffers must be designed to provide a specified travel time through the buffer as described herein. For systems that discharge to receiving water bodies other than OFWs, the VNB must be designed to provide at least 200 seconds of travel time by overland flow through the buffer for the 2-year, 24-hour storm event. Systems which directly discharge to OFWs must be designed to provide at least 300 seconds of travel time by overland flow through the buffer for the 2-year, 24-hour storm event.

A sample calculation for designing a buffer to meet the above requirements is provided in **Section 5 of the Volume II Design Aids**.

5.5.5 Maximum Buffer Slope

The maximum slope of VNB must not be greater than 15%.

5.5.6 Minimum Buffer Length

The length of the buffer (measured perpendicular to the runoff flow direction) must be at least as long as the length of the contributing runoff area (see **Figure 5.5-1**).

5.5.7 Runoff Flow Characteristics

Runoff from the adjacent contributing area must be evenly distributed across the buffer strip to promote overland flow. If channeling of the flow occurs, the buffer is effectively "short-circuited" and will not perform as designed.

5.5.8 Preservation and Maintenance Access

A legal reservation, in the form of an easement or other limitation of use, must be recorded which provides preservation of the existing undeveloped area in its natural state. The reservation must also include access for maintenance of the VNB unless the operation and maintenance entity wholly owns or retains ownership of the property. The legal reservation must include at least the entire area of the VNB. See **Section 12.4(b) ERP A.H. Vol. I** for additional maintenance access requirements.

5.5.9 Maintenance and Inspections

VNBs must be inspected annually by the operation and maintenance entity to determine if there has been any encroachment or violation of the terms and condition of the VNB as described below.

Buffers must be examined for damage by foot or vehicular traffic, encroachment, gully erosion, density of vegetation, and evidence of concentrated flow through or around the buffer. Repairs to the buffer must be made as soon as practical in order to prevent additional damage to the buffer. Repaired areas must be re-established with native vegetation. Invasive plant species such as cattail and primrose willow must be prevented from becoming the dominant species.

5.6 Design Criteria for Stormwater Harvesting

5.6.1 Description

On the average, and in most of the State of Florida, approximately 50% of the potable water delivered to residential units is used for irrigating lawns. The potable water used for irrigation may be supplemented with non-potable water from stormwater detention facilities

Stormwater harvesting systems are designed to prevent the discharge of a given volume of stormwater into surface waters of the state by deliberate application of stormwater runoff for irrigation or other acceptable supplemental water uses. Examples of areas that can be irrigated include golf courses, cemeteries, highway medians, parks, retail nurseries, agricultural lands, and residential and commercial properties. Supplemental uses include hydration of wetlands, low flow augmentation, cooling water, process water, and wash water.

A stormwater harvesting pond is similar to a wet detention system described in **Section ~~8~~5.2 of this Volume** except for the drawdown of the treatment volume storage. For typical wet detention ponds, the treatment volume is released at a controlled rate by a drawdown orifice or weir. However, in a stormwater harvesting system the drawdown structure is replaced by a mechanical harvesting system which recovers the treatment volume storage by withdrawing water from the pond. In a harvesting pond the treatment volume is termed "harvesting volume" and the "control elevation" is the lowest elevation at which water can be withdrawn from the pond by the harvesting system. Like wet detention, stormwater harvesting systems are a recommended BMP for sites with moderate to high ground water table conditions. A schematic a typical harvesting pond is shown in **Figure 5.6 -1**.

The Agency encourages the use of stormwater harvesting systems because of the following benefits they provide:

(a) Reduction of runoff volume discharged to the receiving waters;

(b) Reduction of pollutants discharged to the receiving waters;

(c) Substitution of stormwater use instead of potable ground water withdrawals; and

(d) Potential economic savings from not having to pay user fees for potable water.

Stormwater harvesting systems provide significant removal of both dissolved and suspended pollutants by taking advantage of physical, chemical, and biological processes associated with wet detention systems and the harvesting and recycling of constituents back to the landscape by irrigation with stormwater. Harvesting systems can be utilized to help meet the runoff volume criteria for stormwater management systems and management and storage of surface water (MSSW) projects which discharge to land-locked lakes.

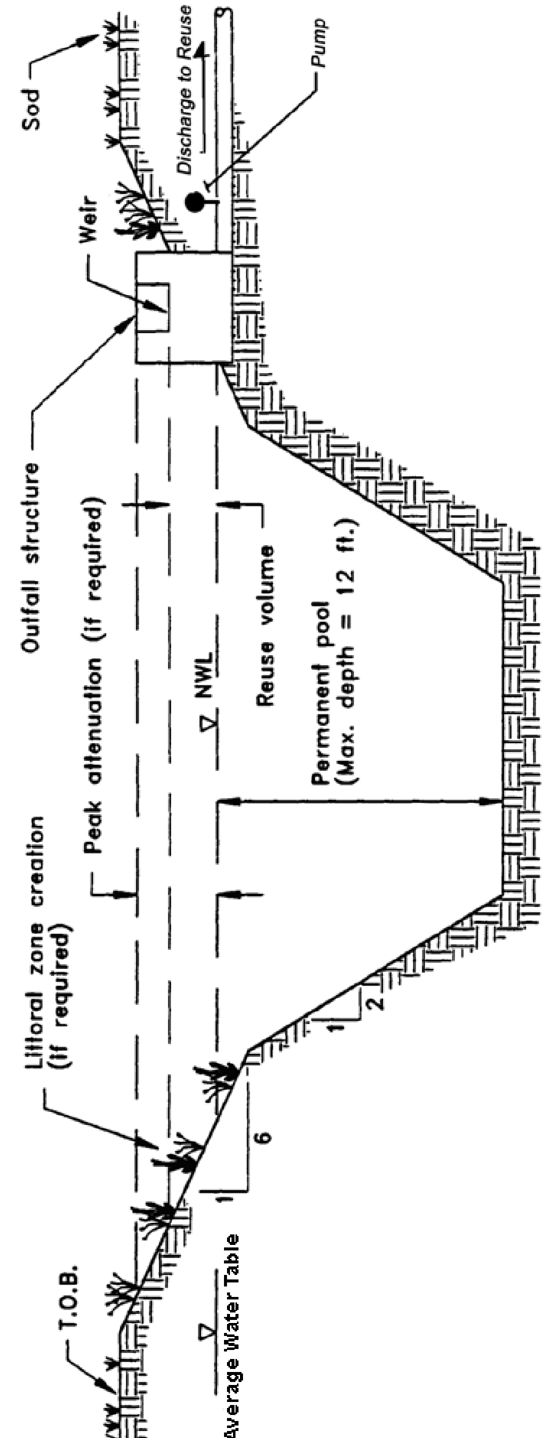

Figure 5.6 -1 Typical stormwater harvesting system (N.T.S.).

In addition, stormwater harvesting ponds also provide other benefits such as flood detention, recreation activities adjacent to ponds, and pleasing aesthetics. As stormwater treatment systems, these ponds must not be designed to promote in-water recreation (i.e., swimming, fishing, and boating).

There are several components in a stormwater harvesting system which must be properly designed to achieve the level of stormwater treatment required by Chapter 62-330, F.A.C. A description of each design feature and its importance to the treatment process is presented below. These criteria are not intended to preclude the harvesting of stormwater from other types of stormwater management systems such as wet detention. Several of these criteria are the same as those for wet detention systems as described in **Section 5.2 of this Volume**.

5.6.2 Harvesting Volume

The required nutrient load reduction will be determined by type of water body to which the BMP treatment train that includes stormwater harvesting discharges and the associated performance standard as set forth in Section 8.3 of Volume I.

5.6.3 Permanent Pool

The permanent pool is that portion of a pond which is designed to hold water at all times (i.e., below the control elevation). The permanent pool shall be sized to provide at least a 21-day residence time during the wet season (June through September).

5.6.4 Littoral Zone

The littoral zone is that portion of a stormwater harvesting pond which is designed to contain rooted aquatic plants. The littoral area is usually provided by extending and gently sloping the sides of the pond down to a depth of 2 to 3 feet below the normal water level or control elevation. Also, the littoral zone can be provided in other areas of the pond that have suitable depths (i.e., a shallow shelf in the middle of the lake).

The littoral zone is established with native aquatic plants by planting and/or the placement of wetland soils containing seeds of native aquatic plants. A specific vegetation establishment plan must be prepared for the littoral zone. The plan must consider the hydroperiod of the pond and the type of plants to be established. Additional information for a list of recommended native plant species is included in the **References and Design Aids for Volume II for wet detention**. In addition, a layer of muck can be incorporated into the littoral area to promote the establishment of the wetland vegetation. When placing muck, precautions must be taken to prevent erosion and turbidity problems in the pond and at its discharge point while vegetation is becoming established in the littoral zone.

The following is a list of the design criteria for stormwater harvesting littoral zones:

(a) The littoral zone shall be gently sloped (6H:1V or flatter). Thirty to forty percent of the stormwater harvesting pond surface area shall consist of a littoral zone. The percentage of littoral zone is based on the ratio of vegetated littoral zone to surface area of the pond at the control elevation.

(b) The treatment volume shall not cause the pond level to rise more than 18 inches above the control elevation unless the applicant provides reasonable assurance that the littoral zone vegetation can survive at greater depths.

(c) Within 24 months of completion of the system, 80 percent coverage of the littoral zone by suitable aquatic plants is required.

(d) Planting of the littoral zone is recommended to meet the 80% coverage requirement. As an alternative to planting, portions of the littoral zone may be established by placement of wetland top soils (at least a four inch depth) containing a seed source of desirable native plants. When utilizing this alternative, the littoral zone must be stabilized by mulching or other means and at least the portion of the littoral zone within 25 feet of the inlet and outlet structures must be planted.

5.6.5 Pond Depth

A maximum pond depth shall not exceed 12 feet. Deeper ponds are allowable, provided the registered professional affirmatively demonstrates that any design for deeper pond depths will not cause stratification within the water column and will prevent resultant anoxic bottom waters and sediments. Many of the nutrients and metals removed from the water column accumulate in the top few inches of the pond bottom sediments. If a pond is deep enough, it will have a tendency to stratify, creating the potential for anoxic conditions developing at the bottom of the pond. An aerobic environment should be maintained throughout the water column in wet detention ponds in order to minimize the release of nutrients and metals from the bottom sediments. The maximum depth criteria minimizes the potential for significant thermal stratification which will help maintain aerobic conditions in the water column that should maximize sediment uptake and minimize sediment release of pollutants.

5.6.6 Pond Configuration

The average length to width ratio of the pond should be at least 2:1. If short flow paths are unavoidable, the effective flow path can be increased by adding diversion barriers such as islands, peninsulas, or baffles to the pond. Inlet structures shall be designed to dissipate the energy of water entering the pond.

5.6.7 Ground Water Table

To minimize ground water contributions which may lower treatment efficiencies, the control elevation should be set at or above the normal on-site ground water table elevation. This elevation may be determined by calculating the average of the seasonal high and seasonal low ground water table elevations. In areas where the seasonal low water table is not determinable, the applicant may propose using the seasonal high water table elevation minus one foot. The decision to use this alternative should be made by a professional with significant history and knowledge of the local areas historic weather patterns and groundwater conditions. Regardless of which method is used, the system cannot cause adverse secondary impacts to adjacent wetlands or other surface waters such as dewatering.

Part VI Special basin Criteria in Northwest Florida

6.0 Sensitive Karst Areas in Northwest Florida

Subparagraph 62-330.301(1)(k)1., F.A.C., provides that a condition for issuance of a permit includes compliance with any applicable special basin or geographic area criteria rules. The only area within the geographical extent of the Northwest Florida Water Management District (NWFWMD) for which additional geographic criteria have been developed are two Sensitive Karst Areas (SKAs). These areas cover portions of the central and eastern regions of the geographical extent of the NWFWMD **(**see **Figure 6.1.-1).** A location description of these areas is contained in **Appendix A of this Volume.** In addition to the design criteria for projects outside of the SKAs, projects located within the SKAs also must meet the additional design criteria of **Sections 6.3 of this Volume**.

6.1 Background of the Sensitive Karst Area Design Criteria

The Floridan Aquifer System is the drinking water source for most of the population in the geographical extent of the NWFWMD. In parts of the NWFWMD, limestone (or dolostone) that makes up or comprise this aquifer system occurs at or near the land surface. Sediments overlying the limestone can be highly permeable. The limestone, due to its chemical composition, is susceptible to dissolution when it interacts with slightly acidic water. “Karst” is a geologic term used to describe areas where landscapes have been affected by the dissolution of limestone or dolostone, including areas where the formation of sinkholes is relatively common. Sensitive Karst Areas reflect areas with hydrogeologic and geologic characteristics relatively more conducive to potential contamination of the Floridan Aquifer System from surface pollutant sources. The formation of karst-related features, such as sinkholes is also more likely to occur in SKAs.

6.2 Hydrogeology of the Sensitive Karst Areas

Throughout the majority of the geographical extent of the NWFWMD the highly porous limestone that comprises the Floridan Aquifer System is generally overlain by tens to hundreds of feet of sands, clays, and other material. Where present, this material may act to protect, to varying degrees, the Floridan Aquifer System from surface pollutants. Surface water seeps through this material slowly, which allows for some degree of filtration, adsorption, and biological transformation or degradation of contaminants.

In SKAs, however, the limestone that comprises the Floridan Aquifer System may occur at or near the land surface (**Figure 6.2-1**), and sand overburden, confining clays, or other confining cover material is absent or discontinuous. As a result, there can be rapid movement of surface water and possibly entrained contaminants into the aquifer. The SKAs are areas of relatively high recharge to the Floridan Aquifer System. Floridan Aquifer System ground water levels vary from land surface to approximately 290 feet below land surface in the SKAs.

One factor that makes the SKAs particularly prone to stormwater contamination is the formation of solution pipe sinkholes within retention ponds. Solution pipe sinkholes are common in these areas and form due to the collapse of surficial material into vertical cavities that have been dissolved in the upper part of the limestone (**Figure 6.2-2**). They are also formed by the movement of surface material into the underlying porous limestone. In most cases, the solution pipes are capped by a natural plug of sands and clays (**Figures 6.2-1 and 6.2-2**). If the cap is washed out (as may happen if a large volume of water is stored over the solution pipes), the resulting solution pipe

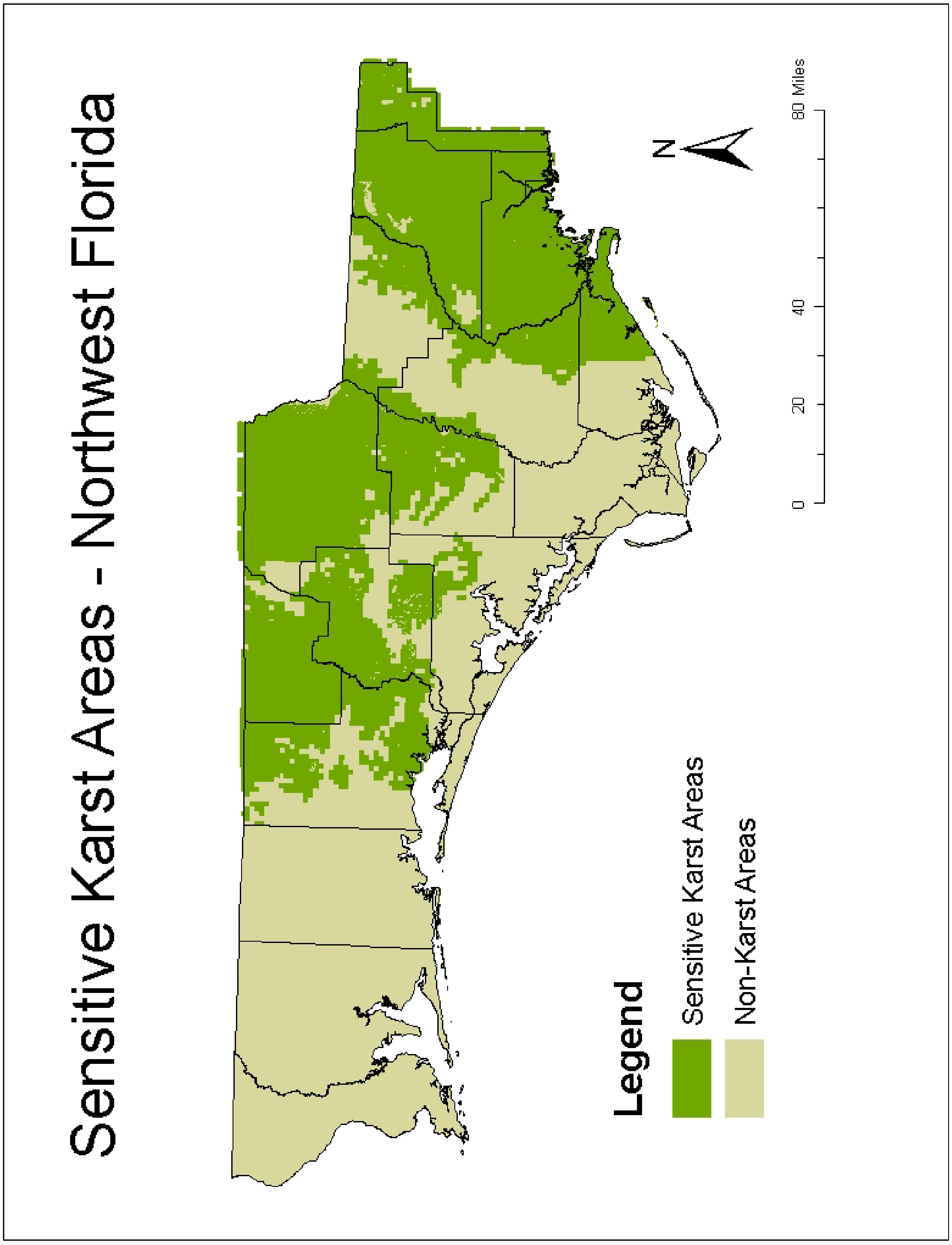

Figure 6.0-1 Sensitive Karst Areas within the NWFWMD

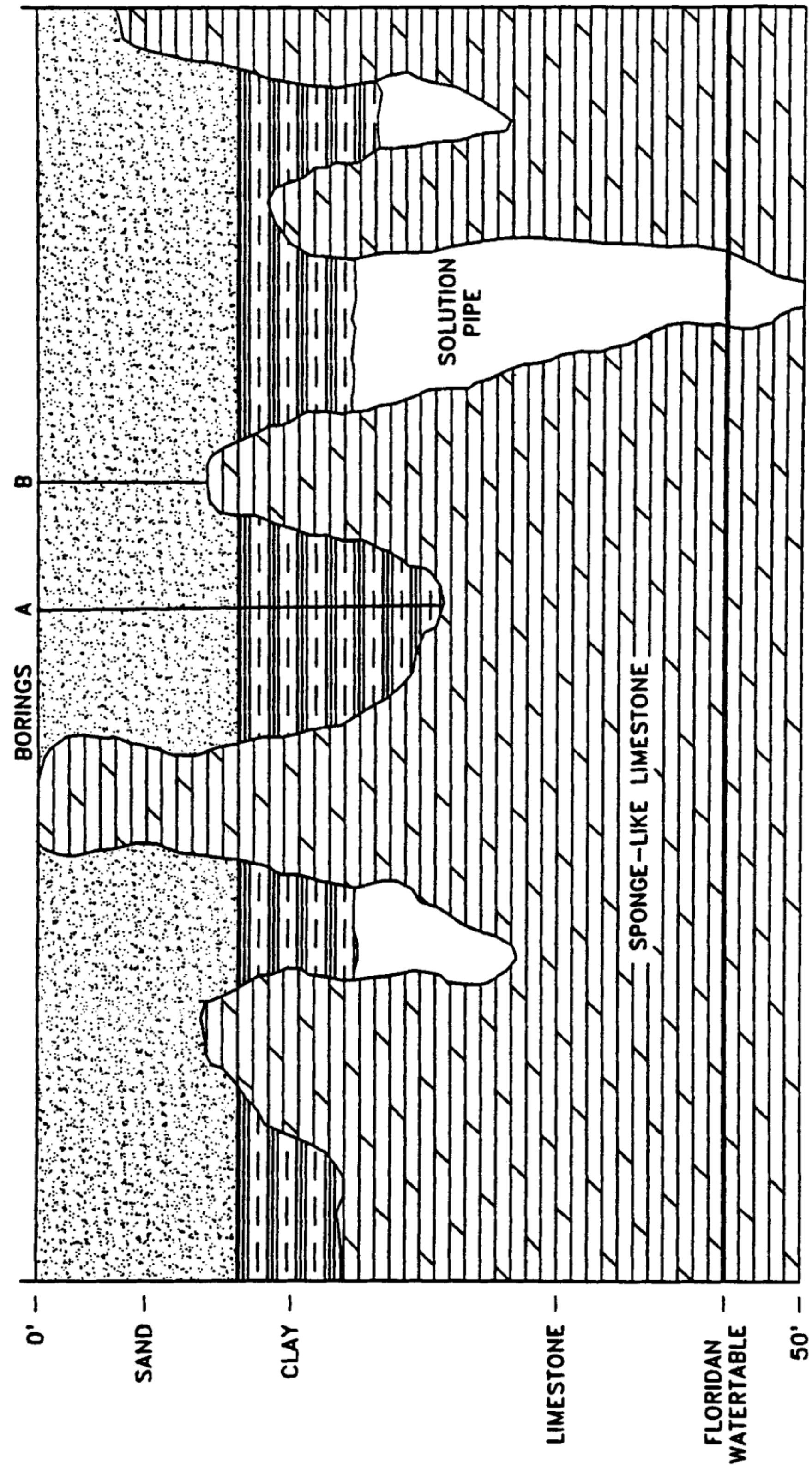

Figure 6.2-1 Generalized geologic section in Sensitive Karst Area with limestone at and near land surface.

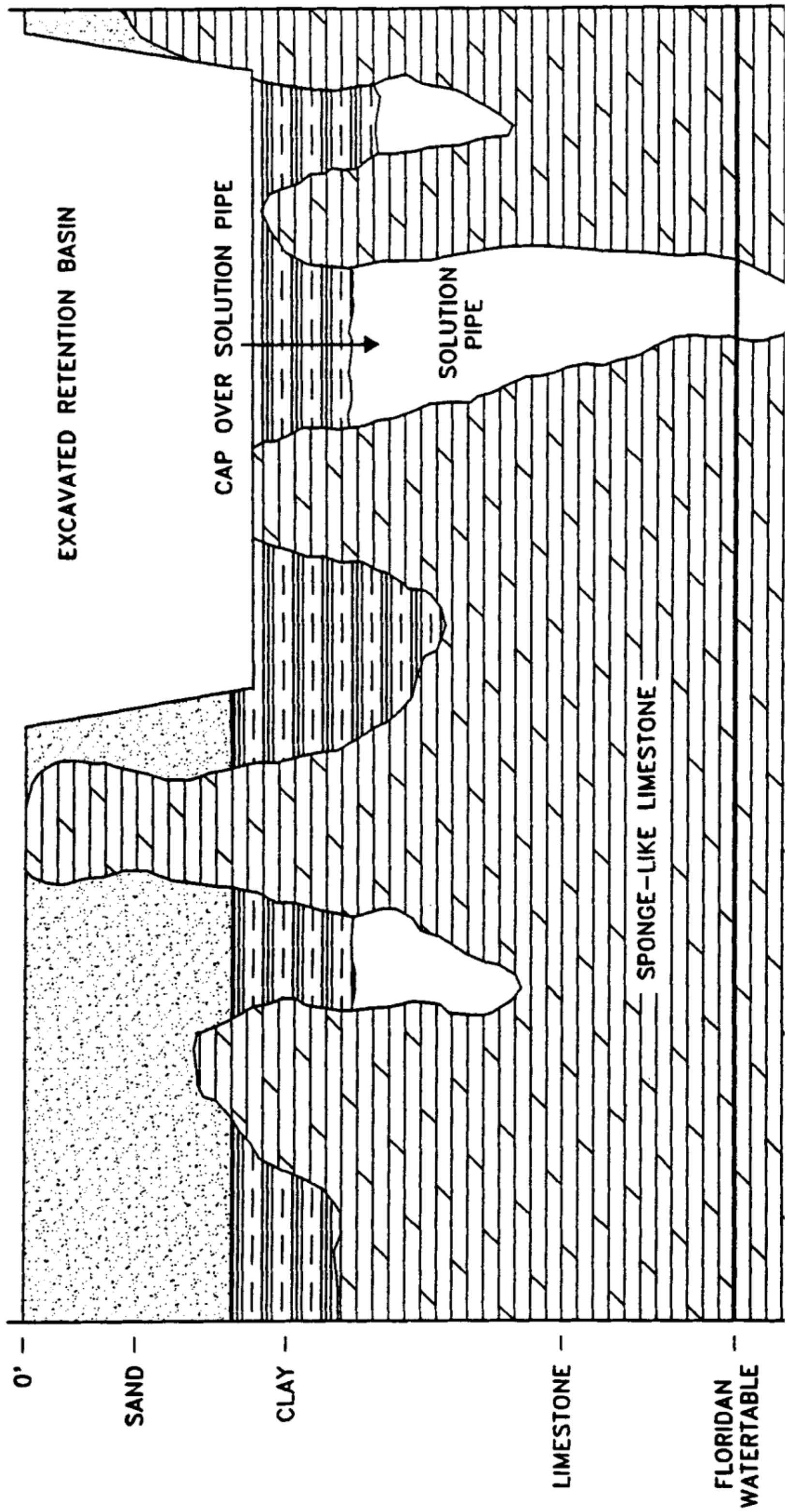

Figure 6.2-2 Retention pond added to Figure 6.2-1.

sinkhole (**Figure 6.2-3**) can act as a direct pathway for the movement of surface water into the Floridan Aquifer System.

Solution pipe sinkholes and other types of sinkholes may open in the bottom of stormwater retention ponds. The capping plug or sediment fill may be reduced by excavation of the pond. Stormwater in the pond may increase the hydraulic head on the remaining material in the pipe throat. Both of these factors can wash material down the solution pipe. Solution pipes act as natural drainage wells and can drain stormwater ponds.

The irregular weathering of the limestone surface in the SKAs contributes to uncertainty and errors in predicting the depth from land surface to limestone. For example, in **Figure 6.2-1**, boring A would show limestone much deeper than it would actually be encountered during excavation, shown at boring B. This potential for error must be considered for site investigations when evaluating site borings, and load-specific geological analyses must be included to base site designs.

6.3 Additional Design Criteria for Sensitive Karst Areas

6.3.1 Stormwater management systems shall be designed and constructed to prevent direct discharge of untreated stormwater into the Floridan Aquifer System. Such stormwater management systems also shall be designed and constructed in a manner that avoids breaching an aquitard and such that construction excavation will not allow direct mixing of untreated water between surface waters and the Floridan Aquifer System. The system shall also be designed to prevent the formation of solution pipes or other types of karst features in the SKAs. Test borings located within the footprint of a proposed stormwater management pond must be plugged in a manner to prevent mixing of surface and ground waters.

6.3.2 Except as provided in **Section 6.3.5 of this Volume**, systems that are designed as follows are presumed to comply with **Section 6.3.1 of this Volume**:

(a) A minimum of three feet of unconsolidated sediment or soil material between the surface of the limestone bedrock and the complete extent of the bottom and sides of the stormwater pond at final completion of the project. Excavation and backfill of unconsolidated sediment or soil material shall be conducted, if necessary to meet these criteria. As an alternative, an impermeable liner can be used to ensure that stormwater is isolated from communication with groundwater (e.g., for wet detention). This provision is presumed to provide reasonable assurance of adequate treatment of stormwater before it enters the Floridan Aquifer System;

(b) To reduce the potential for solution pipe sinkhole formation caused by newly created additional hydraulic head conditions, stormwater storage areas are limited to a maximum of 10 feet of vertical staging (shallower depths are encouraged), as measured for dry ponds from the bottom of the pond to the design high water level; and for wet ponds 10 feet of vertical staging as measured from the seasonal high ground water table to the design high water level, and shall have a horizontal bottom (no deep spots); and

(c) Pond side slopes and bottom (if not a wet pond) must be fully vegetated or otherwise stabilized.

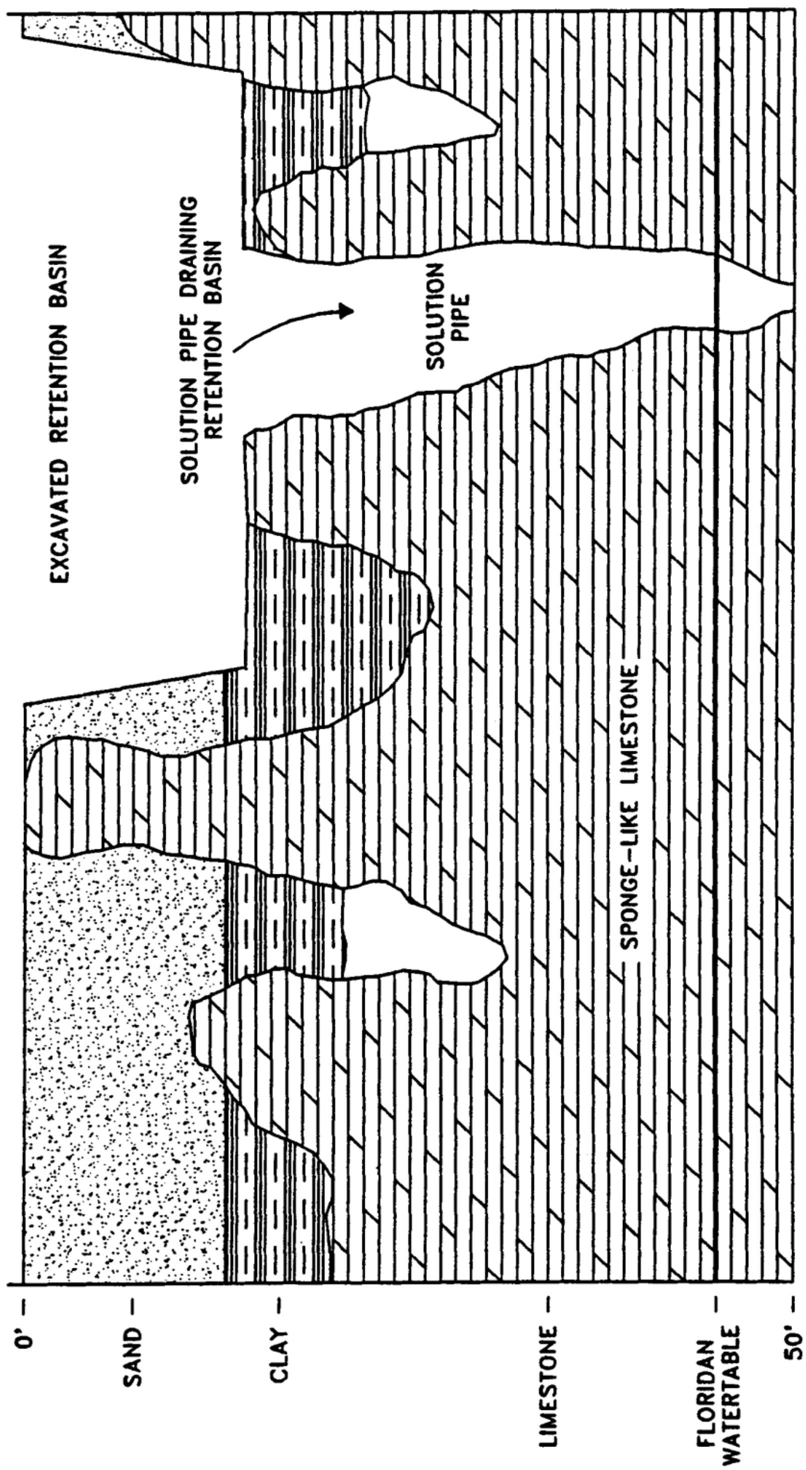

Figure 6.2-3. Potential sinkhole resulting from change in physical conditions due to constructed retention pond depicted in Figure 6.2-2.

6.3.3 Applicants who believe that their proposed system is not within the influence of a karst feature, notwithstanding that it is within the SKAs designated by **Figure 6.0-1** and **Appendix A of this Volume**, and therefore wish to design their system other than as provided in **Section 6.3.2 of this Volume,** shall furnish the Agency with reasonable assurance that the proposed system complies with **Section 6.3.1 of this Volume**. Such reasonable assurance shall consist of:

(a) A geotechnical analysis consisting of existing soil, geologic, and lithologic data of the project area that demonstrates the presence of an aquitard consisting of at least 20 feet of unconsolidated low permeability material [clay (particle size less than 0.002mm, or material passing No. 200 sieve) content >10%] below the pond bottom that will not be breached by the proposed design and construction;

(b) The presence of a minimum of 100 ft. of unconsolidated sediment or soil material from the bottom of the pond and the top of the limestone as demonstrated by core borings within the proposed pond area; or

(c) Other site specific geologic information demonstrating the presence of a confining layer below the pond bottom that provides protection equivalent to that set forth in (a) or (b), above.

A registered professional shall be required to certify that the submitted information, the site characteristics, and the project design provide reasonable assurance of compliance with **Section 6.3.1 of this Volume.**

6.3.4 In addition to sites identified on **Figure 6.0-1, and Appendix A of this Volume,** the Agency shall require compliance with the criteria in **Section 6.3.2 of this Volume** when available data and information indicate that a substantial likelihood exists that a proposed stormwater management system on a site has the potential to be located within the influence of a karst feature based on methodologies generally accepted by registered professionals, and has the potential to adversely affect the Floridan Aquifer System.

6.3.5 If during construction or operation of the stormwater management system, a structural failure is observed that has the potential to cause the direct discharge of surface water into the Floridan Aquifer System, corrective actions designed or approved by a registered professional shall be taken as soon as practical to correct the failure. A report prepared by a registered professional must be provided as soon as practical to the Agency for review and approval that provides reasonable assurance that the breach will be permanently corrected.

6.4 Considerations for Mining and Certain Other Excavation Activities

Reasonable assurance must be provided demonstrating that groundwater quality standards will not be violated by excavation activities, including mining, that have the potential to penetrate confining layers or, that by their nature, must be in direct communication with limestone. Applicants for such activities must demonstrate that runoff entering the excavated area is sufficiently treated prior to discharge to any surface or ground waters. For example, site grading or other water management practices must direct runoff from areas that are potential sources of pollutants into stormwater treatment areas that are designed, constructed, operated and maintained in compliance with **Part II, Part IV, and Part V of Volume I as well as Parts IV and V of this Volume** prior to discharge to the excavated area or off-site. Entrance roads, parking areas, vehicle maintenance and wash areas, and storage areas for petroleum and hazardous substances are examples of areas that have the potential for generating and discharging such pollutants and, as such, require such treatment. However, areas associated with material processing, such as washing associated with grading and sorting of sand or limestone extracted from the site, are not considered potential sources of pollutants, provided that no chemicals, except water conditioners or pH adjusters which, are added to the process water used for transporting, washing, or processing of the sand or limestone.

Applicants are advised that such excavated areas shall not be presumed to be suitable for treating stormwater associated with any future change in land use or development of the site. For example, stormwater from future development may require treatment separate from any impoundment or other surface water created by the excavation. However, such created waters may be suitable for hydrograph attenuation provided that the 10 ft. criteria of **Section 6.3.2(b) above**, is not exceeded.

Impoundments created by mining activities, excluding borrow pits, will not be required to have a horizontal bottom, as provided in **section 6.3.2(b), above**.

APPENDIX A
LOCATION DESCRIPTION OF SENSITIVE KARST AREAS

The following provides a location description of all lands included in the Sensitive Karst Areas, based on the Federal Section, Township, and Range system. Parcels are described to the "Section" level, with Section lines forming the boundary for Sensitive Karst Areas. All lands within the boundaries of a listed Section are included.

In some cases, all of the Sections within the boundary of a Township and Range are included. In those cases, only the Township and Range are specified; these listings appear at the end of the table for the county. The lack of a specified Section means that all Sections within such a Township and Range are included.

Additionally, if included parcels are located only within one county, they are listed under the title: "Part 1: Parcels Wholly Contained Within One County Boundary." If included parcels cover areas located over two or more counties, they are listed under the title: "Part 2: Parcels Contained Within Multiple County Boundaries." **Please be sure to check both Parts when searching for included parcels.**

Part 1: Parcels Wholly Contained Within One County Boundary

BAY COUNTY

TOWNSHIP	RANGE	SECTION
1N	12W	6
1S	12W	5
1S	12W	6
1S	12W	8
1S	12W	9
1S	12W	16
1S	12W	17
1S	12W	21
1S	12W	25
1S	12W	25
1S	12W	27
1S	12W	28
1S	12W	32
1S	12W	33
1S	12W	35
1S	12W	36
1S	13W	3
1S	13W	4
1S	13W	5
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2S	13W	14
2S	13W	15
2S	14W	1

CALHOUN COUNTY

TOWNSHIP	RANGE	SECTION
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2N	7W	6
2N	7W	7

TOWNSHIP	RANGE	SECTION
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2S	8W	29

TOWNSHIP	RANGE	SECTION
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3S	10W	12
1N	9W	

FRANKLIN COUNTY

TOWNSHIP	RANGE	SECTION
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6S	4W	36
8S	3W	72
8S	4W	73

TOWNSHIP	RANGE	SECTION
8S	5W	3
8S	5W	4
8S	5W	5
8S	5W	8
8S	5W	9
8S	5W	24
8S	5W	25
8S	5W	26
8S	5W	34
8S	5W	35
9S	5W	3
4S	5W	
6S	1W	
6S	1W	
6S	1W	
6S	2W	
6S	2W	
6S	3W	
6S	3W	
6S	3W	
6S	4W	
6S	5W	
7S	1W	
7S	3W	
7S	4W	
7S	4W	
7S	4W	
7S	5W	

FRANKLIN COUNTY
REMAINDER OF DOG ISLAND NOT ALREADY LISTED

GADSDEN COUNTY

TOWNSHIP	RANGE	SECTION
1N	3W	1
1N	3W	2
1N	3W	3
1N	3W	4
1N	3W	10
1N	3W	11
1N	3W	12
1N	3W	13
1N	3W	14
1N	3W	15
1N	3W	16
1N	3W	21
1N	3W	22
1N	3W	23
1N	3W	24
1N	3W	25
1N	3W	25
1N	3W	26
1N	3W	26
1N	3W	26
1N	3W	27
1N	3W	27
1N	3W	28
1N	3W	32
1N	3W	32
1N	3W	32
1N	3W	32
1N	3W	32
1N	3W	32
1N	3W	32
1N	3W	33
1N	3W	33

TOWNSHIP	RANGE	SECTION
1N	3W	33
1N	3W	33
1N	3W	34
1N	3W	34
1N	3W	34
1N	4W	25
1N	4W	26
1N	4W	27
1N	4W	28
1N	4W	29
1N	4W	33
1N	4W	34
1N	4W	35
1N	4W	36
1S	4W	1
1S	4W	1
1S	4W	2
1S	4W	3
1S	4W	4
1S	4W	5
1S	4W	6
1S	4W	7
1S	4W	8
1S	4W	9
1S	4W	9
1S	4W	10
1S	4W	10
1S	4W	10
1S	4W	11
1S	4W	11
1S	4W	12
1S	4W	12
1S	4W	16
1S	4W	16
1S	4W	16
1S	4W	17
1S	4W	17
1S	4W	17
1S	4W	17
1S	4W	17
1S	4W	18
1S	4W	18
2N	2W	1
2N	2W	2
2N	2W	3
2N	2W	4
2N	2W	5
2N	2W	6
2N	2W	6
2N	2W	7
2N	2W	7
2N	2W	8
2N	2W	10
2N	2W	11
2N	2W	12
2N	2W	13
2N	2W	13
2N	2W	14
2N	2W	15
2N	2W	16
2N	2W	17
2N	2W	18
2N	2W	19
2N	2W	20
2N	2W	21
2N	2W	22
2N	2W	23

TOWNSHIP	RANGE	SECTION
2N	2W	24
2N	2W	24
2N	2W	24
2N	2W	24
2N	2W	24
2N	2W	25
2N	2W	25
2N	2W	26
2N	2W	27
2N	2W	28
2N	2W	29
2N	2W	30
2N	2W	31
2N	2W	32
2N	2W	33
2N	2W	34
2N	2W	35
2N	2W	36
2N	2W	36
2N	3W	1
2N	3W	2
2N	3W	3
2N	3W	4
2N	3W	5
2N	3W	8
2N	3W	9
2N	3W	10
2N	3W	11
2N	3W	12
2N	3W	13
2N	3W	14
2N	3W	15
2N	3W	21
2N	3W	22
2N	3W	23
2N	3W	24
2N	3W	25
2N	3W	26
2N	3W	27
2N	3W	28
2N	3W	32
2N	3W	33
2N	3W	34
2N	3W	35
2N	3W	36
2N	4W	35
2N	6W	3
2N	6W	5
2N	6W	6
2N	6W	7
2N	6W	8
2N	6W	17
2N	6W	18
2N	6W	18
2N	6W	19
2N	6W	20
3N	1E	93
3N	1E	95
3N	1E	97
3N	1E	98
3N	1E	99
3N	1E	100
3N	1E	258
3N	1E	258
3N	1E	258
3N	1E	259
3N	1E	259

TOWNSHIP	RANGE	SECTION
3N	1E	260
3N	1E	260
3N	1E	260
3N	1W	8
3N	1W	9
3N	1W	10
3N	1W	11
3N	1W	11
3N	1W	11
3N	1W	11
3N	1W	14
3N	1W	14
3N	1W	15
3N	1W	16
3N	1W	17
3N	1W	19
3N	1W	20
3N	1W	21
3N	1W	22
3N	1W	26
3N	1W	26
3N	1W	26
3N	1W	27
3N	1W	27
3N	1W	28
3N	1W	29
3N	1W	30
3N	1W	31
3N	1W	32
3N	1W	33
3N	1W	34
3N	1W	34
3N	2W	7
3N	2W	8
3N	2W	17
3N	2W	18
3N	2W	19
3N	2W	20
3N	2W	24
3N	2W	25
3N	2W	28
3N	2W	29
3N	2W	30
3N	2W	31
3N	2W	32
3N	2W	34
3N	2W	35
3N	2W	36
3N	2W	15
3N	2W	22
3N	2W	23
3N	2W	25
3N	2W	26
3N	2W	35
3N	2W	36
3N	2W	7
3N	2W	1
3N	2W	2
3N	2W	3
3N	2W	4
3N	6W	5
3N	6W	8
3N	6W	9
3N	6W	10
3N	6W	11
3N	6W	12

TOWNSHIP	RANGE	SECTION
3N	6W	13
3N	6W	15
3N	6W	16
3N	6W	17
3N	6W	18
3N	6W	19
3N	6W	20
3N	6W	21
3N	6W	22
3N	6W	23
3N	6W	25
3N	6W	26
3N	6W	27
3N	6W	28
3N	6W	29
3N	6W	30
3N	6W	31
3N	6W	32
3N	6W	33
3N	6W	34
3N	6W	35
3N	6W	36
61N	61E	1
61N	61E	2
61N	61E	3
61N	61E	5
61N	61E	6
61N	61E	7
61N	61E	8
61N	61E	9
61N	61E	10
61N	61E	11
61N	61E	12
61N	61E	13
61N	61E	14
61N	61E	18
61N	61E	19
61N	61E	20
61N	61E	21
61N	61E	22
61N	61E	23
61N	61E	28
61N	61E	29
61N	61E	30
61N	61E	31
61N	61E	32
61N	61E	33
61N	61E	37
61N	61E	39
61N	61E	40
61N	61E	44
61N	61E	76
61N	61E	77
61N	61E	78
61N	61E	78
61N	61E	78
61N	61E	78
61N	61E	78
61N	61E	78
61N	61E	79
61N	61E	80
61N	61E	80
61N	61E	80
61N	61E	80
61N	61E	80
61N	61E	81
61N	61E	81

TOWNSHIP	RANGE	SECTION
61N	61E	81
61N	61E	81
61N	61E	81

HOLMES COUNTY

TOWNSHIP	RANGE	SECTION
3N	17W	33
3N	18W	1
3N	18W	2
3N	18W	3
3N	18W	10
3N	18W	11
3N	18W	12
3N	18W	13
3N	18W	14
3N	18W	15
3N	18W	15
3N	18W	15
3N	18W	22
3N	18W	22
3N	18W	23
3N	18W	24
3N	18W	25
3N	18W	26
3N	18W	27
3N	18W	36
5N	14W	5
5N	14W	6
5N	14W	7
5N	14W	8
5N	14W	11
5N	14W	12
5N	14W	12
5N	14W	14
5N	14W	14
5N	14W	14
5N	14W	18
5N	14W	19
5N	14W	23
5N	14W	23
5N	14W	23
5N	14W	23
5N	14W	26
5N	14W	26
5N	14W	27
5N	14W	28
5N	14W	29
5N	14W	30
5N	14W	31
5N	14W	32
5N	14W	33
5N	14W	34
5N	14W	35
5N	14W	35
5N	14W	35
6N	13W	4
6N	13W	4
6N	13W	4
6N	13W	8
6N	14W	2
6N	14W	3
6N	14W	4
6N	14W	5
6N	14W	6
6N	14W	7
6N	14W	8
6N	14W	9

TOWNSHIP	RANGE	SECTION
6N	14W	10
6N	14W	11
6N	14W	15
6N	14W	16
6N	14W	17
6N	14W	18
6N	14W	19
6N	14W	20
6N	14W	22
6N	14W	27
6N	14W	28
6N	14W	29
6N	14W	30
6N	14W	31
6N	14W	32
6N	14W	33
6N	14W	34
7N	13W	19
7N	13W	22
7N	13W	23
7N	13W	23
7N	13W	26
7N	13W	26
7N	13W	27
7N	13W	27
7N	13W	28
7N	13W	30
7N	13W	33
7N	13W	33
7N	13W	33
7N	13W	34
7N	13W	34
7N	14W	19
7N	14W	20
7N	14W	22
7N	14W	23
7N	14W	24
7N	14W	25
7N	14W	26
7N	14W	27
7N	14W	29
7N	14W	30
7N	14W	31
7N	14W	32
7N	14W	33
7N	14W	34
7N	14W	35
7N	14W	36
7N	15W	22
7N	15W	23
7N	15W	24
7N	15W	25
7N	15W	26
7N	15W	27
7N	15W	28
7N	15W	29
7N	15W	31
7N	15W	32
7N	15W	33
7N	15W	34
7N	15W	35
7N	15W	36
4N	17W	
5N	15W	
5N	16W	
5N	17W	
6N	15W	

TOWNSHIP	RANGE	SECTION
6N	16W	
6N	17W	
7N	16W	
7N	17W	

JACKSON COUNTY

TOWNSHIP	RANGE	SECTION
2N	9W	6
2N	9W	6
2N	9W	6
2N	9W	7
2N	9W	7
2N	9W	18
2N	9W	18
2N	11W	1
2N	11W	11
2N	11W	12
2N	11W	13
2N	11W	14
2N	11W	14
2N	11W	14
3N	6W	6
3N	6W	6
3N	6W	7
3N	6W	7
3N	9W	1
3N	9W	2
3N	9W	3
3N	9W	4
3N	9W	5
3N	9W	6
3N	9W	7
3N	9W	8
3N	9W	9
3N	9W	10
3N	9W	11
3N	9W	12
3N	9W	13
3N	9W	14
3N	9W	15
3N	9W	16
3N	9W	17
3N	9W	18
3N	9W	19
3N	9W	20
3N	9W	21
3N	9W	22
3N	9W	23
3N	9W	24
3N	9W	25
3N	9W	26
3N	9W	27
3N	9W	28
3N	9W	29
3N	9W	30
3N	9W	31
3N	9W	31
3N	9W	32
3N	9W	32
3N	9W	33
3N	9W	34
3N	9W	36
3N	9W	36
3N	10W	1
3N	10W	2
3N	10W	3
3N	10W	4

TOWNSHIP	RANGE	SECTION
3N	10W	5
3N	10W	6
3N	10W	7
3N	10W	8
3N	10W	9
3N	10W	10
3N	10W	11
3N	10W	12
3N	10W	13
3N	10W	14
3N	10W	15
3N	10W	16
3N	10W	17
3N	10W	18
3N	10W	19
3N	10W	20
3N	10W	21
3N	10W	22
3N	10W	23
3N	10W	24
3N	10W	25
3N	10W	26
3N	10W	27
3N	10W	30
3N	10W	31
3N	10W	34
3N	10W	35
3N	10W	36
3N	11W	1
3N	11W	2
3N	11W	3
3N	11W	4
3N	11W	5
3N	11W	6
3N	11W	7
3N	11W	8
3N	11W	9
3N	11W	10
3N	11W	11
3N	11W	12
3N	11W	13
3N	11W	14
3N	11W	15
3N	11W	16
3N	11W	17
3N	11W	18
3N	11W	19
3N	11W	20
3N	11W	22
3N	11W	23
3N	11W	24
3N	11W	25
3N	11W	26
3N	11W	27
3N	11W	28
3N	11W	29
3N	11W	30
3N	11W	34
3N	11W	35
3N	11W	36
3N	12W	1
3N	12W	2
3N	12W	3
3N	12W	10
3N	12W	10
3N	12W	11
3N	12W	12

TOWNSHIP	RANGE	SECTION
3N	12W	13
3N	12W	14
3N	12W	15
3N	12W	15
3N	12W	15
3N	12W	22
4N	7W	4
4N	7W	4
4N	7W	5
4N	7W	6
4N	7W	7
4N	7W	8
4N	7W	8
4N	7W	8
4N	7W	9
4N	7W	15
4N	7W	16
4N	7W	17
4N	7W	17
4N	7W	18
4N	7W	19
4N	7W	20
4N	7W	21
4N	7W	22
4N	7W	23
4N	7W	25
4N	7W	26
4N	7W	27
4N	7W	28
4N	7W	29
4N	7W	30
4N	7W	31
4N	7W	32
4N	7W	33
4N	7W	34
4N	7W	35
4N	7W	36
4N	12W	1
4N	12W	2
4N	12W	3
4N	12W	10
4N	12W	10
4N	12W	11
4N	12W	12
4N	12W	13
4N	12W	14
4N	12W	15
4N	12W	15
4N	12W	22
4N	12W	22
4N	12W	23
4N	12W	24
4N	12W	25
4N	12W	26
4N	12W	27
4N	12W	34
4N	12W	35
4N	12W	36
5N	7W	4
5N	7W	5
5N	7W	6
5N	7W	7
5N	7W	8
5N	7W	9
5N	7W	9
5N	7W	9
5N	7W	9

TOWNSHIP	RANGE	SECTION
5N	7W	9
5N	7W	9
5N	7W	16
5N	7W	16
5N	7W	16
5N	7W	16
5N	7W	17
5N	7W	18
5N	7W	19
5N	7W	20
5N	7W	21
5N	7W	28
5N	7W	28
5N	7W	29
5N	7W	29
5N	7W	30
5N	7W	31
5N	7W	32
5N	7W	33
5N	7W	33
5N	14W	13
5N	14W	13
6N	13W	1
6N	13W	2
6N	13W	3
6N	13W	3
6N	13W	9
6N	13W	9
6N	13W	9
6N	13W	9
6N	13W	9
6N	13W	10
6N	13W	12
7N	13W	24
7N	13W	25
7N	13W	35
7N	13W	36
3N	8W	
4N	10W	
4N	11W	
4N	8W	
4N	9W	
5N	10W	
5N	11W	
5N	8W	
5N	9W	
6N	10W	
6N	11W	
6N	12W	
6N	7W	
6N	8W	
6N	9W	
7N	10W	
7N	11W	
7N	12W	
7N	8W	
7N	9W	

JEFFERSON COUNTY

TOWNSHIP	RANGE	SECTION
1N	4E	1
1N	4E	2
1N	4E	3
1N	4E	4
1N	4E	5
1N	4E	6
1N	4E	7
1N	4E	8
1N	4E	9
1N	4E	11
1N	4E	12
1N	4E	13
1N	4E	14
1N	4E	15
1N	4E	16
1N	4E	17
1N	4E	18
1N	4E	19
1N	4E	20
1N	4E	21
1N	4E	24
1N	4E	25
1N	4E	27
1N	4E	28
1N	4E	29
1N	4E	30
1N	4E	31
1N	4E	32
1N	4E	33
1N	4E	34
1N	4E	35
1N	4E	36
1N	5E	2
1N	5E	4
1N	5E	5
1N	5E	6
1N	5E	7
1N	5E	18
1N	5E	19
1N	5E	30
1N	5E	31
1S	3E	1
1S	3E	2
1S	3E	3
1S	3E	4
1S	3E	5
1S	3E	6
1S	3E	7
1S	3E	8
1S	3E	9
1S	3E	10
1S	3E	11
1S	3E	14
1S	3E	15
1S	3E	16
1S	3E	17
1S	3E	18
1S	3E	19
1S	3E	20
1S	3E	21
1S	3E	22
1S	3E	23
1S	3E	26
1S	3E	27
1S	3E	28
1S	3E	29
1S	3E	30
1S	3E	31
1S	3E	32
1S	3E	33
1S	3E	34
1S	3E	35
1S	4E	1
1S	4E	2
1S	4E	3
1S	4E	4
1S	4E	5
1S	4E	6
2N	5E	1
2N	5E	2
2N	5E	3
2N	5E	4
2N	5E	5
2N	5E	6
2N	5E	7
2N	5E	8
2N	5E	9
2N	5E	10
2N	5E	11
2N	5E	12
2N	5E	13
2N	5E	14
2N	5E	15
2N	5E	16
2N	5E	17
2N	5E	18
2N	5E	19
2N	5E	20
2N	5E	21
2N	5E	22
2N	5E	23
2N	5E	24
2N	5E	25
2N	5E	26
2N	5E	27
2N	5E	28
2N	5E	29
2N	5E	31
2N	5E	32
2N	5E	33
2N	5E	34
2N	5E	35
2N	5E	36
2N	6E	6
2N	6E	18
2N	6E	19
2S	3E	2
2S	3E	3
2S	3E	4
2S	3E	5
2S	3E	6
2S	3E	7
2S	3E	8
2S	3E	9
2S	3E	10
2S	3E	14
2S	3E	15
2S	3E	16
2S	3E	17
2S	3E	18
2S	3E	19
2S	3E	20
2S	3E	21
2S	3E	22
2S	3E	27
2S	3E	28
2S	3E	29
2S	3E	30
2S	3E	30
2S	3E	31
2S	3E	32

TOWNSHIP	RANGE	SECTION
2S	3E	33
2S	3E	34
2S	3E	35
3N	3E	136
3N	3E	137
3N	3E	140
3N	3E	141
3N	4E	134
3N	4E	135
3N	4E	142
3N	4E	143
3N	4E	144
3N	4E	145
3N	4E	146
3N	4E	147
3N	4E	148
3N	4E	176
3N	5E	149
3N	5E	150
3N	5E	151
3N	5E	152
3N	5E	153
3N	5E	154
3N	5E	169
3N	5E	170
3N	5E	171
3N	5E	172
3N	5E	173
3N	5E	174
3N	5E	175
3N	6E	19
3N	6E	30
3N	6E	31
3N	6E	168
3S	3E	2
3S	3E	3
3S	3E	4
3S	3E	5
3S	3E	6
3S	3E	7
3S	3E	8
3S	3E	9
3S	3E	10
3S	3E	11
3S	3E	14
3S	3E	15
3S	3E	16
3S	3E	17
3S	3E	18
3S	3E	19
3S	3E	20
3S	3E	21
3S	3E	22
3S	3E	23
3S	3E	26
3S	3E	27
3S	3E	28
3S	3E	29
3S	3E	30
3S	3E	30
3S	3E	31
3S	3E	32
3S	3E	33
3S	3E	34
4S	3E	2
4S	3E	3
4S	3E	4

TOWNSHIP	RANGE	SECTION
4S	3E	5
4S	3E	6
4S	3E	6
4S	3E	7
4S	3E	7
4S	3E	8
4S	3E	9
4S	3E	10
4S	3E	11
4S	3E	14
4S	3E	15
4S	3E	16
4S	3E	17
4S	3E	18
4S	3E	18
4S	3E	19
4S	3E	20
4S	3E	21
4S	3E	22
4S	3E	23
4S	3E	26
4S	3E	26
4S	3E	27
4S	3E	27
4S	3E	28
4S	3E	29
4S	3E	30
2N	4E	
3N	4E	
3N	5E	

LEON COUNTY

TOWNSHIP	RANGE	SECTION
1N	3W	31
1N	3W	31
1N	3W	31
1N	3W	31
1N	3W	31
1N	3W	35
1N	3W	36
1S	3W	1
1S	3W	2
1S	3W	3
1S	3W	4
1S	3W	5
1S	3W	6
1S	3W	6
1S	3W	6
1S	3W	6
1S	3W	6
1S	3W	7
1S	3W	8
1S	3W	9
1S	3W	10
1S	3W	11
1S	3W	12
1S	3W	13
1S	3W	14
1S	3W	15
1S	3W	16
1S	3W	17
1S	3W	18
1S	3W	19
1S	3W	20
1S	3W	22
1S	3W	23
1S	3W	24

TOWNSHIP	RANGE	SECTION
1S	3W	25
1S	3W	26
1S	3W	27
1S	3W	28
1S	3W	32
1S	3W	33
1S	3W	34
1S	3W	35
1S	3W	36
1S	4W	13
1S	4W	14
1S	4W	15
1S	4W	21
1S	4W	22
1S	4W	24
1S	4W	28
1S	4W	29
1S	4W	29
1S	4W	29
1S	4W	30
1S	4W	30
1S	4W	30
1S	4W	30
1S	4W	31
1S	5W	25
1S	5W	25
1S	5W	35
1S	5W	35
1S	5W	36
1S	5W	36
2S	3W	1
2S	3W	2
2S	3W	3
2S	3W	4
2S	3W	8
2S	3W	9
2S	3W	10
2S	3W	11
2S	3W	12
2S	3W	13
2S	3W	13
2S	3W	14
2S	3W	14
2S	3W	15
2S	3W	16
2S	3W	17
2S	3W	18
2S	3W	18
2S	4W	6
2S	4W	7
2S	4W	8
2S	4W	13
2S	4W	14
2S	4W	17
2S	4W	17
2S	4W	18
2S	4W	18
2S	5W	2
2S	5W	2
3N	1E	261
3N	1W	12
3N	1W	13
3N	1W	23
3N	1W	23
3N	1W	24
3N	1W	25
3N	1W	35

TOWNSHIP	RANGE	SECTION
3N	1W	36
3N	1W	36
3N	3E	138
3N	3E	138
1N	1E	
1N	1W	
1N	2E	
1S	1E	
1S	1W	
1S	2E	
1S	2W	
2N	1E	
2N	2E	
3N	1E	
3N	2E	

LIBERTY COUNTY

TOWNSHIP	RANGE	SECTION
1N	7W	2
1N	7W	3
1N	7W	4
1N	7W	5
1N	7W	6
1N	7W	7
1N	7W	8
1N	7W	9
1N	7W	17
1N	7W	18
1N	7W	19
1S	4W	19
1S	4W	19
1S	4W	19
1S	4W	20
1S	4W	20
1S	4W	20
1S	5W	13
1S	5W	21
1S	5W	22
1S	5W	24
1S	5W	26
1S	5W	27
1S	5W	28
1S	5W	30
1S	5W	31
1S	5W	32
1S	5W	33
1S	5W	34
1S	6W	13
1S	6W	20
1S	6W	21
1S	6W	22
1S	6W	23
1S	6W	24
1S	6W	25
1S	6W	26
1S	6W	27
1S	6W	28
1S	6W	29
1S	6W	30
1S	6W	35
1S	6W	36
2N	7W	1
2N	7W	2
2N	7W	3
2N	7W	3
2N	7W	9
2N	7W	10

TOWNSHIP	RANGE	SECTION
2N	7W	11
2N	7W	12
2N	7W	12
2N	7W	13
2N	7W	14
2N	7W	15
2N	7W	16
2N	7W	17
2N	7W	20
2N	7W	21
2N	7W	22
2N	7W	23
2N	7W	26
2N	7W	27
2N	7W	28
2N	7W	29
2N	7W	31
2N	7W	32
2N	7W	33
2N	7W	34
2N	7W	35
2S	6W	1
2S	6W	24
2S	6W	25
2S	7W	6
2S	7W	7
2S	8W	1
2S	8W	2
2S	8W	3
2S	8W	10
2S	8W	11
2S	8W	12
2S	8W	15
2S	8W	21
2S	8W	28
2S	8W	33
3S	4W	19
3S	5W	2
3S	5W	2
3S	5W	2
3S	5W	3
3S	5W	3
3S	5W	4
3S	5W	8
3S	5W	9
3S	5W	10
3S	5W	11
3S	5W	11
3S	5W	11
3S	5W	12
3S	5W	12
3S	5W	13
3S	5W	13
3S	5W	14
3S	5W	15
3S	5W	16
3S	5W	17
3S	5W	18
3S	5W	19
3S	5W	20
3S	5W	21
3S	5W	22
3S	5W	23
3S	5W	24
3S	5W	24
3S	5W	25
3S	5W	26

TOWNSHIP	RANGE	SECTION
3S	5W	27
3S	5W	28
3S	5W	29
3S	5W	30
3S	5W	31
3S	5W	32
3S	5W	33
3S	5W	34
3S	5W	35
3S	5W	36
3S	6W	24
3S	6W	25
3S	6W	36
4S	6W	1
4S	6W	12
4S	6W	13
4S	6W	22
4S	6W	23
4S	6W	24
4S	6W	25
4S	6W	26
4S	6W	27
4S	6W	33
4S	6W	34
4S	6W	35
4S	6W	36
5S	6W	1
5S	6W	2
5S	6W	4
5S	6W	5
5S	6W	8
5S	6W	9
5S	6W	11
5S	6W	12
5S	6W	13
5S	6W	14
5S	6W	15
5S	6W	16
5S	6W	22
5S	6W	23
5S	6W	24
5S	6W	25
5S	6W	25
5S	6	26

WAKULLA COUNTY

TOWNSHIP	RANGE	SECTION
2S	3W	19
2S	3W	20
2S	3W	20
2S	3W	21
2S	3W	21
2S	3W	22
2S	3W	22
2S	3W	23
2S	3W	23
2S	3W	24
2S	3W	24
2S	3W	25
2S	3W	26
2S	3W	27
2S	3W	28
2S	3W	29
2S	3W	30
2S	3W	31
2S	3W	32
2S	3W	33

TOWNSHIP	RANGE	SECTION
2S	3W	34
2S	3W	35
2S	3W	36
2S	4W	19
2S	4W	20
2S	4W	20
2S	4W	21
2S	4W	22
2S	4W	23
2S	4W	24
2S	4W	24
2S	4W	25
2S	4W	26
2S	4W	27
2S	4W	28
2S	4W	29
2S	4W	31
2S	4W	32
2S	4W	33
2S	4W	34
2S	4W	35
2S	4W	36
3S	2E	25
3S	2E	36
3S	5W	1
4S	1E	10
4S	2E	12
4S	2E	34
4S	2E	35
4S	2E	36
5S	1W	31
5S	1W	32
5S	2W	31
5S	2W	32
5S	3W	33
5S	3W	34
6S	2W	5
3S	1W	
3S	2E	
3S	2W	
3S	3W	
4S	1E	
4S	2E	
4S	2W	
4S	3W	
5S	1E	
5S	1W	
5S	2E	
5S	2W	
6S	2W	
6S	2W	
6S	3W	

WAKULLA COUNTY – SPANISH LAND GRANT

TOWNSHIP	RANGE	SECTION
60N	60E	1
60N	60E	2
60N	60E	3
60N	60E	4
60N	60E	5
60N	60E	5
60N	60E	6
60N	60E	7
60N	60E	8
60N	60E	9
60N	60E	10
60N	60E	11
60N	60E	11
60N	60E	12
60N	60E	13
60N	60E	14
60N	60E	15
60N	60E	16
60N	60E	17
60N	60E	18
60N	60E	19
60N	60E	20
60N	60E	21
60N	60E	22
60N	60E	23
60N	60E	24
60N	60E	25
60N	60E	26
60N	60E	27
60N	60E	28
60N	60E	29
60N	60E	30
60N	60E	31
60N	60E	32
60N	60E	33
60N	60E	34
60N	60E	35
60N	60E	36
60N	60E	37
60N	60E	38
60N	60E	39
60N	60E	40
60N	60E	41
60N	60E	42
60N	60E	43
60N	60E	44
60N	60E	45
60N	60E	46
60N	60E	47
60N	60E	48
60N	60E	49
60N	60E	50
60N	60E	51
60N	60E	52
60N	60E	53
60N	60E	54
60N	60E	55
60N	60E	56
60N	60E	57
60N	60E	58
60N	60E	59
60N	60E	60
60N	60E	61
60N	60E	62
60N	60E	63
60N	60E	64
60N	60E	65
60N	60E	66
60N	60E	67
60N	60E	68
60N	60E	69
60N	60E	70
60N	60E	71
60N	60E	72
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60N	60E	74
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60N	60E	77
60N	60E	78
60N	60E	79
60N	60E	80
60N	60E	81
60N	60E	82
60N	60E	83
60N	60E	84
60N	60E	85
60N	60E	86
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60N	60E	94
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60N	60E	96
60N	60E	97
60N	60E	98
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60N	60E	102
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60N	60E	105
60N	60E	106
60N	60E	107
60N	60E	108
60N	60E	109
60N	60E	110
60N	60E	111
60N	60E	112
60N	60E	113
60N	60E	114
60N	60E	115
60N	60E	116
60N	60E	117
60N	60E	118
60N	60E	119
60N	60E	120
60N	60E	121
60N	60E	121
60N	60E	121
60N	60E	125
60N	60E	126
60N	60E	127
60N	60E	128
60N	60E	501
60N	60E	502
60N	60E	503
60N	60E	504
60N	60E	505
60N	60E	700
60N	60E	701
60N	60E	701
60N	60E	801
60N	60E	802
60N	60E	803
60N	60E	901
60N	60E	902
60N	60E	903
60N	60E	904
60N	60E	906

TOWNSHIP	RANGE	SECTION
60N	60E	907
60N	60E	908
60N	60E	909
60N	60E	910
60N	60E	911

WAKULLA COUNTY
ALL OF PINEY ISLAND

WALTON COUNTY

TOWNSHIP	RANGE	SECTION
1N	16W	6
1N	16W	6
1N	16W	7
1N	16W	7
1N	16W	8
1N	16W	8
1N	16W	8
1N	16W	17
1N	16W	17
1N	16W	17
1N	16W	17
1N	16W	18
1N	16W	19
1N	16W	19
1N	16W	19
1N	16W	19
1N	16W	20
1N	16W	20
1N	16W	30
1N	16W	30
1N	16W	30
1N	16W	30
1N	17W	1
1N	17W	2
1N	17W	3
1N	17W	7
1N	17W	8
1N	17W	9
1N	17W	10
1N	17W	11
1N	17W	12
1N	17W	13
1N	17W	14
1N	17W	15
1N	17W	16
1N	17W	17
1N	17W	18
1N	17W	19
1N	17W	20
1N	17W	21
1N	17W	22
1N	17W	23
1N	17W	24
1N	17W	25
1N	17W	25
1N	17W	25
1N	17W	26
1N	17W	27
1N	17W	28
1N	17W	35
1N	17W	36
1N	17W	36
1N	17W	36
1N	18W	1
1N	18W	2
1N	18W	3

TOWNSHIP	RANGE	SECTION
1N	18W	4
1N	18W	5
1N	18W	6
1N	18W	9
1N	18W	10
1N	18W	11
1N	18W	12
1N	18W	13
1N	18W	14
1N	18W	15
1N	18W	17
1N	18W	18
1N	18W	19
1N	18W	22
1N	18W	23
1N	18W	24
1N	18W	25
1N	18W	28
1N	18W	29
1N	18W	30
1N	18W	31
1N	18W	32
1N	18W	33
1N	18W	34
1N	19W	5
1N	19W	6
1N	19W	8
1N	19W	9
1N	19W	10
1N	19W	16
1N	19W	17
1N	19W	18
1N	19W	19
1N	19W	20
1N	19W	24
1N	19W	25
1N	19W	26
1N	19W	27
1N	19W	28
1N	19W	29
1N	19W	30
1N	19W	31
1N	19W	32
1N	19W	33
1N	19W	34
1N	19W	35
1N	19W	36
1N	20W	11
1N	20W	12
1N	20W	13
1N	20W	14
1N	20W	24
1N	20W	27
1N	20W	33
1N	20W	34
1N	20W	35
1N	20W	36
1S	18W	3
1S	18W	4
1S	18W	5
1S	18W	6
1S	18W	7
1S	18W	8
1S	18W	9
1S	18W	10
1S	18W	11
1S	18W	12

TOWNSHIP	RANGE	SECTION
1S	18W	13
1S	18W	14
1S	18W	15
1S	18W	16
1S	18W	17
1S	18W	18
1S	18W	19
1S	18W	20
1S	18W	21
1S	18W	22
1S	18W	23
1S	18W	24
1S	18W	25
1S	18W	26
1S	18W	27
1S	18W	28
1S	18W	29
1S	18W	30
1S	18W	31
1S	18W	32
1S	18W	33
1S	18W	34
1S	18W	35
1S	20W	1
1S	20W	3
1S	20W	4
1S	20W	9
1S	20W	10
1S	20W	11
1S	20W	12
1S	20W	13
1S	20W	14
1S	20W	15
1S	20W	16
1S	20W	23
1S	20W	24
1S	20W	25
2N	16W	19
2N	16W	19
2N	16W	19
2N	16W	30
2N	16W	30
2N	16W	31
2N	16W	31
2N	17W	2
2N	17W	2
2N	17W	3
2N	17W	4
2N	17W	5
2N	17W	6
2N	17W	7
2N	17W	8
2N	17W	9
2N	17W	10
2N	17W	11
2N	17W	11
2N	17W	13
2N	17W	13
2N	17W	14
2N	17W	14
2N	17W	15
2N	17W	16
2N	17W	17
2N	17W	18
2N	17W	19
2N	17W	20
2N	17W	21

TOWNSHIP	RANGE	SECTION
2N	17W	22
2N	17W	23
2N	17W	24
2N	17W	24
2N	17W	25
2N	17W	26
2N	17W	27
2N	17W	28
2N	17W	29
2N	17W	30
2N	17W	32
2N	17W	34
2N	17W	35
2N	17W	36
2N	18W	13
2N	18W	14
2N	18W	15
2N	18W	22
2N	18W	23
2N	18W	24
2N	18W	25
2N	18W	26
2N	18W	27
2N	18W	29
2N	18W	30
2N	18W	31
2N	18W	32
2N	18W	33
2N	18W	34
2N	18W	35
2N	18W	36
2N	19W	4
2N	19W	5
2N	19W	6
2N	19W	7
2N	19W	27
2N	19W	28
2N	19W	29
2N	19W	32
2N	19W	33
2N	20W	1
2N	20W	2
2N	20W	3
2N	20W	4
2N	20W	9
2N	20W	10
2N	20W	11
2N	20W	12
2N	20W	13
2N	19W	1
2N	19W	2
3N	17W	1
3N	17W	2
3N	17W	3
3N	17W	4
3N	17W	5
3N	17W	6
3N	17W	8
3N	17W	9
3N	17W	10
3N	17W	11
3N	17W	12
3N	17W	13
3N	17W	14
3N	17W	15
3N	17W	16
3N	17W	17

TOWNSHIP	RANGE	SECTION
3N	17W	18
3N	17W	19
3N	17W	20
3N	17W	21
3N	17W	22
3N	17W	23
3N	17W	23
3N	17W	24
3N	17W	24
3N	17W	24
3N	17W	25
3N	17W	25
3N	17W	25
3N	17W	26
3N	17W	26
3N	17W	26
3N	17W	26
3N	17W	27
3N	17W	28
3N	17W	29
3N	17W	30
3N	17W	31
3N	17W	32
3N	17W	34
3N	17W	35
3N	17W	35
3N	18W	4
3N	18W	5
3N	18W	6
3N	18W	7
3N	18W	8
3N	18W	9
3N	18W	16
3N	18W	17
3N	18W	18
3N	18W	19
3N	18W	20
3N	18W	21
3N	19W	1
3N	19W	5
3N	19W	6
3N	19W	6
3N	19W	7
3N	19W	7
3N	19W	8
3N	19W	8
3N	19W	11
3N	19W	12
3N	19W	13
3N	19W	14
3N	19W	15
3N	19W	18
3N	19W	23
3N	19W	24
3N	19W	31
3N	20W	1
3N	20W	1
3N	20W	2
3N	20W	3
3N	20W	4
3N	20W	5
3N	20W	6
3N	20W	9
3N	20W	10
3N	20W	11
3N	20W	12
3N	20W	14

TOWNSHIP	RANGE	SECTION
3N	20W	25
3N	20W	26
3N	20W	34
3N	20W	35
3N	20W	36
4N	19W	2
4N	19W	4
4N	19W	5
4N	19W	6
4N	19W	7
4N	19W	8
4N	19W	9
4N	19W	10
4N	19W	11
4N	19W	13
4N	19W	15
4N	19W	16
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4N	20W	11
4N	20W	12
4N	20W	13
4N	20W	14
4N	20W	16
4N	20W	17
4N	20W	20
4N	20W	21
4N	20W	22
4N	20W	23
4N	20W	24
4N	20W	25
4N	20W	26
4N	20W	27
4N	20W	28
4N	20W	29
4N	20W	31
4N	20W	32
4N	20W	33
4N	20W	34
4N	20W	35
4N	20W	36
4N	21W	36
5N	19W	1
5N	19W	2
5N	19W	3
5N	19W	4
5N	19W	5
5N	19W	6

TOWNSHIP	RANGE	SECTION
5N	19W	7
5N	19W	8
5N	19W	9
5N	19W	10
5N	19W	11
5N	19W	12
5N	19W	13
5N	19W	14
5N	19W	15
5N	19W	16
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5N	21W	14
5N	21W	15
5N	21W	16
5N	21W	18
5N	21W	23
6N	20W	25
6N	20W	26
6N	20W	27
6N	20W	28
6N	20W	31
6N	20W	32
6N	20W	33
6N	20W	34
6N	20W	35
6N	20W	36
6N	21W	28
6N	21W	32
6N	21W	33
6N	21W	34
6N	30W	29
1S	19W	
6N	19W	

WASHINGTON COUNTY

TOWNSHIP	RANGE	SECTION
1N	14W	1
1N	14W	2
1N	14W	3
1N	14W	4
1N	14W	5
1N	14W	6
1N	14W	7
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1N	14W	9
1N	14W	10
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2N	15W	4
2N	15W	5
2N	15W	6
2N	15W	7
2N	15W	8
2N	15W	9
2N	15W	10
2N	15W	11

TOWNSHIP	RANGE	SECTION
2N	15W	16
2N	15W	17
2N	15W	18
2N	15W	19
2N	15W	24
2N	15W	36
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2N	16W	2
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3N	13W	2
3N	13W	3
3N	13W	4
3N	13W	5
3N	13W	6
3N	13W	7
3N	13W	8
3N	13W	9

TOWNSHIP	RANGE	SECTION
3N	13W	12
3N	13W	13
3N	13W	14
3N	13W	17
3N	13W	18
3N	13W	19
3N	13W	19
3N	13W	20
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4N	12W	7
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4N	12W	9
4N	12W	9
4N	12W	9
4N	12W	16
4N	12W	16
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4N	12W	19
4N	12W	20

TOWNSHIP	RANGE	SECTION
4N	12W	21
4N	12W	21
4N	12W	28
4N	12W	28
4N	12W	29
4N	12W	30
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4N	12W	32
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5N	14W	24
5N	14W	24
5N	14W	24
5N	14W	24
5N	14W	25
5N	14W	36
1N	13W	
3N	15W	

Part 2: Parcels Contained Within Multiple County Boundaries

CALHOUN / LIBERTY COUNTIES

TOWNSHIP	RANGE	SECTION
1N	8W	
1S	8W	
2N	8W	

HOLMES / WASHINGTON COUNTIES

TOWNSHIP	RANGE	SECTION
4N	14W	
4N	15W	
4N	16W	

JACKSON / CALHOUN COUNTIES

TOWNSHIP	RANGE	SECTION
2N	10W	

JACKSON / GADSDEN COUNTIES

TOWNSHIP	RANGE	SECTION
3N	7W	
4N	6W	

JACKSON / WASHINGTON COUNTIES

TOWNSHIP	RANGE	SECTION
5N	12W	

LEON / GADSDEN COUNTIES

TOWNSHIP	RANGE	SECTION
1N	2W	
2N	1W	

LEON / JEFFERSON COUNTIES

TOWNSHIP	RANGE	SECTION
1N	3E	
2N	3E	
3N	3E	

LEON / WAKULLA COUNTIES

TOWNSHIP	RANGE	SECTION
2S	1E	
2S	1W	
2S	2E	
2S	2W	

LEON / WAKULLA/LIBERTY COUNTIES

TOWNSHIP	RANGE	SECTION
2S	5W	

LIBERTY / WAKULLA COUNTIES

TOWNSHIP	RANGE	SECTION
5S	5W	
5S	4W	
3S	4W	
4S	4W	
5S	3W	

WAKULLA / JEFFERSON COUNTIES

TOWNSHIP	RANGE	SECTION
3S	1E	

WALTON / HOLMES COUNTIES

TOWNSHIP	RANGE	SECTION
4N	18W	
5N	18W	
6N	18W	

WALTON / WASHINGTON COUNTIES

TOWNSHIP	RANGE	SECTION
3N	16W	

APPENDIX B

DETENTION WITH FILTRATION GUIDANCE

APPENDIX B-1
CAUTIONS CONCERNING THE USE OF DETENTION WITH FILTRATION SYSTEMS

Detention with filtration systems used for stormwater treatment are prone to failure and high operation and maintenance (O & M) cost. Such filters should only be considered where no other Best Management Practice (BMP) is feasible. Sand-filters are only allowed as a treatment BMP in the geographic area of the Northwest Florida Water Management District under the Alternative Design Section of Applicant's Handbook Volume I (Section 8.4.3), Chapter 62-330, Florida Administrative Code (F.A.C.). Applicants who propose the use of filtration systems must provide reasonable assurance that the stormwater management system will satisfy the conditions for issuance listed in Rule 62-330.301, F.A.C., and the general and special limiting conditions listed in Rule 62-330.350, F.A.C. Below, for informational purposes, are several of the issues related to the construction and operation of filtration systems of which applicants should be aware.

Specifically, applicant should be mindful that detention with filtration systems necessitate:

1. Periodic in-situ testing for the life of the system.
2. Routine inspection and cleaning by pressure back-washing to ensure full functionality as designed. Frequently, filters cannot be effectively cleaned or repaired and must be replaced. Annual reports prepared by a registered professional are necessary to ensure continued functionally.
3. Sufficient budgeting for O & M costs, that accounts for inspection and maintenance activities as well as periodic replacement of the filter (e.g., one filter replacement within the first 10 years of operation).
4. Heightened design considerations when proposed for construction in poorly drained soils due to frequent clogging and excessive O & M costs for the owner/operator.
5. Placement of filter systems above the wet season water table; or be designed to eliminate groundwater infiltration.
6. Use being limited to small drainage areas (less than five acres) such as highly impervious commercial/industrial sites that are well stabilized with little potential for erodible soils, organic matter, or other materials to negatively impact the filter function. Filters are not recommended for subdivisions where homeowners' associations are responsible for maintenance.
7. Special protection of the filter bed from the time of construction until the project area has been stabilized to prohibit covering or blinding with silty material, excessive compaction, or damage to the sub-surface drainage system.

APPENDIX B-2
DETENTION WITH FILTRATION CRITERIA CHECK-LIST

Source: Part II, Section 3.10 of Chapter 6 of the *Florida Development Manual* (June 1988)

Filter Construction

1. Geotextile fabrics:
   a. A permeable fabric shall be wrapped around the gravel pack
   b. If the SHGWT is above the filter pipe invert, then designed controls shall be used to the greatest extent practical to prevent or minimize inflow into the filter.

2. The minimum separation between the 'top' of filter and the gravel pack shall be two feet.

3. For a mounded or side-bank filter
   a. Filter side slope no steeper than 3:1
   b. Filter stabilization achieved by at least 3" of gravel (FDOT #57stone), or other suitable material (not sod or seed)

Filter-Drains

4. Filter-drain pipes shall slope towards the discharge pipe (day-lighted pipe) unless site-specific conditions dictate otherwise. In this case, the registered professional must demonstrate that the system will function under these conditions.

5. A gravel-pack shall be required around the filter-drain pipe that meets the following:
   a. The gravel-pack is approximately parallel to the pond floor or side-bank (FDOT)
   b. Use #57 stone or equivalent
   c. An average thickness of at least six inches from the underdrain-pipes (FDOT) shall be used.
6. Filter systems that depress the ground water table shall be allowed, provided that lowering of the groundwater table is restricted to the immediate vicinity of the system. Water tables shall not be lowered to a level that would decrease the flows or levels of surface water bodies below any minimum level or flow established by a water management district Governing Board pursuant to Section 373.042, F.S. or cause negative impacts to the functions provided by water resources on site and adjacent to the project.

Clean-outs

7. Clean-outs shall be provided every 400 feet, at every bend, and at the terminus.

8. A detail of the clean-outs shall be provided on the construction plans that meets the following:
   a. Vertical portions shall be non-perforated
   b. Clean-out cap shall be water-tight.
   c. Clean-outs shall incorporate fittings (wye fittings or bends) that have an angle no less than 45 degrees as measured from the upstream end of the filter pipes.

Erosion control and scour prevention

9. A Registered Professional shall provide an erosion control plan specifically for preventing migration of native soils into the surface of the filter. This may include:
   a. Providing controls around perimeter of filters until drainage basin is stabilized, with subsequent removal;
   b. Providing an overburden of clean filter sand (six inches minimum) above the filter until all upstream areas are stabilized; and with subsequent removal;
   c. Stabilizing the pond floor with suitable vegetation, except for the filter bed, immediately after construction; and
   d. Improving the energy dissipation at any mitered end sections or similar inflow pipes.

Construction Drawings

10. Provide a scaled drawing or detail of the filter's cross-section.
    a. Sufficient detail shall be provided in order to evaluate flow-path lengths.
    b. Dimensions should represent the midpoint or average between the high and low end of the filter-drain.

11. For the filter-drain pipes, drawings shall show invert elevations at the beginning and termination points as well as the bends.

12. If an overflow structure is present, a detail shall indicate the invert elevation(s) at which the filter-pipe(s) connect into the structure.

13. Filter media specifications are to be consistent with the media that meets the minimum performance standards to provide the required removal efficiencies in accordance with Section 8.3 of Applicant's Handbook Volume I.

14. A depth gauge should be provided for each filter system. See comment No. 23, below.

Maintenance Plan

15. The Registered Professional shall develop a site-specific maintenance plan that will include, as a minimum, (a) yearly checklist or equivalent, for use by the maintenance entity, and (b) any other pertinent instructions in order to perform adequate maintenance.

Calculations

16. As a minimum, provide same treatment volume that is required with retention systems.

17. Recovery calculations are provided confirming recovery of the required treatment volume in 72 hours with at least a safety factor of 2 in the design.

18. Filter-drain-pipe capacity analysis is required.
    a. Pipes, flowing full, will carry the maximum developed flow rate.
    b. Use the design hydraulic conductivity without the safety factor.
    c. The minimum pipe diameter shall be six inches.
    d. Manning's solution analysis (or similar analysis with a basis of open channel flow formulas) can only be used when there is essentially no tailwater (other than that

produced by the hydraulic grade-line). When a tailwater exists, Bernoulli or equivalent is required. It must take into account:

i. Head loss from flow through orifices in the filter pipe
ii. Minor and major losses in the pipe.

19. Provide table specifying (a) the individual flow-path lengths and (b) average path lengths for each stage. The individual path lengths should be indicated on the scaled drawing.

20. Provide a table of the 'Darcy' areas for each stage.

21. Incremental draw-down analysis can use either FDM or FDOT methodology or equivalent methodology. Proper use of Modified MODRET or PONDS is also allowable.

22. Hydraulic head must be taken from the stage to the middle of the filter-drain pipe, not the invert of that pipe. However, if a tailwater is present, the hydraulic head must be taken from the stage to the tailwater elevation.

23. A geotechnical analysis must be provided to demonstrate that a retention pond will not function due to impermeable native soil conditions.

Other

24. In order to facilitate the required drawdown test, a depth gauge, staff gauge or equivalent shall be installed and maintained inside each filter pond. The gauge shall meet the following guidelines:
    a. The gauge shall be installed in the pond bottom or any permanent vertical surface extending from the pond bottom, and rise vertically to a height equal to the treatment volume stage elevation, plus one foot;
    b. The gauge shall be marked in one tenth foot increments with vertical elevations indicated at one foot intervals, or be marked with a similar style approved by the District. The elevation representing the treatment volume stage shall be clearly indicated, and shall be verified by survey; and
    c. The gauge shall be mounted and stabilized to prevent movement due to reasonably anticipated conditions.

APPENDIX C

GUIDANCE FOR EVALUATING MINES AND BORROW PIT ACTIVITIES

Background
Borrow pits are considered mining activities by the Department of Environmental Protection (DEP) when the extracted shell, sand, or clay are taken offsite for commercial, industrial, or construction use. According to the Operating Agreement, DEP performs environmental resource permit (ERP) review and agency action for all mines, including sand, shell, and clay mines that have some form of onsite material processing associated with the extraction activity. Therefore, the Northwest Florida Water Management District (NWFWMD) should only be permitting sand, shell, and clay mines that do not involve onsite processing of material, other than very basic screening or scalping for removal of large rocks or debris. These facilities are commonly known as "borrow pits."

General
Borrow pits as well as other types of mines are somewhat unique in that the entire project activity consists of a short-term or long-term excavation of a pit (or pits). Typically, much of the runoff from the activity is directed to the mine pit itself.

Prior to the start of mining, a "Notice of Intent to Mine or Mining Other Resources" must be provided to DEP if the mining will exceed 20 acres. Upon completion of the mining, there is a DEP requirement that lands disturbed by mining operations be reclaimed (see Chapter 378, F.S. for reclamation requirements). The reclamation requirement applies to mines of any size. This is in addition to, and separate from, any ERP permitting requirements. The finished project is typically a borrow "lake" that may be planted with littoral zone vegetation. Some pits may be excavated in-the-dry~~,~~ and may ultimately be stabilized with upland vegetation per Department reclamation rules. Additional information about the Mining and Mitigation Program is available at: Department of Environmental Protection, Bob Martinez Center, 2600 Blair Stone Road, MS 3577, Tallahassee, Florida 32399-2400, 850-245-8336 and website https://floridadep.gov/Water/Mining-Mitigation. Additional information, such as applicable rules, may be found at https://floridadep.gov/water/water/content/water-resource-management-rules#Mining.

There is the potential for confusion on what criteria apply to mines and borrow pits, particularly with respect to activities in Karst areas, and also for use as stormwater management facilities for future development. There are also issues related to mines and borrow pits that may be exempt from ERP permitting.

Permits Required and Grandfathering
Section 373.4145(6)(a), F.S. provides that the operation and maintenance of any activity legally in existence prior to October 1, 2007, may continue without the need for an ERP permit. For the purposes of evaluating mines and borrow pits, this means that a mine or borrow pit operator may continue to extract material from a pit that was ***existing*** prior to October 1, 2007, provided they do not encroach beyond the limits of land that has been prepared for excavation prior to October 1, 2007. Land prepared for excavation includes those lands intended for immediate excavation and may involve preparation such as land clearing, root raking, removal of top soil, etc. A pit existing prior to October 1, 2007 that has no additional land prepared for excavation, may also continue to extract material in the vertical direction within the footprint of the existing disturbed area.

Any new mines or borrow pit~~s~~, or expansion of existing mines or borrow pits that necessitates additional preparation of land for excavation that occurs after October 1, 2007, must obtain an ERP permit prior to initiating construction or land clearing activities. Compliance and enforcement activities related to such facilities that were legally in existence prior to October 1, 2007, and that have not expanded the operation such that an ERP is required, remain the responsibility of DEP. Similarly, compliance and enforcement activities for borrow pits that require an ERP permit will typically be the responsibility of the NWFWMD, subject to the terms of the Operating Agreement.

A permit is not required for certain borrow pits that fall within the general criteria listed under Subsection 62-330.051(17), F.A.C. This includes the construction, alteration, operation, maintenance, repair, reclamation, or abandonment of a dry borrow pit for excavation of sand and other soil materials. Borrow pits that fall within this exemption are to provide notice to the Agency 30 days prior to the commencement of construction, the area of excavation for the borrow pit is to be less than five acres, when measured at the natural land surface grade of the pit, and the borrow pit is to be constructed entirely in uplands for the purpose of using the borrow materials as appropriately permitted, authorized, or as exempted.

Borrow pits that fall under this exemption are subject to the mine reclamation requirements under Part III of Chapter 378, F.S. if excavated materials will be used offsite for commercial, industrial, or construction use.

Mines and Borrow Pits NOT Within Sensitive Karst Area

Generally, for mines and borrow pits, much or all of the stormwater runoff is directed to the pit. Often, the pit operator will construct a perimeter berm in order to keep all runoff onsite during all phases of construction. Many times, there will be no discharge of surface water from the site, even for extreme storms, and freeboard measuring several feet may be provided. In most cases, a no-discharge condition may be asserted if an applicant can demonstrate that available storage at any time during excavation exceeds the cumulative volume from back-to-back 25-year, 24-hour storm events.

Where there is a reasonable assertion that the site will operate under a no-discharge condition, the pit acts as retention for water quality purposes. This is generally the case whether the activity is a wet pit or excavated in-the-dry. The conditions for issuance for water quality are met without further demonstration. However, the applicant must demonstrate that this excessive storage is available throughout the life of the project by showing various construction phases and stages. Alternatively, the applicant may demonstrate that the pit has sufficient capacity to meet the water quality requirements in Part II of Volume I and Part IV of Volume II, including the volume recovery components. Any activities such as access roads or office and parking areas that discharge off-site, must meet stormwater treatment requirements prior to discharge, similar to other types of development. Locations where petroleum products or other hazardous materials are stored, and equipment maintenance areas may need separate containment system to prevent direct stormwater flow to the pit.

For those pits that trip the thresholds in Part III of Volume II, the applicant must demonstrate that post-development peak discharge rates do not exceed pre-development peak discharge rates. For no-discharge pits, this is a fairly simple demonstration as the site will easily contain the entire storm volume in the post condition.

Any proposed mine or borrow pit application that cannot demonstrate a no-discharge condition, must be reviewed under the same criteria as any other type of development.

If a pit is owned entirely by one person, surface water quality standards will not apply in the mine or borrow pit, except with respect to potential discharges to groundwater and to offsite waters. However, if the mine or borrow pit is intended to become a stormwater management system for future development, the system is required to meet the water quality standards listed under Part II of Volume I and Part IV of Volume II. A permit modification would be required to convert the system to a standard best management practice, calculate the projected pollutant loading of the future development, and demonstrate that the system has met the requirements of Section 8.3 and Section 9.0 of Volume I.

Mines or Borrow Pits WITHIN the Sensitive Karst Area

The criteria as described above for mines or borrow pits not within Karst areas apply to pits within Karst areas with a few important exceptions. Reasonable assurance must be provided demonstrating that groundwater quality standards will not be violated by excavation activities that have the potential to penetrate confining layers~~,~~ or are working directly in the limestone. Breaching of confining layers or working directly in the limestone presents a high potential for direct discharge of untreated stormwater in the aquifer. In such cases, site grading must direct runoff from areas that are potential sources of pollutants into stormwater management areas that are designed in compliance with Part II of Volume I and Part IV of Volume II prior to discharge to the pit. Entrance roads, parking areas, buildings, vehicle maintenance and wash areas, and any storage of petroleum, etc., are examples of areas that are potential sources of pollutants. Normal activities associated with excavating, stockpiling, and loading of material are not considered potential sources for pollutants. **Section 6.4 of Volume II** contains additional information on measures required to protect Karst areas.

Storage within mines or borrow pits in karst areas that breach confining layers or directly contact limestone may be used in future development to ***attenuate peak discharge only***, and then only if stormwater treatment is provided prior to discharge to the pit. Finished pits may only be used for stormwater treatment if the pit meets the criteria in **Section 6.3.2. of Volume II**, or if the applicant can demonstrate that groundwater standards will not be violated. The applicant must also demonstrate that confining layers have not been breached.

Additionally, the potential for inducing sinkholes or solution pipes within a mine or borrow pit must be evaluated for all proposed pits within the Karst area. For dry pits, vertical staging of stormwater is limited to a maximum of 10 feet of water depth. For wet pits, vertical staging of stormwater is similarly limited to a maximum of 10 feet of water depth above the seasonal high groundwater table elevation.

Other Hydrological Considerations

Mines or borrow pits that are excavated in wet conditions have the potential to affect adjacent wetlands, particularly those pits employing multiple work areas, and where pumping is involved. Particular attention should be paid to significant changes in water table elevations when adjacent wetlands may be considered within the zone of influence. Depending on soil permeability, this zone of influence may range from 100 to 500 feet, or more. The applicant must demonstrate that any significant decrease in onsite water table elevations will not negatively affect adjacent wetland systems. Similarly, the influence of onsite depressed water tables must be evaluated for any negative impacts on any shallow well systems located nearby.

APPENDIX D

PROCEDURES FOR EVALUATING PROPOSED ACTIVITIES FOR SITES PREVIOUSLY PERMITTED UNDER CHAPTERS 62-25 AND 62-346, F.A.C.

Background
Authorizations for activities regulated under Chapter 62-25, Florida Administrative Code (F.A.C.), have been issued in the geographic area of the NWFWMD since the early 1980's by DEP. Activities legally in existence prior to the implementation of Chapter 62-25, F.A.C., were considered "grandfathered."
In 1996, delegation to issue authorization for stormwater general permits under Chapter 62-25, F.A.C., was granted by DEP to the City of Tallahassee. The City has not written any 62-25 permits under Chapter 62-25, F.A.C. since October 1, 2007, when the delegation ended in conjunction with the implementation of the Environmental Resource Permit program rule for Northwest Florida, Chapter 62-346, F.A.C., on October 1, 2007. The Environmental Resource Permit program rule was updated to Chapter 62-330, FAC, on October 1, 2013. These rules, including the Northwest Florida Water Management District AH Vol 2, were again updated in 2023 pursuant to legislative direction in 2020. [Date]

Along with specific exemptions for systems serving single family residences, Chapter 62-25, F.A.C. provided two authorizations — construction permits (Rule 62-25.040, F.A.C.) and general permits (Rule 62-25.035, F.A.C.). Construction permits were required for projects that did not meet the conditions for issuance for a general permit. Authorizations to use the general permit were verified for those projects that met specific criteria in Rule 62-25.035, F.A.C. These authorizations were not modifiable and also do not expire. As of October 1, 2007, authorizations under Chapter 62-25, F.A.C., will no longer be issued, except for activities that meet the grandfathering provisions of Section 373.4131(4), F.S. Activities already constructed in conformance with Chapter 62-25, F.A.C., and Section 373.4131(4), F.S., may continue to be operated and maintained without a new permit under Chapter 62-330, F.A.C. In addition, as of October 1, 2013, permits for projects will no longer be issued under Chapter 62-346, F.A.C. Those activities that qualified for grandfathering under Section 373.4131(4), F.S. will continue to qualify.

New Permit Required
Modifications that are proposed to a system authorized by a Chapter 62-25, F.A.C., general or construction permit, or under Chapter 62-346, F.A.C., will require a new permit or permit modification under Chapter 62-330, F.A.C. and subsequent updates, under the following circumstances:

- ***Project Area*** – Whenever an~~y~~ increase is proposed in the treatment area from that originally authorized under the Chapter 62-25, F.A.C., permit.
- ***Project Design*** — When the permitted project design is altered, such as ~~by~~ adding new impervious area within the area served by the treatment BMP, or in a way that can be expected to result in more than minimal or insignificant individual and cumulative adverse impact on the water resources of the District.
- ***Treatment BMP*** – When the treatment BMP is altered in a manner that:
    - Changes the treatment type;
    - Modifies the treatment train by adding or removing other BMPs;
    - Changes the capacity of the treatment facility; or
    - Changes the discharge in a manner that may result in un-permitted impacts~~,~~ or relocates the point of discharge for the treatment system.
- ***Alteration in the Activity*** – A change in the system on site that may increase the pollutant load.

- ***Alterations to an Activity Previously Permitted***
  Where activities for a proposed project require a permit under Chapter 62-330, F.A.C., for the conditions described above, and the project area is contained wholly in or extends into an existing project that was permitted under Chapter 62-25 or 62-346, F.A.C., the new project area shall be considered a separate project requiring authorization under Chapter 62-330, F.A.C. The remaining project area for the activity permitted under Chapter 62-25 or 62-346, F.A.C., may continue to be operated and maintained under the existing authorization.

The permit required under Chapter 62-330, F.A.C., will be limited only to the modified project area, provided:
- Runoff from the modified project area is served by its own system, and does not co-mingle with or discharge to treatment systems authorized by the Chapter 62-25 or 62-346, F.A.C., permit; and
- The final plans in the Chapter 62-25 or 62-346, F.A.C., permit provide a description of the changes to the permitted activity.

Activities that do not increase the authorized treatment area, do not alter the treatment BMP, and result in no change in activity type, remain authorized under the previous Chapter 62-25 or 62-346, F.A.C., permit and do not require a new Chapter 62-330, F.A.C., permit. The applicant is recommended to reach out to the District and DEP to confirm and ensure no further permitting is required.

Guidance
When trying to determine if a proposed activity is authorized under Chapter 62-25, F.A.C., identify the following:
- ***Construction date of the facility***
  - Pre-1982, the site may have been grandfathered and no permit would exist.
  - Between 1982 and September 30, 2007, the site may have a Chapter 62-25, F.A.C., permit.
  - Between October 1, 2007 and October 1, 2013, the site may have a Chapter 62-346, F.A.C., permit.
- ***Location of the facility***
  - Facility is located in Tallahassee:
    - Permitted between 1996 and 2007; permit may have been issued by the City of Tallahassee.
    - Permitted prior to 1996, or for State or City projects; permit may have been issued by DEP.
    - Originally constructed between 1996 and-2007, and does not have a City issued Chapter 62-25, F.A.C., permit — a permit may have been issued by DEP due to annexation into the City after the time of permitting. This is because the limits of the City Tallahassee are subject to change.
  - Facility is located outside the City of Tallahassee, regardless of the construction date — a permit may have been issued by DEP (i.e., the activity required a construction permit, and did not qualify for a general permit under Chapter 62-25, F.A.C.).

- ***Permitting Agency***
  - The original permitting Agency should conduct the initial review of the proposed activities to determine if they are covered under the previous authorization. When such a determination is made, the original Agency shall notify the permittee as to the entity that needs to review and authorize the proposed modification.
  - If it is determined that the proposed activities are not authorized by the original permit, then the permittee/applicant shall be directed to the Agency currently responsible for regulating that activity under Chapter 62-330, F.A.C.
    - Agency responsibilities are defined in the operating agreement between DEP and the NWFWMD, which can be found in Appendix A, of Volume I

Where No Previous Chapter 62-25, F.A.C., Permit Exists

- If a previous permit cannot be located, and proof of one having been issued cannot be provided by the applicant, the proposed activities will be reviewed under Chapter 62-330, F.A.C., as new construction. Treatment is only required under Chapter 62-330, F.A.C., for the system serving the new activity. Where the new activity cannot be isolated from larger drainage basins, including run-off from off-site sources, treatment must be provided for the entire drainage basin that includes the new activity.

NOTE: Generally, new activities that exceed permit thresholds for a site that was in existence prior to the implementation of Chapter 62-25, F.A.C., (February 1, 1982) that was considered grandfathered under Chapter 62-25, F.A.C., will result in the need for a new ERP permit.

APPENDIX E

Part II, Section 3.10 of Chapter 6 of the *Florida Development Manual: A Guide to Sound Land and Water Management* (June 1988)

PART-II

Design Criteria for Stormwater Filtration Facilities

Filter systems for stormwater quality renovation may be used in conjunction with either wet or dry detention facilities. The bottom elevation of the former is below the grade line of the underdrain pipe. Conversely, subsurface drains are normally located in the lowest portion and be ow the bottom of dry detention facilities.

Examples of stormwater filtration systems include:

1) Filter systems in the banks of wet detention facilities A typical cross section is illustrated in Figure 6-44. A slightly modified version of this particular style of discharge control structure is shown in Figure 6-45. The major difference between the two consists of a "flash board" type riser for adjustable depths of detention and flood control. Also notice that underdrains enter at the base of the riser pipe in the system shown in Figure 6-45 as opposed to entering somewhere along the outlet pipe as shown in Figure 6-44.

2) Bank filter systems used in conjunction with online or offline wet detention facilities which use the natural in place soils for filtration in conjunction with underdrain pipe for drainage. (See Figure 6-46).

3) Raised filtration beds projecting outward toward the center or extending along the sides of wet or dry detention facilities. (See Figures 6-47, 48, 49 and 50).

FIGURE 6-44

Cross-Section of Stormwater Discharge Structure with "Mixed Media" Bank Filter System (Wet Detention Facility)

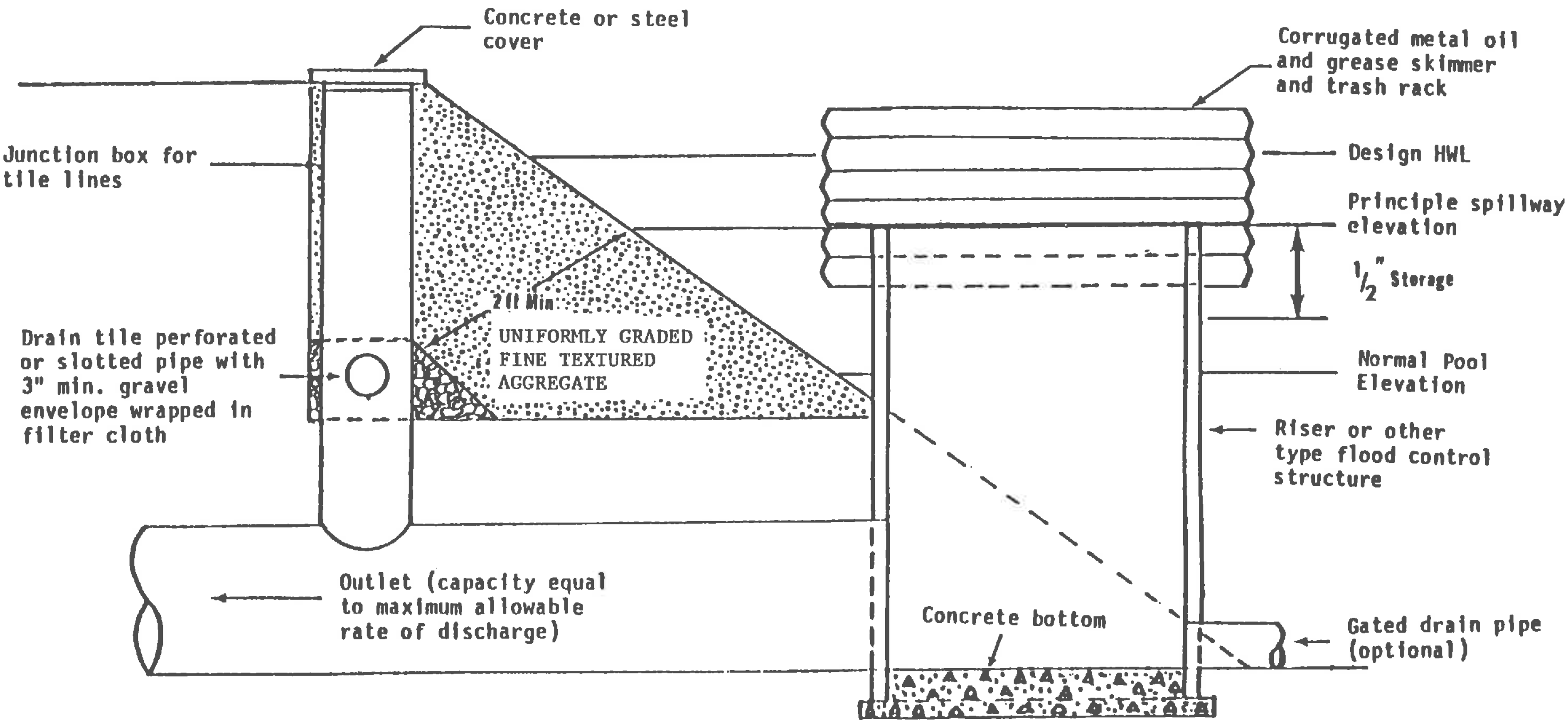
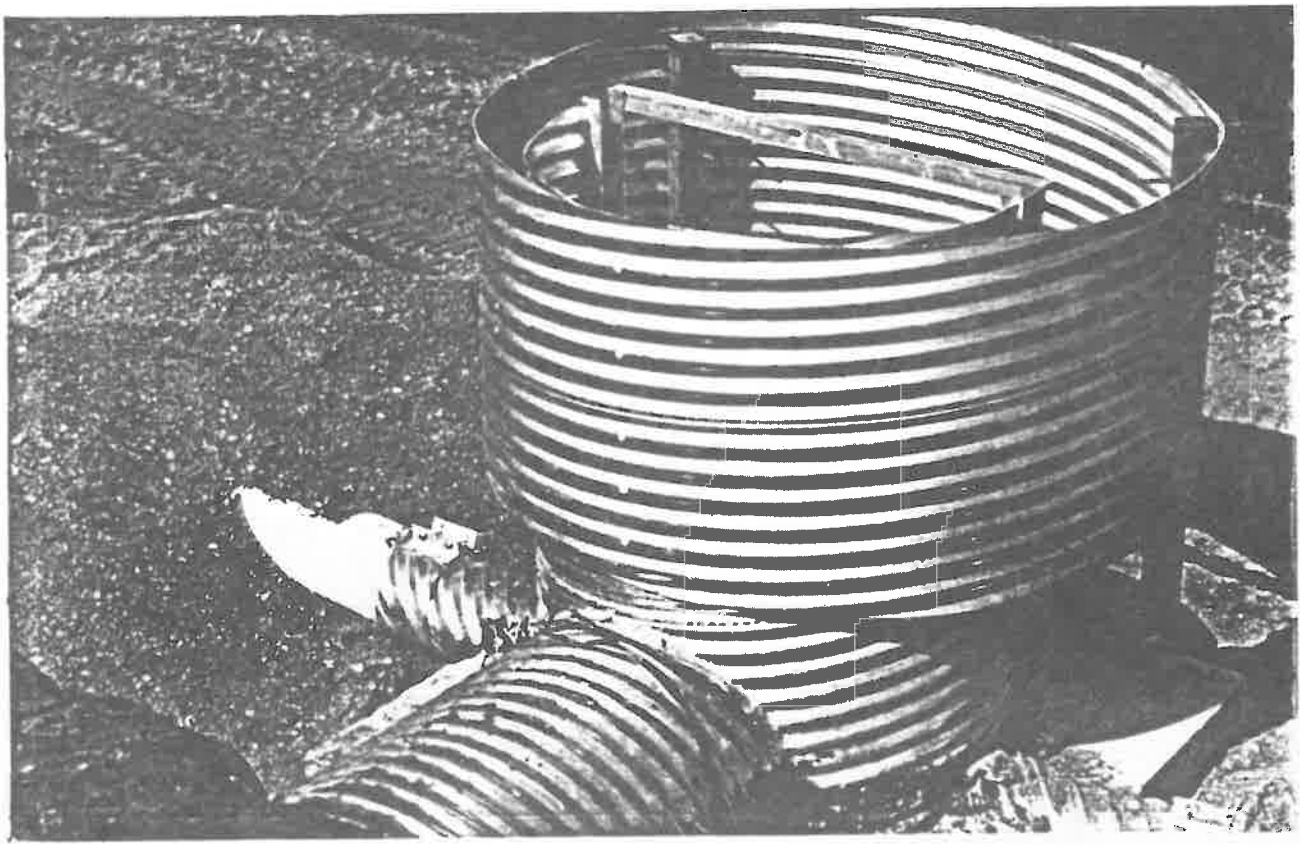


FIGURE 6-45

Typical Stormwater Control Structure @ Orlando Jetport with Bank Filter or Underdrain Pipe to Treat Runoff and Flash Board Riser for Adjustable Levels of Retention and Flood Control



This unit is a custom prefab. The structure consists of a 30-inch outlet pipe, 48-inch riser, and 12-inch underdrain headers; all aluminum construction. (It is suitable for both wet or dry detention facilities).

(Courtesy of Mr. Charles King, P.E., Greiner Engineering Sciences, Tampa, Florida).

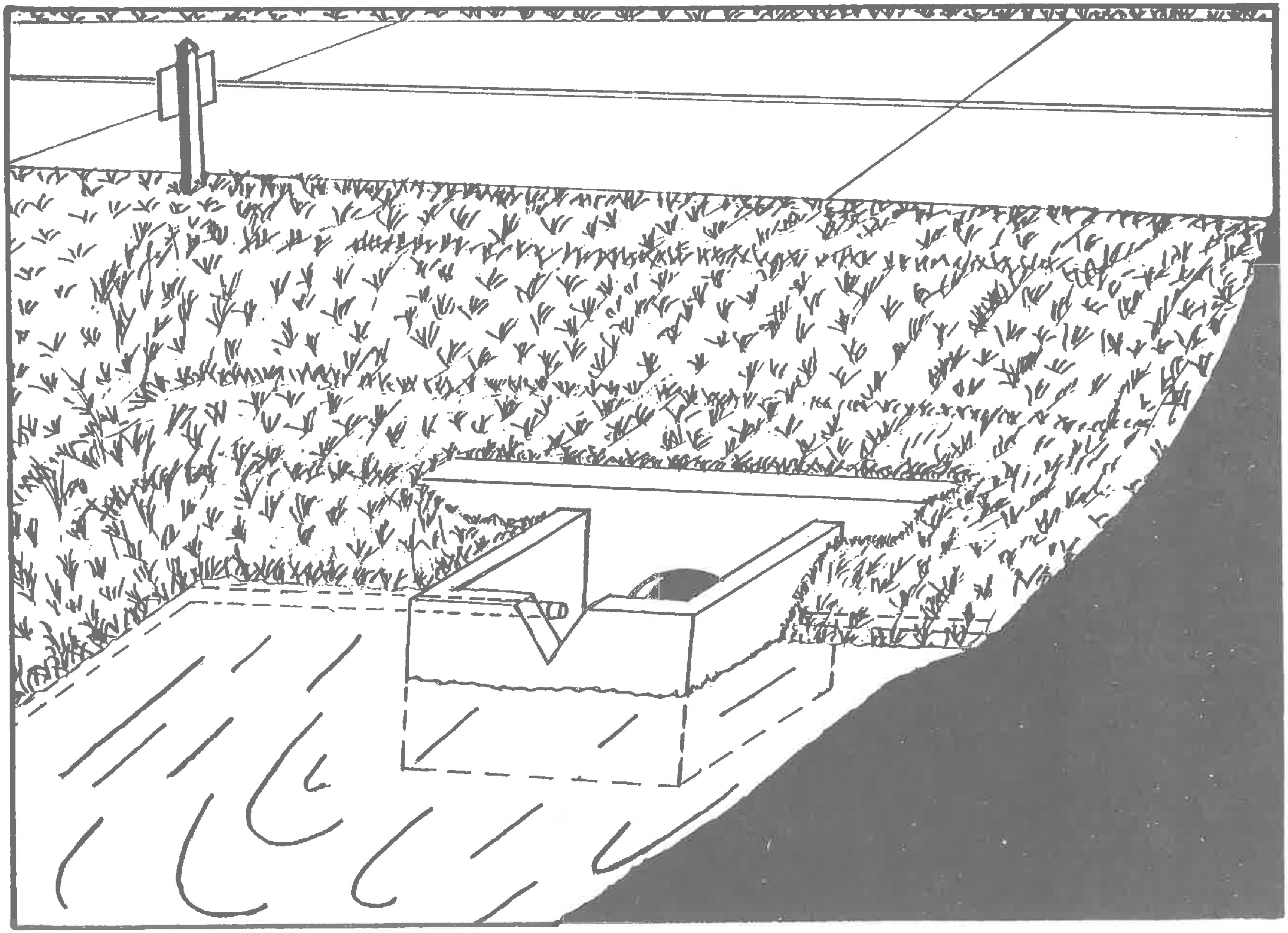


FIGURE 6-46

Illustration of Typical "Natural Soil" Bank Filtration System with Box Inlet Drop Spillway and "V" Notched Weir. (Wet Detention Facility)

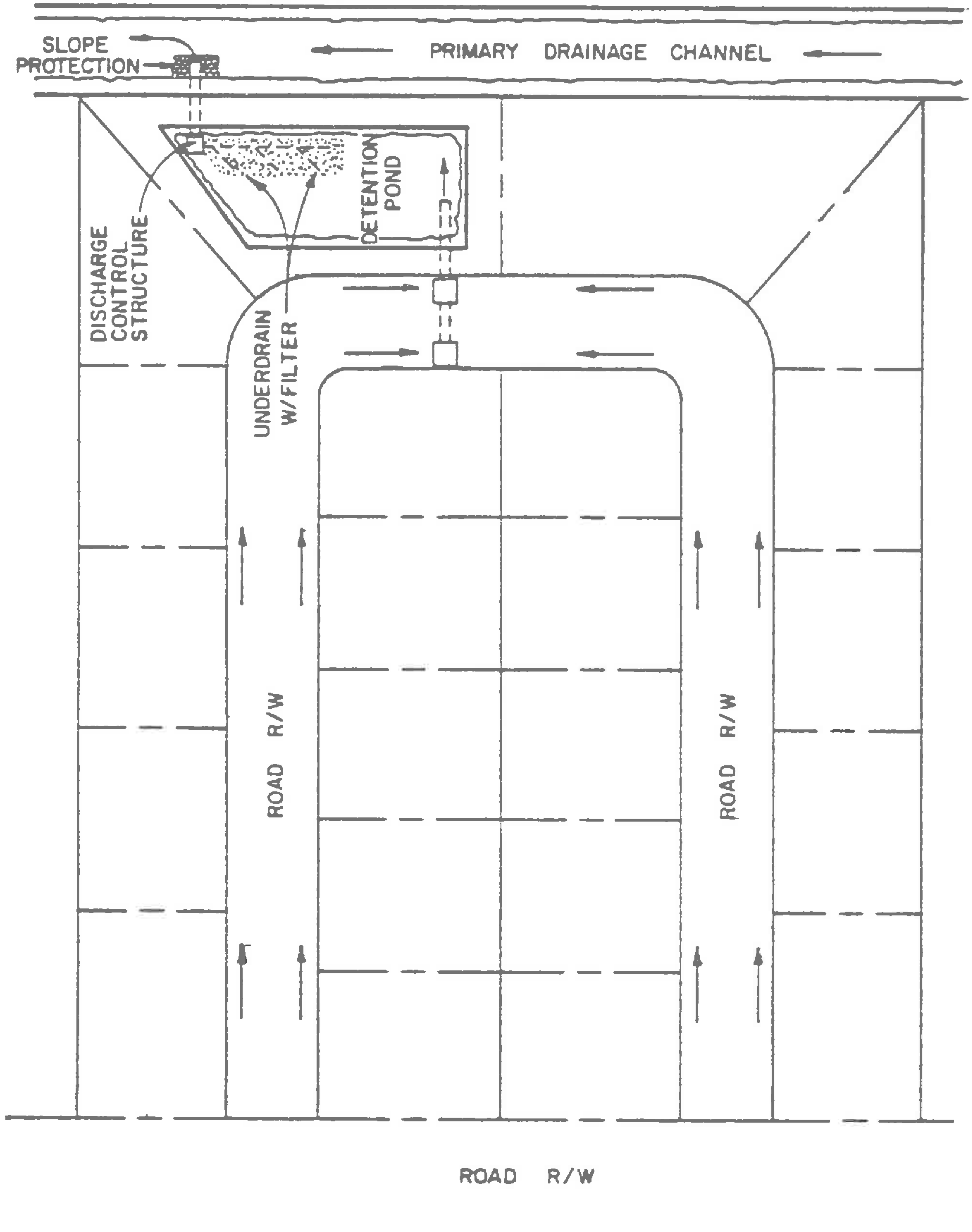


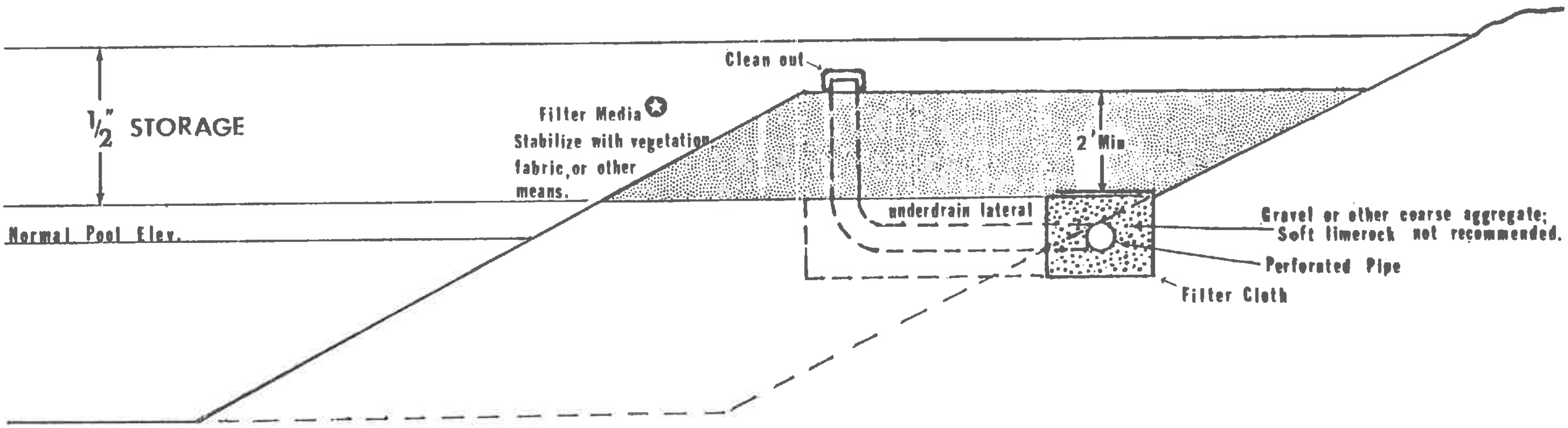
FIGURE 6-47

TYPICAL SUBDIVISION LAYOUT SHOWING ON-LINE DETENTION POND AND OUTFALL
(Courtesy of Pinellas Park Water Management District)

FIGURE 6-48

Typical Cross-Section of Elevated Bank Filtration Bed Used in Conjunction with Wet Stormwater Detention Facilities

SECTION A-A



✪ Effective Size $\leq$ 0.55mm.
Uniformity Coefficient $\geq$ 1.5
Washed; no more than 1% fines recommended

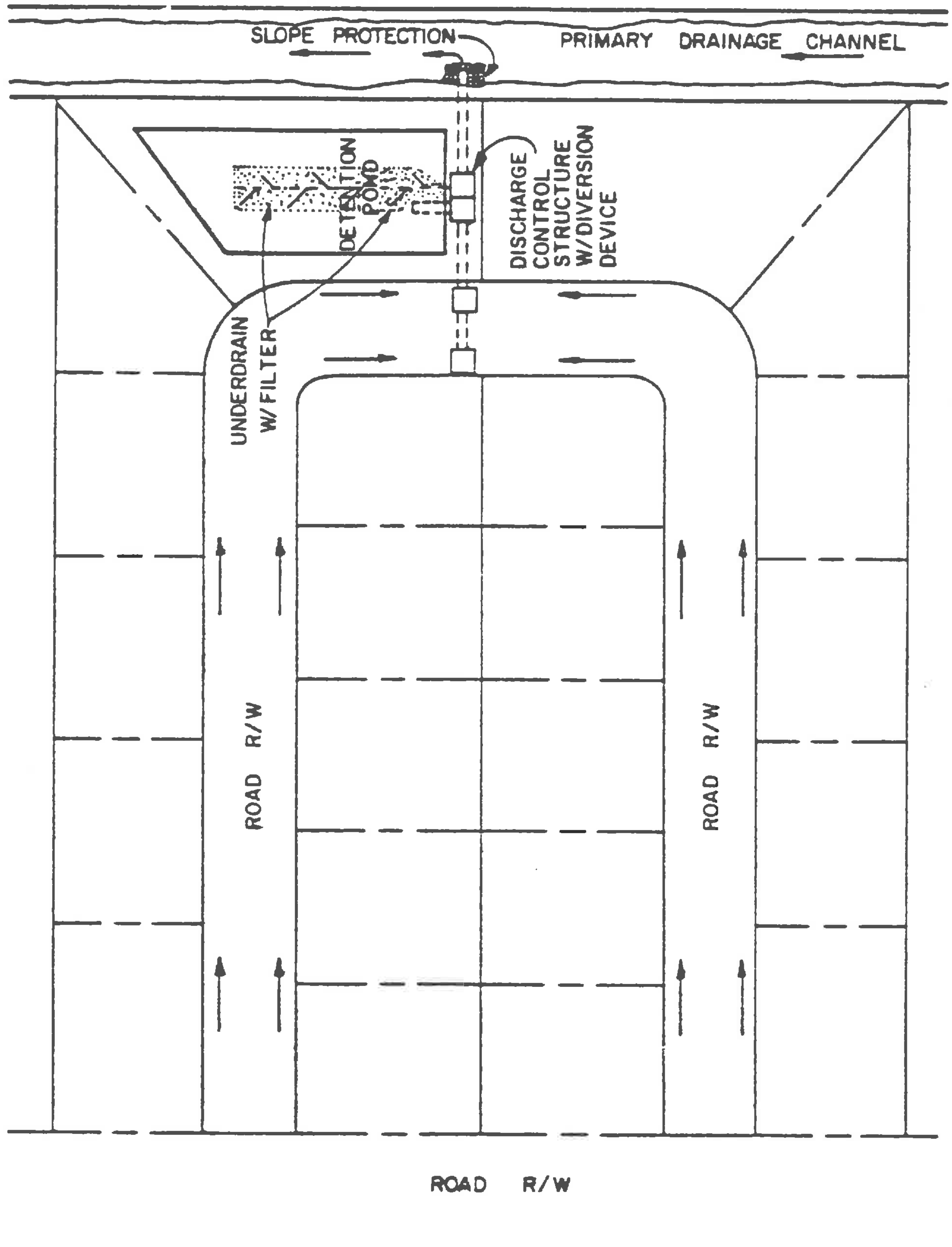
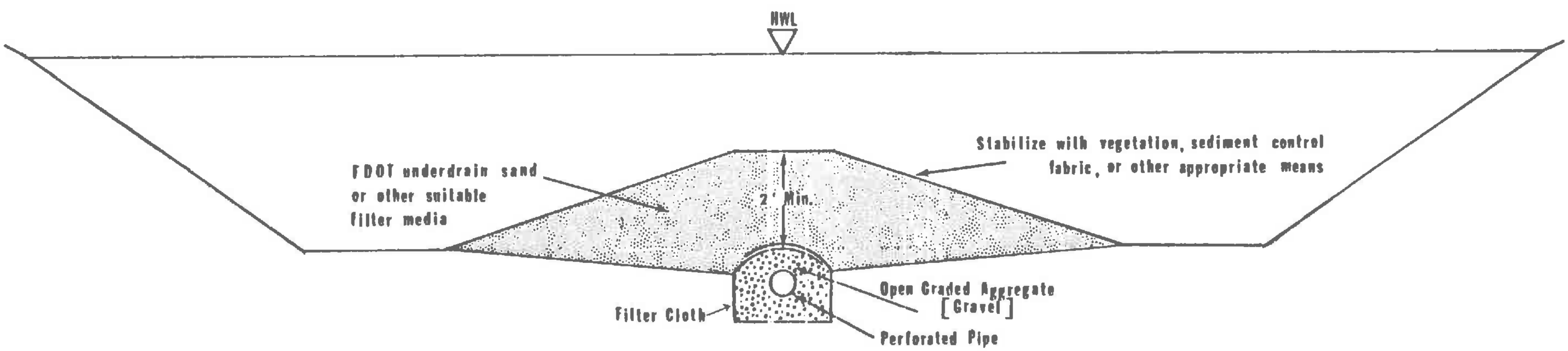


FIGURE 6-49

TYPICAL SUBDIVISION LAYOUT SHOWING OFFLINE DETENTION POND AND OUTFALL
(Courtesy of Pinellas Park Water Management District)

FIGURE 6-50

Typical Cross-Section of Elevated Sand Filter for Stormwater Treatment Used in Conjunction with Dry Detention Facility



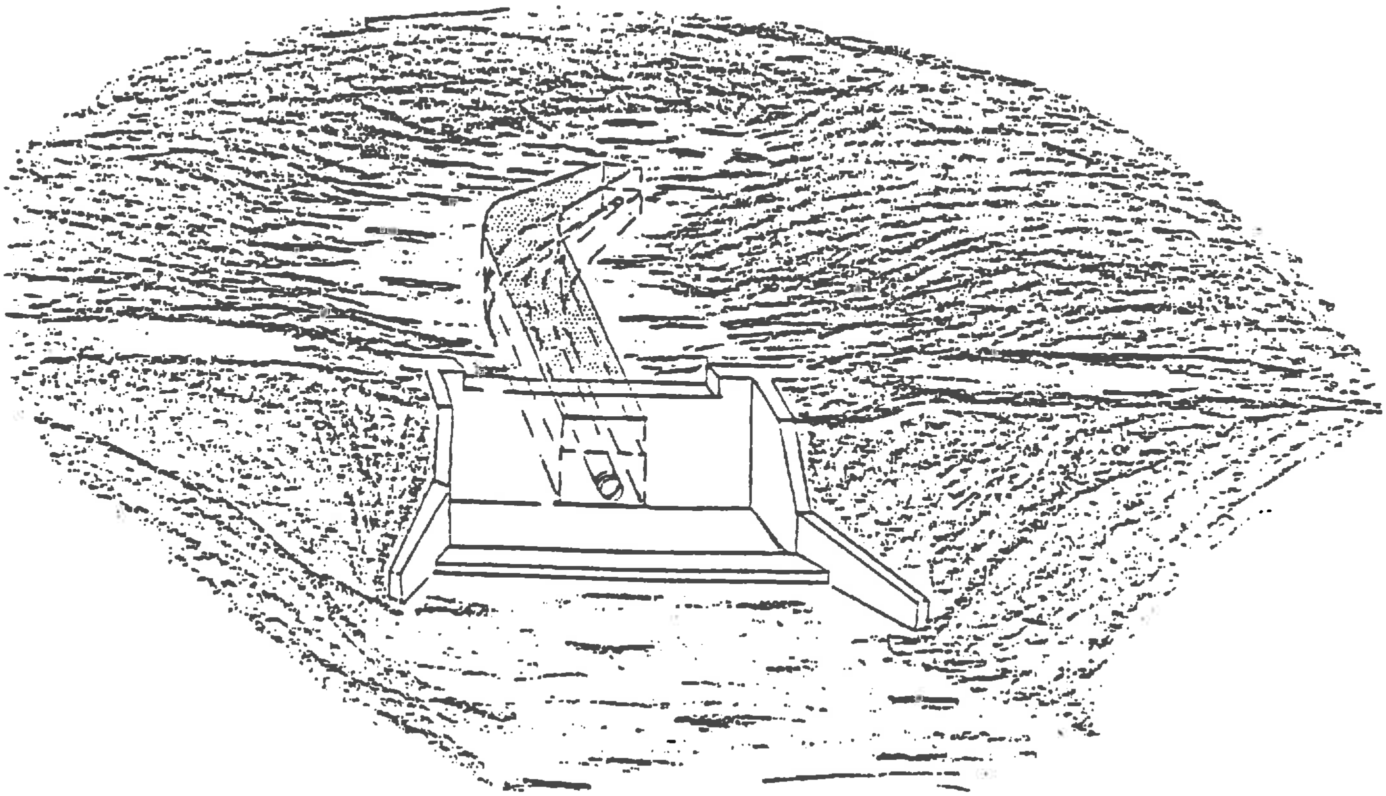


FIGURE 6-51

Illustration of Bottom Filter or Underdrain System in Conjunction with Rectangular Weir and Drop Spillway (Normally Used with Dry Detention Facilities and Swales on Tight Soils and/or Steep Slopes)

4) Sand filter systems installed in the bottom of swales to improve percolation. (See Figure 6-51).

How does the Stormwater Rule standards listed in Chapter 17-25, F.A.C., affect these systems?

Underdrain systems and bank filters which use natural soil for filtration do not have to meet the requirements in section 17-25.025(2), F.A.C. pertaining to effective grain size, uniformity coefficient, etc.

However, these systems must be designed to prevent piping both within and through the filter. They are also subject to the 2 feet minimum flow requirements specified in Section 17-25.02(8), F.A.C.

Filter systems which use an aggregate other than natural soil for filtration must satisfy the standards listed in Section 17-25.025(2), F.A.C. The current standards for filter media are summarized and noted at the bottom of Figure 6-48.

Material with effective sizes less than .20 millimeters are acceptable to the Department for pollution control purposes. However, applicants may find the permeability to be restrictive.

Likewise, material mixed with organic matter or colloidal material may improve pollutant removal. However, anything more than slight amounts of material less than 0.074 millimeters in size has the potential to reduce hydraulic capacity quite substantially. The improvement in removal efficiency of such practices is still being tested by Dr. Wanielista and others at this time.

Design Procedures for Sizing Stormwater Filtration Systems

Underdrain design procedures will often involve the use of "spacing equations" to determine the area over which the drainage network can be expected to function to drain the proper amount of water in the required time frame.

Filter systems are usually designed by trial and error. In this procedure drainage capacity is checked for compliance with various regulations until a suitable configuration (e.g., trench area, depth, pipe diameter, and hydraulic conductivity of filter media) is achieved to meet drawdown time and grain size requirements. In terms of stormwater treatment, the Department is interested in the various design procedures from the standpoint that underdesign will result in reduced hydraulic capacity. This, in turn, will result in a reduction in storage between subsequent rainfall events and an associated decrease in the annual average volume of stormwater treated resulting in a reduction of pollutant removal. Such circumstances also reduce the aesthetic value of the system and may promote mosquito production.

In most cases, various forms of the Darcy Equation for saturated flow through porous media are used to design filters. The equation is written:

$$Q = K\ i\ A$$

Where:

Q = Flow in ft^3/hr
K = Permeability rate of filter media (ft/hr)
i = Hydraulic gradient (ft/ft)
A = Area of the aquifer or water bearing strata intersected (ft^2)

The basic equation is applied in a number of different ways.

1) **Calculating the length of a bottom filter and determining drain pipe size.**

a) Possibly the most simplistic application of the Darcy Equation involves a slight manipulation in the formula such that the designer may determine the length (L) of a bottom filter, as illustrated in Figure 6-52, to treat and dewater an area sized to hold either the first one-half inch of runoff or the runoff from the first inch of rainfall. The flow (Q) of water reaching the underdrain pipe is assumed equal to its average velocity as it moves through the filter profile multiplied by the cross sectional area of the aquifer or filter trench intersected.

The velocity of flow is assumed proportional to the soil hydraulic conductivity (K) at a hydraulic gradient of unity (i.e. i=1).

The cross-sectional area intersected (A) is usually assumed equal to the average width of the drain field or trench (W) times the length of the drain (L). In mathematical form:

$$A = WL \text{ and therefore, } Q = KiWL.$$

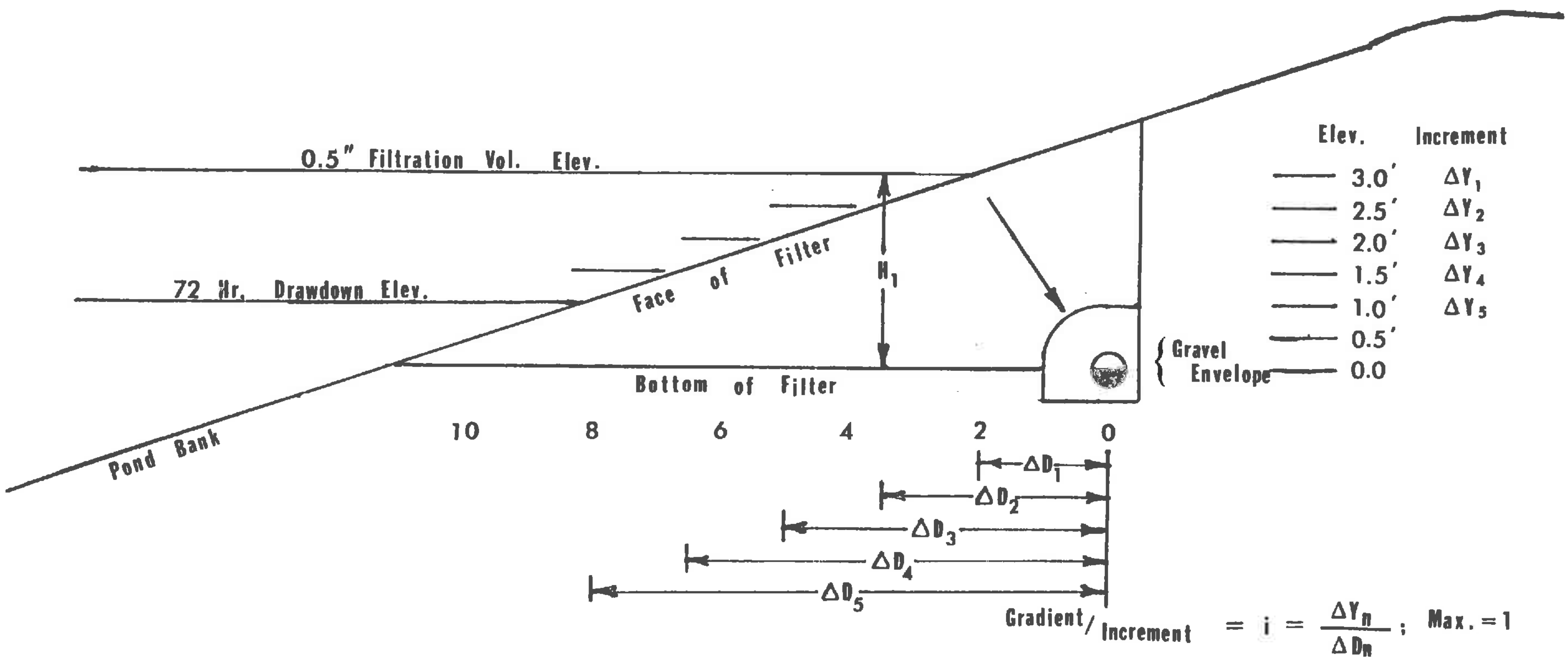
The drain length is unknown but can be determined by rearranging the equation if the width of the trench is known:

$$L = \frac{Q}{KiW}$$

The flow (Q) is based on the storage volume which must be removed in the time frame desired. (K) is determined based on field permeability test or other information. The value of (W) is determined at the discretion of the designer. Its value normally will vary depending on the depth of the drain and the size pipe required.

FIGURE 6-52

Cross-Section of Bank Filter Illustrating Parameters Used in Calculating Drawdown Time with Darcy's Equation for Lateral Flow Situations



Example:

Suppose a facility is required to store one-half inch of runoff water per acre of project area as per state design criteria. Further, suppose the project is twenty acres in size with the hydraulic conductivity or permeability of the filter media (K) equal to ten inches per hour. The design calls for a three feet wide filter trench with the underdrain system installed two and one-half feet below the bottom of the detention facility for drawdown.

The length of filter and underdrain pipe needed to satisfy FDER criteria may be derived using the equation:

$$L = \frac{Q}{KiW}$$

Based on information presented in the paragraph above the one-half inch volume of runoff which must be temporarily stored and treated would equal ten acre-inches or 36,300 cu. ft. To drain the entire amount (36,300 cu. ft.) in three days (72 hrs.) would require an average rate of outflow equal to 504 cu. ft./hour. The length (L) of underdrain needed to satisfy the three day requirement is:

$$L = \frac{Q}{KiW} = \frac{504 \text{ cu. ft./hr.}}{(.833 \text{ ft./hr.})(1)(3 \text{ ft.})}$$

$$L = \underline{202 \text{ ft.}}$$

However, let's suppose the detention area was also planned for use as open space such as a park or recreation facility. In this instance it would be desirable to discharge the stored water within a day. The length (L) needed to provide sufficient drawdown in 24 hours would be:

$$L = \frac{1513 \text{ cu. ft./hr.}}{(.833 \text{ ft./hr.})\ (1)\ (3 \text{ ft.})}$$

$$L = \underline{606 \text{ ft.}}$$

In either instance, the discharge capacity of the underdrain pipe must be equal to or greater than the flow intercepted (Q). Given the latter circumstances, the pipe should be sized to carry at least a flow equal to 1513 cu. ft./hr. Converting this value to 0.42 cfs., the pipe size may be determined in accordance with procedures mentioned earlier (see Part I). Using the SCS Drain Capacity Charts included in this section, 8" corrugated polyethylene pipe (CPEP) with a roughness coefficient (n = 0.015) at a grade of 0.5% or .005 ft./ft. would be capable of conducting the proper amount of water.

The results of this analysis are based on several simplifying assumptions that would rarely, if ever, occur. The hydraulic gradient (i), for example, does not remain constant. The value will change as the water level in the facility rises and falls. A more detailed assessment procedure capable of ascertaining the difference in drainage capacity under variable head conditions would reduce the amount of drain required.

Likewise, the assessment also presumes there are no other contributions from sources such as groundwater. Artesian type conditions would be expected to occur should high water table elevations surround the treatment area during any portion of the year. The size of the underdrain pipe would need to be increased. An increase of 1.5 to 2 times should be used in these situations.

b) A more complex method to determine if a specific design will satisfy the drawdown requirement under various head conditions is currently used by several engineering companies. The procedure combines Darcy's Law and the Falling Head Equation into a form similar to that used to determine the hydraulic conductivity (K) from falling head permeability testing techniques. The equations may be rearranged to solve for either drawdown time (t) or filter area (A) if the hydraulic conductivity (K) is known, and certain simplifying assumptions are made.

$$K = \frac{2.3\ aL}{A\ dt} \quad \mathrm{Log}_{10} \ \frac{h_0}{h_i}$$

$$dt = \frac{2.3\ aL}{AK} \quad \mathrm{Log} \ \frac{h_0}{h_i} \text{ and,}$$

$$A = \frac{2.3\ aL}{Kdt} \quad \mathrm{Log} \ \frac{h_0}{h_i}$$

In these equations <u>a</u> is the average cross-sectional area of the pond or reservoir; <u>A</u> is the cross-sectional area of the soil profile or filter served by the drain tube; <u>L</u> is the length or depth of the soil profile (filter media) through which the water must travel to reach the gravel envelope or perforated pipe; in most cases that value will be a minimum or two feet; and <u>dt</u> is the time interval (hrs.) during which the elevation drops from its initial value (h_o) to some lower value (h_i) as the water approaches the pond bottom.

Example: Assume our objective is to estimate the time to remove the 36,300 ft^3 of runoff discussed in the previous problem. The area of the facility is assumed to remain constant as the water receeds. The head (h_o) at the time when the facility was full would equal 5.0 feet. The average area (a) may be calculated by dividing the pond volume by the depth. In this case presume the pond was designed 3 feet deep.

Working through the equation when the facility was full to determine the area (A) of filter needed:

a = 12,100 square ft.
L = 2.0 ft. (average length of travel)
K = 10 in/hr. or .833 ft. hr.
h_o = 5.0 ft.
h_i = 2.0 ft.
dt = 24 hrs.

Therefore:

$$A = \frac{2.3\ a\ L}{Kdt} \quad \text{Log}\ \frac{h_o}{h_i}$$

$$A = \frac{2.3\ (12{,}100)(2.0)}{.833\ (24)} \quad \text{Log}\ \frac{5.0}{2.0}$$

$$A = \frac{55{,}660}{20} \times 0.40$$

$$\underline{A = 1{,}113\ ft^2}$$

Given no accretion from other sources, a filter three ft. wide by 371 ft. long should be capable of draining the facility. As may be seen, the results using this procedure will be much more favorable to the applicant since the drain length is reduced. This is because the previous assumption relative to a hydraulic gradient of unity (i = 1) is not used in this procedure. The procedure is much less conservative than the former.

The designer should also notice that this analysis is dependent on the presumption that the size and slope of the drain tube as well as the number and size of pipe orifices or openings will not restrict the maximum peak flow delivered to the drain tube via the filter media.

In using the equation above, pipe size must be checked using the pipe flow capacity charts mentioned previously. Likewise, orifice area must be checked using the orifice equation which may be written:

$$Q = C_d A\ (2gh)^{1/2}$$

Where:
Q = Orifice discharge (cfs)
C_d = Coefficient of discharge (usually assumed to be 0.6)
A = Orifice cross-sectional area (ft.^{2})
g = Gravitational acceleration (32.2 ft./sec.^{2})
h = Hydraulic head above the orifice (ft.)

The maximum peak flow expected from the filter system must first be calculated once again using Darcy's Law.

$$Q = KiA = K \frac{Y}{D} A$$

Where:

K = Coefficient of soil permeability (ft./hr.)

$i = \frac{Y}{D}$ = Instantaneous hydraulic gradient

A = Area of trench or filter (ft.^{2})

Y = Head difference between water level elevation behind the structure at any point in time and the flow line of the underdrain pipe or top or gravel envelope if used

D = Depth of soil column or filter to flow line of underdrain or top of envelope material

Q = Instantaneous rate of discharge (ft.^{3}/hr.)

Continuing to work through this example problem:

K = 0.83 ft./hr.
Y = 5.0 ft. (when the detention area is full)
D = 2.0 ft.
A = 1113 ft.^{2}

Therefore:

$$Q_{max} = K \frac{Y}{D} A = 0.833(2.5)(1113)$$
$$= 2318 \text{ ft.}^3\text{/hr.}$$
$$Q_{max} = 0.64 \text{ cfs.}$$

Checking this rate of flow with the SCS pipe capacity charts mentioned previously for n = 0.015 and hydraulic grade (0.005 ft./ft.) indicates that an 8-inch pipe will remain adequate to handle the maximum peak flow.

The minimum orifice area required is then determined using the following equation:

$$A = \frac{Q}{C_d\sqrt{2gh}}$$

Where:

A = total orifice area required
Q = .64 cfs
C_d = .6
g = 32.2 ft./sec.^{2}
h = 5.0 ft.

$$A = \frac{.64}{.6\sqrt{64.4 \times 5}}$$

= .059 ft.^{2}/371 ft. of pipe

= 1.6 X 10^{-4} ft.^{2}/ft. of pipe

A = .023 in^2 ft. of underdrain pipe

Therefore, a pipe is required that contains at least eight or more 1/16 inch diameter holes per foot.

2) Calculating the length of a bank filter and determining pipe size required.

Either of the two basic procedures discussed above may also be used to determine the length of a bank filter system. However, the designer should notice that both the cross-sectional area of the filter media intersected by the drain and the hydraulic gradient which was presumed to be unity or larger in the previous analysis decrease with time in these systems. These factors must be taken into account and act to complicate the more simplistic procedures discussed previously. The length of bank filters is usually established by trial and error. The designer chooses the underdrain length desired and subsequently checks drawdown time against state regulations or land use requirements until a suitable configuration is reached.

a) Procedure for sizing bank filters based on Darcy's Equation

The most simple and easily understood method for sizing bank filters is primarily applicable to conditions wherein lateral flow is predominant (see Figure 6-52).

Once again, the rate of flow should be in accordance with Darcy's Law which states that the flow velocity of water through porous media media is proportional to the hydraulic conductivity and the hydraulic gradient. The relationship may be stated:

$$V = Ki$$

Where: V = velocity of flow

K = the hydraulic conductivity

i = the hydraulic gradient $\frac{\Delta Y}{\Delta D}$

Y = the change in elevation between the free water surface in the reservoir and a horizontal reference plane usually passing through the flow line of the underdrain pipe.

D = the horizontal distance from the edge of the free water surface to the verticle reference plane (usually chosen passing through the center of the underdrain pipe).

The flow of water delivered to the drain is equal to the velocity (Ki) as it moves through the media, multiplied by the cross sectional area (A) of the filter. Contrary to the more simplistic situations analyzed earlier, this area changes not only in relation to the depth of the free water surface but also decreases in relation to the upper line of seepage as it moves toward the underdrain.

Hence, Darcy's Law is usually applied in the design of bank filters in much the same way as it is for determining seepage through an embankment. The media is assumed to be homogenous throughout and located on an impervious foundation (e.g., the bottom and sides of filter trenches are presumed impermeable). Since the depth of the saturated zone varies as it approaches the drain, the mean width is used to calculate the area factor used in these determinations.

When the bottom of the filter and the horizontal reference plane coincide, (A) is assumed to equal one-half the vertical distance (H) shown in Figure 6-52 multiplied by the length of the filter (L). Stated in mathematical form:

$$A = \frac{H\,L}{2}$$

Where: A = Mean cross sectional area of the saturated zone (ft^2)
H = Change in elevation or depth of the filter from the free water surface to the bottom of the filter (ft)
L = Length of the filter (ft)

The instantaneous rate of discharge (Q) is subsequently calculated at the various stages of drawdown or storage elevations in the pond. The greater the number of increments the more accurate the assessment is likely to be.

The formula for expressing the discharge through a unit length of filter media for each increment is:

$$q = K\left(\frac{\Delta Yn}{\Delta Dn}\right)\left(\frac{Hn}{2}\right)$$

When the elevation of the free water surface does not exceed the top of the filter, the value of (Hn) is equivalent to the change in elevation per increment of rise or fall in the storage area (i.e., Hn = Δ Yn). The equation subsequently may be written:

$$q = \frac{K}{2}\left(\frac{Hn^2}{\Delta Dn}\right)$$

Where: q = discharge rate per unit length
K = hydraulic conductivity of the filter material comprising the least permeable section
Hn = change in elevation from the flow line of the drain or other reference point to the water level in the reservoir
Dn = horizontal distance measured from the edge of the free water surface to the vertical plane of reference (in this instance, the middle of the perforated pipe).

Values of (q) are multipled by the total length (L) of the filter system to determine the total discharge capacity (Q) of the facility. In equation form the relationship may be stated:

$$Q = (q)(L)$$

In some situations, the bottom of the filter is located below the flow-line of the pipe. In these instances, the value of (H) will exceed (Yn). The mean discharge area may be determined as follows:

$$A = \frac{(H - \Delta Y) + H}{2} L = (H - \frac{\Delta Y}{2})L$$

In either case, the instantaneous discharge is averaged between each increment. The drawdown time is then determined by dividing the volume of storage available in the reservoir between stages by the mean rate of outflow projected through the filter.

Similar to previous examples presented in this section, Table 6-15 shows how the drawdown time would be calculated for a project designed to treat approximately 36,300 ft^3 or 10 acre inches of runoff. By comparison, it may be seen that bank filtration is not nearly as hydraulically efficient as a means of treatment as bottom filters or underdrains. Earlier it was shown that only 200 ft. of bottom filter would be needed to drain an equivalent amount of water within 72 hours. However, a 75% increase in length (350 ft.) of bank filter is required to accomplish the same task even though the hydraulic conductivity of the media is presumed to be more than five times greater than the earlier example.

The designer should be aware that the configuration of the system itself can have substantial influence on the hydraulic efficiency of these facilities. Most of these systems are relatively low head since they are normally designed with little more than 2 feet of elevation difference between the maximum stage in the facility and the bottom of the filter. Consequently, there is often only a very slight energy gradient to move water toward the drain. In such situations elongated envelopes (see Figure 6-53) are often used to provide higher internal gradients and improve the flow of water through these structures. The higher discharge capacity decreases the length of filter needed to satisfy drawdown requirements.

TABLE 6-15

Incremental Method for Calculating Drawdown Time for Bank Filters Using Darcy's Equation

Increment	Storage Vol. (ft^3)	Storage (ft^3)	Horizontal (1) Dist. "ΔD" (ft)	Change in Elev per (1) Increment "ΔY" (ft)	Hydraulic (2) Gradient "i" (ft/ft)	Hydraulic Conductivity "K" (ft/hr)	Avg Discharge (3) Area "A" (ft^2)	Instantaneous Discharge Q = KiA (ft^3/hr)	Avg. Discharge (4) Per Increment "Q" (ft^3/hr)	Drawdown Time (5) "t"/Increment (Hrs)
1	36,312	9,690	2.0	3.0	1.0	5.0	525	2625	2089.07	4.64
2	26,622	9,276	3.5	2.5	.71	5.0	437.5	1553.13	1126.57	8.23
3	17,346	8,871	5.0	2.0	.40	5.0	350	700.00	500.94	17.71
4	8,475	8,475	6.5	1.5	.23	5.0	262.5	301.88	207.82	40.78
5	0		8.0	1.0	.13	5.0	175	113.75		
		36,312 TOTAL								71.36 TOTAL

Notes: Pond with Top Length = 150'
Top Width = 132'
Depth 1/2" Storage = 2.0'
Side Slopes 3:1
Rectangular Shaped
Assume 350' of filter is used (l = 350 ft)

(1) Values of D and Y are illustrated with Figure 6-52.

(2) Hydraulic Gradient $i = \frac{\Delta Y}{\Delta D}$, i (Max) = 1.0

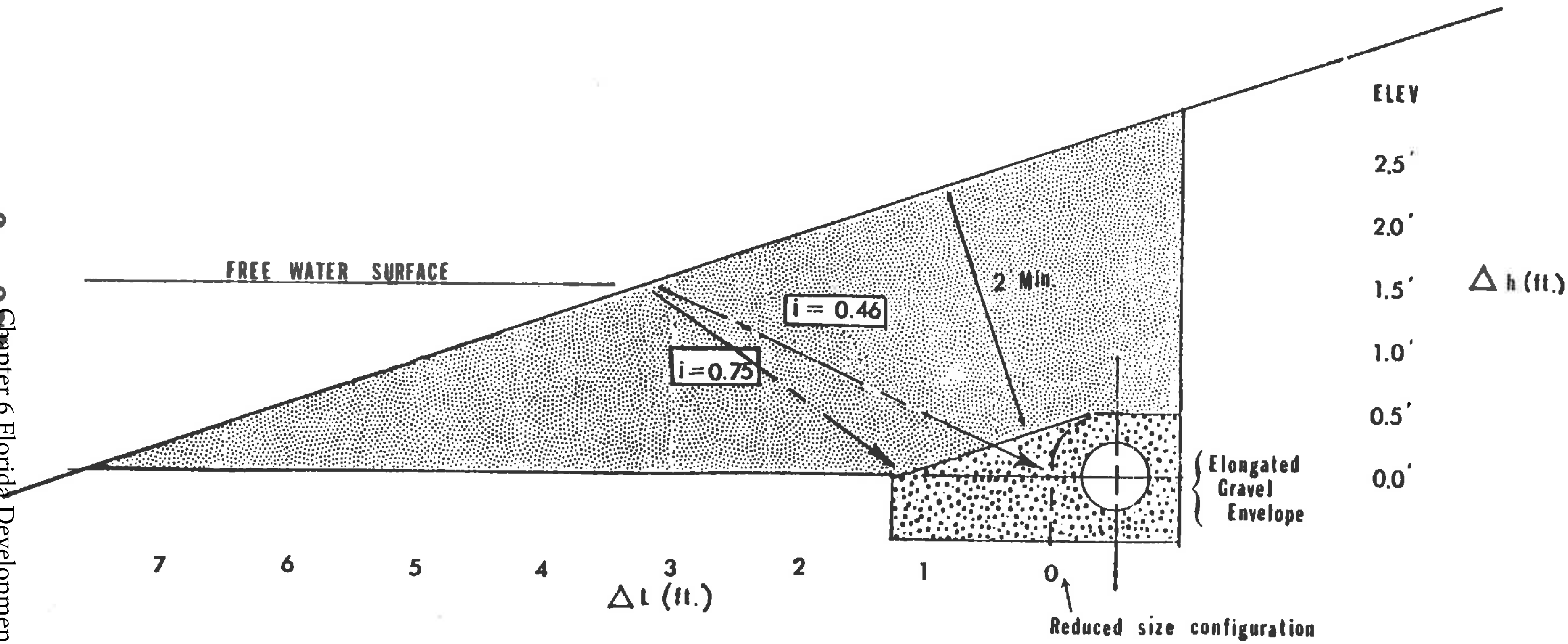
(3) Avg. "A" = $\frac{\Delta Y \times l}{2}$

(4) Avg. Discharge $Q' = \frac{Q(n) + Q(n+1)}{2}$

(5) Drawdown time = $\frac{\text{Storage}}{Q'}$

FIGURE 6-53

Elongated Gravel Envelope to Provide Improved Internal Gradient for Low Head Bank Filtration Systems



b) Flow Nets

Another commonly used method for evaluating problems related to seepage through porous media involves the construction of flow nets. This type of analysis can also prove to be a valuable tool for the design of stormwater filtration systems. A diagram of a flow net constructed for a rather commonly used bank filter design is illustrated in Figure 6-54. Those interested in developing skills in flow net analyses for this and other configurations of bank filters may find Seepage, Drainage, and Flow Nets by H.R. Cedergren to be a vaulable reference.

Flow nets may be applied in sizing a bank filter in the same manner as Darcy's Law was used earlier. A number of diagrams are constructed, each correlating to various stages within the reservoir. The individual diagrams are then used to determine the discharge relationship per linear foot of filter. The flow net equation may be written:

$$q = KH \frac{nf}{nd}$$

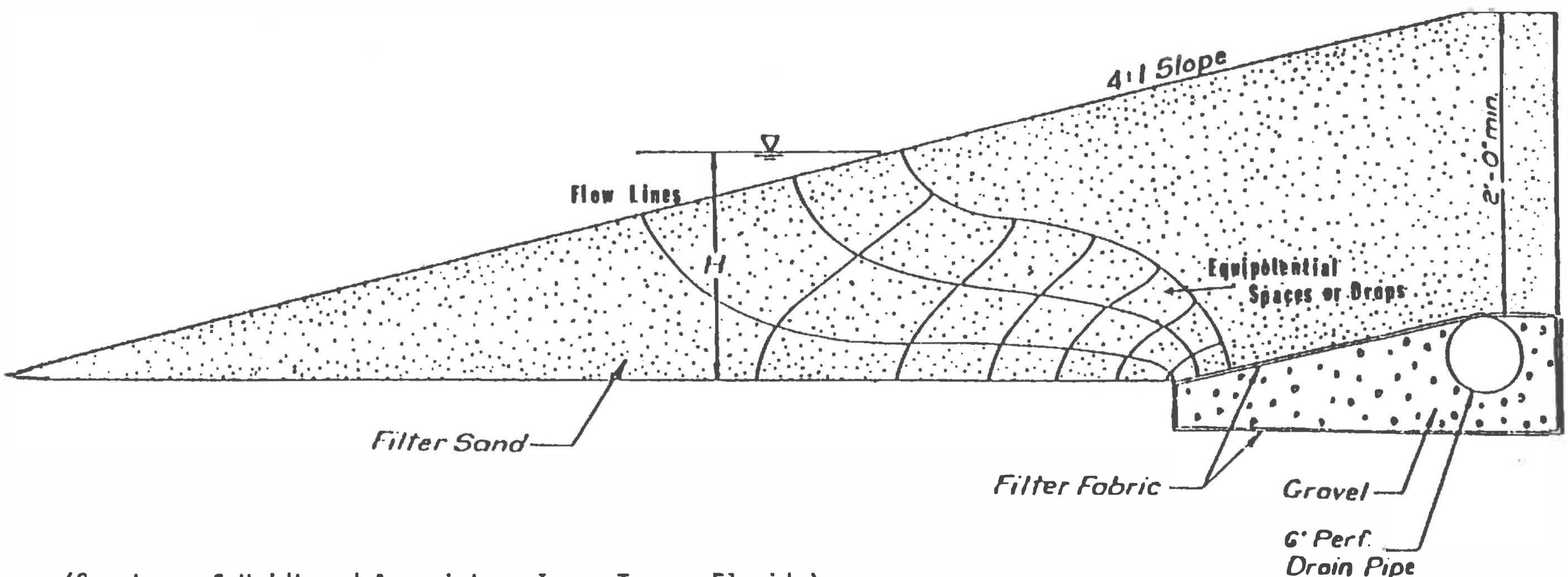
Where: q = seepage quantity (L^3/T/ft. of filter)
K = permeability of the media (L/T)
H = net head (L) as illustrated in Figure 6-54
nf = number of flow channels
nd = number of equipotential drops

The ratio nf/nd is otherwise known as the shape factor. As noted by Cedergren, the number of flow channels and the corresponding number of equipotential spaces depends on the shape of the cross section, and will not necessarily be a whole number. A different shape will produce a different number of spaces and channels. He goes on to warn those who are just learning by stating that novices "sometimes overlook one or more of the basic rules. The resulting flow net can be so filled with errors that a grossly distorted picture of a seepage pattern will be given". However, he also points out that any number of flow nets for a given problem will agree when the work has been done correctly. There is but one solution to a given problem. Consequently, although this work may be quite cumbersome at first, once flow nets have been constructed for a given configuration of filter they will continue to apply to the specific design or shape as long as they are not changed.

Once the unit rate of discharge (q) is established at each increment of drawdown the total stage discharge relationship may be determined by multiplying the value of (q) at each stage by the length of the filter (L). The instantaneous discharge Q is averaged between each increment. Assuming the volume of storage between increments is known, the drawdown time (t) may be calculated by dividing the volume by the average rate of outflow in the same manner as illustrated in Table 6-15. Here again, the preparation of such a table is an aid to any reviewing agency and can help speed up project approval. Preliminary evaluations seem to show that flow net analyses may

FIGURE 6-54

Flow Net Diagram Illustrating Lines of Seepage Through
a Typical Bank Filtration System



(Courtesy of Heidt and Associates, Inc., Tampa, Florida)

(Not to scale, for illustration only)

offer benefits over the procedure described in section 2(a) when gradients of 100% or more are likely to occur over much of each filtration cycle.

c. Other Analytic Approaches

Currently several other methodologies being used to design underdrains that are not incremental in nature. In these instances several designers have modified the designs illustrated previously. This has enabled the use of equations in which drawdown time (t) or filter length (L) may be calculated in one step based on a single formula. Figure 6-55 illustrates the general design and important dimensions of two such systems.

The form of the expression used to estimate the length of filter trench (L) required to accomplish drawdown in the time desired is written as follows:

$$L = \frac{1.33 \; Ar \; D}{KtW} \ln (Y_1/Y_2) \text{ for system (a), and;}$$

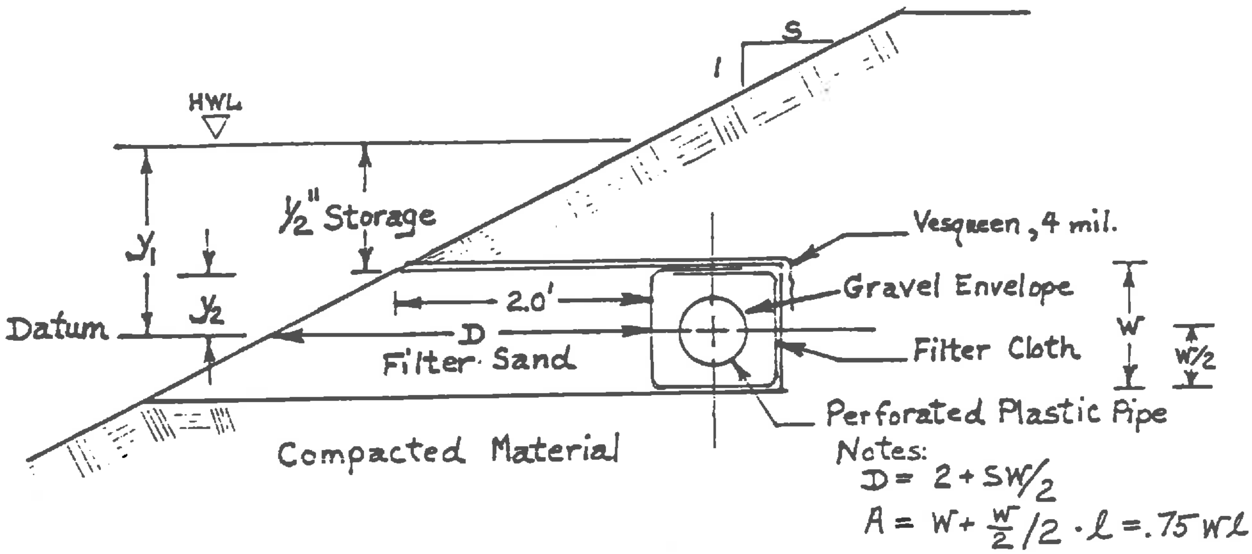
$$L = \frac{Ar \; D}{KtW} \ln (Y_1/Y_2) \text{ for system (b) Figure 6-55.}$$

A_r = Average area of reservoir between elevation Y_1 and Y_2 (ft^2)

Where:
- L = length of filter required (ft)
- K = hydraulic conductivity of the filter media (ft/hr)
- t = allotted drawdown time (hrs)
- W = trench width as illustrated in Figure 6-55 (ft)
- D = average distance which water must travel through the filter profile as shown in Figure 6-55 (ft)
- Y_1= Difference in elevation between the flow-line of the underdrain pipe and the water level in the reservoir at the appropriate volume of storage (ft)
- Y_2= Difference in elevation between the flow-line of the underdrain and the stage in the reservoir following discharge of the treatment volume required (ft)

It should be noted that the mean distance (D) traveled is used for calculations involving systems similar to that illustrated in Figure 6-55(a) while for the system shown in Figure 6-55(b) the distance is equal to 2.0 feet. However, the difference in the form of each equation is primarily due to differences in the magnitude of the cross section visualized to be intersected by the drain in each situation. In Figure 6-55(b) the trench is perpendicular to the face of the bank. Flow through the filter, toward the drain is predominantly vertical. The entire cross-section or width (W) of the filter is intersected by the drain and its surrounding gravel envelope. Flow is primarily normal to the plane of reference. In these circumstances, the discharge area remains relatively constant as water moves toward the drain. The phreatic surface will be parallel to the upper trench wall, intersecting nearly the entire width of the drain before curving toward the drain tube itself. The presumption that A = WL is primarily correct in this circumstance.

(a) Reduced Head System



(b) Side-of-the-Bank System

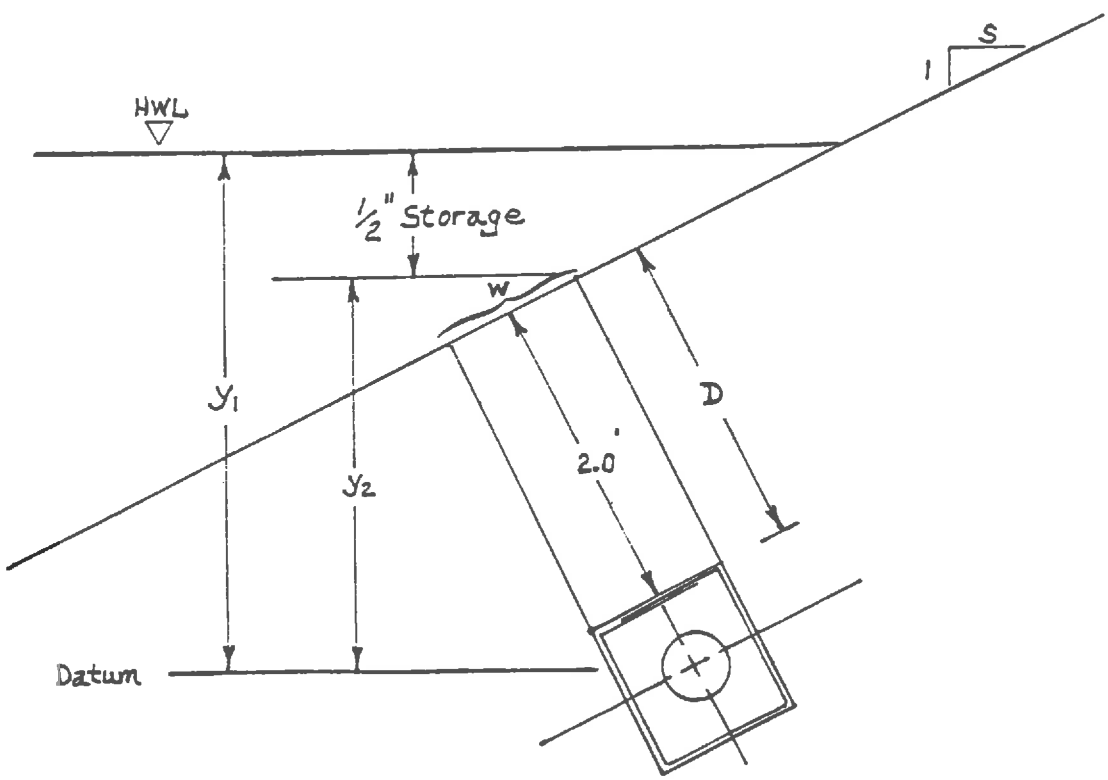


FIGURE 6-55

Sketch of Bank Filter Designs Illustrating Symbols for Use With Single Step Evaluation of Trench Length and Drawdown Time

(Courtesy of Post, Buckley, Schuh and Jernigan Engineers, Clearwater, Florida)

On the other hand, this assumption does not pertain in the horizontal flow situation shown in Figure 6-55(a). The reference plane (passing through the flow line of the pipe when full flow is assumed) does not intersect the entire cross-section or width of the filter. Flow is primarily parallel to the plane of reference. In these circumstances the width of the saturated zones is reduced as water moves toward the drain. The average discharge area (see notation with figure) should be used in this situation. The presumption that the discharge area (A) is equal to the entire width (W) of the drain times its length (L) would be correct only in the event that the water level was expected to be at the top of the trench during operation. This assumption is pertinent to submerged drains only. The latter situation would not be recommended because of added difficulties in the design, installation, operation and maintenance of these systems due to constant anaerobic conditions.

When the engength of the system is known, the equations may be rearranged to solve for the drawdown time associated with the system as fo lows:

$$t = \frac{1.33\ Ar\ D}{K\ W\ L} \ln Y_1/Y_2 \text{ for system (a)}$$

$$t = \frac{Ar\ D}{K\ W\ L} \ln Y_1/Y_2 \text{ for system (b)}$$

Similar to using other forms of Darcy's Law when the length and drawdown constraints are predetermined, the equation may be used to establish the required permeability. In this instance the designer would establish a trial thickness of the filter and calculate the hydraulic conductivity needed to satisfy these requirements. Likewise, the designer may select one or more permeabilities that represent commercially available filter materials within acceptable degrees of uniformity and effective size (as outlined in 17-25.025(2) F.A.C.) and calculate their required thickness.