STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK

EXFILTRATION SYSTEMS

OFFICE OF DESIGN, DRAINAGE SECTION FEBRUARY 2012
TALLAHASSEE, FLORIDA

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The Florida Department of Transportation Drainage Office coordinated the development of this Exfiltration Systems Handbook. The consultant Project Manager and co-author was Robert T. Carballo, P.E., Vice President and Principal in charge of the Transportation Division of C3TS, PA.

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Assistance for the preparation of this Handbook was provided by the FDOT State Drainage Office and District Drainage Engineers and staff, who improved the contents of this document with comments and suggestions during the preparation of this Exfiltration Systems Handbook.

TABLE OF CONTENTS

APPENDICES

Chapter 1

Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume 1 – Policy; Volumes 2A and 2B – Procedures; Volume 3 – Theory. On October 1, 1992, Volume 1 – Policy was revised to Volume 1 – Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume 1 – Standards was revised in January 1997 and was renamed to simply the "Drainage Manual". No revisions have been made, nor will be made to Volume 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several the Central Office Drainage section is developing to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a "standards" document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of FDOT projects, and to provide guidelines for the analysis of exfiltration systems (trenches and wells). Pertinent sections of the 1987 Drainage Manual have been incorporated into this handbook.

The guidance and values in this handbook are suggested or preferred approaches and values, not requirements or standards. The values provided in the Drainage Manual are the minimum standards. In cases of discrepancy, the Drainage Manual standards shall apply. As the Drainage Manual states about the standards contained in it, situations exist where the guidance provided in this handbook will not apply. The inappropriate use of and adherence to the guidelines contained herein does not exempt the engineer from the professional responsibility of developing an appropriate design.

1.3 Distribution

This handbook is available for downloading from the Department's Internet site at **www.dot.state.fl.us/MapsAndPublications** and from the Drainage Section website at **http://www.dot.state.fl.us/rddesign/Drainage/Default.shtm**

1.4 Revisions

Any comments or suggestions concerning the handbook can be made by mailing them to:

Florida Department of Transportation
Office of Design – Drainage Section
Mail Station 32
605 Suwannee Street
Tallahassee, FL 32399

1.5 Definitions of Terms and Acronyms

AASHTO	American Association of State Highway and transportation Officials.
ASTM	American Society of Testing Materials.
CFS	Cubic Feet per Second
Curve number	A dimensionless site-specific runoff parameter developed by the (former) Soil Conservation Service (now National Resource Conservation Service) to empirically estimate rainfall excess; it accounts for infiltration losses and initial abstractions.
Design Tailwater (DTW)	The elevation of the hydraulic gradient (or water surface) at the outlet of a storm drain system during the design storm event.
Drainage Basin	A subdivision of a watershed.
Duration	The time of a rainfall hyetograph used to perform runoff calculations.
Environmental Resource Permit (ERP)	Conceptual approval, individual or general permits for a surface water management system issued pursuant to part IV Chapter 373, F. S.
Exfiltration	The loss of water from a drainage system as a result of percolation or absorption into the surrounding soil.

Exfiltration Trench	A subsurface system consisting of a conduit, such as a perforated pipe, surrounded by natural or artificial aggregate which temporarily stores and exfiltrates stormwater runoff.
FM	Florida Method of Testing Materials. This is the standard FDOT method of testing materials.
Hydraulic Grade Line (HGL)	In pressure flow, it is a theoretical line connecting hydraulic gradient points along the flow path.
Hydraulic Gradient (HG)	In pressure flow it is the elevation to which the water would rise in a tube or inlet connecting the flow pipe to atmospheric pressure.
Infiltration	Abstraction process in which water flows or is absorbed into the ground.
Intensity	The rate of precipitation, usually in inches/hour.
Minor Losses	All losses that are not due to friction. Generally these are energy losses due to changes or disturbances in the flow path. Minor losses include such things as entrance, exit, bend, and junction.
Overland flow	That water which travels over the ground surface to the stream channel, usually limited to a maximum length of 100 feet.
Runoff	Precipitation remaining after appropriate hydrologic abstractions have been accounted for.
Runoff coefficient	Empirical parameter used to calculate rainfall excess as a fixed percentage of precipitation; it accounts for interception, surface storage, and infiltration.
Seasonal High Water Table (SHWT)	Elevation to which the ground and surface water can be expected to rise due to a normal wet season.
State Water Quality Standards	Water quality standards adopted pursuant to Chapter 403, F.S.

Stormwater Injection Wells	Wells used for stormwater runoff disposal into pervious underground soils or the water table.
Tailwater	The hydraulic gradient (water surface elevation) downstream of a pipe section.
Time of concentration (t_c)	The time required for runoff to travel from the hydraulically most distant point of a watershed to the design point.
Underground Source Drinking Water	A USDW is defined as an aquifer that contains a total of dissolved solids concentration of less than 10,000 milligrams per liter or parts per million (ppm).
Well Casing	Well casings serve as a lining to limit discharge to the desired aquifer. It also provides structural support against caving materials outside the well. Materials commonly used are wrought iron and steel.

Chapter 2

General

2.1 Hydrology

The Department's general guidance regarding hydrology is included in the Drainage Manual and the Drainage Handbook, Hydrology. The design criteria and calculation methods for each project should be previously coordinated and approved by the District Drainage Engineer.

2.1.1 Time of Concentration

The Hydrology Handbook defines and provides methods to calculate the time of concentration. A longer time of concentration usually reduces the calculated peak discharge. The Rational Method is very sensitive to changes in the time of concentration (i.e. if the time of concentration increases from 10 minutes to 60 minutes; the calculated peak discharge could exhibit up to 60 percent reduction in discharge). The Flow Hydrograph Methods are less sensitive to the time of concentration charges (i.e. up to 15 percent peak discharge reduction if the time of concentration increases from 10 minutes to 60 minutes).

2.2 Hydro-Geology

2.2.1 Darcy's Law

Darcy's Law characterizes the flow through porous media, assuming that the viscosity, temperature and density of the fluids are constants. The flow rate is a function of the flow area, the hydraulic gradient and proportionality constant (Refer to Figure 2.2.1):

$$Q = k\ i\ A$$

Where: Q = Flow Rate (cfs)
k = Permeability Constant (ft / sec)
i = Hydraulic Gradient ($i = \Delta H / L$)
A = Cross-sectional area of soil conveying flow (ft^2).
ΔH = Change in the hydraulic grade line (ft)
L = Distance between points of interest (ft)

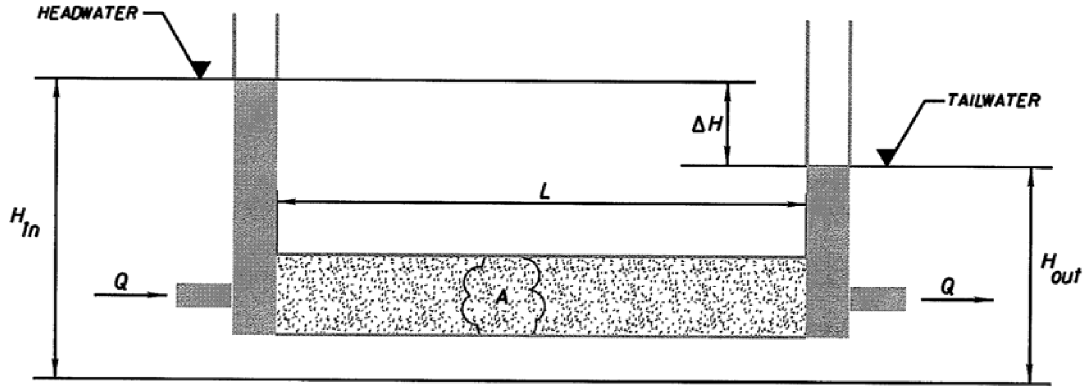

FIGURE 2.2.1 SATURATED FLOW THROUGH POROUS MEDIA.

The Darcy's Law was established for saturated flow. As such, it may be adjusted for unsaturated and multiphase flows.

2.2.2 Soil Permeability

The coefficient of permeability (k) in the Darcy's Law is a measure of the rate of water flow through a saturated soil under a given hydraulic gradient in length unit over time unit (i.e. ft/day). The soil permeability is dependent on the grain-size distribution and void ratio. The coefficient of permeability (k) typically varies from 0.03 ft/sec (43 ft/day) for gravels to less than 10^{-8} ft/sec (1.44×10^{-5} ft/day) for clays (Refer to Appendix B).

2.2.3 Hydraulic Conductivity of Soils

The hydraulic conductivity of a soil (K) measures the relative ease of water transmission through the soil:

$$K = \frac{Q}{A \, \Delta H}$$

Where: Q = Flow Rate (cfs)
A = Flow Area (ft^2)
ΔH = Hydraulic Head (ft)

The hydraulic conductivity of a soil is the ratio between the discharge through the unit area of soil perpendicular to the flow per unit of head (i.e. cfs/ft^2 – ft of head). This is the primary factor used to determine the exfiltration rate of a system.

The flow transmission through a soil increases as the water content increases. As such, the maximum hydraulic conductivity occurs under saturated conditions. The hydraulic conductivity for horizontal-saturated flow (K_s) is usually several times greater than the hydraulic conductivity for vertical-unsaturated flow (K_u).

2.2.4 Hydro-Geologic Tests

The hydro-geotechnical properties of a soil can be measured in tests under falling or constant head, either in the laboratory or the field. The most effective soil testing is a combination of laboratory and field methods. Laboratory tests on undisturbed soil samples usually provide accurate results representative of only a point among the soil stratum.

The hydrologic and geologic characteristics of the site where the exfiltration system will be installed have to be evaluated in order to define the test procedures to be used. The following tests are suggested:

a) Laboratory Permeameter Test for saturated hydraulic conductivity on undisturbed soil samples. (ASTM D 5084)

b) Double Ring Infiltrometer Test to estimate the initial vertical unsaturated permeability data of the upper soil layer. (ASTM D 3385)

c) Constant Head Test in soils with permeabilities that allow keeping the test hole filled with water during the field test. (AASHTO T 215)

d) Falling Head Test in areas with excellent soil percolation where keeping the test hole filled with water is not feasible during the test. (FM 5-513)

e) D.O.T. Standard Test (constant head) that can be used for the Department's projects. (FM 1-T 215)

f) Well Test Holes are performed to determine relative permeability and water quality characteristics of the aquifer (ASTM D4050). Through continuous water quality testing the test hole will indicate the depth at which a minimum of 10,000 milligrams per liter Total Dissolved Solids (TDS) concentration is found. The test will also indicate the most favorable depth for stormwater discharge.

g) A Pumping Test is performed at the most favorable depth for stormwater discharge to determine the design discharge capacity normally in gallons per minute per foot of head. The test is normally performed in conjunction with the well test hole.

2.2.5 Seasonal High Water Table

Published information (such as the Soil Conservation Service publications) provide preliminary guidance related to the water table at a specific location, but site specific water table information is required to design exfiltration systems.

The initial data to determine the seasonal high water table (SHWT) elevation is the measurement of the stabilized ground water level in a boring or well. The initial (encountered) water table elevation should be adjusted in order to estimate the SHWT based on antecedent rainfall, examination of the soil profile (color variations, depth to hardpan, etc.), consistency with water levels of the adjacent water bodies, vegetative indicators, etc.

2.2.6 Average Antecedent Moisture Conditions

The Antecedent Moisture Condition (AMC) indicates the wetness of a soil and its availability to infiltrate water. The soil moisture ranges from dry to saturate depending on the rainfall amount prior to the moisture measurement. The average AMC means that the soil is neither dry nor saturated, but at an average moisture condition at the beginning of the design storm event.

2.3 Data Collection

The design of an exfiltration system requires a good understanding of the site conditions. The following is a summary of information required and potential sources:

a) Topographic Data: Preliminary topographic data is usually found in the United States Geological Survey (USGS) quadrangle maps and previous project construction plans. This information should be supplemented with a detailed topographic survey of the project area.

b) Geotechnical Data: The Soil Survey Reports by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS) provide general geological and geotechnical properties information. Previous geotechnical reports for the project area and adjacent developments can provide more specific information regarding the geotechnical conditions at the project location. After preliminary evaluation of the available data a detailed geotechnical study should be requested. The site specific geotechnical report should provide the types of soils within the project location; soil engineering characteristics including hydraulic conductivity (Refer to Section 2.2.3), ground water elevations, etc.

c) Receiving Water Bodies: The Water Management Districts and some local agencies can provide information regarding the water elevations under different storm frequencies for lakes, rivers, canals, and reservoirs. These agencies can also provide potentiometric surface maps to assist in determining the ground water elevations. Tidal information is available in the National Oceanic and Atmospheric Administration (NOAA) website. The design tailwater elevation can be determined from the above sources.

d) Permit Data: Previous permit information for the site and surrounding developments can provide information related to design criteria, existing wetlands, possible outfalls, discharge limitations, control elevations, off-site contributors, geotechnical data, prior soil testing results, etc.

e) Right of Way Data: The right of way information can be obtained from the Department's right of way maps and county and city right of way documents.

f) Field Reviews: Visiting and inspecting the site provides first-hand updated information to the designer regarding the existing drainage system, off-site contributors, water management facilities, outfalls, and other site conditions.

2.4 Permitting Considerations

The drainage design in general is to be developed in compliance with all applicable Federal, State, and Local environmental regulatory programs. The respective permitting authorities have to be identified and contacted early in the design process. These agencies include local water control districts, county and state water management districts. Each permit agency has specific water quality requirements and may impose restrictions on the construction of exfiltration trenches and well systems.

2.5 Construction and Maintenance Considerations

Stormwater exfiltration systems should be installed no less than 2 feet from parallel underground utilities and 20 feet from existing large trees to remain in place. Careful evaluation of the existing soils and the excavation method is necessary if the exfiltration system is located in close proximity to the right of way line to avoid damages to the adjacent properties. Erosion control measures should be implemented in order to impede the access of sediments and debris into the exfiltration system during construction, which can clog the filter fabric diminishing the capacity of the exfiltration trench.

Exfiltration systems should not typically be used within any type of manmade, compacted embankment since there is little to no percolation in compacted fill as compared to natural soils.

Do not install exfiltration systems in close proximity to and behind MSE walls. Install solid conveyance pipes behind the MSE wall and install exfiltration systems away from the walls. Do not use exfiltration trenches in locations where a 1H:1V mound could allow the filtrate to impact the MSE soil reinforcements due to the potential for accelerated corrosion of metallic reinforcements to occur without warning. Furthermore, seepage forces would need to be included in the design of the wall, and daylighting filtrate could result in soil washouts, unsightly mildew, vegetation, staining and other maintenance problems.

Physical access devices must be provided with the stormwater exfiltration systems in order to facilitate maintenance activities. Minimum pipe sizes and maximum spacing between drainage structures (Refer to Sections 3.2.8 to 3.2.13) should be considered for the efficient operation of maintenance equipment. Consideration should be given to future expansion of the facilities and to possible increase of maintenance requirements.

In the case of drainage wells the injection well chamber must be provided with physical access devices for maintenance activities. Maintenance of the injection well includes cleaning of the well, removal of debris and in some cases the redevelopment of the well to reestablish discharge capacity. The well location needs to be accessible from the surface to allow these activities to take place.

Chapter 3

Exfiltration Trenches

3.1 Description

An exfiltration trench is an underground drainage system consisting of a perforated pipe surrounded by natural or artificial aggregate, which stores and infiltrates runoff (Refer to Figure 3.1). The stormwater runoff is collected by catch basins located at the end of each exfiltration trench segment; the perforated pipe delivers the stormwater into the surrounding aggregate through the pipe perforations. The stormwater ultimately exfiltrates into the ground water aquifer through the trench walls and bottom. As the treatment volume is not discharged into surface waters, exfiltration trench systems are considered a type of retention treatment.

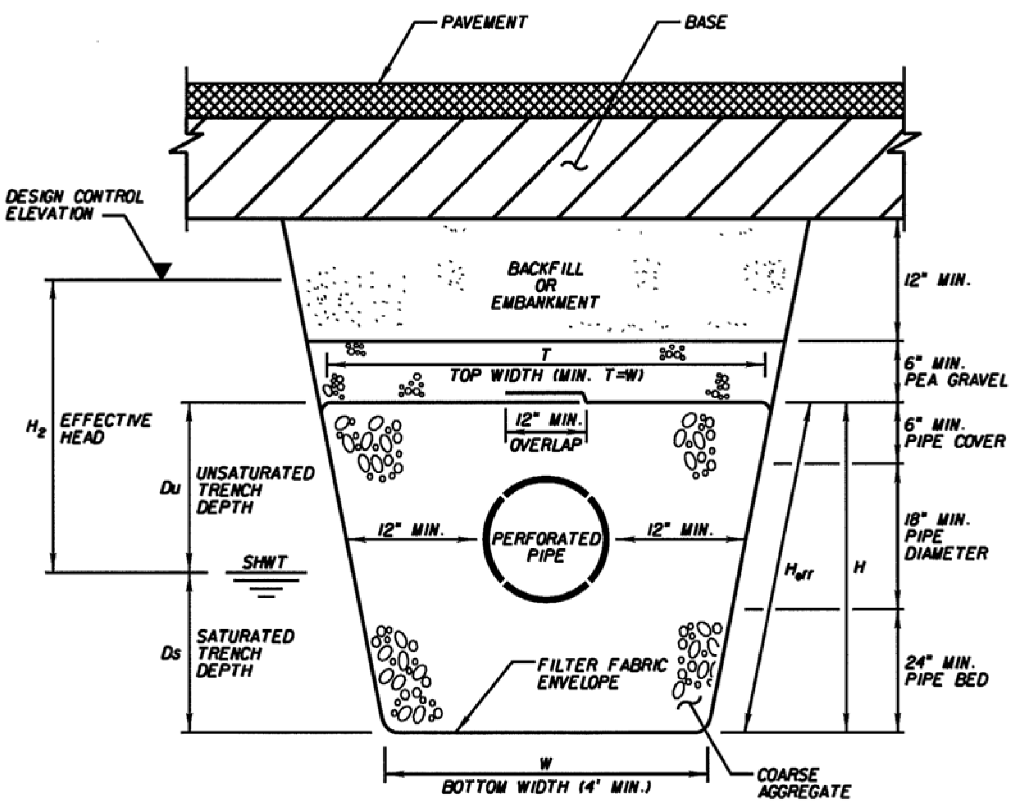

FIGURE 3.1 TYPICAL EXFILTRATION TRENCH

The permeability of the soils at the exfiltration trench location and the anticipated water table elevation determine the applicability and performance of the exfiltration trench system, which has to be able to exfiltrate the required stormwater treatment volume and drawdown the treatment volume to return to its normal condition within a specific time after the design storm event. When the trench Bottom is located at or above the average wet season water table, the exfiltration trench is considered a dry system.

3.1.1 Use

The exfiltration trench systems can provide the required stormwater treatment, if the hydro-geological conditions are suitable for runoff infiltration (i.e. permeable soils with hydraulic conductivity exceeding 1×10^{-5} cfs / ft^2 per ft of head), for projects where the areas available for water management facilities are limited and high right of way acquisition costs are anticipated. Exfiltration trenches, like other types of retention systems, are able to efficiently remove the stormwater pollutants. Additionally, exfiltration trenches contribute to recharge of the ground water aquifer thus assist in combating saltwater intrusion in coastal areas.

Due to the direct infiltration of the surface runoff with its associated pollutant load into the ground water aquifer, the exfiltration trench systems must not be installed in the proximity of potable water supply wells. Usually the installation of exfiltration trenches is not allowed within the ten-day well field protection contour but specific requirements for each well field should be verified with the permitting authorities. Exfiltration trenches should not be proposed within or near contaminated ground water areas to avoid the potential migration of the polluted plume due to the direct injection of surface runoff into or adjacent to the contaminated groundwater plume. In areas with high ground water elevation the available hydraulic head for exfiltration trenches operation is minimal but the required hydraulic head can be obtained by pumping if feasible.

The limited life span of the exfiltration trenches is their main disadvantage. The accumulation of sediments and clogging of the filter fabric and the void spaces of the aggregates usually shorten the operational life of the exfiltration trenches. Replacement of the exfiltration trenches in order to restore the treatment capacity of a system is typically considered to evaluate the cost effectiveness of any proposed drainage solutions. The remaining treatment capacity of existing exfiltration trenches should be tested, if feasible, prior to their replacement or if the existing exfiltration trenches are to be included in the design of a new drainage system.

3.2 Design Criteria

An exfiltration trench transmits the inflow runoff hydrograph into the groundwater during small storm events or in land-locked conditions; but in drainage areas with positive outfall the fraction of the runoff hydrograph that is not transmitted into the groundwater and retained within the exfiltration trench is transmitted downstream, usually through an outfall control structure.

The Standard Specifications for Road and Bridge Construction (Section 443 French Drains) includes directions, provisions, and requirements for exfiltration trenches. Standard exfiltration trenches are detailed in the standard index drawings (Index 285, French Drains). In the cases where the standard index drawings are not suitable for a specific project need, a detailed design should be developed and this information must be included in the design documentation. The following is the Department's general design criteria. It is recommended that additional specific criteria from the permitting agencies be evaluated in the pre-application conference early in the design process.

3.2.1 Water Quality

The exfiltration trenches to provide water quality treatment to a watershed can be installed off-line or on-line in the drainage system.

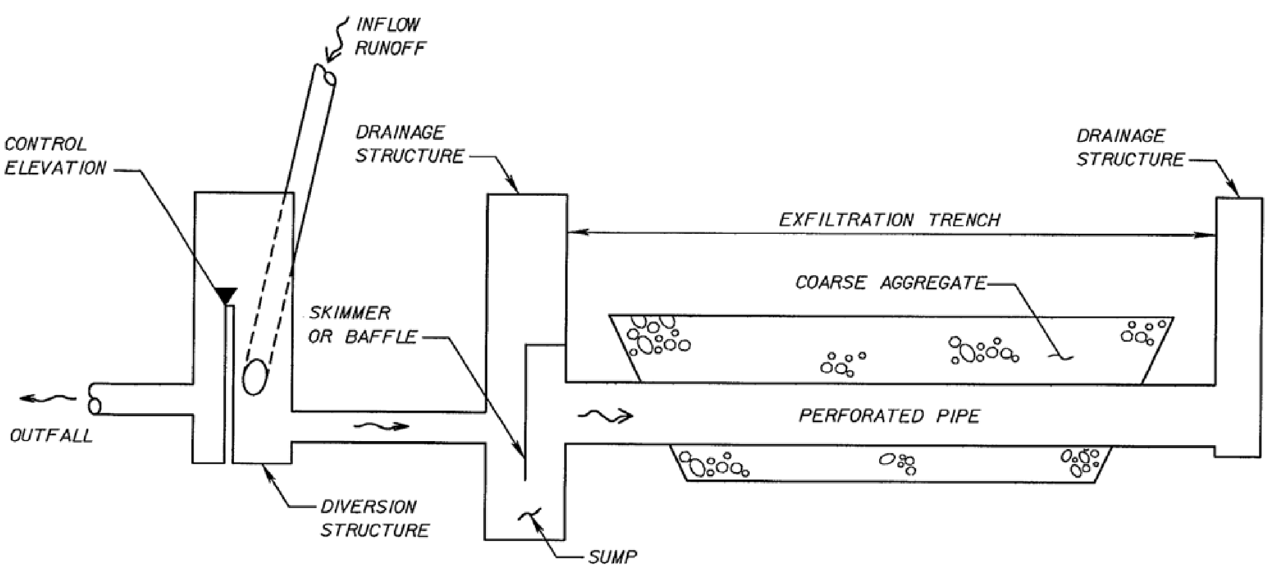

FIGURE 3.2.1 (A) OFF-LINE EXFILTRATION SYSTEM.

The off-line treatment method (Figure 3.2.1 A) diverts runoff into the exfiltration trench designed to provide the required treatment volume; subsequent runoff in excess of the treatment capacity bypass the off-line exfiltration trench towards the outfall. A diversion drainage structure is usually required for off-line systems.

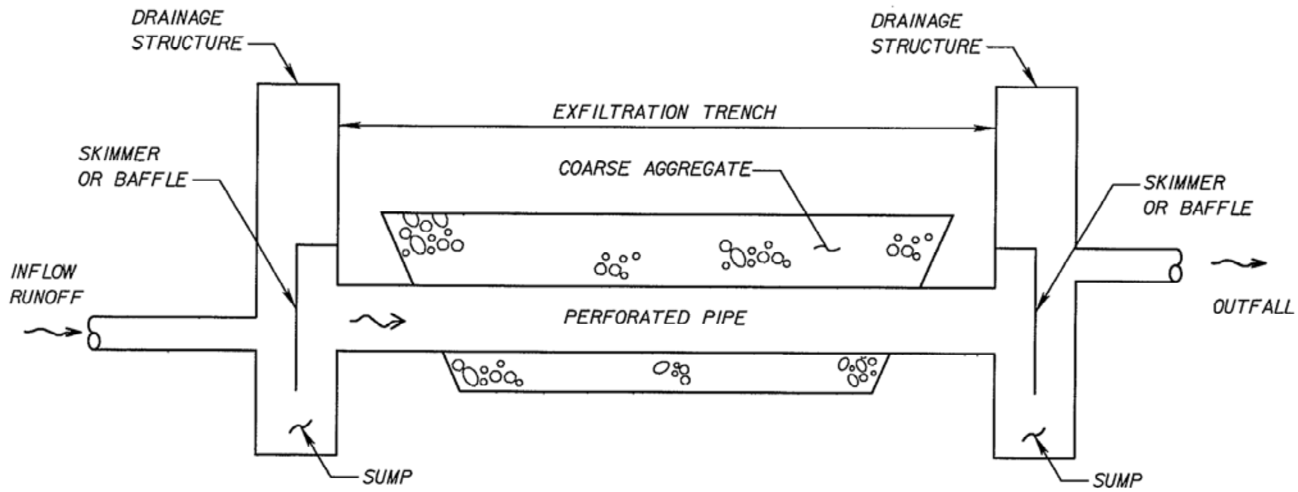

FIGURE 3.2.1 (B) ON-LINE EXFILTRATION SYSTEM.

The on-line exfiltration trench (Figure 3.2.1 B) provides the required water treatment but the treatment volume is mixed with the total runoff volume. As such, runoff volume in excess of the treatment capacity carries a portion of the pollutant load to the receiving water body.

3.2.2 Water Quantity

For exfiltration trenches designed to satisfy water quality requirements rather than provide flood protection, only the fraction of the overall exfiltration trench storage volume, including pipe and aggregate voids, located above the design ground water elevation and below the outfall control elevation should be considered for discharge attenuation.

In some special locations (i.e. Miami-Dade County and Monroe County) with very limited area available for water treatment the exfiltration trench systems could be credited for discharge attenuation if the ground water is considered variable, rising from the Seasonal High Water Table along with the design storm event.

3.2.3 Design Ground Water Elevation

The elevation to which the ground water can be expected to rise during a normal wet season should be used to calculate the required exfiltration trench length.

3.2.4 Control Elevation

The minimum control elevation for an exfiltration trench system should be at the same elevation as the top of the perforated pipe. The maximum control elevation should not violate the base clearance criteria for the project or produce changes in the land use value of the properties located upstream and downstream of the drainage system. A specific site survey, the permit files and the field reviews are the main source to determine the design control elevation.

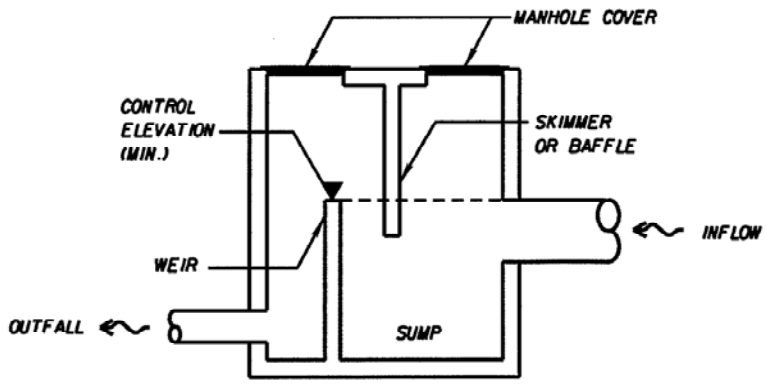

FIGURE 3.2.4 OUTFALL CONTROL STRUCTURE

3.2.5 Effective Head

a. Positive Discharge: The effective head of the exfiltration trenches (H_{eff}) with discharge to an outfall should be the average vertical distance from the SHWT to the outfall control elevation:

$$H_{eff} = \frac{\text{Control Elevation} - \text{SHWT}}{2}$$

b. Closed Systems: The effective head for exfiltration trenches with no outfall (self-contained system) should be the vertical distance from the SHWT to the average distance between the SHWT and the design high water (DHW):

$$H_{eff} = \frac{\text{DHW} + \text{SHWT}}{2} - \text{SHWT} = \frac{\text{DHW} - \text{SHWT}}{2}$$

3.2.6 Recovery Time

If the permitting authorities with jurisdiction over the project location do not have specific recovery time requirements, the water treatment capacity of the exfiltration trench systems should be recovered within the 72 hours following the design storm event, assuming average AMC.

3.2.7 Safety Factor

A safety factor of two or more shall be used to calculate the required length of exfiltration trenches in order to consider possible geotechnical uncertainties.

3.2.8 Dimensions

The minimum pipe diameter is 18 inches but 24 inches is preferable; and the minimum trench width is 4 feet. The maximum dimensions should depend on site specific conditions and construction methods. In general, exfiltration trenches with bottoms wider than 8 feet and or deeper than 20 feet are not recommended and perforated pipes over 36-inch diameter should be approved by the District Drainage Engineer.

Pipe perforations can be slotted or perforated. Standard locations and dimensions of the pipe perforations are included in the Standard Specifications Section 443, French Drains

3.2.9 Maximum Length

a. The maximum length of exfiltration trenches with access through both ends should be:

For 18” to 30” pipes	300 feet
For 36” and larger pipes	400 feet

b. The maximum length of the exfiltration trenches with access through only one end should be half of the maximum length of exfiltration trenches with access through both ends.

3.2.10 Pipe Invert

The invert elevation of the perforated pipe should be at least one foot above the trench bottom elevation.

It is desirable to locate the pipe invert above the SHWT to facilitate maintenance operations. This criterion may not be feasible in sites where the water table is close to the ground surface or where a deep permeable stratum underlies low permeability soils.

3.2.11 Aggregates

Uniform-graded, natural or artificial coarse aggregate with no more than 3 percent weight of material passing the Number 200 Sieve at the point of use should be used.

3.2.12 Filter Fabric

The coarse aggregate in the exfiltration trench must be enclosed in filter fabric. The perforated pipe could also be enclosed in filter fabric to increase the life span of the exfiltration trench if approved by the District Drainage Engineer.

The filter fabric shall comply with the requirements established in the latest FDOT Design Specification 985. Additionally, the permeability of the filter fabric shall be equal to or more than the permeability of the surrounding soil.

3.2.13 Drainage Structures

The minimum side dimension of the drainage structures for exfiltration trenches shall be four feet. Inlets shall include sediment sumps to collect sediments and skimmers / baffles (Refer to Standard Index 241) to prevent oil and floating debris from exiting the catch basin into the exfiltration trench. The minimum clear distance between baffles in the same drainage structure will be two feet. Fiberglass skimmers and baffles are not recommended due to possible damages from debris impact.

Drainage structures have to provide adequate access to the exfiltration trench for maintenance operations; the minimum grate size should be two feet and 2-piece cast iron covers (Standard Index 201) are recommended. Manholes should be provided for inspection and clean out at the end of each exfiltration trench with no inlet. If double-chamber outfall control structures are used, access to both sides of the weir is to be provided. Inlets Type 1 to 4 (Standard Index 210) are recommended for exfiltration trenches installed along or from a gutter line.

3.3 Boundary Conditions

The design and performance of an exfiltration trench system depends on the specific boundary conditions of the site, which are: the ground water elevation, the tailwater elevation if positive outfall and the allowable headwater.

3.3.1 Ground Water Elevation

The ground water elevation to design exfiltration trench systems will be the high ground water table as defined in Section 2.2.5 above.

3.3.2 Tailwater Elevation

The receiving water body defines the tailwater conditions for exfiltration trench systems with positive discharge. The design tailwater should be defined as per the latest FDOT Drainage Manual, Section 3.4.

3.3.3 Headwater Elevation

The maximum allowable stage upstream of the exfiltration trench system will limit the design high water elevation. The drainage design in general should limit the design high water during the design storm event to meet the base clearance requirements and cause no adverse impact to the land use value of the surrounding properties.

3.4 Methodologies to Design Exfiltration Trenches

There are several methodologies used to design exfiltration trench systems. All methods are similar in nature with specific criteria and requirements placed by regulatory agencies FDOT District Drainage Offices. As such, the methodology to be used in each specific project should be coordinated and previously approved by the District Drainage Engineer and the Permitting Authorities with jurisdiction over the area where the proposed drainage system will be installed.

The equations and formulas included in the following sections present the conceptual development of the procedures and calculations, which is applicable with any unit system. As such, the conversion factors are not included, but the designer has to convert the units of each parameter as required to be consistent.

In order to illustrate the calculation methods a sample roadway segment (Figure 3.4) with a contributing drainage area of 2.3 acres (including 0.8 acres of pavement) with on-line treatment will be used in the following Sections 3.4.1, 3.4.2 and 3.4.3.

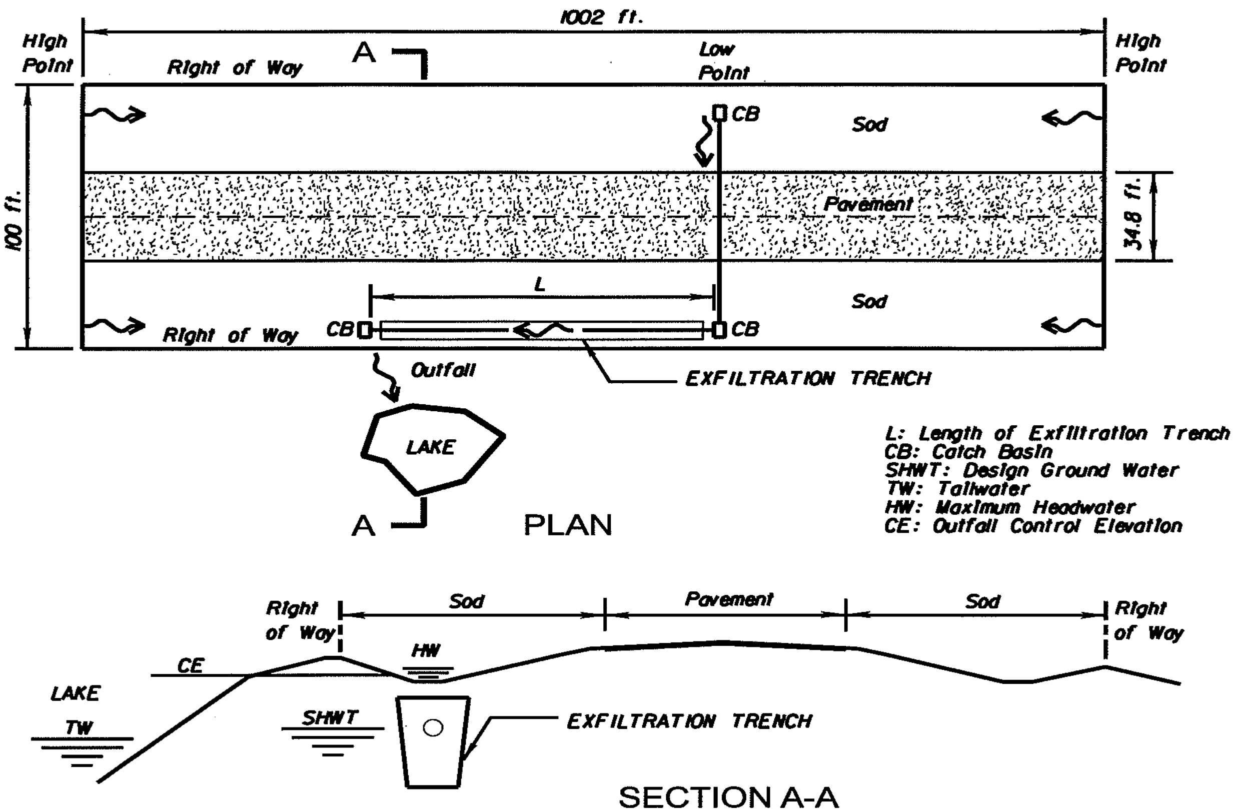

FIGURE 3.4 SAMPLE PROBLEM

3.4.1 Storage-Recovery Method

This method is acceptable for the most of the permitting agencies and FDOT Districts because it provides the required water quality storage capacity within the exfiltration trench and assures that the treatment capacity will be available again within the required recovery time.

The storage-recovery method is recommended for exfiltration trenches with bottom elevation, or at least the perforated pipe invert above the design ground water elevation. This method is not applicable if the top of the proposed exfiltration trench is below the design ground water elevation.

Storage-Recovery Design Procedure:

<u>Step 1:</u> Data Collection

Total Drainage Area: A_T = 2.3 acre

Pavement Area: A_I = 0.8 acre

Ground water elevation: SHWT = 6.00 ft

Tailwater elevation: TW = 10.00 ft

Headwater elevation: HW = 14.00 ft

Fillable porosity of soil: $f_s = 0.3$

Fillable porosity of aggregate: $f = 0.45$

Unsaturated hydraulic conductivity of soil: $K_u = 0.00007 \dfrac{ft^3 / sec}{ft^2\text{-ft of head}}$

Saturated hydraulic conductivity of soil: $K_s = 0.00025 \dfrac{ft^3 / sec}{ft^2\text{-ft of head}}$

Step 2: Calculate the required storage for water quality

Note: The water quality criterion from Saint Johns River Water Management District (SJRWMD) is used for the sample problem.

If off-line treatment is proposed use the greatest of the following volumes:

$V_T = 0.5"/12\ A_T$ $V_T = 0.096$ acre-ft

$V_I = 1.25"/12\ A_I$ $V_I = 0.083$ acre-ft $V_{off} = 0.096$ acre-ft

If on-line treatment is proposed increase 0.5-inch runoff on the total drainage area to the off-line treatment volume above:

$V_{on} = V_{off} + V_T = 0.096 + 0.096;$ $V_{on} = 0.192$ acre-ft

If discharging into shellfish harvesting areas or other receiving water bodies with specific regulations provide the water quality volume as required: $V_{spec} = 0$ acre-ft

Total Required water quality volume: $V_{WQ} = V_{on};$ $V_{WQ} = 0.192$ acre-ft

Step 3: Define the preliminary characteristics of the exfiltration trench

Outfall control elevation: CE = 13.00 ft Perforated pipe diameter: D = 24 in

Trench bottom width: $W_{tr} = 5$ ft Perforated pipe invert: $P_{inv} = 10.00$ ft

Trench bottom elevation: B_{el} = 8.00 ft

Trench top width: T_{tr} = 5 ft

Trench top elevation: T_{el} = 13.00 ft

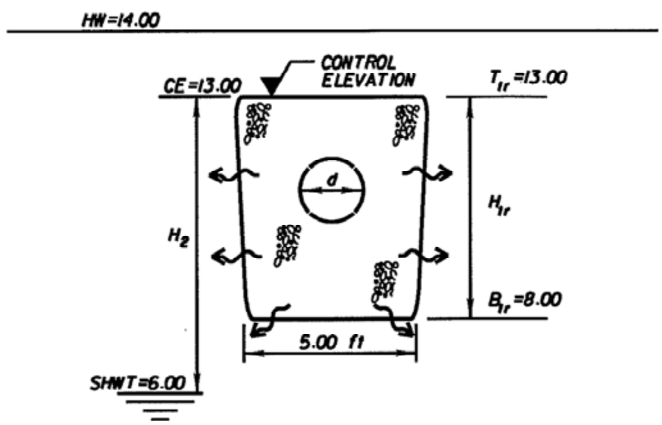

FIGURE 3.4.1 SAMPLE EXFILTRATION TRENCH

<u>Step 4:</u> Calculate the required length of exfiltration trench

The storage capacity of an exfiltration trench is the available void space above the seasonally high ground water elevation (SHWT), including pipe and aggregate voids.

Storage within pipe:

If the pipe invert is higher than the SHWT
the storage area in pipe is: $A_{full} = \dfrac{\pi D^2}{4}$ $A_{full} = 3.142\ ft^2$

Storage within aggregate:

Storage height in trench: $D_u = T_{el} - SHWT$ $D_u = 7.00$ ft
(unsaturated depth)

Storage area in trench: $A_{trench} = f\,(W_{tr} \cdot D_u - A_{pipe})$ $A_{trench} = 14.34\ ft^2$

Note: Engineering judgment will be used to consider or not the thickness of the pipe walls to calculate A_{trench}.

Net trench length: $L_{net} = \dfrac{V_{WQ}}{A_{pipe} + A_{trench}}$ $L_{net} = 477.7$ ft

Safety factor to account for hydro-geotechnical uncertainties ($SF \geq 2$) SF = 2

Required trench length: $L_{req} = SF\ L_{net}$ $L_{req} = 955$ ft

Note: If the required length does not fit within the available locations, return to Step 2 and modify the preliminary characteristics of the exfiltration trench as required.

Step 5: Calculate the recovery time of the treatment volume

Effective head: $H_{eff} = D\frac{HW - SHWT}{2}$ $H_{eff} = 4.00$ ft

Trench height: $H_{tr} = T_{el} - B_{el}$ $H_{tr} = 5.00$ ft

Unsaturated exfiltration will occur only through walls (2) and bottom if the trench bottom is above the SHWT:

$$A_{uwb} = L_{net}\ (2D_u + W_{tr}) \qquad A_{uwb} = 9076\ ft^2$$

Determine the unsaturated exfiltration capacity of the exfiltration trench:

$$Q_u = K_u\ A_{uwb}\ H_{eff} \qquad Q_u = 2.54\ \frac{ft^3}{sec}$$

Recovery time ($T_{rec} \leq 72hr$): $T_{rec} = \frac{V_{WQ}}{Q_u}$ $T_{rec} = 0.91hr$ OK

3.4.2 Empirical Equations Method

The following exfiltration trench design formulas have been developed by the South Florida Water Management District (SFWMD) and are included in the SFWMD Permit Application Manual, Volume IV. This method calculates the required length of exfiltration trench based on a one-hour exfiltration time, which is representative of the majority of small magnitude and short duration rainfall events.

Empirical Equations Design Procedure:

Step 1: Data Collection (Refer to Figures 3.1 and 3.4)

Total Drainage Area: A = 2.3 acre

Pavement Area:	A_i = 0.8 acre
Ground water elevation:	SHWT = 11.00
Tailwater elevation:	TW = 10.00
Headwater elevation:	HW = 14.00
Saturated hydraulic conductivity:	$K = 0.00025 \dfrac{ft^3 / sec}{ft^2 - \text{ft of head}}$

Step 2: Calculate the required storage for water quality

The empirical equations have incorporated the adjustment to consider that exfiltration trenches are retention systems (50% of the treatment volume required for wet detention systems) and a safety factor of 2. As such, the treatment volume to be used in these formulas is the greatest of 1 inch runoff on the contributing area or 2.5 inches on the pavement area:

V_T = 1.0”/12 A $\qquad$ V_T = 0.19 acre·ft

V_I = 2.5”/12 A_i $\qquad$ V_I = 0.17 acre·ft $\qquad$ V_{WQ} = 0.19 acre-ft

Step 3: Define the preliminary characteristics of the exfiltration trench

Outfall control elevation:	CE = 13.00	Perforated pipe diameter:	D = 24 in
Trench bottom width:	W = 5.00 ft	Perforated pipe invert:	P_{inv} = 10.00 ft
Trench bottom elevation:	B_{el} = 4.00		
Trench top elevation:	T_{el} = 13.00		

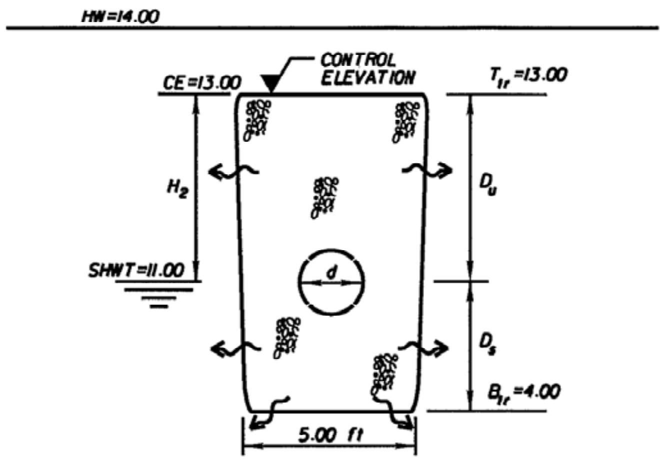

FIGURE 3.4.2 SAMPLE EXFILTRATION TRENCH

<u>Step 4:</u> Calculate the required length of exfiltration trench

Treatment volume in acre-inches:	$V = 12V_{WQ}$	V = 2.30 acre-in
Effective head:	$H_{eff} = \dfrac{CE - SHWT}{2}$	H_{eff} = 2.00 ft
Trench height:	$H = T_{el} - B_{el}$	H = 9.00 ft
Unsaturated depth of trench:	$D_u = T_{el} - SHWT$	D_u = 2.00 ft
Saturated depth of trench:	$D_s = SHWT - B_{el}$	D_s = 7.00 ft

When either non-saturated depth of trench is greater than saturated depth or the trench width is lesser than 2 times depth:

$$L_1 = \frac{V}{K\,(H_2W + 2H_{eff}\,D_u - D_u^2 + 2H_{eff}\,D_s) + 0.000139W\,D_u} \qquad L_1 = 193.44 \text{ ft}$$

When either saturated depth of trench is greater than non-saturated depth or the trench width is greater than 2 times depth:

$$L_2 = \frac{V}{K\,(2H_{eff}\,D_u - D_u^2 + 2H_{eff}\,D_s) + 0.000139 \cdot W \cdot D_u} \qquad L_2 = 244.94 \text{ ft}$$

Required length of trench $L_{req} = L_1$ if $D_u \geq D_s$ or L_2 if $D_u < D_s$ (use the greatest of these $\qquad L_{req}$= 244.94 ft two lengths): $L_{req} = L_1$ if $W \leq 2H$ or L_2 if $W > 2H$ $L_{req} = 193.44$ ft

For the sample problem the saturated depth of trench is greater than the non-saturated depth but the trench width is lesser than 2 times the trench depth. As such, the required length of trench is 245 feet.

3.4.3 FDOT District VI Method

The technical paper SUBSURFACE DRAINAGE WITH FRENCH DRAINS, prepared by the District VI Drainage Section on June 20, 1991 (Refer to Appendix A) includes criteria and procedures to design exfiltration trenches. This method is acceptable for projects within the FDOT District VI jurisdiction, mainly within Miami-Dade County, and other areas if approved by the permitting authorities.

This method considers that there is no flow through the bottom of the trench (assuming that the bottom is the first portion of the trench to get clogged) but only through the vertical areas (walls) of the exfiltration trench. The hydraulic conductivity of the existing soils at the depth where the exfiltration trench will be located is considered to calculate the dimensions and required length of trench. A test procedure has been developed to determine the hydraulic conductivity (K) of the soils at different depths: The initial investigation includes soils from 0 to 10-foot depth; if the test hole exfiltration rate is less than 6 gpm then soils from 10-foot to 15-foot depth are investigated. If the accumulated exfiltration rate is still less than 6 gpm soils from 15-foot to 20-foot depth are investigated. Deeper exfiltration trenches are not considered economically practical. The exfiltration trench should be constructed with its bottom elevation coinciding with the depth of the selected test results.

FDOT District VI Design Procedure: (Refer to Appendix A)

<u>Step 1</u>: Data Collection (Refer to Figure 3.4)

Total Drainage Area:	$A = 2.3$ acres	Time of Concentration:	$t_c = 11$ min
Pavement Area:	$A_i = 0.8$ acres	Runoff Coefficient	
Pervious Area:	$A_p = A - A_i$	Impervious:	$C_i = 0.9$
Design Frequency:	F =10 years	Pervious:	$C_p = 0.3$
Ground water elevation:	SHWT = 11.00	Tailwater elevation:	TW=10.00

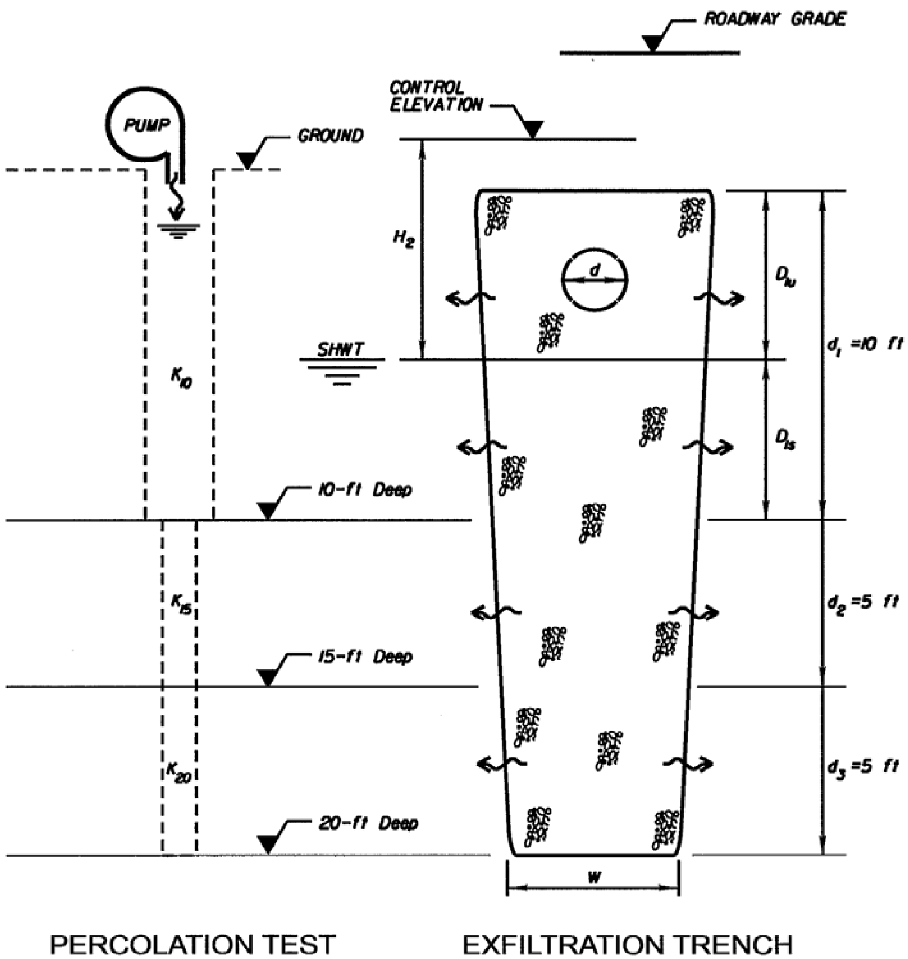

FIGURE 3.4.3 SAMPLE EXFILTRATION TRENCH

Saturated hydraulic conductivity in cfs / square foot per foot of head:

Depth from 0' to 10':	d_1 = 10 ft	K_{10} = 0.000152
Depth from 10' to 15':	d_2 = 5 ft	K_{15} = 0.000211
Depth from 15' to 20':	d_3 = 5 ft	K_{20} = 0.000349

Step 2: Determine the maximum polluted volume

Note: The following procedure is only applicable in Miami-Dade County, and it is required by the Miami-Dade Department of Environmental Resources Management (DERM).

Weighted runoff coefficient: $C = \dfrac{C_i \cdot A_i + C_p \cdot A_p}{A}$ $\qquad$ C = 0.51

Time to Generate 1" runoff: $t_1 = \dfrac{2940\ F^{-0.11}}{308.5\ C - 60.5(0.5895 + F^{-0.67})}$ $t_1 = 21.07$ min

Polluted runoff duration: $t_T = t_1 + t_c = 21.07 + 11$ $t_T = 32.07$ min

Rainfall Intensity: $i = \dfrac{308.5}{48.6\ F^{-0.11} + T_T(0.5895 + F^{-0.67})}$ $i = 4.86\ \dfrac{in}{hr}$
(For Miami-Dade-County)

Peak discharge $Q = C\ i\ A$ $Q = 5.70$ cfs

Maximum polluted volume: $V = 60\ Q\ T_T$ $V = 10939\ ft^3$

Note: All the runoff generated from a storm event lasting T_T or less is assumed to be polluted or contaminated. As such the maximum polluted volume (V) is usually greater than the treatment volume required by the Water Management Districts.

Step 3: Define the preliminary characteristics of the exfiltration trench

Outfall control elevation: CE = 13.00

Trench bottom width: W = 5.00 ft

Trench bottom elevation: $B_{el} = -7.00$

Trench top elevation; $T_{el} = 13.00$

Perforated pipe diameter: D = 24 in
Perforated pipe invert: $P_{inv} = 10.00$

Fillable porosity of aggregate: f = 0.5

Effective head: $H_{eff} = \dfrac{CE - SHWT}{2}$ H_{eff}=2.00 ft

Trench height: $H = T_{el} - B_{el}$ H=20.00 ft

Unsaturated depth of trench: $D_u = T_{el} - SHWT$ D_u=2.00 ft

Saturated depth of trench (0' to 10') $D_s = SHWT - B_e$ D_s=8.00 ft

Step 4: Determine the trench storage

The storage capacity of an exfiltration trench is the available void space above the design ground water elevation (SHWT), including pipe and aggregate voids.

Storage within pipe:

If the pipe invert is higher than the SHWT the storage area in pipe is: $A_{full} = \frac{\pi(D/12)^2}{4}$ $A_{full}=3.14\ ft^2$

If the pipe invert is lower than the SHWT the available storage depth in pipe is: $D_{pipe} = P_{inv} + \frac{D}{12} - SHWT$ $D_{pipe}=1.00ft$

Central angle of the circle segment: $\theta = 2acos[\frac{(D - 2D_{pipe})}{D}]$ $\theta = 0.82\ rad$

Storage area in pipe segment: $A_{seg} = \frac{(D/12)^2}{8}(\theta - \sin\theta)$ $A_{seg}=0.04\ ft^2$

$A_{pipe} = A_{full}$ if $P_{inv\ l} \geq SHWT$
$A_{pipe} = A_{seg}$ if $P_{inv} < SHWT$ In this example $A_{pipe} = A_{full}$ $A_{pipe}=3.14\ ft^2$

Storage within aggregate:

Storage height in trench:
(unsaturated depth) $D_u = T_{el} = SHWT$ $D_u = 2.00\ ft$

Storage area in trench: $A_{trench} = f\ (W\ D_u - A_{pipe})$ $A_{trench}=3.43\ ft^2$

Storage in trench: $S = A_{pipe} + A_{trench}$ $S = 6.57\ \frac{ft^3}{ft}$

Step 5: Determine the exfiltration rate per foot of trench

$$E_T = 2\ K_{10}\ (\frac{D_u}{2} + D_s)\ H_{eff} + 2\ K_{15}\ d_2\ H_{eff} + 2\ K_{20}\ d_3\ H_2$$

$$E_T = 0.0167\ cfs\ /\ foot\ of\ trench$$

Note: The exfiltration rate values are limited by the District VI Drainage Section to 0.15 cfs / linear foot of trench.

Step 6: Determine the length of exfiltration trench for water quality

$L_{net} = \underline{\quad V \quad}$ $L_{net} = 283\ ft$

$$S + 60 \cdot E_T \cdot T_T$$

Safety factor to account for hydro-
Geotechnical uncertainties ($SF \geq 2$): $SF = 2$

$$L_{req} = SF\ L_{net} \qquad\qquad L_{req} = 566 \text{ ft}$$

3.4.4 Other Design Methods

a) Design Curves for Exfiltration Systems: The exfiltration curves provide the ratio between the trench storage and the exfiltration rate from the trench to the soil. This method is based on the long term mass balance of an exfiltration system under local rainfall conditions. As such, it should only be acceptable for the areas where the exfiltration curves have been developed.

b) Exfiltration Trenches for Discharge Attenuation: With the exception of Miami-Dade County and Monroe County, exfiltration trenches are not approved for discharge attenuation by most of the permitting agencies. As such, only under very special conditions the use of exfiltration trenches to attenuate the outfall discharge will be negotiated with the permitting authorities, prior approval by the FDOT District Drainage Engineer.

   The exfiltration trench design procedure for discharge attenuation should be similar to the procedures described above. The difference is the required treatment volume: For closed basins the treatment volume should be the pre-development versus post-development discharge increase instead of the required water quality volume. The same criteria could be applied for basins with positive outfall if the Rational Method is used because this method considers the rainfall intensity constant along the storm duration. If hydrograph methods are used, the runoff hydrograph has to be combined with the exfiltration hydrograph to determine the outflow from the drainage system. Spreadsheets and modeling programs are available to perform hydrograph calculations.

Chapter 4

Drainage Wells

4.1 Description

The term drainage wells include all wells that are used to inject surface water directly into an aquifer, or transfer shallow ground water directly into a deeper aquifer. By definition, an injection well is any bored, drilled, or driven shaft or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system (Refer to Figure 4.1).

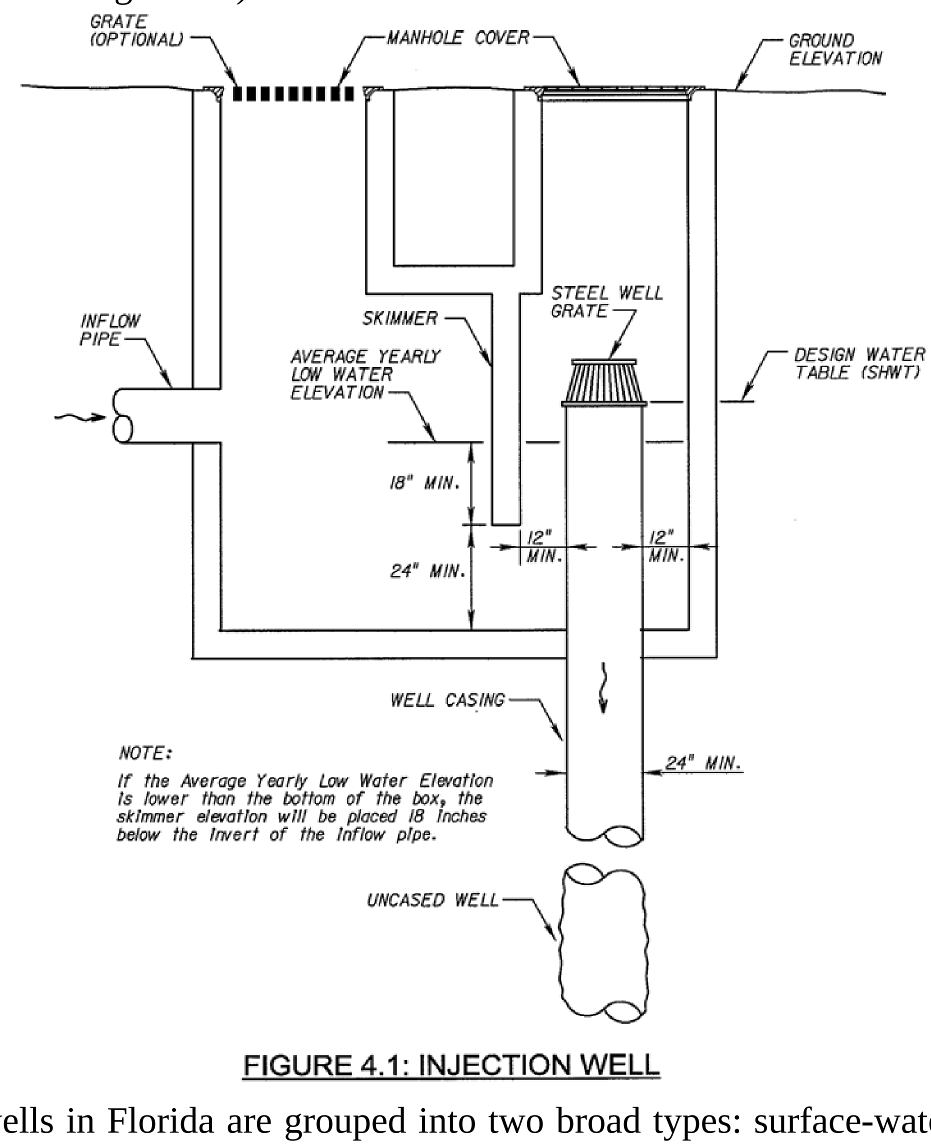

FIGURE 4.1: INJECTION WELL

Drainage wells in Florida are grouped into two broad types: surface-water injection wells and inter-aquifer connector wells. Surface-water injection wells are further categorized as either Floridan aquifer drainage wells or Biscayne aquifer drainage wells.

4.1.1 Use

The Floridan aquifer drainage wells are generally effective as a method of urban drainage and lake level control. They emplace more recharge into the Floridan aquifer than the recharge it would receive under natural conditions. The most common use of Floridan aquifer drainage wells is to supplement surface drainage for urban areas in the karst terrains of the topographically higher areas of central and north Florida. Caution, however, is suggested in regard to the water quality aspects of these wells because they often inject surface runoff into the same aquifer from which public water supplies are withdrawn.

Biscayne aquifer drainage wells are used locally to dispose stormwater runoff and other surplus water, mainly in southeast Florida. The majority of these wells are used to dispose water from swimming pools or to dispose heated water from air-conditioning units. The remainders are used for disposal of urban runoff or wastewaters from business and industry in the area. The use of Biscayne aquifer drainage wells should have minimal effect on aquifer water quality so long as the injection of runoff and industrial wastes is restricted to zones where chloride concentrations exceed 1,500 milligrams per liter and 10,000 milligrams per liter Total Dissolved Solids (TDS).

4.2 Design Criteria

The design of a of storm sewer system using drainage wells requires the hydrologic analysis and determination of the design peak runoff rate of discharge from the project area. The procedures for determining the peak runoff rates are included in the Department's Drainage Handbook, Hydrology.

The data collection for drainage well design should include the research of similar installed wells within the project area. Local well contractors can provide an estimate of the discharge capacity of wells based on previous drainage well installations and pumping tests. In cases where there is no available data, test holes and pumping tests should be performed. The exfiltration capacity of a well will be determined in gallons per minute per foot of head.

4.2.1 Water Quality

Water quality requirements have to be addressed before discharging into injection wells. The typical design of drainage wells provides for the use of a retention basin with baffles or skimmers prior to discharging into the drainage well. The size of the retention basin is determined based on a 90-second detention time. Other options include the use of exfiltration trenches and detention/retention treatment swales.

Stormwater pollutant load is very dependent on the climatic and topographic features such as storm intensity and duration, distribution over the basin, land use, and topographic features such as hills, swamps, and soil types. Therefore the type of stormwater treatment should be designed to meet the needs of the particular location (i.e. more stringent water quality measures should be required for wells discharging into the Floridan aquifer). Pretreatment methods include physical, chemical, and biological control measures. Physical treatment includes typical operations as settling and screening. An example of chemical treatment would be the injection of alum into the stormwater on a storm by storm basis. Biological treatment might be accomplished by using plants, fish, or other types of treatment in retention ponds. In many instances there is a combination of the above methods prior to the discharge of stormwater into the freshwater injection well.

The discharge of the wells needs to occur below the 10,000 ppm Total Dissolved Solids (TDS) level.

4.2.2 Fresh Water-Salt Water Hydrostatic Balance

One major consideration in the design of drainage wells in coastal areas is the difference in density between fresh water and saline water. The hydrostatic balance between fresh and saline water can be illustrated by the U-tube shown in Figure 4.2.2 (A). Pressures on each side of the tube must be equal, therefore:

$$P_s \, g \, H_f = P_f \, g \, (Z + H_f)$$

Where:
P_s = Density of the saline water (lb/ft^3)
P_f = Density of the fresh water (lb/ft^3)
g = acceleration of gravity (ft/sec^2)
Z = Head difference (ft)
H_f =Fresh water height above the Mean Sea Level (ft)

Solving for Z the Ghyben-Herzberg relation is obtained:

$$Z = (P_f / P_s - P_f) \, H_f$$

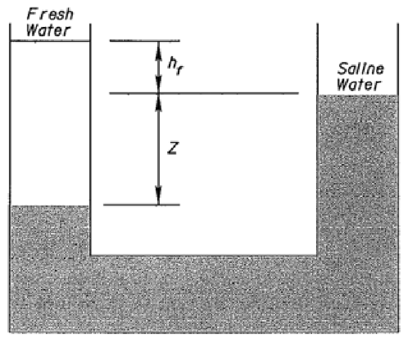

FIGURE 4.2.2 (A): U-TUBE HYDROSTATIC BALANCE BETWEEN FRESH AND SALINE WATER

Translating the U-tube to a coastal situation as shown in Figure 4.2.2 (B), H_f is the elevation of the water table above the sea level and Z is the depth to the fresh-saline water interface below the sea level.

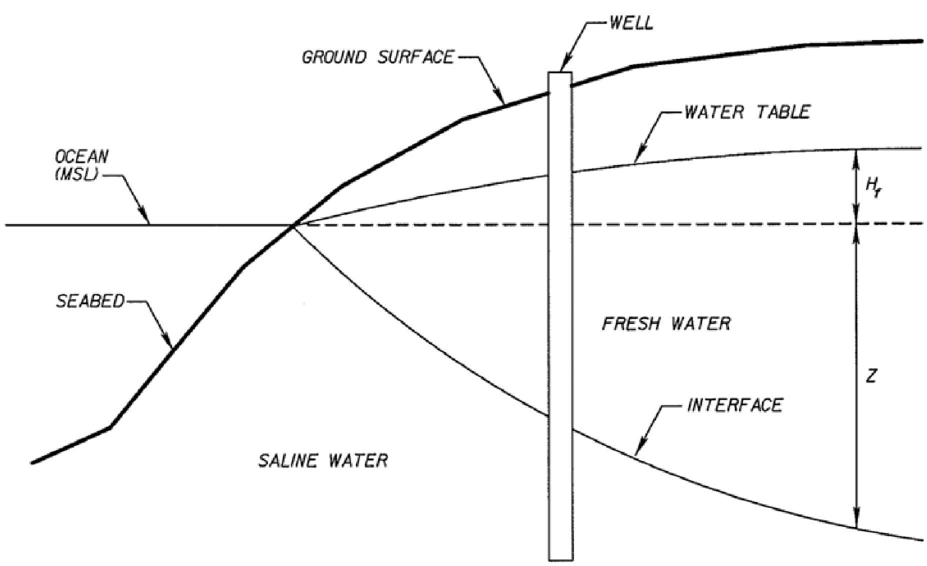

FIGURE 4.2.2 (B): FRESH-SALINE WATER INTERFACE

For typical seawater conditions (P_s = 63.9 lb/ft^3 and P_f = 62.4 lb/ft^3) the hydrostatic head balance is approximately: $Z = 40\ H_f$

The cased portion of the drainage wells in coastal areas is placed at least up to the fresh-saline water interface. The head loss due to the difference in the density of salt water and freshwater could be approximated as one foot of head loss per 40 feet of casing based on the relationship $Z = 40\ H_{f,}$. As such, a typical design with 60-foot average casing up to the interface would require 1.5 feet of head to displace the salt water, which is the usual rule of thumb value used to design drainage wells discharging into the Biscayne aquifer. This additional head is not required for wells discharging into a freshwater aquifer.

4.2.3 Hydraulic Head

The design hydraulic head for gravity drainage wells is limited by the maximum allowable stage upstream of the drainage wells. The design high water for the design storm event should meet the base clearance requirements and cause no adverse impact to the land use value of the surrounding properties.

In areas where gravity head is not available, the required hydraulic head can be obtained artificially by pumping. Pressurized drainage wells have basically the same design requirements as the gravity wells but the hydraulic head is produced by a lift station. Runoff pretreatment is necessary prior to the lift station, which is usually provided by a retention basin with baffles and a bar screen for protection of the pumps. These features are typically combined with a finer screen to remove smaller debris from discharging into the well. The FDEP has waived the requirement of the Reasonable Assurance Report (Refer to Appendix C) when a passive by-pass at or below 8.00-foot NGVD'29 is provided from the pumps common header. This requires a vertical stack pipe with a top elevation at 8.00-foot NGVD'29. If the head on the pumps is larger there will be overflow in this stack. The stack is normally provided with a cap/bird screen to avoid tampering.

4.2.4 Safety Factor

Due to some uncertainty issues related to geological and other factors a Safety Factor of 1.5 is recommended for the design of Drainage Wells.

4.2.5 Dimensions

Drainage (deep) wells are usually 24-inch diameter and 100 feet to 150 feet in depth. When more than 1 well is necessary for the system a 75-foot to 100-foot separation between wells is recommended.

4.2.6 Exfiltration Rate

The drainage well is drilled as an open hole until the desired level of exfiltration is found based on the results of the well test holes and the pumping tests described in Section 2.2.4 f and g above. Common values of well exfiltration rates (in Miami-Dade County) range from 500 to 1500 gpm per foot of head.

4.2.7 Casing

The casing point is usually determined by finding the required minimum total dissolved solid levels in the aquifer or by finding structurally stable rock formation. Approximately 70 percent of the well depth is usually provided with a steel encasement.

4.3 Methodologies of Calculation

The calculation methods to design gravity wells (Section 4.3.1) and pressurized wells (Section 4.3.2) are illustrated based on a sample roadway segment (Figure 4.3) with a contributing drainage area of 2.3 acres including 0.8 acres of pavement to be drained into an injection well system.

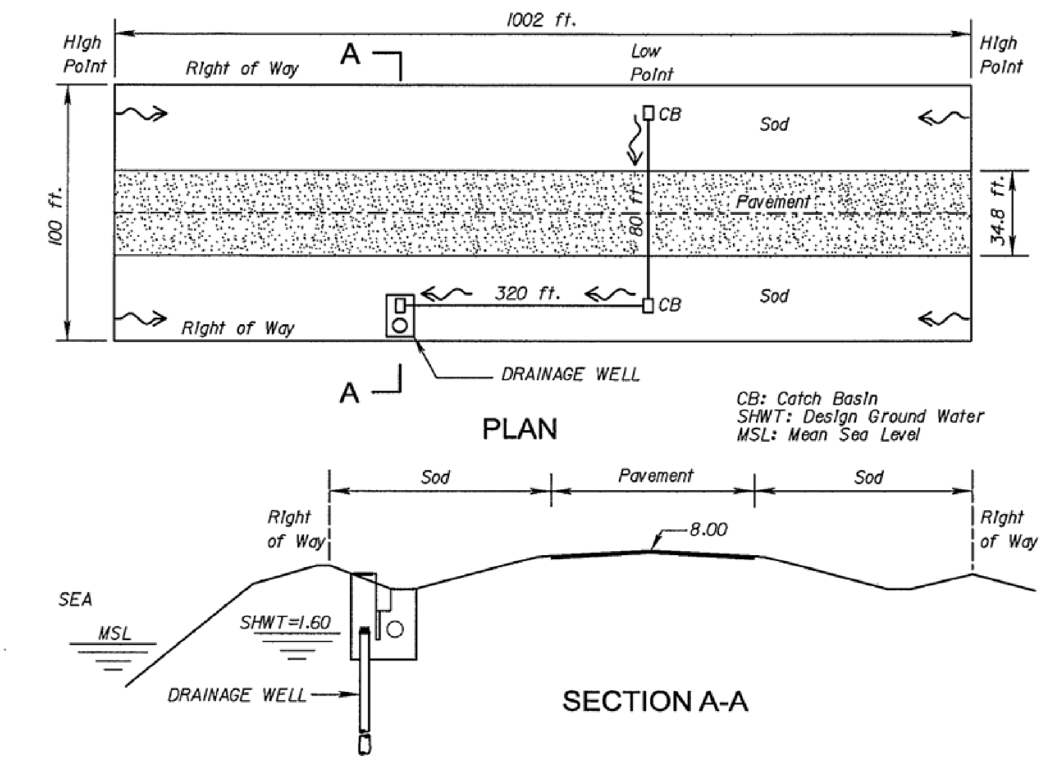

FIGURE 4.3: SAMPLE PROBLEM

4.3.1 Gravity wells

<u>Step 1</u>: Data Collection (Refer to Figure 4.3)

Total Drainage Area:	$A = 2.3$ acres	Hydrologic Location:	Zone 10
Pavement Area:	$A_i = 0.8$ acres	Design Frequency:	$F = 3$ years
Pervious Area:	$A_p = A - A_i$	Runoff Coefficient	

Ground Water Elevation: SHWT = 1.60 Impervious: $C_i = 0.95$

Minimum Roadway Grade: G = 8.00 Pervious: $C_p = 0.30$

Well Design Data: (Based on information from other wells near the project site).

Well Capacity: Q_W = 750 gpm / ft of head

Depth to Interface: Z = 60 ft

<u>Step 2</u>: Determine the peak discharge rate into the gravity well system.

Weighted runoff coefficient: $C = \frac{C_i \cdot A_i + C_p \cdot A_p}{A}$ C = 0.53

Time of concentration (t_c):

Note: Methods to calculate the time of concentration can be found in the Department's Hydrology Handbook, Section 1.2.2.2. The Rational method is used to solve the sample problem but for larger projects the Unit Hydrograph Method could be used to design the system.

Inlet time: t_i = 10 min (Based on the minimum t_c)

Travel time (pipe): $t_t = \frac{\text{Flow Path Length}}{\text{Flow Velocity}} = \frac{80\text{ ft} + 320\text{ ft}}{2\text{ ft/sec}} = 3.3\text{ min}$

$t_c = t_i + t_t$ $t_c = 13.3$ min

Rainfall Intensity:
(For Miami-Dade County)

$i = \frac{308.5}{48.6\,F^{-0.11} + T_T\,(0.5895 + F^{-0.67})}$ $i = 5.39\ \frac{in}{hr}$

Note: The rainfall intensity for other project locations can be determined using the Rainfall Intensity-Duration- Frequency Curves from the Department's Hydrology Handbook, Figures F-21 to F-32.

Peak discharge $Q = C\,i\,A$ Q = 6.52 cfs

Note: If substantial runoff storage capacity exists or is to be provided within the contributing area, the attenuation of the peak discharge due to the runoff storage has to be considered. In these cases the SCS Method is recommended.

<u>Step 3</u>: Calculate the exfiltration capacity of one gravity well.

Effective head: $H_{eff} = CE - SHWT - \Delta H$ $H_{eff} = 0.50$ ft

Note: The head loss ($\Delta H = 1.5$ ft.) used above is based on the rule of thumb described in Section 4.2.2. Additional head loss due to friction along the well casing (approximately 0.001 foot per foot of casing) can be considered but it is usually disregarded. More accurate head loss calculation may be necessary when dealing with deep casings and very high flow rates coupled with low available hydraulic heads.

One-well capacity: $Q_w = 0.00223\, q_w\, H_{eff}$ $Q_w = 0.84$ cfs

<u>Step 4</u>: Determine the required number of gravity wells.

Safety Factor: SF = 1.5

Number of gravity wells: $N_w = \frac{SF\, Q}{Q_w}$ $N_w = 11.69$ Wells

Recommendation: Use 12 gravity wells at a minimum spacing of 75 ft. These wells could be distributed within the project area or at one location. Head losses due to pipe lines have to be considered to design the stormwater system.

<u>Step 5</u>: Determine the 90-second retention volume for **each** gravity well.

Required detention volume: $V_{90sec} = \frac{90\, Q}{N_w}$ $V_{90sec} = 48.9\ ft^3$
(for each gravity well)

NOTE: The 90-second retention volume has to be provided right before the drainage wells, below the top of the well (usually at the SHWT).

4.3.2 Pressurized Wells

<u>Step 1</u>: Data Collection (Refer to Figure 4.3)

(The same as Section 4.3.1 Step 1)

<u>Step 2</u>: Determine the peak discharge rate into the gravity well system.

(The same as Section 4.3.1 Step 2)

<u>Step 3</u>: Determine the net hydraulic head required for a pressurized well system.

Number of pressurized wells: $N_p = 1$ well

Safety Factor: SF = 1.5

Pressurized net head: $H_p = \dfrac{SF\ Q}{0.00223\ q_w\ H_2} + \Delta H$ $H_p = 7.34$ ft

Note: The head loss ($\Delta H = 1.5$ ft.) used above is based on the rule of thumb described in Section 4.2.2. Additional head loss due to friction along the well casing (approximately 0.001 foot per foot of casing) can be considered but it is usually disregarded. More accurate head loss calculation may be necessary when dealing with deep casings and very high flow rates coupled with low available hydraulic heads.

Required head elevation: $HEAD = SHWT + H_p$ HEAD = 8.94 ft NGVD

Note: The sample problem would require approximately 12 wells to function by gravity (See Section 4.3.1, Step 4 above), while by pressurizing the system with 7.34 ft head (at 8.94 ft NGVD) only one well would be needed. Notice that a Reasonable Assurance Report will be required by FDEP (See Section 4.2.2) because the head elevation at 8.94 ft NGVD is higher than 8.00 ft NGVD.

<u>Step 4</u>: Determine the 90-second retention volume for **eac**h pressurized well.

Required detention volume: $V_{90sec} = \dfrac{90\ Q}{N_p}$ $V_{90sec} = 586.8\ ft^3$
(for each gravity well)

Note: The 90-second retention volume has to be provided right before the pump station or stations for the pressurized drainage wells.

<u>Step 5</u>: Design the pump station.

Required pump discharge: $Q_p = SF\ Q$ $Q_p = 9.78$ cfs

Required head elevation (Step 3 above): HEAD = 8.94 ft NGVD

The pump station would have to deliver a flow of 9.78 cfs with a maximum head of 8.94 ft NGVD. The system design will then consist of pump selection, design of the pump pit and the forced line, which is not in the contents of this handbook. The procedure for the lift station design could be done by manual methods, spreadsheets, and other computer software commercially available for lift station design.

Chapter 5

Modeling Exfiltration Systems

5.1 Basic Modeling Concepts

Hydraulic modeling idealizes existing and proposed hydraulic systems for calculation purposes. The hydraulic models generated for a specific project may vary in complexity as a function of the accuracy desired by the designer. Several spreadsheets and modeling software are available to design or evaluate open and closed stormwater systems, but the designers have to use software accepted by the Department and the permitting agencies with jurisdiction over the area where the project is located.

The main elements in a hydraulic model setup are nodes and links. A node is a point with a defined location in the drainage system used to simulate inlets, manholes, grade breaks, bends, outlets, etc. Nodes are usually points where the conservation of mass is maintained during the calculation process. Links are the connections between nodes used to simulate pipes, ditches and channels. Links are used to transfer or convey water through the drainage system.

Other important modeling elements are the boundary conditions, which usually simulate the tailwater conditions as time-stage nodes, and the drainage areas, which are closed boundaries contributing into their respective nodes. Rainfall and peak runoff computations, usually related to the drainage areas, are calculated using the rational or the SCS methods.

5.2 Exfiltration Trenches Modeling

In order to simulate the exfiltration trench performance in a hydraulic model the trench pipe is modeled as a normal pipe link between nodes. The stage-area or the stage-volume characteristics of the exfiltration trench are usually assigned to one of the nodes where the trench is connected. If the modeling program computes the emptiness of the links only the voids in the aggregate will be considered in the stage-area or the stage-volume capacity of the exfiltration trench. A boundary node, usually as a time-stage node, is included to simulate the design groundwater elevation, which could be constant or variable with time as per the designer's criteria.

It is necessary to include a special link to model the transferring of runoff from the node simulating the exfiltration trench to the node simulating the design ground water (See Figure 5.2). This special link is usually a head-discharge ratio or rating curve, which could be defined by giving different head values (ΔH) in order to calculate their respective discharges (Q) using the following equation (Refer to Section 2.2.3) :

$$Q = (K_u A_u + K_s A_s) \Delta H$$

Where: Q = Flow Rate (cfs)
K_u = Unsaturated Hydraulic Conductivity (cfs/ft^2 – ft of head)
K_s = Saturated Hydraulic Conductivity (cfs/ft^2 – ft of head)
A_u = Unsaturated Flow Area (ft^2) = L_{net} $(2D_u)$
A_s = Saturated Flow Area (ft^2) = L_{net} $(2D_s + W)$
ΔH = Hydraulic Head (ft)

If the unsaturated depth of the exfiltration trench (D_u) is less than one tenth of the total trench depth the unsaturated exfiltration can be disregarded because the unsaturated hydraulic conductivity is usually several times less than the saturated hydraulic conductivity of the soils. If the trench width is greater than 2 times the depth the exfiltration through the trench bottom should be disregarded ($A_s = 2L_{net} D_s$).

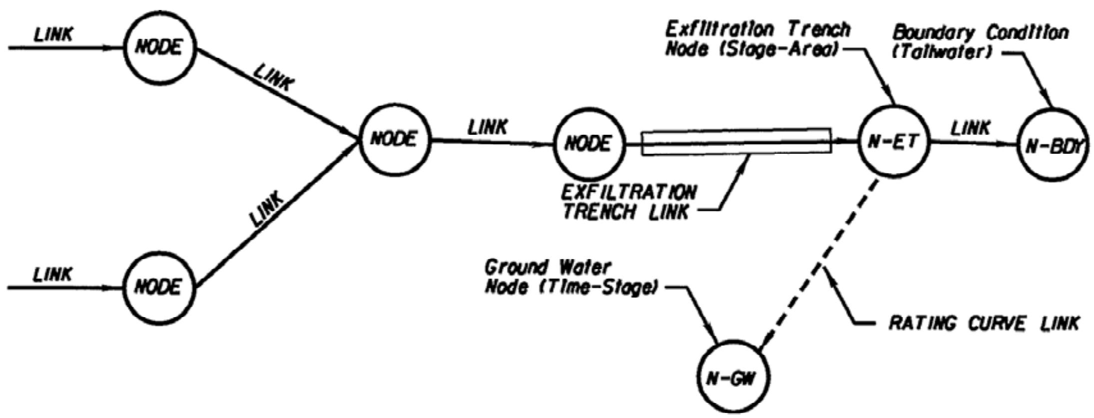

FIGURE 5.2: SAMPLE NODAL DIAGRAM FOR EXFILTRATION TRENCHES

The head-discharge ratio or rating curve may not be accurate if the design ground water is considered variable and the modeling program uses elevations instead of hydraulic head for calculations. The variable ground water is usually considered rising from the average October ground water elevation to the anticipated ground water elevation at the end of the design storm event. In these cases a family of tailwater-headwater-discharge rating curves is necessary to simulate the transferring of runoff from the exfiltration trench to the ground water. The rating curves can be calculated by giving specific values to couples of tailwater and headwater elevations in order to calculate their correspondent head values (ΔH). Having the hydraulic head (ΔH) the discharges (Q) can be calculated using the above equation and the tailwater-headwater-discharge relationship would be complete.

5.3 Drainage well Modeling

The inflow discharge is calculated using the approved hydrologic method and is assigned to a node which could simulate the retention box. The drainage well or a series of them could be analyzed using rating curves with elevation versus discharge relationship of the wells. The rating curve link will be connected to a time series node representing the water table at the discharge point. Following is a simple schematic nodal diagram of a well system model.

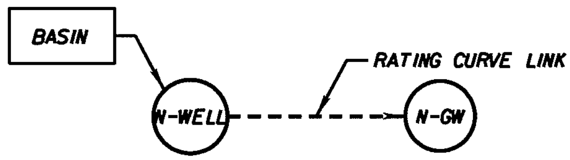

FIGURE 5.3: RATING CURVE LINK FOR DRAINAGE WELLS

Basin represents the drainage area contributing into the drainage well. Node N-WELL simulates the retention box prior to discharge into the well. The rating curve link simulates the well discharge into the water table. Node N-GW is the boundary node representing the ground water elevation.

The rating curve link will be connected to a time series node representing the water table at the discharge point. If the system includes several drainage wells a rating curve would be developed for each well. There are a number of acceptable computer software hydrologic and hydraulic models capable of analyzing the system.

APPENDIX A

SUBSURFACE DRAINAGE WITH FRENCH DRAINS

FLORIDA DEPARTEMENT OF TRANSPORTATION
DRAINAGE SECTION

Florida Department of Transportation
District 6
Drainage Section
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Table of Contents

1 – SUBSURFACE DRAINAGE

The subsurface drainage theory and application explained in the following pages is the standard procedure following by FDOT District VI in Miami for the design of French drains. A French drain is an exfiltration trench that provides conveyance and storage by using a perforated pipe and crushed stone. But, before entering into the French drain design is important to explain the exfiltration theory as it is seen by this drainage section.

2 - TRENCH EXFILTRATION

The exfiltration capacity of a trench is expressed by the formula:

$$Q = E\ L \qquad (1)$$

Q – Exfiltration of trench, in cfs
E – Exfiltration rate, in cfs / foot of trench
L – Length of the trench, in feet

The total exfiltration rate, that is, the exfiltration of 1 foot of trench, E_T, is defined by the general formula:

$$E_T = \sum K \quad A \quad H \qquad (2)$$

K = Hydraulic conductivity, in cfs / (sq ft x foot of head)
A = Area, in sq ft
H = Hydraulic head, in feet

One linear foot of trench is divided into different vertical areas. Each "KAH" represents the exfiltration flow, in cfs, that passes perpendicularly through each one of the vertical areas. It is assumed that no flow exists through the bottom of the trench. The hydraulic head affecting the horizontal movement of water is measured above the ground water elevation. Tests are made where the French drains will be located to determine the hydraulic conductivity of the subsoil.

3 – HYDRAULIC CONDUCTIVITY

The drainage section in Miami has a standard procedure for the determination of the hydraulic conductivity (K) for different depths. The depths investigated are: 1) from 0 to 10 feet deep, 2) from 10 feet to 15 feet deep, and 3) from 15 feet to 20 feet deep. The depths on 2) and 3) are investigated only if the hydraulic conductivity is low, that is, when P is less than 6 gpm, see figure 1.

1) FROM 0 FEET TO 10 FEET DEEP

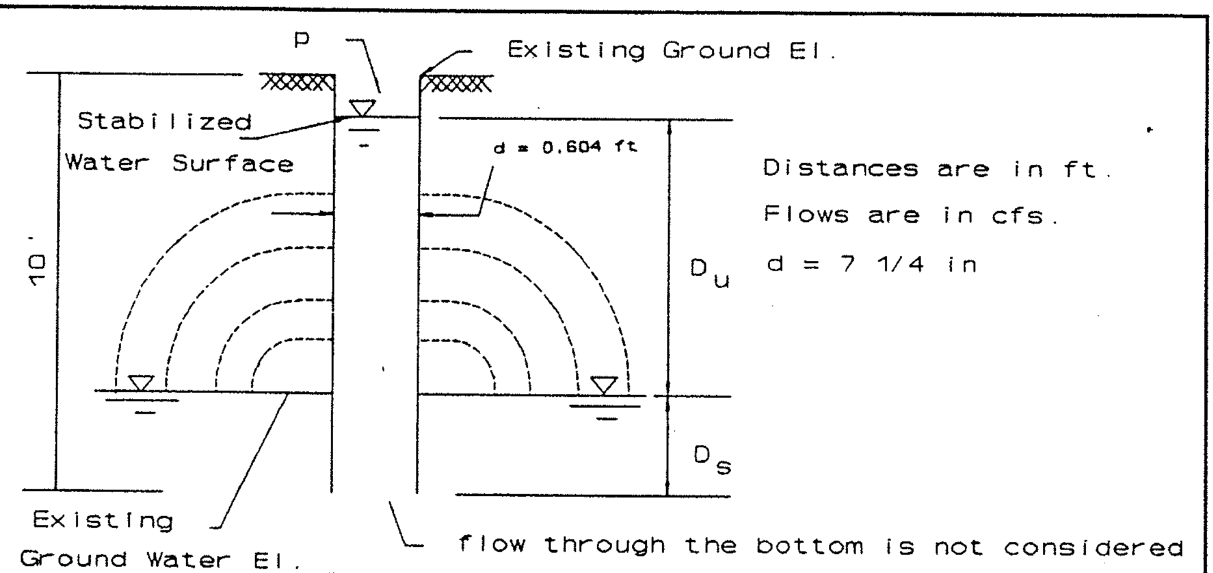


Figure 1
(nts)

The formula for the determination of the hydraulic conductivity for the 10 feet stratum is:

$$K_{10} = \frac{P}{\sum S\,h} \tag{3}$$

P – Pump discharge, in cfs
S – Area of any vertical surface perpendicular to the direction of flow, in sq ft

h – Average hydraulic head affecting the movement of water through S, in feet

It is important to note here that the formula (1), (2), and (3) imply a lineal relationship between flow and hydraulic head even though the flow is directly proportional to the head elevated to a power less than 1.

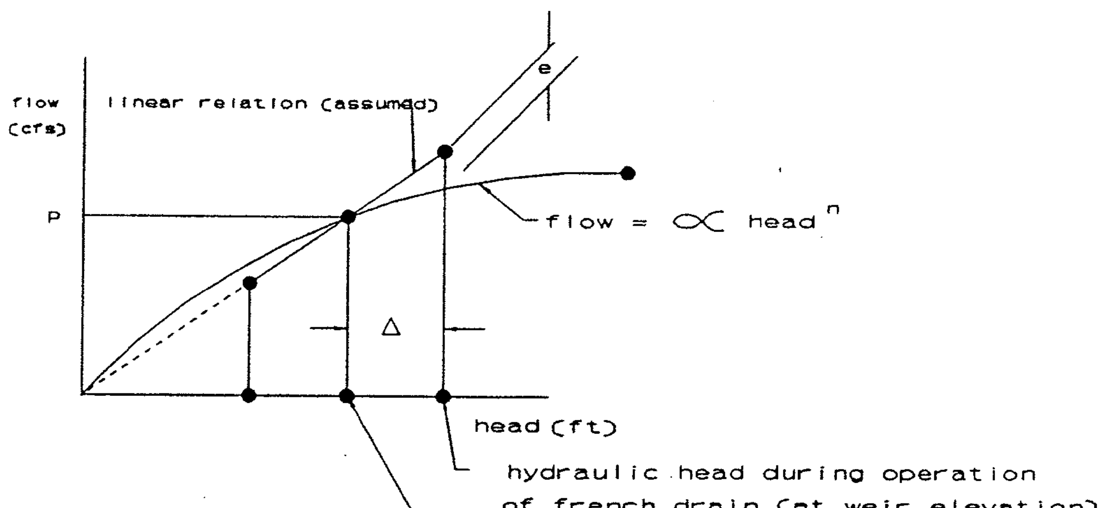


Figure 2
(nts)

a – "Varies directly as"
n < 1

The error, e, assuming a lineal relation and not a true one is small, because Δ is small too, see figure 2.

Formula (3) becomes then:

$K_{10} = P / (S_u h_u + S_s h_s)$

S_u – Unsaturated area, in sq ft = $\pi d D_u$
h_u – Average head for the unsaturated area, in feet; $h_u = D_u / 2$
S_s – Saturated area, in sq ft = $\pi d D_s$
h_s – Average head for the saturated area, in feet; $h_s = D_u$

$$K_{10} = P / ((\pi\ d\ D_u\ (D_u/2)) + (\pi\ d\ D_s\ D_u))$$

$$K_{10} = \frac{P}{\pi\ d\ D_u\ [D_u/2 + D_s]} \qquad (4)$$

Formula (4) determines the hydraulic conductivity for the test from 0 to 10 feet deep. The usual test performed is an uncased open-hole test 7 ¼" in diameter (0.604 ft), but the hole may be cased with a fully perforated casing for unstable soils. The procedure implies to:

a. Determine the elevation of the ground where the test will be performed.
b. Auger the hole to 10 feet deep.
c. Case the hole if necessary with perforated case.
d. Measure the distance from the ground surface to the water table before beginning the pumping.
e. Pump water into the test hole and maintain water level close to the ground surface. Pumping water must be recorded using one minute intervals. The pumping should be maintained for 10 minutes after the stabilization of the pumping rate and the water elevation in the hole. Then, measure the distance between ground surface and water surface in the hole.

Observe that formula (4) implies that the water table is less than 10 feet deep. Average ground water level for developed areas of Dade County is shown in Dade County Design Standard W.C. 2.5. It varies from 1.5 feet to 8.5 feet Mean Sea Level. Ground elevation varies from 5 feet to 12 feet indicating that the ground water is always less than 10 feet deep. The application of the above procedure for the determination of a formula for the case in which the water table is more than 10 feet deep does not give accurate results in the determination of the hydraulic conductivity and is not recommended.

2) FROM 10 FEET TO 15 FEET DEEP TEST

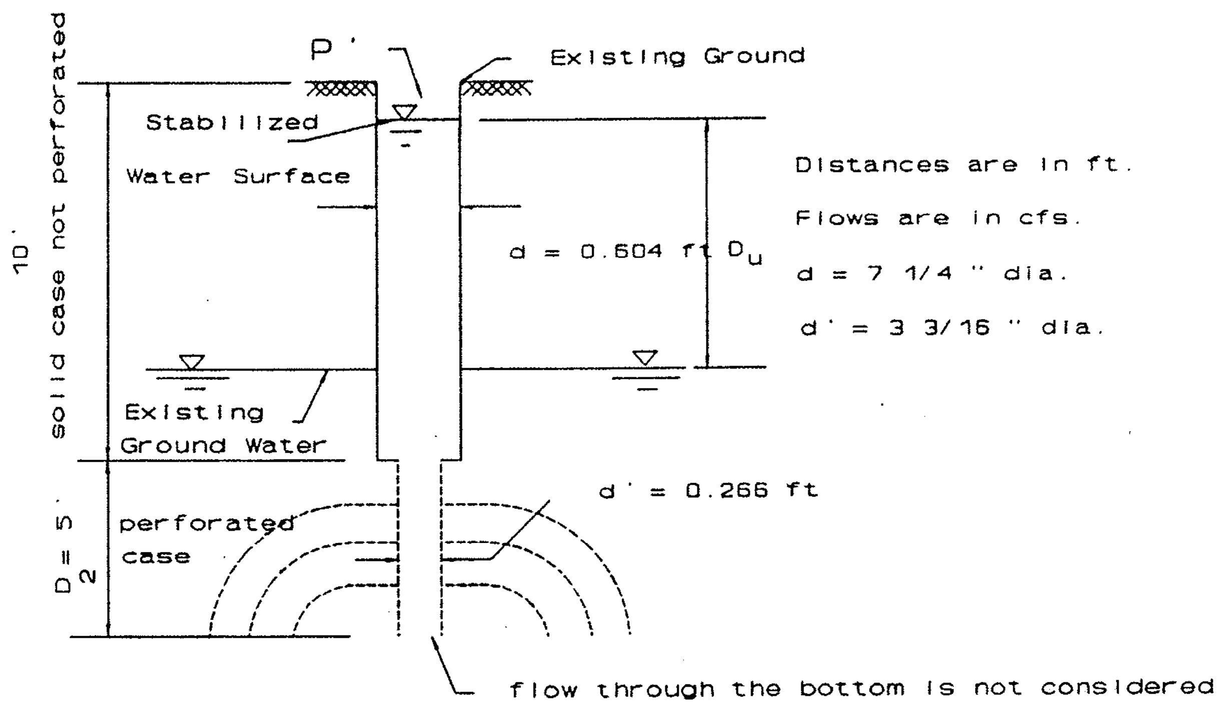


Figure 3
(nts)

The 10 feet to 15 feet deep test should be performed if the rate of pumping during the first 10 feet test is less than 0.0134 cfs (6 gpm) or as directed by the Drainage Engineer (see figure 3). The formula for the determination of the hydraulic conductivity for the stratum 10 feet to 15 feet deep is:

$$K_{15} = P' / S\, h$$

P' – Pump discharge, in cfs

S – Area of the vertical surface perpendicular to the direction of the flow, in sq ft

h – Average hydraulic head affecting the movement of water through S, in feet

$$S = D_2\, \pi\, d'$$

$h = D_u$

$$K_{15} = \frac{P}{\pi\, d'\, D_2\, D_u} \qquad (5)$$

Formula (5) determines the hydraulic conductivity for the stratum of 10 feet to 15 feet deep. The procedure implies to:

a. Case the open hole 10 feet deep with a solid case (not perforated).
b. Auger a hole from 10 feet deep to 15 feet deep for a 3 3/16" dia. perforated case.
c. Lower the perforated 3 3/16" dia. perforated case.
d. Pump test water into the hole with the same method as the one used in the determination of K_{10}.

3) 15 FEET TO 20 FEET DEEP TEST

The French drain may be reasonable constructed to a depth of 20 feet deep. This test should be perforated if the combined flow P + P' (see figures 1 and 3) is still less than 0.0134 cfs (6 gpm), or as directed by the drainage engineer in charge of the project.

The formula for the determination of the hydraulic conductivity for the stratum of 15 feet to 20 feet deep is:

$K_{20} = P'' / S\ h$

P" = Pump discharge, in cfs
S = Area of the vertical surface perpendicular to the direction of the flow, in sq. ft
h = Average hydraulic head affecting the movement of water through S, in feet
$S = D_3\, \pi\, d'$
$h = D_u$

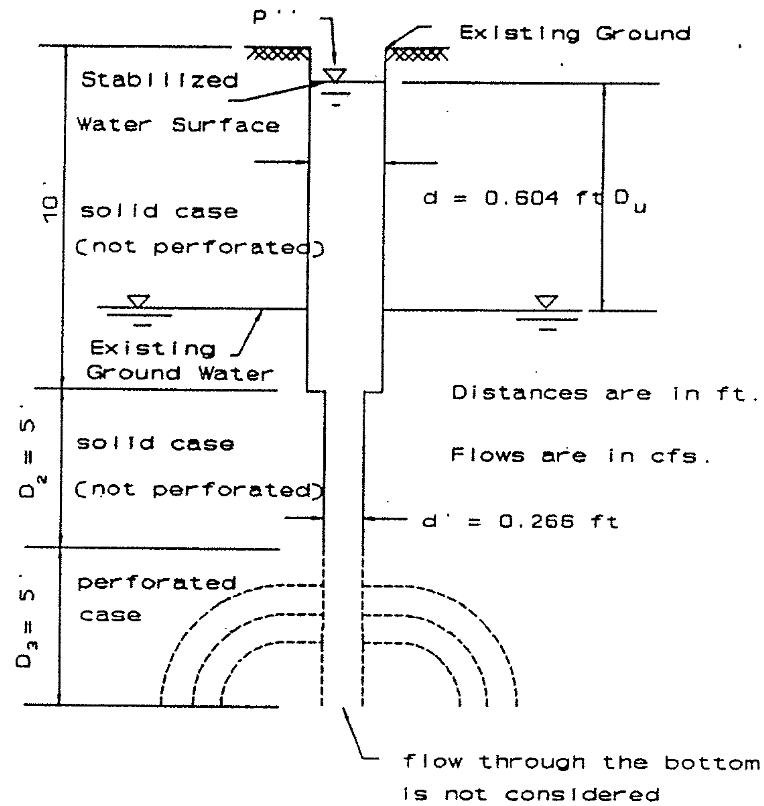


Figure 4
(nts)

$$K_{20} = \frac{P''}{\pi\, d'\, D_3\, D_u} \qquad (6)$$

Formula (6) determines the hydraulic conductivity for the stratum from 15 feet to 20 feet deep. This procedure implies to:

a. Remove the perforated case from 10 feet to 15 feet deep
b. Auger a hole from 15 feet to 20 feet deep for a 3 3/16" perforated case
c. Lower a 10 feet long, 3 3/16" case, with the lower 5 feet perforated
d. Pump the test water into the hole following the same procedure as the one used in the determination of K_{10}

Up to 3 values of hydraulic conductivity may have been determined at the same test hole. These test holes were located as close as possible to the potential sites where the French drains will be located. Hydraulic head was imposed upon these test holes and the exfiltration quality, or hydraulic conductivity, determined. It should be noted that the hydraulic conductivity, as it is defined b its dimensions, implies an inherent condition of the soil and independent of the observed parameters that defined its value. This is not true, because the flow is not linearly proportional to the available head, as explained before. So, the design ground water surface elevation should be provided as close as possible to the stabilized water elevation in the test hole. The design water surface elevation may be different elevation that the ground water elevation during the test, but, that difference is also small. French drain cannot be constructed with a bottom elevation different to the one obtained during the test. This implies that there is no freedom to increase the area arbitrarily, even though the area is none of the dimensions of the hydraulic conductivity.

Having in mind the above restrictions upon the same dimensions that define the hydraulic conductivity, the task of developing formulae for exfiltration rates that are used in French drain design is now explained.

4 - EXFILTRATION RATE

The expansion of formula (2) gives:

$$E_T = E_{10} + E_{15} + E_{20}$$

E_T – Total exfiltration rate, in cfs/foot of trench

E_{10} – Exfiltration rate for 0’ to 10’ deep stratum, in cfs/foot of trench

E_{15} – Exfiltration rate for 10’ to 15’ deep stratum, in cfs/foot of trench

E_{20} – Exfiltration rate for 15’ to 20’ deep stratum, in cfs/foot of trench

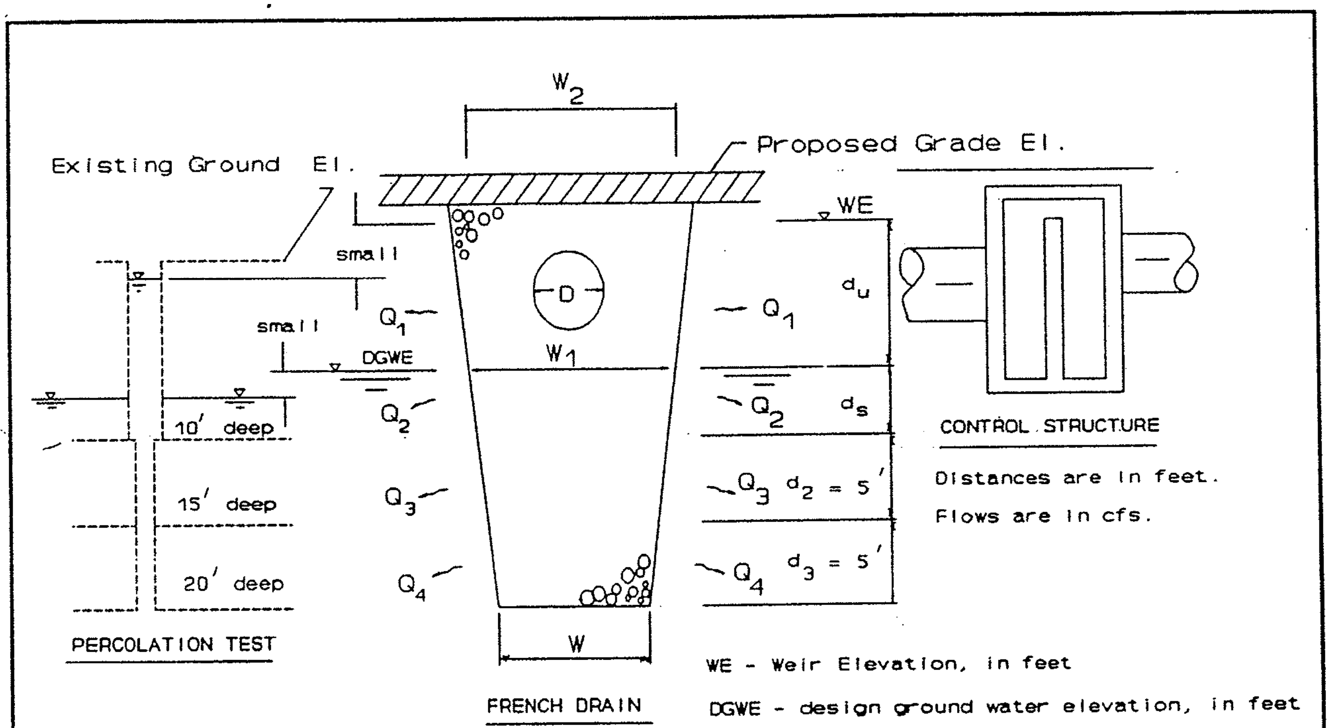


Figure 5
(nts)

WE – Weir elevation, in feet

DWSE – Design ground water surface elevation, in feet

Q_1, Q_2, Q_3, and Q_4 – Exfiltration, in cfs/foot of trench

$E_{10} = 2\ Q_1 + 2\ (Q_1 + Q_2)$
$E_{10} = 2\ (K_{10}\ d_u\ d_u/2 + K_{10}\ d_s\ d_u)$
$E_{10} = 2\ K_{10}\ d_u\ (d_u/2 + d_s)$

$E_{15} = 2\ Q_3$ & $E_{20} = 2\ Q_4$

$E_{15} = 2\ K_{15}\ d_2\ d_u$ & $E_{20} = 2\ K_{20}\ d_3\ d_u$

EXFILTRATION RATE FORMULA

The total exfiltration rate, in cfs/foot of trench, is determined:

$E_T = 2\ K_{10}\ d_u\ (d_u/2 + d_s)$, @ 10 ft deep (7)

$E_T = 2\ K_{10}\ d_u\ (d_u/2 + d_s) + 2\ K_{15}\ d_2\ d_u$, @ 15 ft deep (8)

$E_T = 2\ K_{10}\ d_u\ (d_u/2 + d_s) + 2\ K_{15}\ d_2\ d_u + 2\ K_{20}\ d_3\ d_u$, @ 20 ft deep (9)

Observe that the French drain bottom elevation in each of the three (3) cases above coincides with that of the test. This means that the application of the above formulas implies that the French drain should be constructed with its bottom elevation coinciding with that of the selected test. The DGWSE should be obtained from the Dade County Design Standard W.C. 2.2, “Average October Ground Water Level”.

Part of the runoff that enters is stored, and part is exfiltrated into the ground. The runoff-storage-exfiltration relationship is the basis of the French drain design.

5 – FRENCH DRAIN DESIGN

As soon as the runoff enters into a French drain, exfiltration begins. The exfiltration flow reaches a maximum when the water reaches the maximum elevation. French drains are used:

a. To provide water quality retention requirements enforced by Dade County Water Control Section. After the minimum retention is provided, an overflow is allowed through the control structure into adjacent open masses of water.

b. As a final drainage structure in which no overflow exists. The system using French drains with no overflows is known as closed system and/or self contained system.

Design of a French Drain to Satisfy Water Quality

The following procedure describes the determination of a length of French drain that satisfies the minimum retention required by Dade County.

Dade County's water quality policy requires the retention of 1" of the runoff from the entire drainage area. This does not mean to retain a runoff volume equal to 1" times the entire drainage area. The volume to retain is more than that.

Dade County specifies in its "An Updated Policy for the Design of Storm Runoff Drainage Structures" that for any surface there is a relation between the frequency, the runoff coefficient from the rational formula, and the time that it takes to generate 1" of runoff. This relation is expresses by the formula:

$$* \; t_1 = \frac{2940 \, F^{-0.11}}{308.5 \, C - 60.5 \, (0.5895 + F^{-0.67})} \qquad (10)$$

* – Formula developed by the author after substitutions and eliminations.

t_1 – Time to generate 1” of runoff, in minutes
C – Runoff coefficient (from the rational formula)
F – Frequency, in years.

Dade County assumes that the generated 1” encompasses most of the polluted runoff. But, observe that the last drop of water for the 1” of runoff has been generated at the furthest point from where the French drain inlet/s is/are located. Now, that last drop of the polluted generated by the runoff begins its movement toward the point of collection. In its path, it mixes and contaminates the non-polluted runoff that is being generated after t_1. The time that it takes to that drop of water to reach the point of collection is called time of concentration, t_c, in minutes. The runoff that is collected after that last drop of water reaches the point of collection is assumed to be non-polluted. Then, what is the duration of a storm whose total runoff is polluted and contaminated? Definitively, the duration is equal to $t_1 + t_c$. It is assumed that the French drain will be designed to retain all the runoff from a storm whose duration is:

$$T_T = t_1 + t_c \tag{11}$$

T_T – Duration of a storm whose runoff is polluted and contaminated, in minutes.

Dade County has its own formula for relating intensity, duration, and frequency:

$$i = \frac{308.5}{48.6\, F^{-0.11} + T_T\,(0.5895 + F^{-0.67})} \tag{12}$$

i – Storm intensity, inches/hour
F – Frequency, in years
T_T – Total storm duration, in minutes

The application of the rational formula gives the runoff flow:

$$Q = C\ i\ A_T \tag{13}$$

Q – Runoff flow, in cfs
C – Runoff coefficient
A_T – Tributary area, in acres

The total runoff volume to be retained is:

$$V = 60\ Q\ T_T \tag{14}$$

V – Volume to be retaining, in cubic feet

The storage, S, is formed by holes in the crushed stone and the empty pipe, it is expressed by the formula:

$$S = \left(\frac{W_1 + W_2}{2} d_u - \frac{\pi}{4} D^2\right) 0.5 + \frac{\pi}{4} D^2 \tag{15}$$

D – Diameter of the perforated pipe, in feet
S – Storage in cubic foot/foot of French drain

The computed length of French drain is then obtained by equating the volume:

Volume of runoff = French drain capacity

$$V = S\ L + E_T\ L\ T_T\ 60$$
$$V = L\ (S + 60\ E_T\ T_T)$$

$$L = \frac{V}{S + 60\ E_T\ T_T} \tag{16}$$

L – Computed length of French drain, in feet

The computed length, L, may not be enough for storms durations less than T_T. Any storm duration less than T_T is assumed to be highly polluted and no overflow should be permitted. But, how is it possible that a computed length, L, may retain all the runoff from a storm of duration T_T, and cannot retain all the runoff from storms having shorter duration? The answer to that question can be found by the help of a graphic representation.

To create figure 6:

a. Evaluate formula (12), (13), and (14) for different storms duration, that are less than, and greater than T_T.

b. Plot runoff volume envelope curve, locate point a, with coordinates T_T and V. The reader should be aware that the Runoff Volume Envelope curve does not represent the accumulated runoff vs. time at any particular storm. It is a curve that passes through the total accumulated runoff from storms of different durations. See figure 6.

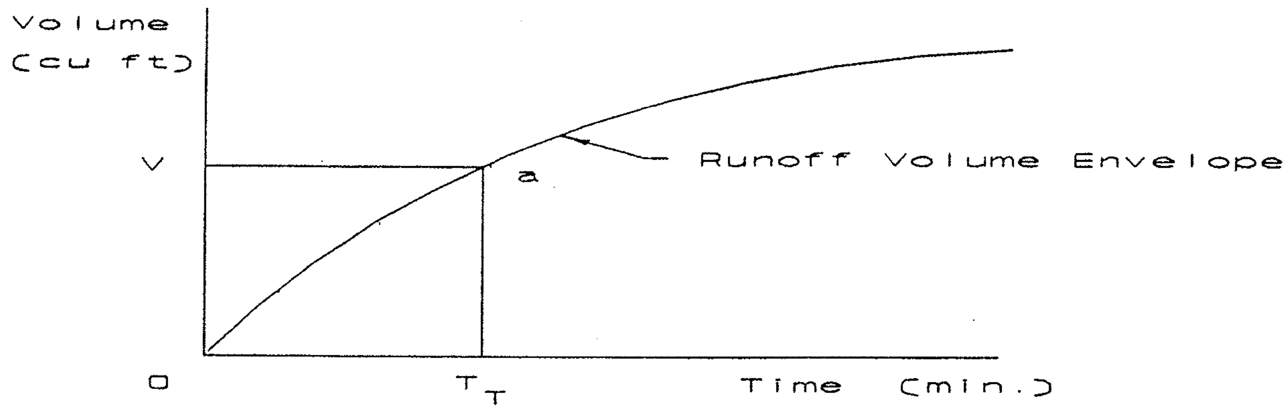


Figure 6
(nts)

c. Plot the volume disposed by the French drain line, see figures 7 and 8. The relative positions of the two mentioned curves could fall in either these cases, called CASE A or CASE B.

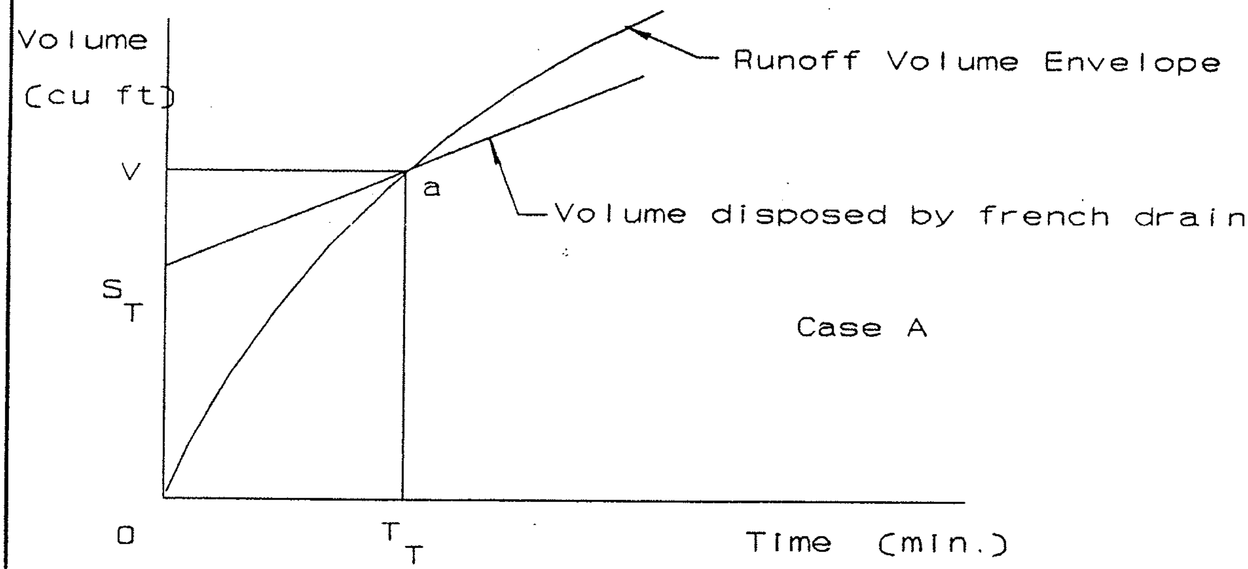


Figure 7
(nts)

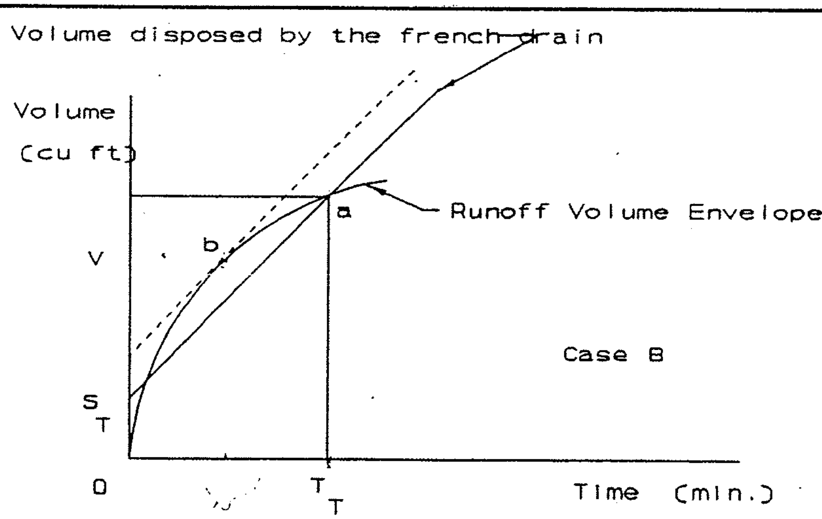


Figure 8
(nts)

S_T = S L
S_T = Total Storage, in cf

CASE A:

All runoff from storms equal or less than T_T is retained by the French drain. There is overflow for storms with durations greater than T_T that does not have to be retained because it is not polluted. The length of French drain as computed by formula (16) is satisfactory.

CASE B:

Runoff from storms with durations less that T_T generate an overflow that has to be retained because it is polluted. The length of French drain as computed by formula (16) is not satisfactory. The length of the French drain has to be increased until the volume represented by the curve and the dash line tangent at point b is achieved.

Any French drain whose Volume Disposed curve is tangent to the Runoff Envelope curve produces no overflow for all storm durations. A formula for the determination of a length of French drain that produces no overflow is deduced in Appendix A. Then, which is the overflow used to compute the weir at a control structure? In CASE A, the overflow is not polluted, in CASE B there is no overflow.

The control structure is not sized using Dade County rainfall curves. FDOT rainfall curves give higher values and are used for hydraulic computations. See example problem below.
To compute the maximum overflow, an explanation is provided in the following pages. But, will the French drain be constructed without using any safety factor?

A safety factor of 2 is used to determine the effective length L_E:

$$L_E = 2\ L \qquad (17)$$

L_E – Effective length of French drain, in feet

The effective length is the physical length of the French drain. This length should never be used for computations.

A computer program has been developed by this Drainage Section to compute the effective length of French drain using the procedure explained in this paper, from the determination of the K's up to the computation of L_E. Storms with duration from 10 to 80 minutes are tabulated. The storm of T_T is highlighted. But, where is the hydraulic grade line located in an operating French drain? It can be assumed that the position of the hydraulic grade line is flat and coincides with the weir elevation or water elevation over the weir in a French drain whose effective length have been computed following the above procedure. But, the position of the hydraulic grade changes when:

1. The length of the French drain constructed is less than L_E. There is overflow over the weir of unknown value. If the French drain is short and overflow small it can still be assumed that the hydraulic grade line is flat and at an elevation equal to that of the water over the weir.

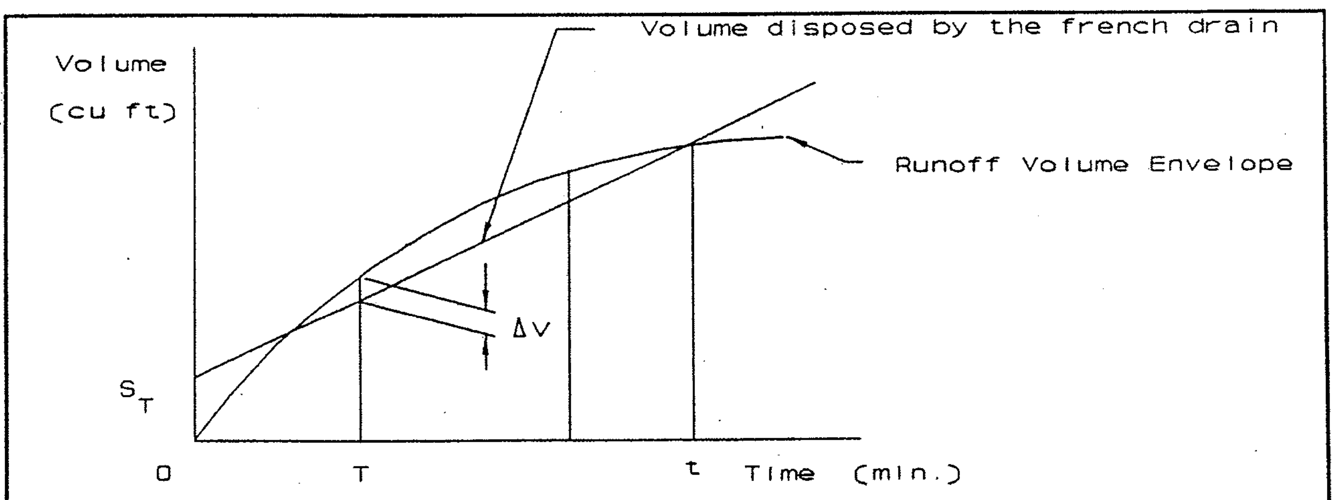


Figure 9
(nts)

A practical procedure to determine the overflow for a storm of duration T is indicated in figure 9, and requires to:

a. Select T and determine ΔV
b. Compute the average overflow for storm of duration T

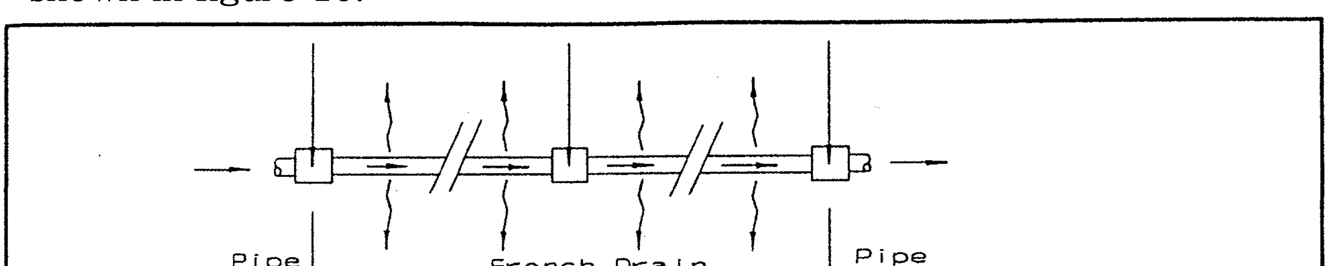
$$OF_{avg} = \Delta V / T$$

OF_{avg} – Average overflow of the storm with duration T, in cu ft/min
ΔV – Total overflow of the storm with duration T, in cu ft
T – Duration of the storm, in minutes.

The procedure explained in a), and b) must be repeated for all the storms that have overflows, whose T comply with the relation $T \leq t$, and the maximum determined.

2. The French drain is only a part of a more complex system. It has its own drainage area but the weir is far and controlling other French drains. There may be a significant difference between the weir elevation and the hydraulic gradient over the weir. The French drain is conveying flow that entered through its end from other parts of the same system. The determination of the hydraulic grade line over this French drain can no longer be determined using the simplistic approach described above. When that is the case, the French drain behavior cannot be isolated from the system and the entire net has to be solved together. Any of the existing commercial water management packages can be used to solve the hydrology and the hydraulics involved. This District has 2: BRN and ADICPR.

A long French drain that is moving a significant flow is modeled as shown in figure 10.



Any model assumes that any drainage system can be transferred into a net of nodes and reaches. The nodes can be of two types:

1. Time type, in this node the stages are related to times

2. Area type, in this node the stages are related to areas

A reach is a path that moves flow among the nodes. A reach may be a pipe, a weir, a channel, a gate, etc.

To model a French drain as the one shown in figure 10, it is necessary to:

1. Transform each manhole into a node type area. The empty volume because of voids in the stone in the adjacent sections of French drain is transformed into a relation of stages vs. areas in such a way that at any elevation the volume under it is equal to the volume of voids under the same elevation. Do not include the empty volume of the French drain pipe because the program computes the emptiness of any reach.

2. Assume the connections between French drain manholes as common pipes.

3. Assume the ground water table as a node time type. Its elevation is constant and equal to the design ground water surface elevation.

4. Assume reaches between all the nodes area type with the single node time type. Each one of these reaches will have a rating curve. A rating curve is a relation discharge vs. elevation. It is computed based on the adjacent sections of French drain and the hydraulic conductivity. To continue with an explanation indicating the operative procedure of the simultaneous routing is beyond the scope of this presentation and will not be discussed any further. This brief introduction to the water management packages has been made with the intention of alerting the reader that there are sophisticated software packages in the market capable of solving the most complicated problems. Presently, complex projects, as the Golden Glades HOV Lanes, the Palmetto Expressway, from NW 122nd St. to NW 154th St. have their drainage designed with the help of the mentioned Water Management Packages.

DESIGN OF A FRENCH DRAIN AS A CLOSED SYSTEM
(Self Contained System)

The absence of any adjacent open water mass imposes additional safety measures. A French drain as a final drainage structure of any closed system must be:

a. Designed following same procedures as the one followed to satisfy the water quality. Ten year frequency and no overflow condition is required. Use FDOT rainfall curves.
b. Constructed to the length not less than the effective length L_E.

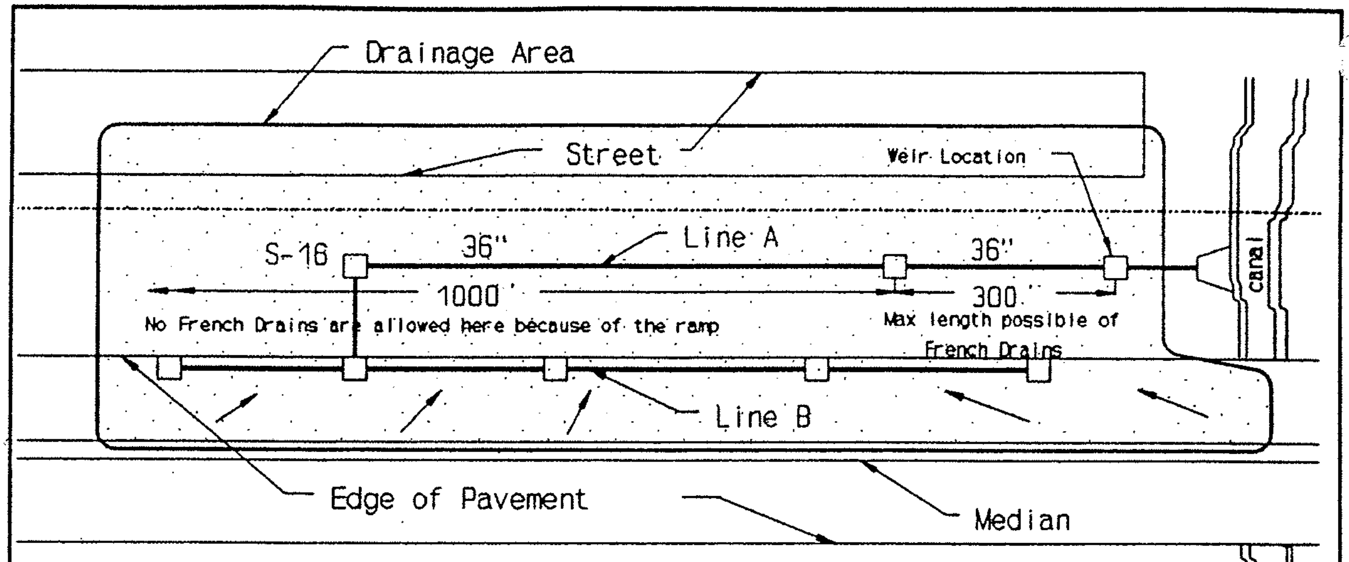
The theory has been covered in the previous pages. But, how does it apply to a simple problem? A simple French drain design, similar to the one found in any FDOT drainage system is now explained.

6 – EXAMPLE PROBLEM

Drainage system 9 (figure 11) of a FDOT project in Dade County will discharge into an adjacent canal using a tentative 36" diameter pipe. Dade County Water Control Section requires the retention of the first 1" of runoff from the entire drainage area before any discharge is made into any open mass of water.
The Normal High Water elevation in the canal (NHW) is the controlling outlet elevation. It is assumed equal to the design ground water elevation and is readily obtained from Dade County Standard W.C. 2.2, "Average October Ground Water level", it is 2.7.

The last section of the 36" pipe crosses through an area of R.O.W. at elevation +8.0 that is grass covered. This is the only section of R.O.W. where a French drain might be constructed. The designer anticipates that the maximum retention required cannot be achieved. Still the designer has decided to continue with the French drain design for this area. The following questions come into mind:

1- What percentage of the required retention will I provide?
2- Will it be satisfactory to Dade County Water Control Section?
3- What will be the maximum overflow?
4- What will be the position of the hydraulic gradient over the entire drainage system?



SYSTEM 9 (N.T.S.)
FIGURE 11

Figure 11 shows 2 mainlines of pipes: Line A and Line B. Line A is lower and located adjacent to the R.O.W, line. Line B is higher and located under the expressway shoulder.

a) Hydraulic Conductivity Determination.
   Test performed from 0 feet to 10 feet, see figure 12:

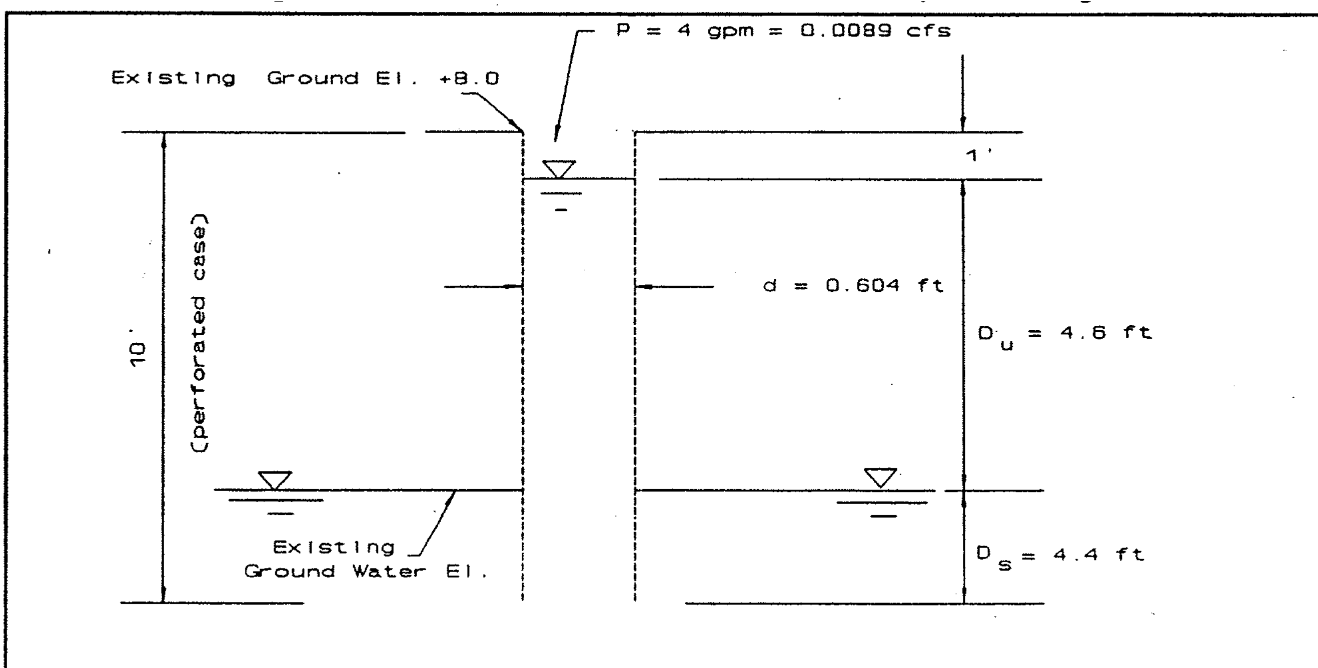


Figure 12
(nts)

$$K_{10} = \frac{P}{\pi\, d\, D_u \left(\frac{D_u}{2} + D_s\right)}$$

$K_{10} = 0.0089 / [\Pi \times 0.604 \times 4.6 \times (4.6/2 + 4.4)]$

$K_{10} = 0.000152$ cfs / (sq ft x foot of Head)

Test Performed from 10 feet to 15 feet, see figure 13:

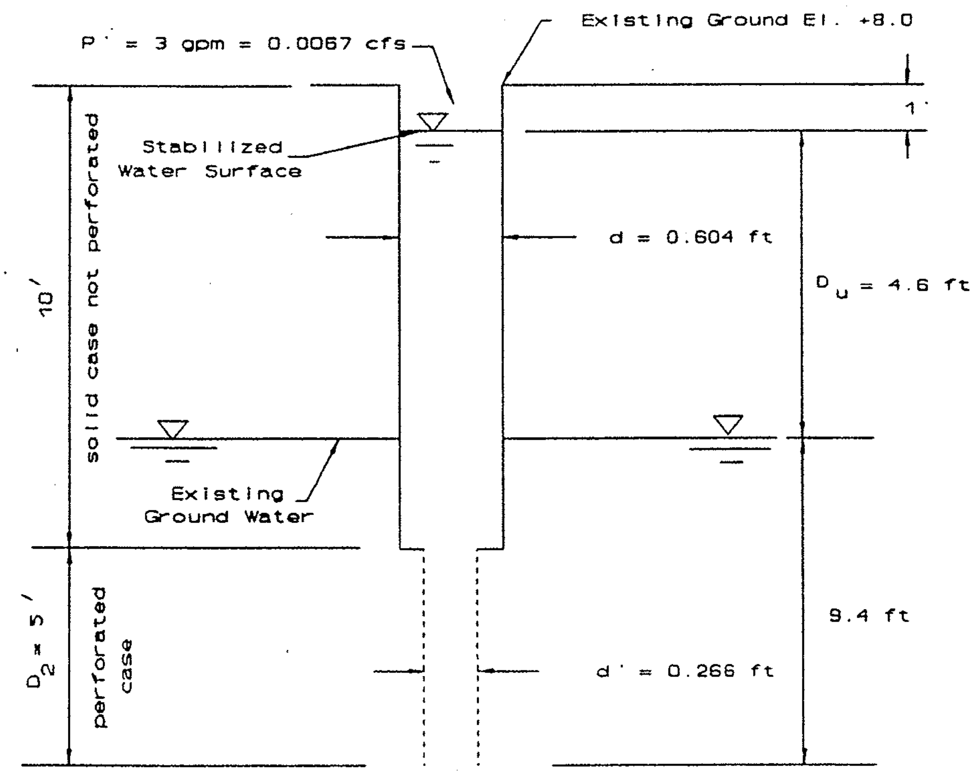


Figure 13
(nts)

Applying formula (5):

$$K_{15} = \frac{P}{\pi\, D_2\, d'\, D_u}$$

$$K_{15} = 0.0067 / [5 \times 3.14 \times 0.266 \times 4.6]$$

K_{15} = 0.000349 cfs / (sq ft per foot of head)

b) Exfiltration rate determination.

The designer has to assume the elevation of the weir of the control structure. It is expected overflow over the weir because of the impossibility of constructing the French drain to the full effective length L_E as defined by formula (17). If the overflow is, high the hydraulic gradient might be over structure S-16, which is the remotest inlet of the lower line. The area where line A is located will be filled to approximately elevation +9.0. The designer finally playing safe decides to assume a weir top of wall at elevation +7.5, see figure 14.

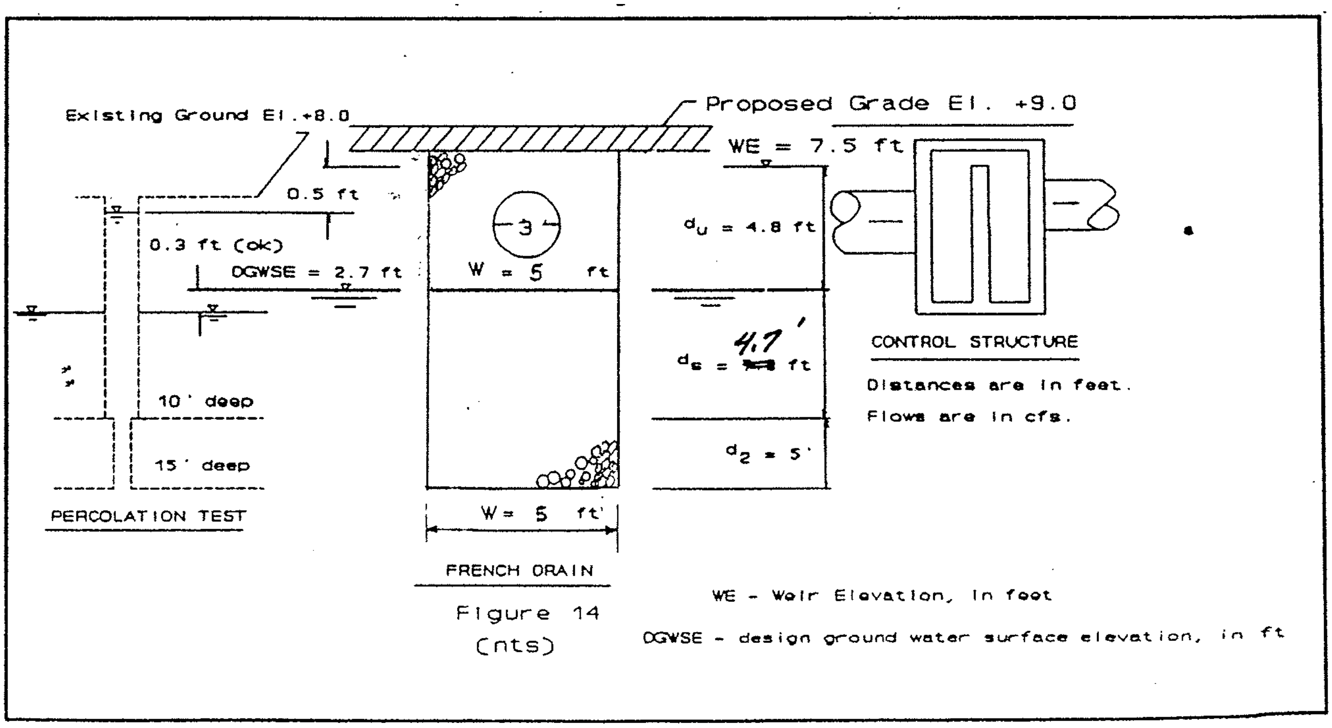


Figure 14
(nts)

Apply formula (8):

$$E_T = 2\ K_{10}\ d_u\ (\frac{d_u}{2} + d_s) + 2\ K_{15}\ d_2\ d_u$$

In this case E_T means $E_{10} + E_{15}$

$E_T = 2 \text{ x } 0.000152 \text{ x } 4.8\ (4.8 / 2 + 4.7) + 2\text{x}0.000349 \text{ x } 5 \text{ x } 4.8$

$E_T = 0.0271$ cfs / foot of French drain

c) Storage determination:

Apply formula (15):

$$S = [(W)\ d_u - \frac{\pi}{4} D^2]\ 0.5 + \frac{\pi}{4} D^2$$

$S = [(5)\ 4.8 - 3.14 / 4 \text{ x } 3^3]\ 0.5 + 3.14 / 4 \text{ x } 3^2$

S = 15.53 cu ft / foot of French drain

d) Design frequency, contributing area, and roughness coefficient criteria.

Design Frequency = F = 10 years
Total Drainage Area = A_T = 4 acres
Paved Area = 3.0 Ac., C_1 = 0.9
Grass Area = 1.0 Ac., C_2 = 0.3

$$C = \frac{3.0 \text{ x } 0.9 + 1.0 \text{ x } 0.3}{4} = 0.75$$

e) Determine the time that it takes to generate 1” of runoff, $t_{1”}$.

Apply formula 10:

$$t_1” = \frac{2940\ F^{-0.11}}{308.5\ C - 60.5\ (0.5895 + F^{-0.67})}$$

$t_{1”} = 2940 \times 10^{-0.11} / 308.5 \times 0.75 - 60.5 \times (0.5895 + 10^{-0.67})$

$t_{1”} = 12.5$ min

f) Determine the maximum polluted volume.

Observe that the C value used above is the average. The $t_{1”}$ time to generate 1” of runoff is over the average runoff. The time of concentration has been computed separately and is not included in this paper.

$t_C = 11$ minutes

$T_T = t_{1”} + t_C = 12.5 + 11 = 23.5$ minutes

All the runoff from a storm lasting 23.5 minutes or less is assumed to be polluted or contaminated.

Apply formula (12):

$$i = \frac{308.5}{48.6\ F^{-0.11} + T_T\ (0.5895 + F^{-0.67})}$$

$i = 308.5 / [(48.6 \times 10^{-0.11}) + 23.5\ (0.5895 + 10^{-0.67})]$

$i = 5.45$ in/hour

Applying formula (13):

$Q = C\ i\ A_T = 0.75 \times 5.45 \times 4 = 16.35$ cfs

Applying formula (14):

$V = 60\ Q\ T_T = 60 \times 16.35 \times 23.5 = 23054$ cf

g) Determine the French drain length to retain the maximum polluted volume.

Applying formula (16):

$$L = \frac{V}{S + 60\ E_T\ T_T}$$

$L = 23054 / (15.53 + 60 \times 0.0271 \times 23.5)$
$L = 429$ ft

The designer confirms his suspicion and will not be able to construct a french drain to retain all polluted runoff. The french drain to be built will only be 300 ft long which has no safety factor. L_E is not computed in this case.

h) Determine the percentage of required retention.
Figure (15) shows the volumes disposed by french drains 429 ft and 300 ft long. Observe also that a french drain 429 ft long is not capable of handling storms of duration less than T_T and that are highly polluted. This is "Case B", see figure 8. The length of french drain should be increased to avoid any overflow. The length of a french drain without any overflow is 427 ft, see Appendix A. The percentage of the required retention that is provided by the french drain 300 ft long is:

$$\text{Perc } \% = \frac{V'}{V} \times 100 = \frac{L'\ (S + 60\ E_T\ T_T)}{L\ (S + 60\ E_T\ T_T)} \times 100$$

$$\text{Perc \%} = \frac{L'}{L} \times 100 = \frac{300}{429} \times 100 = 70\%$$

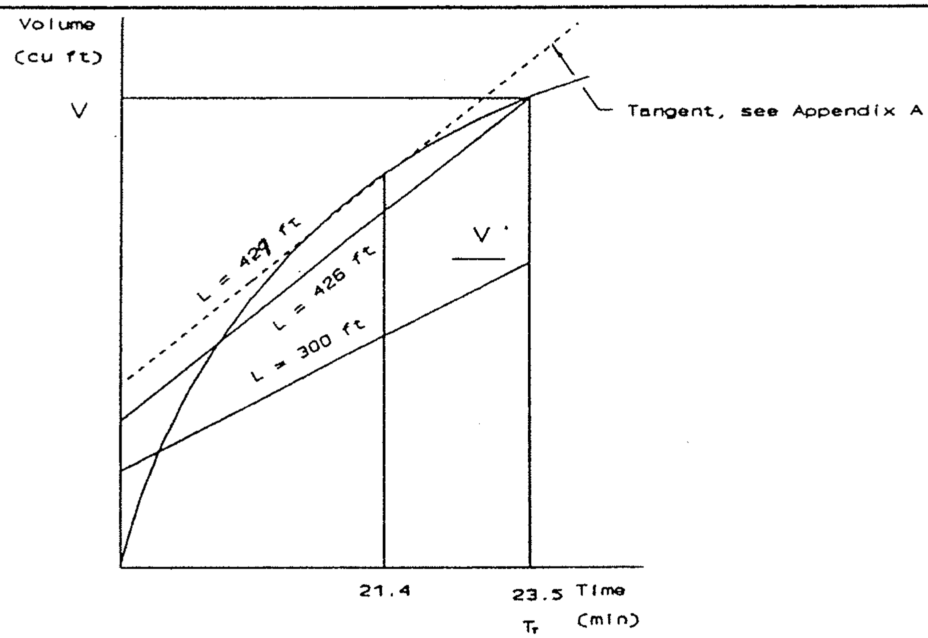


Figure 15
(nts)

The provided retention is only 70% of the required retention.

i) Determine the maximum overflow.

The computations for water quality finished in h).
Hydraulic Computation, that is, location of the hydraulic grade line, maximum overflow, and weir dimensions use the relation Rainfall-Duration-Frequency Curves from FDOT Drainage Manual. Refer to Chapter 5.

Zone 10, frequency = 10 years

$$i = 10.84265 - 0.18976 (\ln t) - 0.69575 (\ln t)^2 + 0.07495 (\ln t)^3$$

[illegible] (ln t) − 0.69575 (ln t)^2 + 0.07495 (ln t)^3

TIME (min)	INTENSITY (in/hr)	FLOW Q (cfs)	TIME (sec)	INFLOW VOLUME (CF)	STORAGE CAPACITY (CF)	ADJUSTED INFLOW (CF)	EXFILTRATION RATE IN THE SYSTEM (CF)	SYSTEM CAPACITY OUTFLOW VOLUME (CF)	SYSTEM CAPACITY OUTFLOW VOLUME (CFS)	DOT VS DERM 1"- Overflow Vol. (CF)	Overflow Vol. (CFS)	TIME (MIN)
0	0.00	0.00	0.00	0.00	4,660.29	-4660.29	0.00	4,660.29	0.00	(4,660.29)	0.00	0
8	8.11	24.34	480.00	11,683.45	4,660.29	7,023.16	3,902.68	8,562.97	17.84	3,120.48	6.50	8
10	7.63	22.90	600.00	13,737.44	4,660.29	9,077.15	4,878.35	9,538.64	15.90	4,198.80	7.00	10
12	7.23	21.68	720.00	15,606.08	4,660.29	10,945.79	5,854.02	10,514.31	14.60	5,091.76	7.07	12
14	6.87	20.62	840.00	17,322.00	4,660.29	12,661.71	6,829.69	11,489.98	13.68	5,832.02	6.94	14
15	6.71	20.14	900.00	18,130.33	4,660.29	13,470.05	7,317.53	11,977.82	13.31	6,152.52	6.84	15
16	6.57	19.70	960.00	18,908.85	4,660.29	14,248.56	7,805.37	12,465.65	12.99	6,443.20	6.71	16
18	6.29	18.87	1,080.00	20,384.50	4,660.29	15,724.22	8,781.04	13,441.32	12.45	6,943.18	6.43	18
20	6.05	18.14	1,200.00	21,762.93	4,660.29	17,102.64	9,756.71	14,416.99	12.01	7,345.94	6.12	20
22	5.82	17.47	1,320.00	23,055.34	4,660.29	18,395.05	10,732.38	15,392.66	11.66	7,662.68	5.81	22
24	5.62	16.85	1,440.00	24,270.91	4,660.29	19,610.62	11,708.05	16,368.34	11.37	7,902.58	5.49	24
26	5.43	16.29	1,560.00	25,417.28	4,660.29	20,756.99	12,683.72	17,344.01	11.12	8,073.28	5.18	26
28	5.26	15.77	1,680.00	26,500.91	4,660.29	21,840.62	13,659.39	18,319.68	10.90	8,181.23	4.87	28
30	5.10	15.29	1,800.00	27,527.32	4,660.29	22,867.03	14,635.06	19,295.35	10.72	8,231.97	4.57	30
35	4.74	14.23	2,100.00	29,872.81	4,660.29	25,212.52	17,074.24	21,734.52	10.35	8,138.28	3.88	35
40	4.44	13.31	2,400.00	31,948.66	4,660.29	27,288.37	19,513.41	24,173.70	10.07	7,774.96	3.24	40
45	4.17	12.52	2,700.00	33,799.14	4,660.29	29,138.86	21,952.59	26,612.88	9.86	7,186.27	2.66	45
50	3.94	11.82	3,000.00	35,458.23	4,660.29	30,797.95	24,391.77	29,052.05	9.68	6,406.18	2.14	50
55	3.73	11.20	3,300.00	36,952.69	4,660.29	32,292.40	26,830.94	31,491.23	9.54	5,461.46	1.65	55
60	3.55	10.64	3,600.00	38,304.09	4,660.29	33,643.80	29,270.12	33,930.41	9.43	4,373.68	1.21	60
65	3.38	10.14	3,900.00	39,530.15	4,660.29	34,869.86	31,709.30	36,369.58	9.33	3,160.56	0.81	65
70	3.23	9.68	4,200.00	40,645.62	4,660.29	35,985.33	34,148.47	38,808.76	9.24	1,836.86	0.44	70
75	3.09	9.26	4,500.00	41,662.97	4,660.29	37,002.68	36,587.65	41,247.94	9.17	415.04	0.09	75
80	2.96	8.87	4,800.00	42,592.84	4,660.29	37,932.55	39,026.83	43,687.11	9.10	(1,094.28)	(0.23)	80
85	2.84	8.52	5,100.00	43,444.37	4,660.29	38,784.09	41,466.00	46,126.29	9.04	(2,681.92)	(0.53)	85
90	2.73	8.19	5,400.00	44,225.56	4,660.29	39,565.27	43,905.18	48,565.47	8.99	(4,339.91)	(0.80)	90
95	2.63	7.88	5,700.00	44,943.35	4,660.29	40,283.06	46,344.36	51,004.64	8.95	(6,061.29)	(1.06)	95
120	2.21	6.64	7,200.00	47,774.26	4,660.29	43,113.97	58,540.24	63,200.53	8.78	(15,426.26)	(2.14)	120
150	1.85	5.56	9,000.00	50,022.06	4,660.29	45,361.78	73,175.30	77,835.59	8.65	(27,813.52)	(3.09)	150
180	1.59	4.77	10,800.00	51,544.79	4,660.29	46,884.50	87,810.36	92,470.65	8.56	(40,925.86)	(3.79)	180

D[illegible] STORM FREQ. CURVES - POLYNOMIAL COEFFICIENTS				
years	A	B	C	D
3	11.32916	-1.38557	-0.36672	0.05012
10	10.84265	-0.18976	-0.69575	0.07495
5	11.19083	-0.93165	-0.48526	0.05836

French Drain Length to Retain the Max. Polluted Volume = V(Tt)/S+60 Et Tt = 428.95 LF

Maximum overflow = 6.5 cfs, see figure 16.

Maximum overflow = 6.5 cfs, see figure 16.

i) Determination of the weir dimensions.

Try a weir 4’ wide

$Q = C\,L\,H^{3/2}$ Q – maximum overflow, in cfs

$H = (Q / CL)^{2/3}$ C – Weir coefficient

$H = (6.5 / 3 \times 4)^{2/3}$ L – Length of weir, in ft

$H = 0.66$ ft H – Depth of water over the weir, in ft

The depth of water over the weir at the end of the french drain is 0.66 ft.

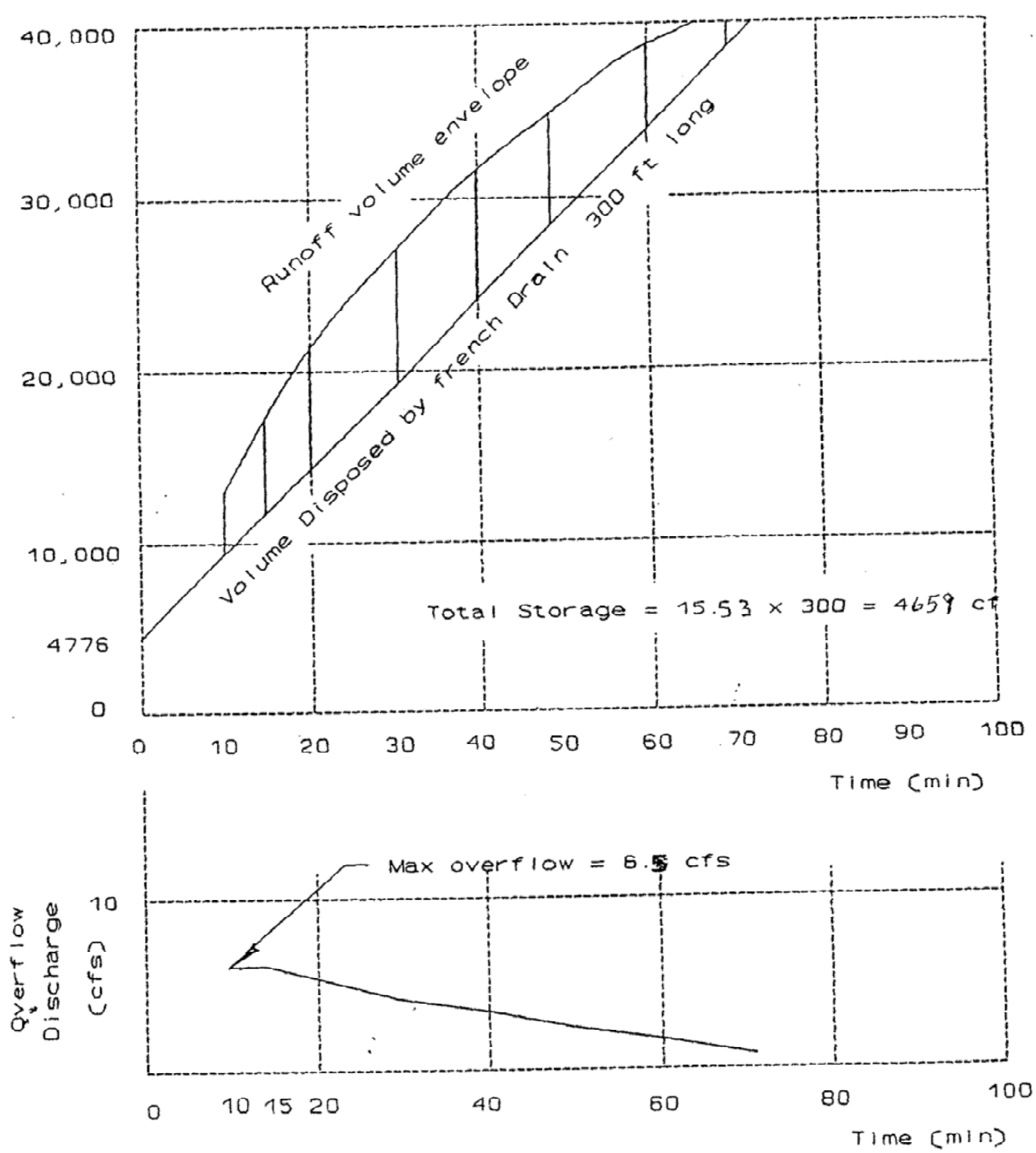


Figure 16
(nts)

j) Determination of the hydraulic gradient over the pipe.

The designer assumes that the hydraulic grade line is flat and at elevation = +7.5 + 0.66 ft = +8.16 ft over the entire length of the french drain. He considers it as a “short” french drain. A long french drain has a sloped hydraulic grade line and should be included in the storm sewer line computations.

The determination of the hydraulic grade over the pipe will be made using the computer program DRAINO. The controlling outlet elevation +8.19 is assumed to be at the entrance to the french drain.

The possibility of flooding S-16 exists. If that is case new runs will have to be made in which:

1) - Pipes diameter may be increased.
2) - Weir elevation may be lowered accomplishing smaller retention percentages. If the weir elevation changes, the procedure begins anew from step b).

The determination of an acceptable hydraulic grade line is beyond the scope of this paper and will not be covered here.

The purpose of this technical paper has been made with the intention of providing guidelines for the design of french drains to the District VI Internal Design Drainage Group and any other interested personnel from other sections.

APPENDIX A

Determination of the formula for a length of french drain without overflow.

The dash line of figure 8 indicates the volume disposed by a french drain with no overflow. It is determined by assuming a length and computing the overflow. The computations end when the overflow is zero. The procedure is time consuming unless you have a computer program. The following analysis determines a length of french drain without overflow.

$$V_1 = 60\ Q\ t$$

V_1 – Total runoff from a storm of duration t, in cf
Q – Average discharge of a storm of duration t, in cfs
t – Duration of a storm, in minutes

Substituting Q from equation (13):

$$V_1 = 60\ C\ i\ A_T\ t$$

Substituting i from equation (12):

$$V_1 = 60\ C\ \frac{308.5}{48.6\ F^{-0.11} + t\ (0.5895 + F^{-0.67})}\ A_T\ t$$

Values assumed constant are assigned to K_1, K_2, and K_3:

$K_1 = 18510\ C\ A_T$
$K_2 = 48.6\ F^{-0.11}$
$K_3 = 0.5895 + F^{-0.67}$

Substituting in V_1:

$$V_1 = \frac{K_1\, t}{K_2 + K_3\, t}$$

The representation of V_1 is shown in figure 17. The reader should realize that curve V_1 is not the accumulated curve of any storm, but a curve that passes through the total accumulated values of many storms, each one have having different durations.

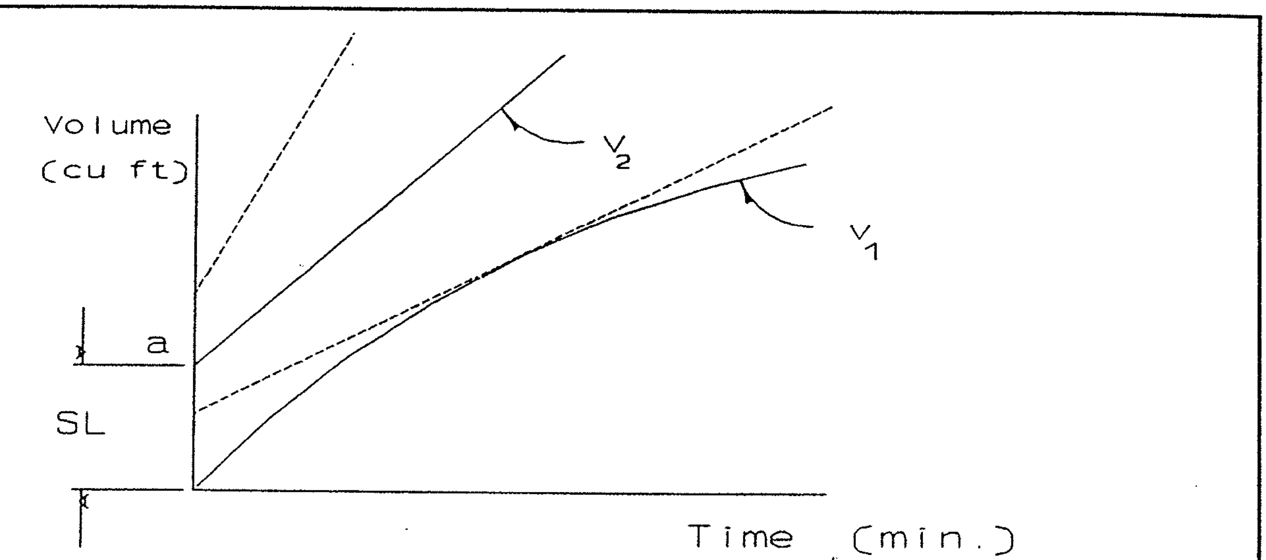


Figure 17
(nts)

dv_1 / dt will give the slope of curve v_1:

$$\frac{dv_1}{dt} = \frac{K_1\,(K_2 + K_3\, t) - K_3\,(K_1\, t)}{(K_2 + K_3\, t)^2}$$

$$\frac{dv_1}{dt} = \frac{K_1\, K_2 + K_1\, K_3\, t - K_1\, K_3\, t}{(K_2 + K_3\, t)^2}$$

$$\frac{dv_1}{dt} = \frac{K_1 K_2}{(K_2 + K_3 t)^2}$$

The volume disposed by the french drain as a function of time may be obtained from equation (16):

$$V_2 = S L + 60 E_T L t$$

v_2 – Volume disposed by the french drain, in cf.

v_2 is shown in figure 17

dv_2 / dt will give the slope of the straight line v_2:

$$\frac{dv_2}{dt} = 60 E_T L$$

Two conditions have to be satisfied to make v_2 tangent to v_1:

a) Slope condition. The straight line should be tangent to the curve.

b) Volumetric condition. The accumulated volumes determined by the straight line and the curve should be equal at the point of tangency.

Slope condition:

$$\frac{dv_1}{dt} = \frac{dv_2}{dt}$$

$$\frac{K_1 K_2}{(K_2 + K_3 t)^2} = 60 E_T L$$

Volumetric condition:

$$v_2 = v_1$$

$$S\,L + 60\,E_T\,L\,t = \frac{K_1\,t}{K_2 + K_3\,t} \qquad (19)$$

(18) and (19) form a determined system: 2 equations and 2 unknowns, L and t.

Solving for L in (18):

$$L = \frac{K_1\,K_2}{60\,E_T\,(K_2 + K_3\,t)^2} \qquad (20)$$

Arranging (19) and substituting L:

$$L\,(S + 60\,E_T\,t) = \frac{K_1\,t}{K_2 + K_3\,t}$$

$$\frac{K_1\,K_2}{60\,E_T\,(K_2 + K_3\,t)^2}\,(S + 60\,E_T\,t) = \frac{K_1\,t}{K_2 + K_3\,t}$$

Eliminating common factors:

$$\frac{K_2\,(S + 60\,E_T\,t)}{60\,E_T\,(K_2 + K_3\,t)} = t$$

Multiplying:

$$K_2\,(S + 60\,E_T\,t) = 60\,E_T\,(K_2 + K_3\,t)\,t$$

$$K_2\,S + 60\,E_T\,K_2\,t = 60\,E_T\,K_2\,t + 60\,E_T\,K_3\,t^2$$

Eliminating common factors:

$$K_2\,S = 60\,E_T\,K_3\,t^2$$

Solving for t:

$$t = \left(\frac{K_2\,S}{60\,E_T\,K_3}\right)^{1/2} \qquad (21)$$

Substituting t of (21) in (20):

$$L = \frac{K_1\,K_2}{60\,E_T\left[K_2 + K_3\left(\frac{K_2\,S}{60\,E_T\,K_3}\right)^{1/2}\right]^2} \qquad (22)$$

Formula (22) determines the length of french drain that does not allow any outflow. Formula (21) determines the duration of the storm most critical.
In the example problem the length of the french drain that did not allow any overflow was not determined. Let us determined that length applying formula (22).

$K_1 = 18510\ C\ A_T = 18510 \times 0.75 \times 4 = 55530$
$K_2 = 48.6\ F^{-0.11} = 48.6 \times 10^{-0.11} = 37.73$
$K_3 = 0.5895 + F^{-0.67} = 0.5895 + 10^{-0.67} = 0.803$
$E_T = 0.0271$ cfs / foot of french drain
S = 15.53 cf / foot of french drain

$$L = \frac{55530 \times 37.73}{60 \times 0.0271\left[37073 + 0.803\left(\frac{37.73 \times 15.92}{60 \times 0.0271 \times 0.803}\right)^{1/2}\right]^2}$$

$$L = 430\ \text{ft}$$

The time of the critical storm can be determined applying formula (21):

$$t = \left(\frac{K_2\, S}{60\, E_T\, K_3} \right)^{1/2}$$

$$t = \left(\frac{37.73 \times 15.53}{60 \times 0.0271 \times 0.803} \right)^{1/2}$$

$$t = 21.2 \text{ minutes}$$

APPENDIX B

HYDRO-GEOTECHNICAL AIDS

COEFFICIENT OF PERMEABILITY (k) FOR SATURATED AND COMPACTED LABORATORY SOIL SPECIMENS

SOIL TYPICAL NAME	SOIL CLASSIFICATION		PERMEABILITY (ft/day)
	UNIFIED	AASHTO	
Well-graded gravels or gravel-sand mixtures with little or no fines	GW	A-3	300 to 0.3 Pervious
Poorly graded gravels or gravel-sand mixtures with little or no fines	GP	A-3	3×10^{4} to 30 Very pervious
Silty gravels, gravel-sand-silt mixtures	GM	A-2-4	3 to 3×10^{-3} Semi-pervious to pervious
Clayey gravels, gravel-sand-clay mixtures	GC	A-2-6	3×10^{-3} to 3×10^{-5} Impervious
Well-graded sands or gravelly sands with little or no fines	SW	A-3	30 to 0.3 Pervious
Poorly graded sands or gravelly sands with little or no fines	SP	A-3	300 to 3 Pervious
Silty sands, sand-silt mixtures	SM	A-2-4	3 to 3×10^{-3} Semi-pervious to pervious
Clayey sands, sand-clay mixtures	SC	A-6	3×10^{-3} to 3×10^{-5} Impervious
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slightly plasticity	ML	A-6	3 to 3×10^{-3} Semi-pervious to pervious
Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays	CL	A-7	3×10^{-3} to 3×10^{-5} Impervious
Organic silts and organic silty clays of low plasticity	OL	A-6	0.3 to 3×10^{-3} Semi-pervious to pervious
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	A-6	0.03 to 3×10^{-4} Semi-pervious to pervious
Organic clays of high plasticity, fat clays	CH	A-8	3×10^{-3} to 3×10^{-6} Impervious

NOTE: Table adapted from Drainage Manual Volume 2, FDOT 1987.

COEFFICIENT OF PERMEABILITY (k) FOR SCS HYDROLOGICAL SOILS

HYDROLOGICAL SOIL CLASSIFICATION		PERMEABILITY (ft/day)
TYPE	**CHARACTERISTICS**	
A	Soils that have high infiltration rates even when thoroughly wetted and a high rate of water transmission	60
B	Soils that have moderated infiltration rates when thoroughly wetted and a moderated rate of water transmission	48
C	Soils that have slow infiltration rates when thoroughly wetted and a slow rate of water transmission	24
D	Soils having very slow infiltration rates when thoroughly wetted and a very slow rate of water transmission	12
A/D	Soils Type A under saturated natural conditions that can be adequately drained, considering that drainage is feasible and practical.	60
B/D	Soils Type B under saturated natural conditions that can be adequately drained, considering that drainage is feasible and practical.	36
C/D	Soils Type C under saturated natural conditions that can be adequately drained, considering that drainage is feasible and practical.	12

NOTE: Table adapted from Applicant’s Handbook: Regulation of Stormwater Management Systems. SJRWMD, 2005

APPENDIX C

REASONABLE ASSURANCE REPORT

Submit a reasonable assurance statement, in a report form, to the Department for review and approval.

1) The report should provide a reasonable assurance that the buoyant stormwater discharge into a Class G-III aquifer (total dissolved solids **greater** than 10,000 mg/L) via the drainage well(s) has a minimum potential to: i) rise due to buoyancy drive into a preferential pathway in a Class G-II aquifer system [Underground Source of Drinking Water (USDW; total dissolved solids less than 10,000 mg/L)], ii) impact any surface water bodies in the vicinity of the project via groundwater discharge, and iii) cause mounding that may rise into USDW and/or manifest on land surface as spring due to pressurization of well that are located or receive stormwater flow from building roof at elevation + XXX'. The report shall be signed and sealed by a Florida Licensed Professional Geologist/Engineer with hydro-geological expertise to:

- Document the existence of sufficient confining strata above the base of well casing, to minimize the potential of injected buoyant stormwater to rise into a G-II aquifer system.

- Show the injected stormwater has minimum potential to impact any surface water bodies in the vicinity of the project that may be affected via groundwater discharge.

- Show the pressurization of the well due to its location at elevation + XXX' or receiving stormwater flow from building roof elevation + XXX', will not cause mounding that may rise into USDW and/or manifest as spring on land surface.

2) The report may be submitted at this time or its submission may be deferred until the stormwater drainage well(s) construction permit has been issued. However, it must be submitted for Department review and approval prior to the Department's authorization to use the stormwater drainage well(s).

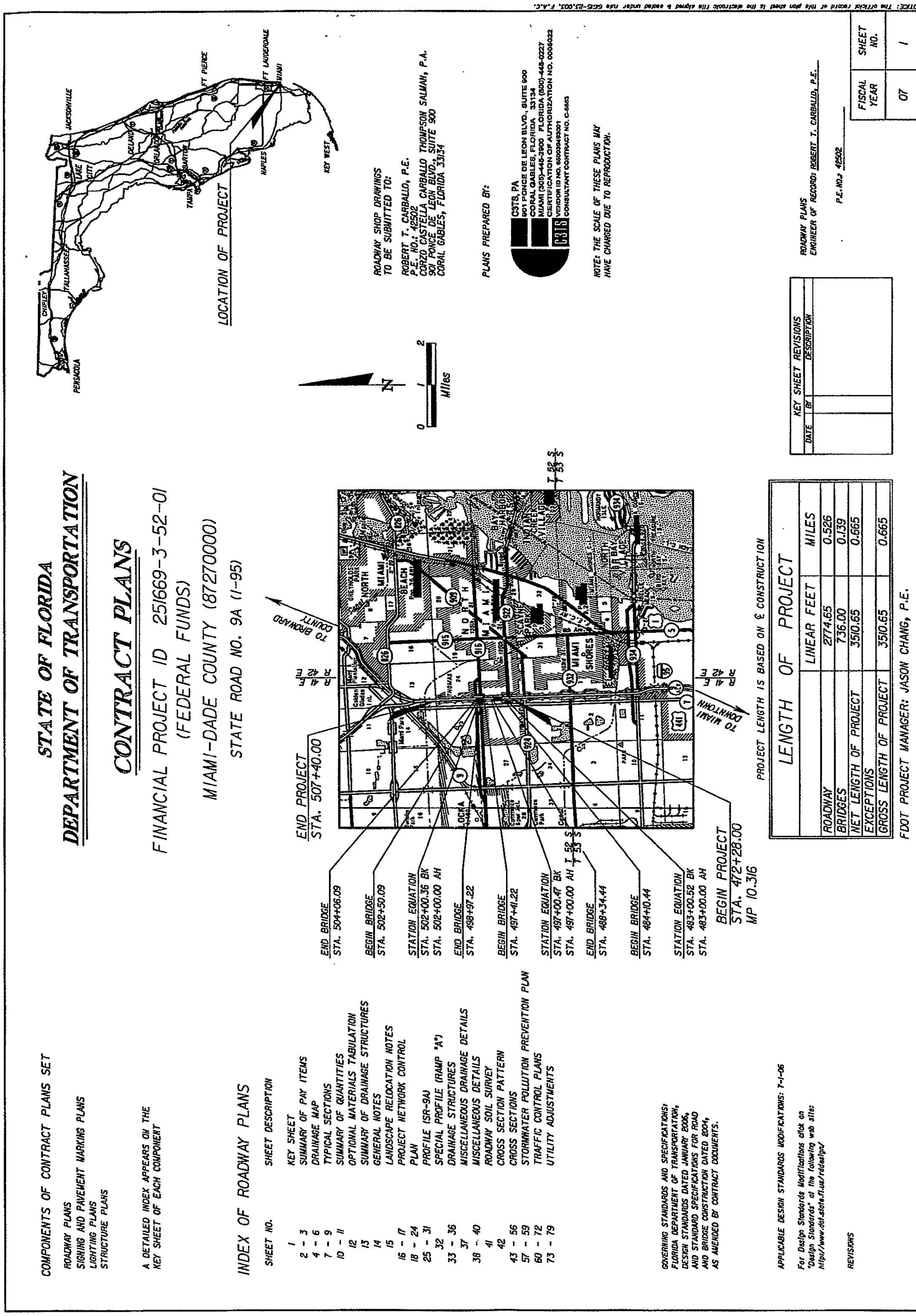
3) Indicate whether you wish to submit this report before or subsequent to Department issuance of a stormwater drainage well construction permit. The Department strongly advises the former option in order to most efficiently achieve the wells operational authorization.

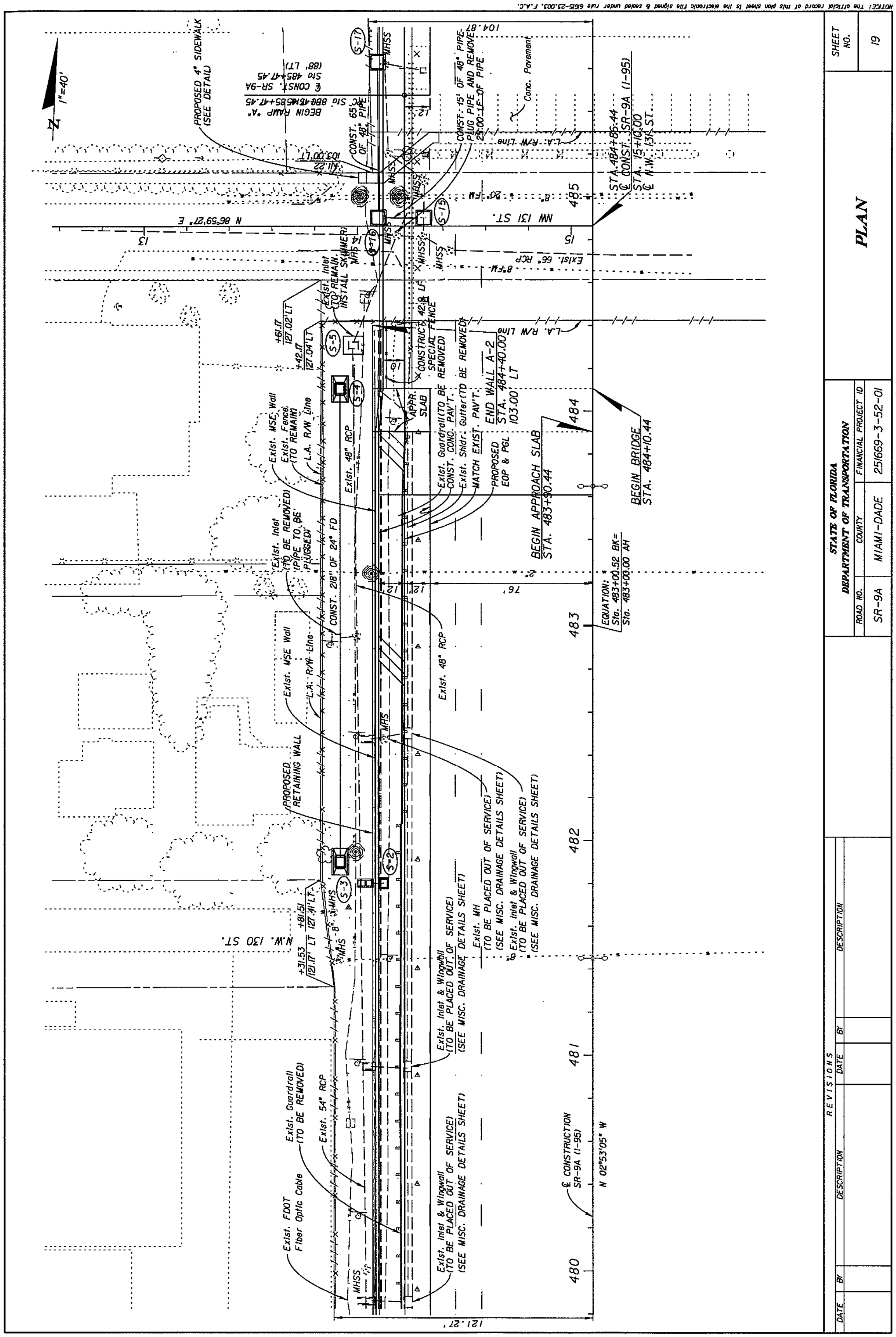
4) Note that no fluid shall be discharged into the stormwater drainage well(s) without written authorization from the Department to use the well(s).

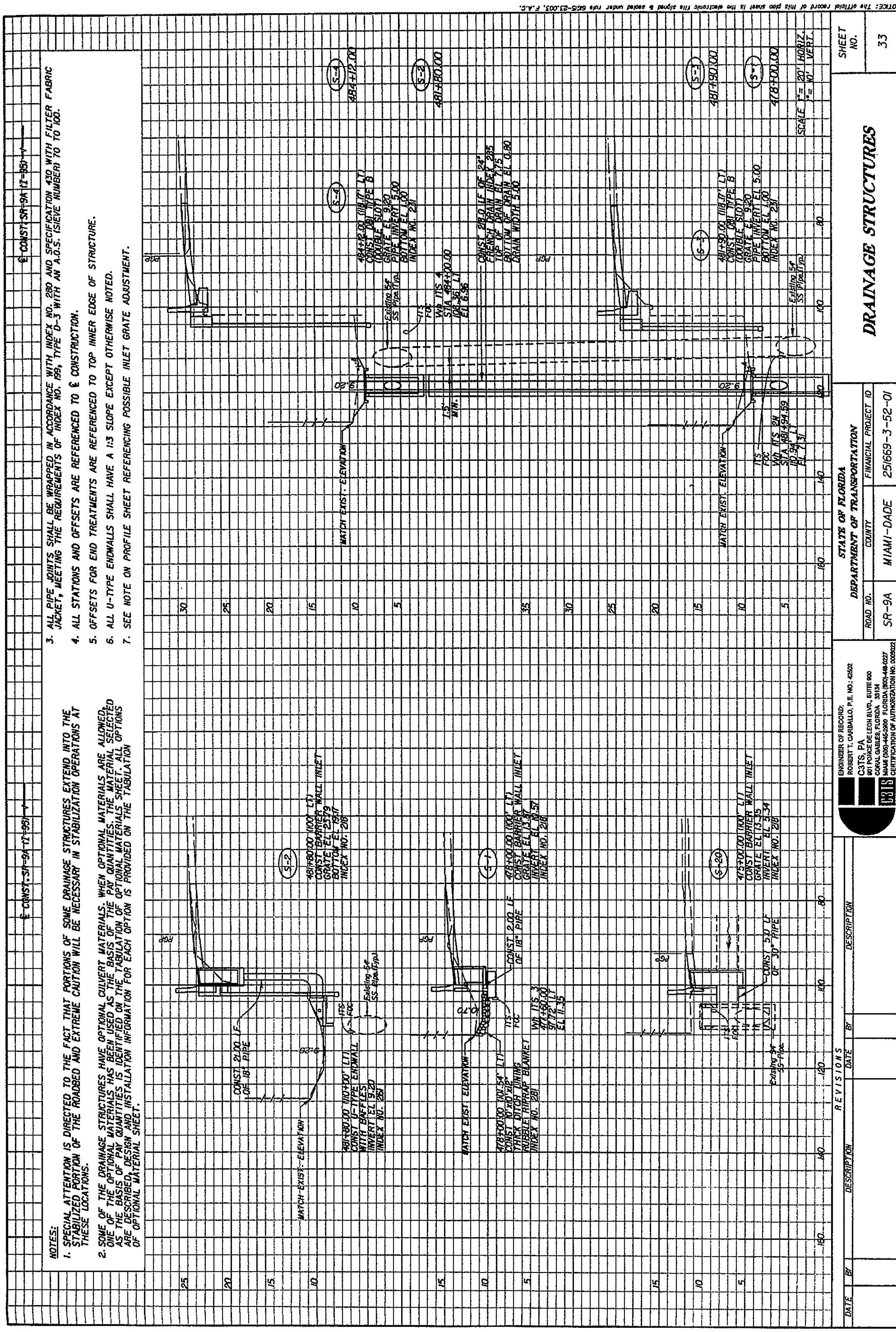
5) Issuance of a well construction permit does not obligate the Department to authorize the use of well(s), unless the well(s), information required by the permit and this report qualifies for an authorization.

APPENDIX D

SAMPLE PLANS




PLAN
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
ROAD NO.
SR-9A
COUNTY
MIAMI-DADE
FINANCIAL PROJECT ID
251669-3-52-01
SHEET NO.
19
REVISIONS


NOTES:
1. SPECIAL ATTENTION IS DIRECTED TO THE FACT THAT PORTIONS OF SOME DRAINAGE STRUCTURES EXTEND INTO THE STABILIZED PORTION OF THE ROADBED AND EXTREME CAUTION WILL BE NECESSARY IN STABILIZATION OPERATIONS AT THESE LOCATIONS.
2. SOME OF THE DRAINAGE STRUCTURES HAVE OPTIONAL CULVERT MATERIALS. WHEN OPTIONAL MATERIALS ARE ALLOWED, ONE OF THE OPTIONAL MATERIALS HAS BEEN USED AS THE BASIS OF THE PAY QUANTITIES. THE MATERIAL SELECTED AS THE BASIS OF PAY QUANTITIES IS IDENTIFIED ON THE TABULATION OF OPTIONAL MATERIALS SHEET. ALL OPTIONS ARE DESCRIBED, DESIGN AND INSTALLATION INFORMATION FOR EACH OPTION IS PROVIDED ON THE TABULATION OF OPTIONAL MATERIAL SHEET.
3. ALL PIPE JOINTS SHALL BE WRAPPED IN ACCORDANCE WITH INDEX NO. 280 AND SPECIFICATION 430 WITH FILTER FABRIC JACKET, MEETING THE REQUIREMENTS OF INDEX NO. 199, TYPE D-3 WITH AN A.O.S. (SIEVE NUMBER) 70 TO 100.
4. ALL STATIONS AND OFFSETS ARE REFERENCED TO ℄ CONSTRUCTION.
5. OFFSETS FOR END TREATMENTS ARE REFERENCED TO TOP INNER EDGE OF STRUCTURE.
6. ALL U-TYPE ENDWALLS SHALL HAVE A 1:3 SLOPE EXCEPT OTHERWISE NOTED.
7. SEE NOTE ON PROFILE SHEET REFERENCING POSSIBLE INLET GRATE ADJUSTMENT.
MATCH EXIST. ELEVATION
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
ROAD NO.
SR-9A
COUNTY
MIAMI-DADE
FINANCIAL PROJECT ID
251669-3-52-01
DRAINAGE STRUCTURES
SHEET NO.
33