ALTERNATIVES FOR THE TREATMENT OF GROUNDWATER CONTAMINANTS:

INFILTRATION CAPACITY OF ROADSIDE SWALES

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Department of Civil Engineering
and Environmental Sciences

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ENVIRONMENTAL RESEARCH

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16. Abstract
In recent years, there have been efforts to develop a design procedure for swale system which could be applied to permeable soils. This design should consider space limitations or any other site restriction.

Five existing swale sites in the Central Florida area were instrumented to deter flow rates and permeabilities. Twenty-two experiments were performed. It was conclu that the use of a double-ring infiltrometer to determine the infiltration rate is pos for existing swale sites. However, the rate from the infiltrometer must be reduced to match existing conditions. Another conclusion is that published values of permeablity are very variable and not accurate, nor specific enough to predict actual infiltration rates. However, conservative values can be used in design.

A swale design procedure, including an equation for the length of triangular and trapezoidal swales, was developed based on a requirement to percolate at least 8(percent of the runoff waters.

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ABBREVIATIONS AND CONVERSION FACTORS

[Factors for converting inch-pound units to International System of units (SI) and abbreviation of units]

Multiply	By	To Obtain
inch (in)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
acre	0.4047	hectare (ha)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
square foot (ft^2)	0.09290	square meter (m^2)
pound (lb)	0.4536	kilogram (kg)
cubic foot (ft^3)	0.02832	cubic meter (m^3)

NOTICE

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Neither the State of Florida nor the United States Government endorse products or manufacturers. Trade or manufacturers names appears herein only because they are considered essential to the object of this report.

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CHAPTER I

INTRODUCTION

In recent years, there has been an extensive effort to analyze and evaluate the different processes by which highway runoff waters and their pollutants can be lessened. Several methods to achieve infiltration of runoff waters from highways have been used. Among the most popular are detention/retention ponds and natural swale systems. Swales are vegetated open channels which both transmit and infiltrate runoff waters.

Several factors affect the effectiveness of swale systems, and proper considerations of such factors are necessary to optimize their design. Swales are an important management practice for control of rainfall excess (runoff volumes). Inherent in the design is the specification that the swale-wetted areas will infiltrate the rainfall excess (Wanielista et al. 1985).

Scope and Objectives

This study is limited to the determination of the infiltration capacity and design of swales. The quality of runoff is not emphasized, while quantity is. However, proper hydraulic design of swales would also increase their effectiveness to remove pollutants.

The swales used in this study are actual highway swales, adjacent to operating highways.

The objectives of this study are to determine:

1. double-ring infiltrometer, laboratory and published permeability measures that may relate to field infiltration rates estimated using a mass balance of volumes into and out of a swale.
2. design equations to size the length of a swale knowing runoff hydrograph volumes, field infiltration rate estimates, slope, cross-sectional area and roughness coefficients.

CHAPTER II

LITERATURE REVIEW

Introduction

A swale, as defined by the Florida State Department of Environmental Regulation (1981), is a shallow trench used to convey stormwater and permit infiltration/percolation. Although several other definitions of swales exist, the most commonly used define swales as broad shallow grassed channels, or grassed roadside ditches (Kent et al. 1983).

Precipitation

The State of Florida Administrative Code, Chapter 17.25, on Stormwater Management (1985), requires that swales be designed to infiltrate the rainfall excess from the one-in-three years design storm. Therefore, rainfall data must be available for design. Several methods have been used to compute the average precipitation, such as Thiessen and arithmetic average methods; however, interpretation of the watershed area and the designer's own experience must be predominant to predict precipitation intensities and volumes.

For Florida, 90% of all storms have a volume of one inch or less. Distribution of storm frequencies show that roughly 120 storms can be expected to occur in any location in Florida during one year. The rainfall mass distribution curves presented by Wanielista (1978) can be used. Specifically, for Orlando, a time of event distribution was developed. It shows that most storms (regardless of volume or intensity) can be expected to last less than 8 hours. Other distributions may be calculated for other areas from hourly precipitation data (Anderson 1982).

Hydrograph Generation

Similarly, hydrograph generation is necessary to determine the rainfall excess (runoff) to reach the receiving streams. Models, such as Santa Barbara unit hydrograph, unit hydrograph, Soil Conservation Service dimensionless hydrograph, and the rational formula are most widely used in calculating hydrograph shapes (Wanielista 1978). Several other methods are available, such as the recession analysis method, and once again, the judgement of the designer must determine their use.

Infiltration

Very special consideration must be given to the determination of the infiltration capacity of a swale. Infiltration is the passage of water through the soil surface into the soil. Although a distinction is made between infiltration and percolation (the movement of water within the soil) the two phenomena are closely related since infiltration cannot continue unimpeded unless percolation removes infiltered water from the surface soil. The soil is permeated by non-capillary channels through which water flows, primarily downward, toward the groundwater, following the path of least resistance. Capillary forces continuously divert gravity effected water into capillary-pore spaces, so that the quantity of gravity water passing successively lower horizons is steadily diminished. This leads to increasing resistance to gravity flow in the surface layer and a decreasing rate of infiltration as a storm progresses. The rate of infiltration in the early phases of a storm is less if the capillary pores are filled with water from a previous storm.

The infiltration volume is a function of the infiltration rate and contact time between the water and the soil, and this contact time is a function of the slope, resistance, length of swale, flow rate and depth of flow.

The infiltration process is non-linear. Several models have been adopted to create a unique mathematical model that can describe it. Darcy's Law and the principle of continuity of flow through porous media dictate the form of equations which best describe the rate of infiltration for a particular soil (Anderson 1982). A description of the existing models follows.

Green-Ampt Model

One of the earliest infiltration equations was developed by Green and Ampt (1911), which in light of subsequent research (Bouwer 1966, Fox 1975, Mein and Farrell 1974, Mein and Larson 1973, Morel-Seytoux and Khanji 1974, Neuman 1976) can be written as:

$$V_i = K \frac{H_w + L_f - h_{cr}}{L_f}$$

where:

V_i = infiltration rate (length/time)

K = hydraulic conductivity of wetted zone

H_w = depth of water

h_{cr} = critical pressure head of soil for wetting

L_f = depth to wetting front of groundwater

This equation is obtained by applying Darcy's equation to the wetted zone, assuming vertical flow and uniform water content and hydraulic conductivity in the wetted zone. The wetting front is considered as an abrupt interface between wetted and non-wetted material. Thus, the infiltration system is treated as "piston flow" (Bouwer 1978).

Mein-Larsen Model

The Mein-Larsen Model (1973) postulates a two-step process to include surface delay effects during infiltration. In this model, infiltration rate is related to hydraulic conductivity, initial moisture content, wetting front positions, rainfall intensity and capillary action at the wetting front. Uniform initial moisture content in the zone of infiltration, constant rainfall intensities and homogeneous soil types are assumptions upon which the model is based. The Mein-Larsen Model has merit and predicts infiltration within its assumptions, but the quantity of parameters which must be measured to define the process may be unattractive to some potential users.

In reality, infiltration-flow systems are much more complex than can be expressed with a simple equation. Interaction between the infiltrating water and the soil, rearrangement of soil particles near the surface due to the impact of raindrops or erosion by runoff or other flowing water, accumulation of fines on the surface, and the inherent non-uniformity of the soil limit the accuracy with which equations based on Darcy's Law or diffusion theory can predict infiltration rates. The other approach then, is to use empirical equations with "constants" calculated to relate infiltration rates and volume to time. One of the simplest empirical equations relates volume of infiltrate (I_t) to time (t), Kostiakov (1932):

$$I_t = Ct^{\alpha}$$

The parameters C and α are readily obtained from a plot of the measured values of I_t and t on double logarithmic paper, which is assumed to yield a straight line.

Horton's Equation

R.E. Horton (1940) proposed representing the infiltration rate by an exponentially decaying formula of the form:

$$f = f_c + (f_o - f_c)\, e^{-K_f t}$$

where:

f = infiltration rate at time t

f_o = initial infiltration rate at time t = 0

f_c = limiting infiltration rate

K_f = a constant

t = time

Horton's method of describing infiltration makes it one of the easier methods to gather rate data for, but there are shortcomings to the use of this equation. Horton's equation is descriptive of ponded conditions and has no direct discernable dependence on rainfall intensity. The equation must be calibrated for different soil types and cover conditions (Musgrave et al. 1964). The relationship of f_o to initial moisture content could use verification. The question of whether the f_c value is the true predictor of saturated hydraulic conductivity needs investigation. The variation of the K factor with soil type, moisture and cover conditions is also subject for future determination (Anderson 1983). Horton's equation is the principle in which the infiltration rate obtained from a double-ring infiltrometer is based.

Mass Balance Method

Probably the most accurate method of determining infiltration is by the mass balance method; however, it has the disadvantage that it is a labor intensive method that requires the movement of a lot of equipment. Two flow-measuring devices, triangular weirs, are placed at a certain distance in the swale, and water is pumped into the swale system measuring the flow in and the flow out, until they both stabilize when the pumping of water is stopped. Then, the remaining water storage is measured and the infiltration volume (I) is calculated knowing flow rates (Q) into and from a swale and its storage or:

$$I = (Q_{in} - Q_{out})\Delta t - \text{Storage}$$

The average infiltration rate can then be calculated by measuring the time and dividing the infiltration volume by the product of the time, the wetted perimeter and the distance between weirs.

Figure 1 lists the factors that affect the infiltration rate into a soil. These factors include the hydraulic conductivity, the moisture content, the soil classification, and the surface conditions.

Permeability

Permeability can be defined as the movement of water in the soil; it has the units of length over time, and typical

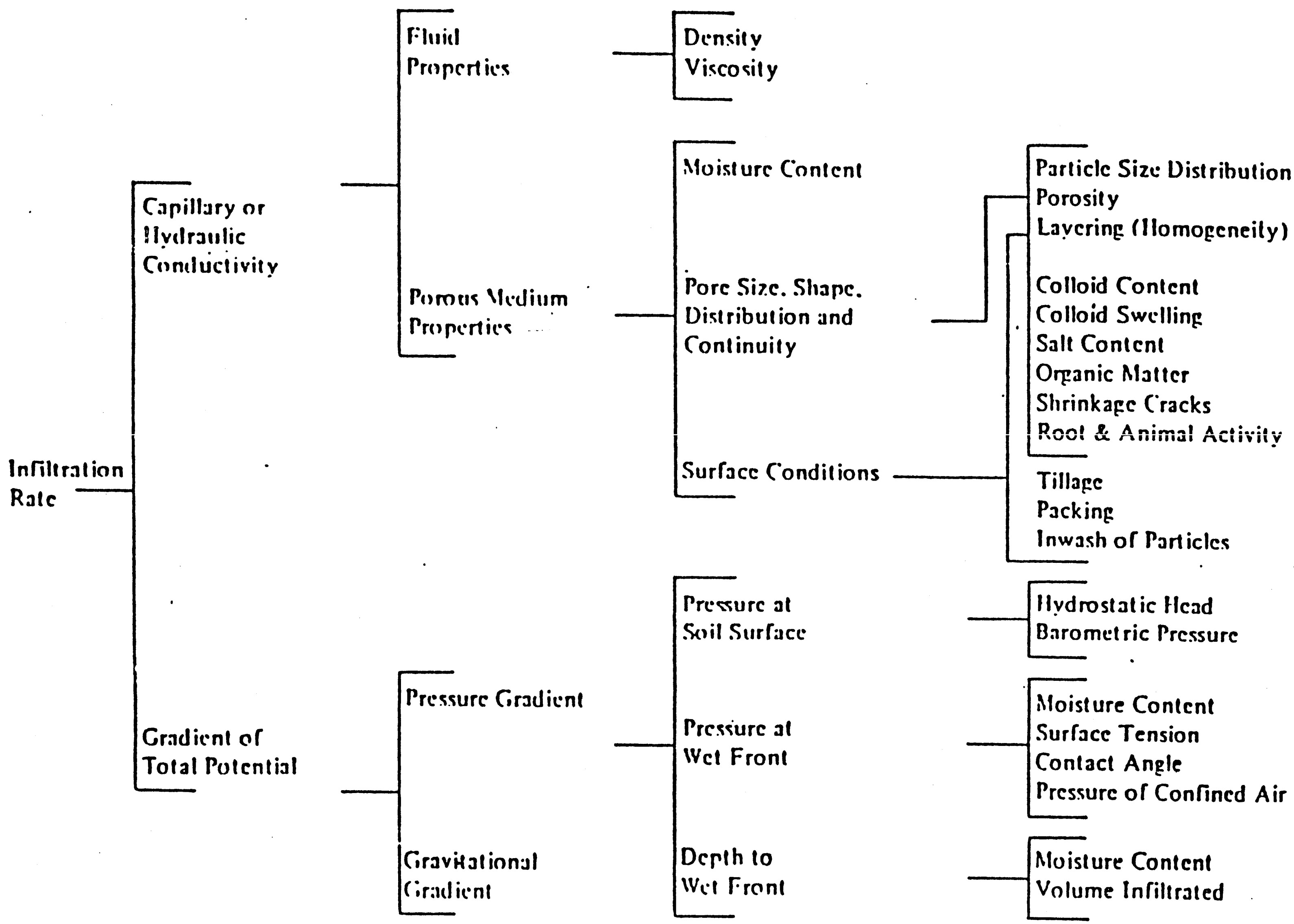


Figure 1. Factors Affecting the Infiltration Rate into Unfrozen Soil.

values of it can be found in Table 1 and in Figure 2 (US and SI units). These approximate average values for permeability listed in Table 1 are to be used only when no other information on the soil can be obtained. Also listed are AASHTO subgrade classification.

There are several methods to define permeability. Some of them are performed in the field, while others are done in the laboratory. Table 2 lists some of these methods. The most common direct methods for determining permeability in the laboratory are the constant and falling head permeameters.

Constant Head Permeameter

The constant head method consists of measuring the time, t, for the volume of water, Q, to pass through a soil sample of length, L, and cross-sectional area, A, while keeping a constant head, Δh. The permeability, K, is then computed as:

$$K = \frac{QL}{\Delta h\ At}$$

where:

L = length of sample

Q = flow rate of water through sample

Δh = head of water which is kept constant

A = cross-sectional area of sample

t = time

TABLE 1

COEFFICIENT OF PERMEABILITY AND AASHTO CLASSIFICATION FOR SOILS

FORMATION	VALUE OF K		AASHTO
	(cm/sec)	(in/hr)	
River deposits			
Rhone at Gennissiat	Up to 0.40	567	A-1
Small streams, eastern Alps	0.02-0.16	28-227	A-1
Missouri	0.02-0.20	28-283	A-1
Mississippi	0.02-0.12	28-170	A-1
*Sands			
St. Lucie, Lakeland, Pomello, Lakeland	0.014 -0.02	20-30	A-3
Blanton, Imokalee, Leon	0.007 -0.014	10-20	A-3
Plumner, Delray, St. Johns, Manatee	0.0035-0.007	5-10	A-3
*Loamy Sand			
Manatee loamy, Delray Hucky, Terra Ceia	0.0018-0.0035	2.5-10	A-2-4
*Clay Loam and Peat			
Iberia, Istokpoga, Okeechobee	0.00014	< .2	A-7
Clay	0.0000001	.00014	A-7

*Florida Soil Associations

SOURCE: Williams 1979 and USDA 1975

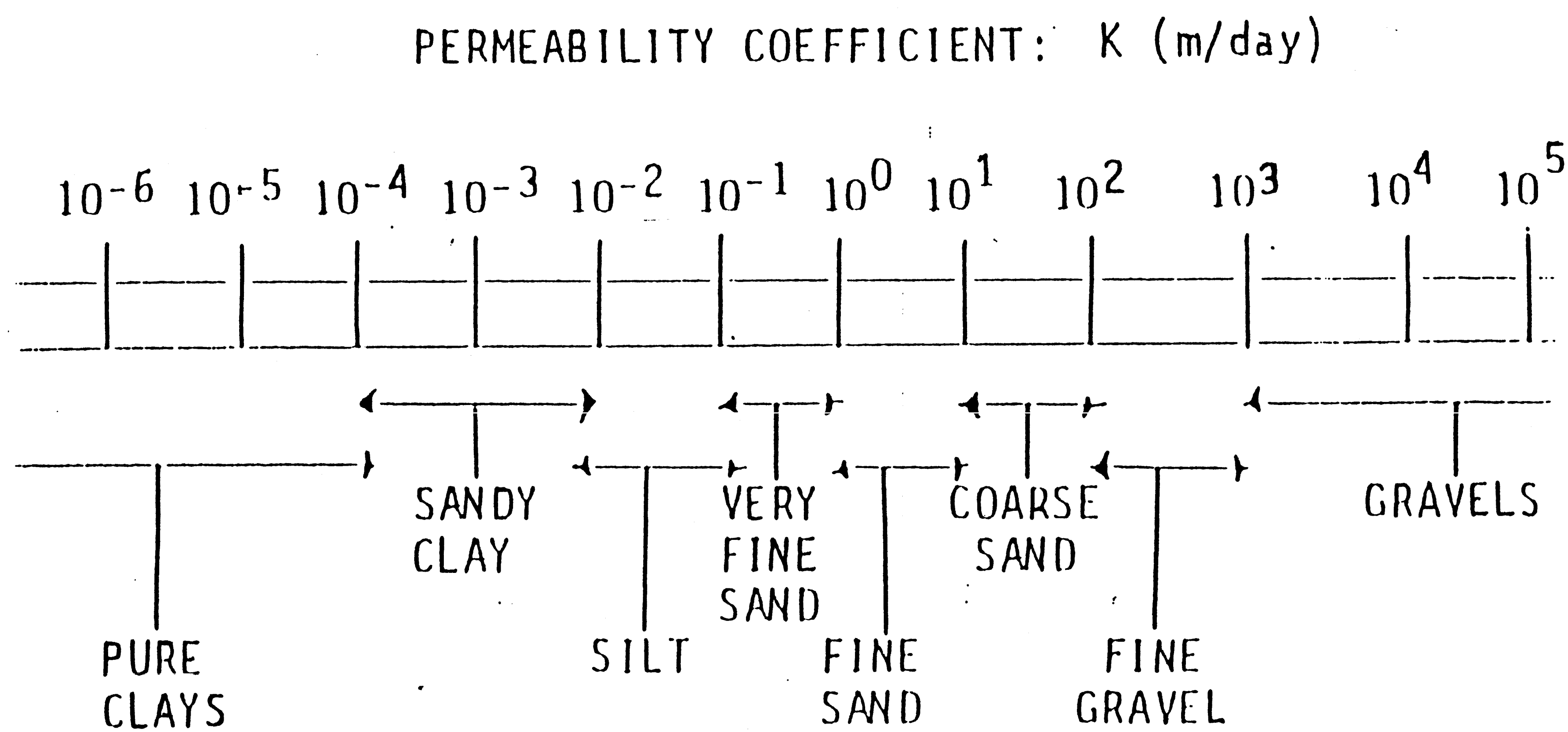


Figure 2. Range of Permeability Coefficient Values

TABLE 2

DETERMINING PERMEABILITY

A. Direct Methods - Laboratory

1. Fixed or constant head permeameter
2. Falling head permeameter

B. Direct Methods - Field

1. Auger hole
2. Tube method
3. Piezometer method
4. Tracers
5. Mass balance

C. Indirect Methods

1. Consolidation testing
2. Horizontal capillary
3. Formulas
4. Design charts

SOURCE: Spangler and Handy 1973

Falling Head Permeameter

For the falling head permeameter, the volume rate of flow, q, can be expressed on the basis of the rate of fall, dH/dt, of the water level in the narrow standpipe as q = A dH/dt, or on the basis of the flow through the sample with Darcy's equation as q = KA H/L. Equating both expressions for q, integrating and solving:

$$K = \frac{a\ L}{A\ \Delta t} \ln \frac{h_1}{h_2}$$

where:

L = length of soil sample

Δt = time interval

a = area of the tube

A = cross-sectional area of sample

h_1, h_2 = H values at beginning and end of time, t

Other Methods

Several direct methods to determine permeability in the field are listed in Table 2. They include the auger hole method, the tube method, the piezometer method, etc. Furthermore, indirect methods to determine permeability are available, and the criteria to choose the method must depend on what type of facilities are available. These methods to obtain permeability can be found in Spangler and Handy (1973).

Several other factors must be considered to analyze a swale system, such as type of vegetation covering the swale, soil classification, antecedent moisture content, slope, maintenance etc. However, only the coefficients of uniformity and curvature, and the roughness or retardance coefficient, are described in this report.

Coefficient of Uniformity and Curvature

A soil that has a nearly vertical grain-size distribution curve (all particles of nearly same size) is called a uniform soil. If the curve extends over a rather large range, the soil is call well-graded. The distinction between a uniform soil and a well-graded soil can be defined numerically by the uniformity coefficient, C_u, and the coefficient of curvature

C_u is defined by the ratio D_{60}/D_{10} and a value smaller than 4 indicates that the soil is uniform. The more uniform the soil, the greater its infiltration capacity becomes. Therefore, a value for C_u less than 4 would indicate that the soil has a relatively high infiltration rate within a specific soil classification. C_z is defined as $D_{30}/D_{10}D_{60}$ and a value of C_z between 1 and 3 would indicate that a soil is well-graded (Dunn et al. 1980). A well-graded soil would have a relatively low infiltration rate for a specified soil group while a poorly graded soil would have a relatively higher rate. The coefficient values can be used to specify soils suitable for swale construction.

Roughness Coefficient

Proper consideration must be given to selecting an accurate value for Manning's Roughness Coefficient. When Manning's equation is used, and the assumptions that the width of flow is much greater than the depth of flow and that the rainfall excess is equivalent to depth of flow per unit time, then the Manning coefficient can be estimated using the equation developed by Wanielista et al. (1983) and is reproduced as:

$$n = \frac{t_e^{5/3} \; S^{1/2} \; (R/t)^{2/3}}{L}$$

where:

R/t = rainfall excess rate (m/hr)

t_e = time to equilibrium (min)

L = length of swale (m)

S = flow slope

n = overland flow Manning's coefficient

An average roughness coefficient of $n = 0.05$ is suggested for use in swale systems with the current regular grass cutting maintenance procedures. Several tables and graphs are available to estimate the value of n, Todd (1980), Wanielista (1988), Florida DOT (1987).

CHAPTER III

FIELD PROCEDURE

Site Descriptions

Swales at five different sites were chosen for the experiments. The choices were based on: (1) number of lanes of traffic from which the swale receives runoff waters, (2) variable permeability, and (3) availability of water. Two of them receive waters from an interstate highway (swales at Interstate 4), two receive waters from two-lane secondary highways (swale a Sunrise Place and swale at Reed Road), and one receives runoff from a state road (swale at SR 426). The soils were classified for each site.

The soil classification is obtained in the laboratory passing the soil through different sieves and using this data, the uniformity and curvature ratios are calculated. The coefficient of permeability is obtained in the laboratory using the constant head permeameter described previously in the Literature Review section.

Figures 3 through 8 show the map location of the sites, the cross-sections, side slopes and soil classificaton for each one of the swales. It can be seen that the soil type is about the same in each of the locations, while the flow slope, the coefficient of permability and the side slope varies from site to site.

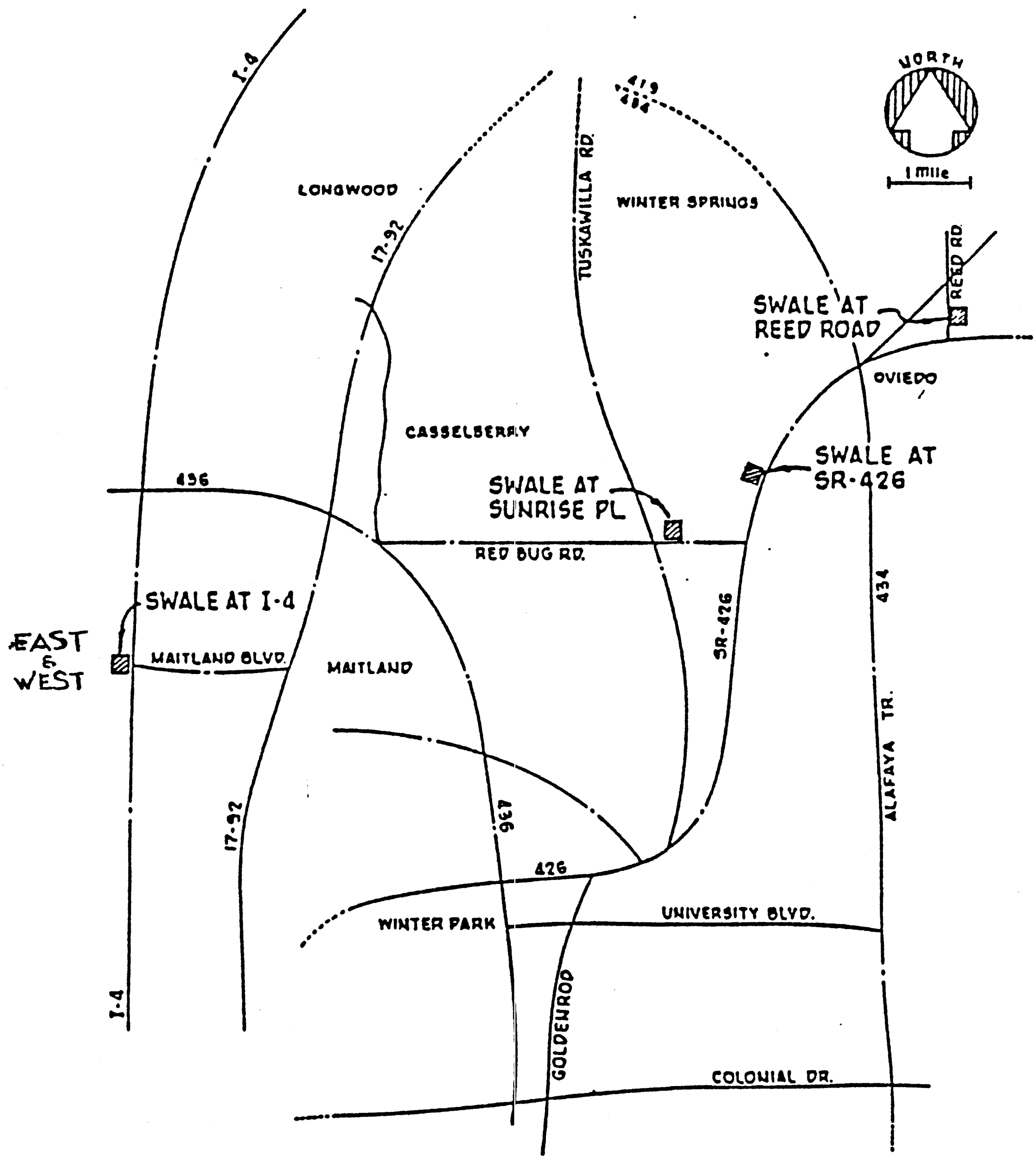
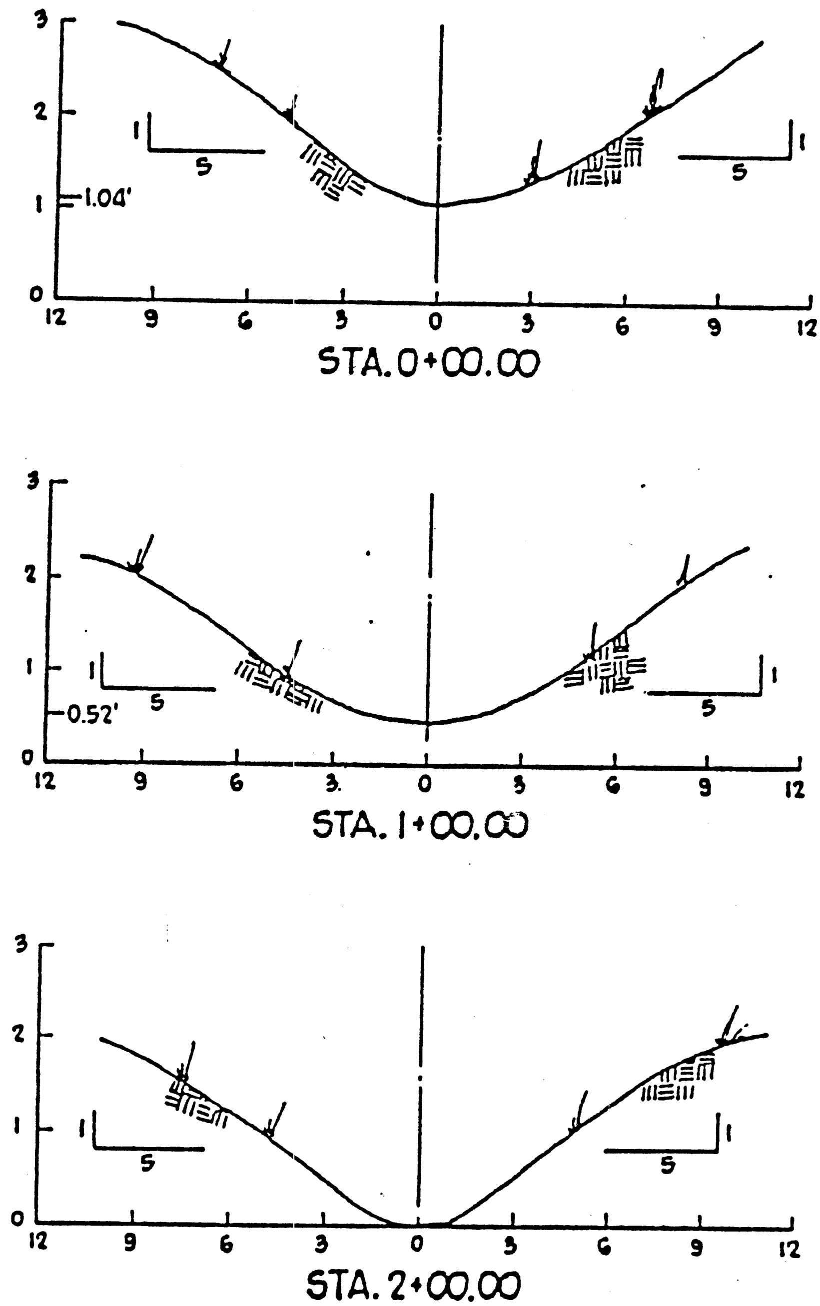


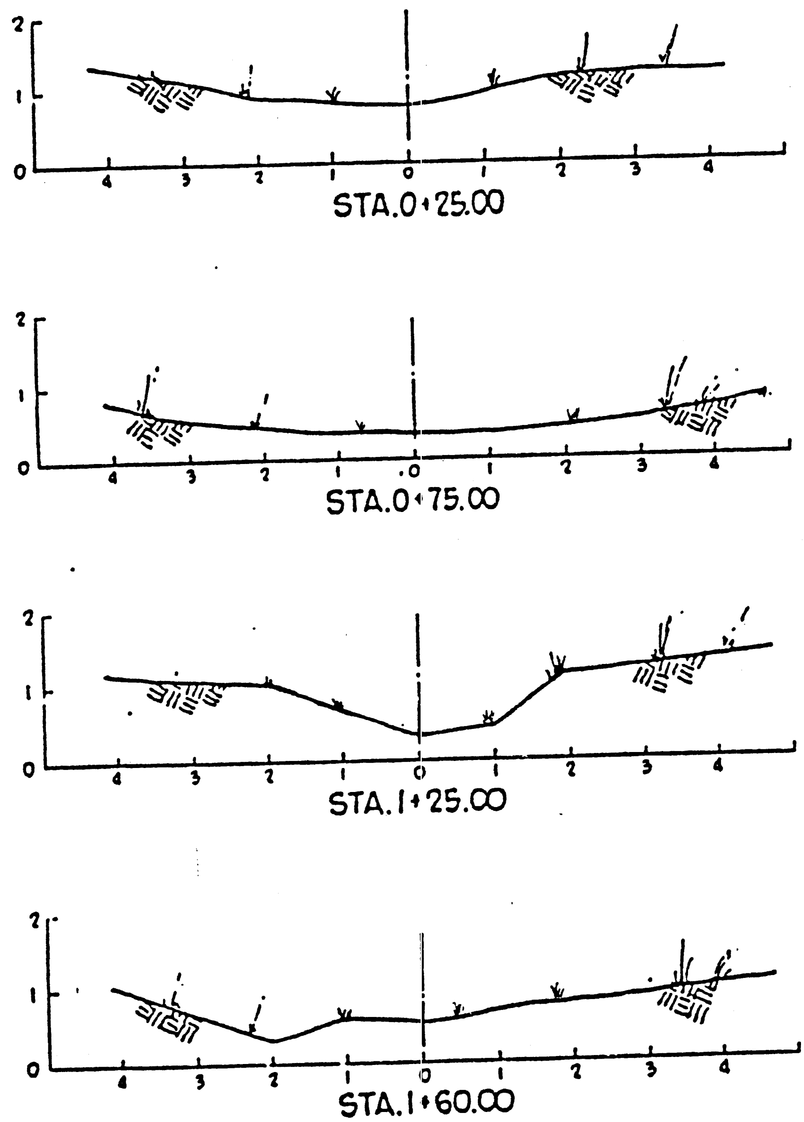
Figure 3. Map Location of Swales.



<u>Site Classification</u>
C_u = 1.38
C_z = 0.96
K = 10.871 in/hr
Longitudinal slope: 0.0052

Horizontal Scale: 6' =
Vertical Scale: 2' =

Figure 4. Swale at Sunrise Place - Red Bug Lake Road.



<u>Site Classification</u>

C_u = 1.966

C_z = 1.412

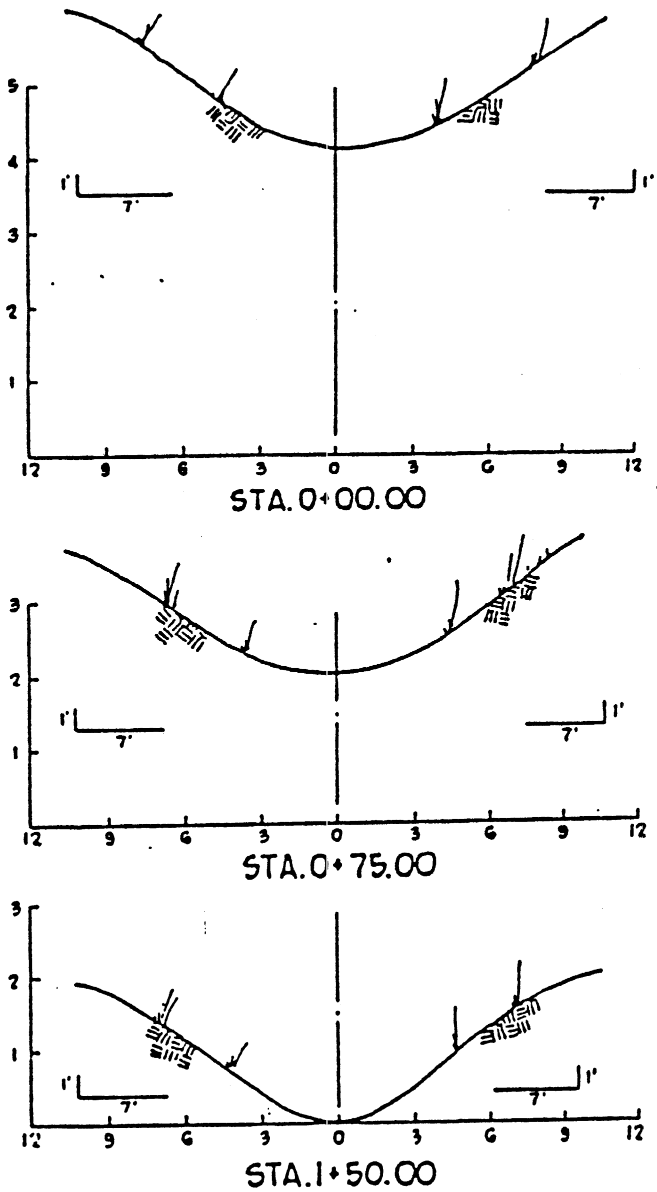
K = 7.228

Longitudinal Slope: 0.0085

Horizontal Scale: 3' =

Vertical Scale: 2' =

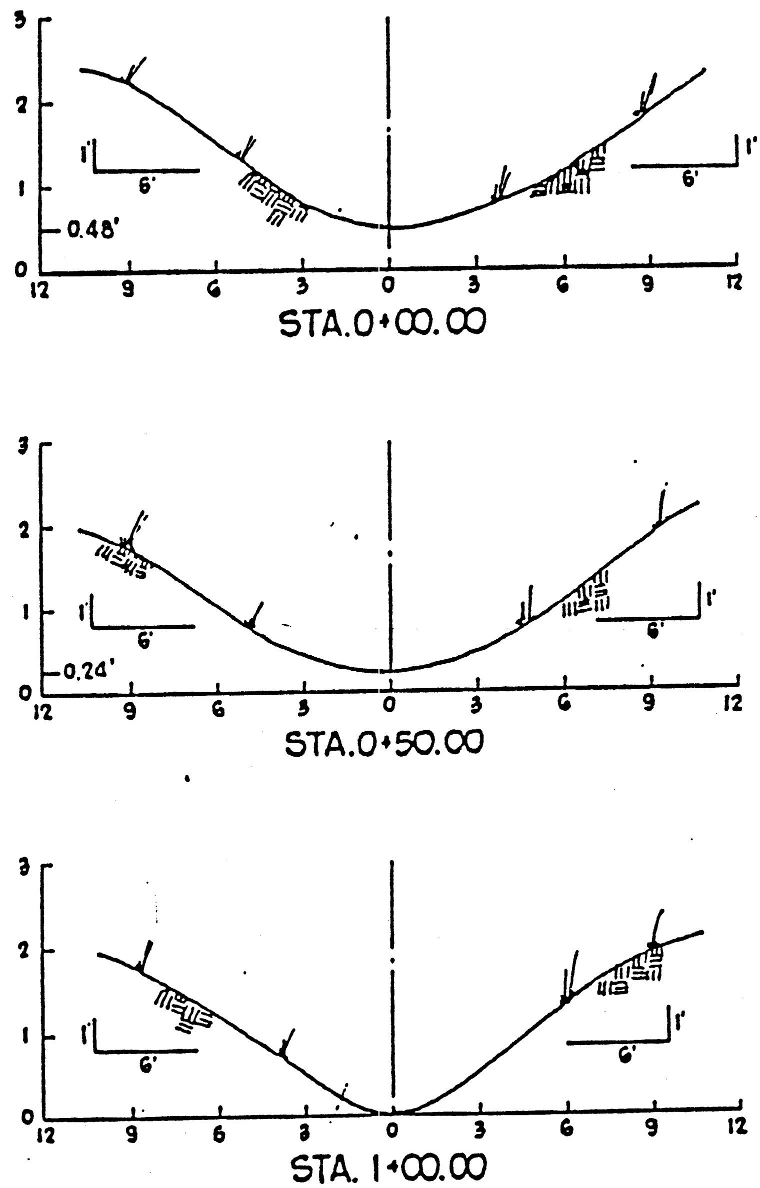
Figure 5. Swale at I-4 Maitland Interchange-West.



Site Classification
C_u = 2.08
C_z = 1.33
K = 4.068 in/hr
Longitudinal Slope: 0.0279

Horizontal Scale: 6' = ├──────┤
Vertical Scale: 2' = ├──────┤

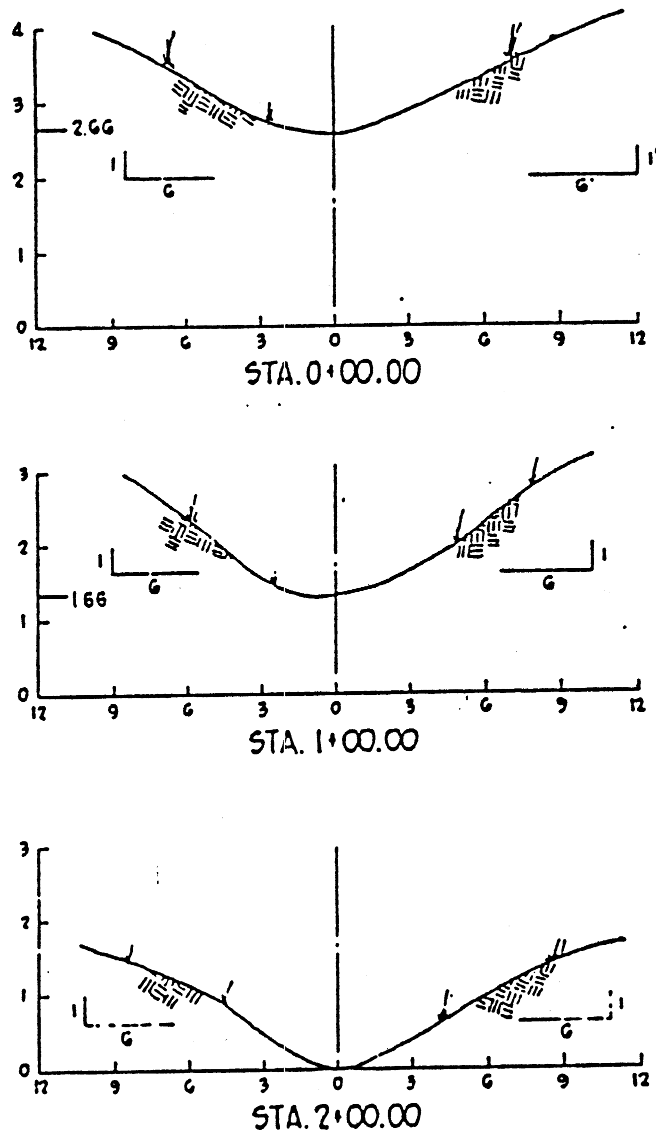
Figure 6. Swale at I-4 Maitland Interchange-East.



<u>Site Classification</u>
$C_u = 1.67$
$C_z = 1.24$
$K = 12.047$ in/hr
Longitudinal Slope: 0.0048

Horizontal Scale: 6' =
Vertical Scale: 2' =

Figure 7. Swale at State Road 426.



Site Classification
C_u = 1.81
C_z = 1.28
K = 3.118 in/hr
Longitudinal Slope: 0.0133

Horizontal Scale: 6' =
Vertical Scale: 2' =

Figure 8. Swale at Reed Road-Chuluota Area.

Swale at Sunrise Place

This swale is located off of Red Bug Lake Road, at the entrance of Sunrise Place, at approximately two miles west of SR 426 in Seminole County, Florida. The swale is in the north side of the road and the water flows from east to west. The swale is totally grassed and has a high level of maintenance. It receives runoff waters from Red Bug Lake Road and from Sunrise Road through an invert located at the eastern section of the swale. The grass cover is approximately three inches. Its slope (longitudinal) is 0.0052, being the second smallest slope of the swales studied. The soil beneath the grass is a uniform soil with gravely to fine sand, with a coefficient of permeability of 10.871 in/hr. The side slope of the swale is approximately 5 horizontal by 1 vertical. The value of 1.38 for the coefficient of uniformity, C_u, shows that the soil under the swale is uniform. The value of 0.96 for the coefficient of curvature, C_z, shows that the soil is poorly graded.

Swale at Interstate 4 - West

Located at Maitland Interchange, in the southwest section of the intersection of I-4 and Maitland Boulevard, in Orange County, Florida, with the water flow from north to south, the swale areas are covered with predominantly Bahia grass, approximately two to four inches in height. It receives runoff waters from Interstate 4 and from Maitland Boulevard. The swale has longitudinal slope of 0.0085, and it is usually well maintained. The soil beneath the grass is a poorly graded soil, with medium to fine sands, and a coefficient of permeability of 7.228 in/hr. C_u is equal to 2.08 and shows that the soil under this swale is uniform. C_z is equal to 1.33 and shows that this soil is better graded than the one in Sunrise Place.

Swale at Interstate 4 - East

Located at Maitland Interchange, in the southeast section of the intersection of I-4 and Maitland Boulevard, in Orange County, Florida, with the water flowing from south to north, this swale presents the same characteristics of the swale located west of I-4, with side slopes of 7:1 and permeability of 4.068 in/hr. This swale has the steepest longitudinal slope of 0.0279. A value for C_u of 2.08 shows that this soil is uniform, and C_z is equal to 1.33 and shows that the soil is better graded than the swale at Sunrise Place.

Swale at State Road 426

This swale is located on State Road 426 in the Slavia Area, in the west side of the road, approximately one mile north of Red Bug Lake Road, in Seminole County, Florida. This swale has water flowing from north to south and has a minimum Bahia grass cover which exposes the swale to some erosion. The maintenance is also very poor, and a large amount of metal cans, bottles and all sorts of trash is usually found on top of the swale. The soil beneath the grass is a uniform soil, with a gravely to fine sand texture. The coefficient of permeability is 12.047 in/hr, which shows the greatest infiltration capability of this swale. It presents a longitudinal slope of 0.0048, with side slopes of 6:1. The values of C_u are equal to 1.67 and C_z is equal to 1.24, which show that the soil is uniform and poorly graded.

Swale at Reed Road

This swale is located at the west side of Reed Road in the Chuluota area, half a mile north of SR 419 in Seminole County, Florida, with water flowing southward. The grass cover is approximately two inches and has poor maintenance. The soil beneath the grass is poorly graded, with gravely to fine sands, and a coefficient of permeability of 3.118 in/hr. It has a steep longitudinal slope of 0.0133, with side slopes of 6:1. The values of C_u is equal to 1.81 and C_z is equal to 1.28, which show that the soil is uniform.

Flow Measurements

The flow through the swale is measured using 90^0 V-notch weirs and the equation:

$$Q = 2.49\ H^{2.48}$$

where:

H = head of flow over the weir (ft)

Q = flow rate (cfs)

and the constants (2.49 and 2.48) were determined in the field using site-related data.

These flow rates were converted to gallons per minute using the conversion factor of 448.8 gpm/cfs. During field experiments the acccuracy of the weir equation was checked by accumulating flows in a graduated cylinder for a short time period and recording the head over the weir during this period (Avellaneda, 1985).

These calculations also were checked using the nomograph presented in Public Works Magazine (Wanielista 1981).

Water Content

The water content is determined using a nuclear gauge density moisture tester, before and after water flows through the swale. Water content can also be determined by collecting samples, before and after the flow of water, in recipients that do not allow water to evaporate; then, weighing the samples in laboratory and drying them in an oven to get the weight of solids in each sample, the content (as a percentage) can be obtained using the equation:

$$W = \frac{W_T - W_S}{W_S} \times 100$$

where:

W_T and W_S = total weight and weight of solids, respectively

Double-Ring Infiltration Rate

A field infiltration rate is obtained by using a double-ring infiltrometer which consists of a 24 inch outer ring that is dug into the ground and an inner, 12 inch diameter smaller ring that is also dug into the ground. The infiltration rate, in inches per hour, is then computed with the change of depth of water in the inner ring in each time interval. The infiltrated water in the outer ring makes the water in the inner ring flow vertically downward, preventing it from flowing horizontally. (ASTM Standard D3385).

Due to the fact that flow in the swale produces dynamic conditions, instead of static, it is not expected that the actual values will be predicted using the double-ring method. The pressure distribution caused by flow is less at the bottom of flow in the open channel relative to equal head experiments using the double-ring infiltrometer. However, one should expect that the double-ring may be useful to estimate a static value and then by comparison to the actual dynamic value, an adjustment factor (safety factor) can be developed. All double-ring experiments were performed in the swale just upstream of the actual flow test area and at the same approximate head as experienced in the swale. Both experiments were conducted simultaneously.

CHAPTER IV

RESULTS

Geohydrologic Data

One of the requirements of the experiments is to compare the actual field geohydrologic conditions, such as soil density, average static infiltration rates and average actual infiltration rates, with published values of the Soil Conservation Service (SCS and with predicted values from laboratory data.

Of specific interest for swale design is the rate of infiltration. In Table 3 are listed some of the geohydrologic site data for swales studied. It can be seen that the SCS values for permeability represent a wide range of values which makes it difficult for the designer to predict a value that accurately describes the infiltration behavior of the swale. In two cases, the value of the laboratory permeability does not coincide with the range presented by the SCS for that site. One of those sites, Reed Road, is described by the SCS classification system as man-made or modified by man, which would explain the offset in the permeability values. For the other cases, Interstate 4 - East, represents an offset of approximately 20% with the actual value. Generally, the lab derived permeability values are lower than or equal to the lower value in the range for the SCS. Also in Table longitudinal slopes and soil densities for each site are listed. A nuclear density meter was used to measure density at the sites.

TABLE 3

GEOHYDROLOGIC SITE DATA

LOCATION OF SWALE	SUBGRADE CLASS *	SCS ** PERMEABILITY (in/hr)	LAB PERMEABILITY (in/hr)	SLOPE	SOIL DENSITY (lb/ft^3)	DOUBLE-RING INFILTRATION RATE (in/hr)	AVERAGE MASS BALANCE INFILTRATION RATE (in/hr)
Sunrise Place	A-3	10-20	10.87	0.0052	108	12.86	9.48
Reed Road	A-3	10-20	9.07	0.0133	112	13.46	8.50
S.R. 426	A-3	10-20	15.38	0.0048	90	19.75	14.39
Interstate 4 (West)	A-2-4	5-10	7.23	0.0075	110	7.41	4.08
Interstate 4 (East)	A-2-4	5-10	4.07	0.0279	104	9.95	6.33

* American Association of State Highway and Transportation Officials (AASHTO) Highway Subgrade Materials Classification, 1970, Edition 10.
General Classification: Granular Materials with basic groups:
A-3 (fine sand), A-2-4 (silty or clayey gravel and sand).
The liquid limit range was 16-25% and zero plasticity.

**SOURCE: SCS Supplement 1975

Infiltration rates determined from the mass balance include the higher infiltration rates at the beginning of flow. Thus, these values may be higher than at saturated conditions, near the end of flow.

The longitudinal slope is a very important parameter to be considered, because the steeper the slope, the less predictable the infiltration rate from permeability data. The soil density is necessary to perform the permeability test, both in the laboratory and in the field. Finally, in Table 3 are listed the average static infiltration rates and the actual average infiltration rates.

Mass Balance and Double-Ring Infiltration Rates

The double-ring infiltration rate refers to the value of the infiltration rate obtained by using the double-ring infiltrometer. It represents the infiltration rate during non-flow (static) conditions. It is termed static because the flow of water does not present any horizontal displacement; the water should move vertically downward and the value of the infiltration is obtained by measuring the amount of water that infiltrates in each time interval. Several factors affect the infiltration behavior of the soil when the double-ring infiltrometer is used. For instance, the head of water over the soil is much greater by using this method than if the mass balance method is used, which increases the value of the infiltration rate. It can be seen in Table 3 that values of average static infiltration rate range from 7.41 in/hr to 19.75 in/hr. Also listed in Table 3 are the average values of the infiltration rates obtained from a mass balance. It is also referred to as the dynamic infiltration rate because it determines the infiltration rate of the soil as water flows through the system. The mass balance method is one of the most accurate methods to determine infiltration. However, it involves a significant movement of equipment and a significant amount of time. It can be seen from Table 3 that average values for mass balance infiltration rates range from 4.08 in/hr to 14.39 in/hr for the sites studied. In Table 4 are listed the ratios of the average mass balance infiltration rate to lab permeability, and to average double-ring infiltration rate for each site. It can be seen that, based on the information of Table 4, the lab permeability does not predict accurately the mass balance infiltration rates. Lab permeability values range from about one-half of the mass balance infiltration rate to almost 94% of the actual value. However, the ratio of mass balance infiltration rate to static infiltration rate seems to be always less than 75%, which confirms the belief that the double-ring infiltrometer yields results that are higher than mass balance values. The ratio varies from 52% to 73% with an average value of 63%. A conservative design should use a value for infiltration of 50% of the value obtained with the double-ring method.

Field Experimentation

Values for mass balance infiltration rate for all sites are listed in tables 5 through 9. In each table are given

TABLE 4

SUMMARY OF INFILTRATION RATIOS DURING STUDY PERIOD

LOCATION	# OF EXP	LABORATORY PERMEABILITY (in/hr)	AVERAGE DOUBLE-RING INFILTRATION (in/hr)	AVERAGE MASS BALANCE INFILTRATION (in/hr)	RATIO MASS BALANCE INF / LAB PERMEABILITY	RATIO MASS BALANCE INFIL / DOUBLE-RING INFIL
Sunrise Place	6	10.87	12.86	9.48	0.87	0.74
Reed Road	6	9.07	13.46	8.50	0.94	0.63
S.R. 426	4	15.38	19.75	14.39	0.94	0.73
Interstate 4						
(West)	3	7.23	7.41	4.08	0.56	0.55
(East)	3	8.07	12.17	6.33	0.78	0.52

TABLE 5

FLOW DATA FOR SWALE AT SUNRISE PLACE

EXP. NO.	DISTANCE BETWEEN WEIRS (ft)	FLOW IN (in)	FLOW OUT (in)	BEGINNING WATER CONTENT (%)	ENDING WATER CONTENT (%)	PEAK FLOW IN (in/hr)	PEAK FLOW OUT (in/hr)	SWALE DISCHARGE COEFFICIENT C_v (volume)	SWALE PEAK FLOW COEFFICIENT C_p	MASS BALANCE INFILTRATION RATE (in/hr)
1	100	23.53	9.62	*	*	8.4	4.92	0.41	0.59	13.90
2	60	34.16	22.89	25.75	52.43	41.4	29.76	0.67	0.72	14.10
3	62	35.58	22.64	23.76	51.94	40.32	27.48	0.64	0.68	8.00
4	180	35.76	24.74	35.50	56.30	18.48	13.92	0.69	0.75	5.60
5	60	62.4	49.	33.60	54.8	41.23	37.97	0.79	0.92	6.37
6	180	62.4	40.1	33.60	54.8	41.23	30.62	0.64	0.74	8.92

* Not measured

NOTES: m/hr = in/hr x 0.0254
m = in x 0.0254
m = ft x 0.305

TABLE 6

FLOW DATA FOR SWALE AT S.R. 426

EXP. NO.	DISTANCE BETWEEN WEIRS (ft)	FLOW IN (in)	FLOW OUT (in)	BEGINNING WATER CONTENT (%)	ENDING WATER CONTENT (%)	PEAK FLOW IN (in/hr)	PEAK FLOW OUT (in/hr)	SWALE DISCHARGE COEFFICIENT C_v (volume)	SWALE PEAK FLOW COEFFICIENT C_p	MASS BALANCE INFILTRATION RATE (in/hr)
1	75	68.45	0.23	9.84	41.28	12.84	0.48	0.01	0.03	12.48
2	75	194.05	5.13	10	29.25	12.84	8.16	0.03	0.16	26.80
3	50	60.96	3.59	14.25	34.85	20.52	2.28	0.06	0.11	16.39
4	50	35.3	4.3	20.25	45.90	15.47	2.63	0.12	0.17	10.9

TABLE 7

FLOW DATA FOR SWALE AT REED ROAD

EXP. NO.	DISTANCE BETWEEN WEIRS (ft)	FLOW IN (in)	FLOW OUT (in)	BEGINNING WATER CONTENT (%)	ENDING WATER CONTENT (%)	PEAK FLOW IN (in/hr)	PEAK FLOW OUT (in/hr)	SWALE DISCHARGE COEFFICIENT C_v (volume)	SWALE PEAK FLOW COEFFICIENT C_p	MASS BALANCE INFILTRATION RATE (in/hr)
1	110	6.14	3.15	17.5	35.0	5.25	2.77	0.51	0.53	0.84
2	100	12.34	1.43	9.5	38.0	12.19	2.57	0.12	0.21	8.84
3	90	15.60	11.58	28.35	41.4	11.41	8.56	0.74	0.75	2.79
4	60	48.13	32.07	26.0	38.5	41.71	29.9	0.67	0.72	11.20
5	60	53.48	25.0	16.8	36.4	41.18	20.32	0.47	0.49	18.90
6	120	26.74	9.91	16.8	36.4	20.59	8.29	0.37	0.41	8.42

TABLE 8

FLOW DATA FOR SWALE AT INTERSTATE 4 - WEST

EXP. NO.	DISTANCE BETWEEN WEIRS (ft)	FLOW IN (in)	FLOW OUT (in)	BEGINNING WATER CONTENT (%)	ENDING WATER CONTENT (%)	PEAK FLOW IN (in/hr)	PEAK FLOW OUT (in/hr)	SWALE DISCHARGE COEFFICIENT C_v (volume)	SWALE PEAK FLOW COEFFICIENT C_p	MASS BALANCE INFILTRATION RATE (in/hr)
1	66	7.29	3.34	24.25	40	0.38	0.19	0.46	0.50	2.15
2	66	36.89	16.25	19.61	23.27	3.16	1.54	0.44	0.49	7.63
3	145	19.92	11.28	29.75	37.75	0.70	0.42	0.57	0.60	2.47

TABLE 9

FLOW DATA FOR SWALE AT INTERSTATE 4 - EAST

EXP. NO.	DISTANCE BETWEEN WEIRS (ft)	FLOW IN (in)	FLOW OUT (in)	BEGINNING WATER CONTENT (%)	ENDING WATER CONTENT (%)	PEAK FLOW IN (in/hr)	PEAK FLOW OUT (in/hr)	SWALE DISCHARGE COEFFICIENT C_v (volume)	SWALE PEAK FLOW COEFFICIENT C_p	MASS BALANCE INFILTRATION RATE (in/hr)
1	72	13.37	0.59	8.3	37.2	3.57	0.67	0.04	0.19	3.19
2	30	21.4	14.26	32.3	46	40.6	32.7	0.67	0.81	9.52
3	60	10.69	5.35	32.3	46	20.3	12.8	0.50	0.63	6.29

details on length of swale in which the mass balance method is performed. It can be noted that different lengths of swales are chosen. Also, in these tables are listed the flow of water input into the swale system in each experiment, as well as the flow of water out of the system. These values are necessary to compute the volumetric discharge coefficient for each experiment. Both the peak flow and the volumetric coefficient of discharge can be defined as the amount of the inflow (in percent) that goes out of the system (runoff). In tables 5 through 9, information for the water content before and after each experiment is provided. The results of the experiments tend to indicate that the higher the value of the beginning water content, the smaller the value of the infiltration rate, and thereby, the higher the value of the volumetric coefficient of discharge, C_V. Finally, in tables 5 through 9 are listed the values of the peak value of the flow into the system, and the peak value of the flow out of the system from which the peak flow coefficient of discharge is computed. These peak flow coefficients of discharge increase with an increase in beginning water content.

These values of flow in and flow out listed in tables 5 through 9 are computed by obtaining the area under the hydrographs (flow of water versus time). These hydrographs were presented by Avellaneda (1985). The peak flow in and the peak flow out are also obtained directly from these graphs. It can be easily seen that the discharge coefficient increases as the moisture content increases, and the coefficient of discharge decreases as the length of swale is increased.

Several sources of error are faced in this experiment. Regardless of measurement errors that are always encountered in field experimentations, it should be pointed out that flow measuring devices such as weirs are a source of errors because they have to be placed perpendicular to the flow line, in a position that produces minimum backwater conditions. Also, each weir has to penetrate the ground approximately the same depth, and this is usually difficult to achieve. The fact that the longitudinal slope of the swale varies along its length is minimized by computing the average flow of water through the swale. Therefore, the discharge coefficient based on volume of flow yields a more representative value than the coefficient of discharge based on peak flow.

CHAPTER V

DESIGN METHODOLOGY

Introduction

In recent years, it has been considered a priority to develop a procedure for proper design of swale systems. It is the intention of this report to present design procedures for swales that considers many factors affecting the swale infiltration volume. This design procedure is developed for the two most common swale shapes, namely trapezoidal and triangular shapes. The procedure has been developed so that a choice can be made for each site-specific variable. This is very important because geometric and soil conditions vary from place to place. If space is limited, or if an uncommon value of the roughness coefficient (n) is found, this design procedure can be followed to estimate the proper geometry of a swale. It should also be noted that this design procedure has been checked two times in the field with margins of error less than 10% in both cases. Finally, these design equations are expressed in both the SI system of units and the US customary system of units. Therefore, the designer should be very careful in choosing the appropriate units for each variable.

Triangular Shape Swale

From the definition of depth of flow for triangular sections:

$$D = \frac{P}{Z\sqrt{1 + Z^2}} \qquad (1)$$

where:

D = depth of flow

P = wetted perimeter

Z = horizontal distance per one-foot change in side slope

and, from Manning's equation for uniform flow:

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \qquad (2)$$

where:

n = Manning's roughness coefficient

A = cross-sectional area of flow

R = hydraulic radius

S = longitudinal slope

The length of swale (L) necessary to percolate the runoff (Q) for the given conditions can be defined as:

$$L = \frac{K\ Q^{5/8}\ S^{3/16}}{n^{3/8}\ i} \qquad (3)$$

where the constant K is a function of the side slope "Z"

$$K = \frac{21{,}032\ Z^{5/8}}{(1 + Z^2)^{5/8}} \qquad (4)$$

if the US customary system of units is used, and:

$$K = \frac{151{,}361\ Z^{3/8}}{(1 + Z^2)^{5/8}} \qquad (5)$$

if the SI system of units is used.

In Table 10, there are listed the values for K in both systems of units for different values of Z.

TABLE 10

CONSTANT (K) FOR DESIGN EQUATION FOR TRIANGULAR SHAPE

Z	K (US UNITS) i(in/hr); Q(CFS)	K (SI UNITS) i(cm/hr); Q(m^3/s)
1	10,516	98,145
2	9,600	71,787
3	8,446	54,192
4	7,514	43,327
5	6,784	36,122
6	6,203	31,024
7	5,730	27,231
8	5,337	24,300
9	5,006	21,965
10	4,722	20,059

The derivation of equation (3) can be found in Appendix A.

A sensitivity analysis for the retardance coefficient (n) shows that the value of n is also very significant. This sensitivity analysis, shown in Table 11, is done for the case study at Interstate 4 - East, near Maitland, Florida. It can be seen that by doubling the value of n from 0.02 to 0.04, the necessary length of swale changes (decreases) by approximately 25%.

TABLE 11

SENSITIVITY ANALYSIS FOR ROUGHNESS COEFFICIENT AT INTERSTATE 4 - EAST

n	L (ft)
.02	319
.03	273
.04	245
.05	226
.06	211
.07	199
.08	189

Trapezoidal Shape Swale

Applying the same principles used for the triangular section, and the fact that the most efficient trapezoidal section can be defined by:

$$\frac{B}{D} = 2\ [(1 + Z^2)^{1/2} - Z] \qquad (6)$$

the following equations are derived:

$$L = \frac{43,200\ Q}{\{B + 2\ [\frac{1.068\ n\ Q\ (1 + Z^2)^{1/3}}{S^{1/2}\ Z^{2/3}\ \ 2[(1 + Z^2)^{1/2} - Z]}]^{3/8}\ (1 + Z^2)^{1/2}\}\ i} \qquad (7)$$

if the US customary system of units is used, and for the SI system

$$L = \frac{360{,}000Q}{\dfrac{B\sqrt{1 + Z^2}}{\left[B + \dfrac{2Q^{3/5}\,(B + \sqrt{1 + Z^2} - Z)^{2/5}\, n^{3/5}\sqrt{1 + Z^2}}{B + \dfrac{BZ}{2(1 + Z^2 - Z)}}\right] i}} \qquad (8)$$

where:

Q = average flow rate (m^3/s), (CFS)

L = length of swale (m), (ft)

i = actual infiltration rate (cm/hr), (in/hr)

n = Manning's roughness coefficient

Z = horizontal distance per one-foot change in side slope

S = longitudinal slope

B = bottom width of swale (m), (ft)

Derivations of equations (7) and (8) can be found in Appendix A.

CHAPTER VI

CASE STUDIES

Two sites were chosen to calibrate and verify the equations for calculating the necessary length of swale to infiltrate an average runoff (Q), for the given soil geometric conditions. It must be noted that the values of the infiltration rate were obtained by using field hydrographs and the mass balance method. Therefore, its value should be considered very accurate. Also, the flow rate was an average from the input hydrograph developed in the field. The average was used because the equations developed to calculate the length of swale were based on a mass (volume) using average flow rate per time period. The two sites (a swale at Interstate 4 - East, near Maitland, Florida, and a swale at Reed Road in the Chuluota area of Florida) were chosen based on the long length of the swale at each site.

Experiment at Interstate 4 - East

From Table 3, it can be seen that the longitudinal slope (S) is 0.0279, and from Table 2, a value for the side slope (Z) of 7 is obtained. A triangular shape can best represent the geometry of the swale. For this location, a value of 0.05 for the roughness coefficient (n) is recommended (Wanielista et al. 1985). From a field experiment (Avellaneda, 1985) a runoff volume (160.4 ft^3 in 51 minutes) is obtained, with a value for infiltration rate (i) of 6.29 in/hr (Table 9). For as actual swale, double-ring infiltrometer data with actual operating head can be obtained and then divided by two. For a design situation, actual infiltration rates from this report for associated sub-base materials can be used.

Summarizing triangular shape:

$n = 0.05$

$S = 0.0279$

$$Q = \frac{160.4 \text{ ft}^3}{(51 \text{ min})(60 \text{ sec/min})} = 0.052 \text{ cfs}$$

$i = 6.29$ in/hr

$Z = 7$

Equations (3) and (4) may be utilized. Also, Table 10 can be used to obtain a value of K for Z = 7, which for US customary units is:

$$K = 5{,}730$$

and

$$L = \frac{K\, Q^{5/8}\, S^{3/16}}{n^{3/8}\, i}$$

Plugging in the values:

$$L = \frac{5{,}730\ (.052 \text{ cfs})^{5/8}\ (.0279)^{3/16}}{(.05)^{3/8}(6.29 \text{ in/hr})}$$

Solving:

$$L = 226 \text{ ft}$$

The length of swale necessary to infiltrate this runoff (Q) is measured in the field as L = 225 ft. This represents an error of approximately 0.29%.

Experiment at Reed Road

What follows is a summary of the different parameters for this experiment at Reed Road.

Triangular shape:

$n = 0.05$

$$Q = \frac{802.1 \text{ ft}^3}{(120 \text{ min})(60 \text{ sec/min})} = 0.111 \text{ cfs}$$

$i = 8.43$ in/hr

$Z = 6$

$S = 0.0133$

From Table 10, it is found that for Z = 6, K = 6,203. Using equation (3):

$$L = \frac{K\, Q^{5/8}\, S^{3/16}}{n^{3/8}\, i}$$

For this case:

$$L = \frac{6{,}203\ (0.11 \text{ cfs})^{5/8}\ (.0133)^{3/16}}{(.05)^{3/8}\ (8.42 \text{ in/hr})}$$

Solving:

$$L = 253.67$$

The length of swale necessary to infiltrate an average runoff of 0.111 cfs, for the site at Reed Road, was measured in the field to be L = 278 ft. This represents an error of approximately 8.75% with the value obtained with the design equation.

The two cases studied appear to verify the validity of the design equation to predict the length of swale required to percolate a certain runoff volume $Q(\Delta t)$.

If published permeability values must be used, then the most conservative value of published permeability is recommended. If this approach is used for the experiment at Interstate 4 - East, using the design equation, the length of swale is found to be L = 305.7 ft. This length of swale is based on using the lower limit of the range of values presented by the SCS report listed in Table 3.

Using the design equation for the case study at Reed Road, with the lower limit of the SCS published permeability as the actual value, a length of swale of 234 ft is found. This approach for design shows that the values for permeability listed by the SCS report could lead the designer to under-design the swale. Double-ring infiltrometer determinations with site related density would produce more accurate results.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Introduction

This study provides an analysis of several different parameters affecting swale design and operation, based on existing literature and experience, and on the field observation of such parameters to culminate with a design procedure for swales. Five different swales, representing different soil conditions and topography, were chosen to obtain specific information which could be used to specify a range of design conditions. Twenty-two experiments were performed in these locations and the hydrograph and soil information of each were documented (Avellaneda, 1985). Field studies to determine infiltration rates from a mass balance of flow volumes as well as laboratory studies to determine soil physical parameters, are used to investigate their effect on the infiltration efficiency of the swales.

Infiltration

Twenty-two experiments were performed in five different sites to determine a field rate of infiltration. It was concluded that the field rate of infiltration can be estimated from a mass balance of flows. It varied among sites and experiments at a site. It was also concluded that the infiltration rate varies inversely with the moisture content of the soil. For design situations, or when field infiltration data are not available, infiltration can be estimated from published data similar to that of this report. As an example, actual infiltration rates are related to subgrade classification (AASHTO) schemes as noted in Table 3. Another example of published data would be the soil surveys for a region performed by the Soil Conservation Service (SCS). However, it is important to note that the SCS publications present an extensive range of permeability values for each location. Therefore, the designer should use the most conservative value for the infiltration rate. When few estimates are available, the lower of each estimate would be used to determine how each value would affect the geometry of the swale.

For an existing swale system, the rate of infiltration was predicted more accurately from the double-ring infiltrometer than from lab permeability. If the double-ring infiltrometer can be used, a value for infiltration rate equal to one-half of the value obtained with the double-ring method is recommended. From Table 4, the laboratory rates vary from 56% to 94% of the lab permeability. The laboratory density approximated the field conditions. This range of values seems to indicate that actual infiltration rates cannot be predicted accurately from lab permeability.

Design Equation

In appendix A is the derivations for the design equations for triangular and trapezoidal shapes. For both shapes, the SI system and the US customary system of units were used for the design equations. It is obvious from those equations that one of the most important paramters for the design of swales is the infiltration rate (i). The length of swale is inversely related to the infiltration rate. If the estimate of the infiltration rate is twice the actual infiltration rate, then the design equation would yield a value of the length of swale of one-half of the needed length. From Table 10, it can also be seen that the value of the side slope (Z) has a great effect on the length of the swale. An increment of ten times in the side slope (Z) results in a decrease of three times in the length of the swale. Therefore, if possible, the designer should use several geometries and decide on the best from a practical and economical standpoint. Parameters such as slope (S) and rainfall excess (Q) would depend more on the topography and watershed area of the sites, and on the storm events used for design.

It can be concluded that all the parameters are very significant to compute the length of swale needed. However, emphasis must be made on predicting a reasonable rate of infiltration because it has the greatest effect on the accuracy of the design. It is recommended that the length of swale equations be used for hydrologic design of swales.

Future Recommendations

1. Develop a computerized model to do all of the mathematics and specify the optimum design (length and side slopes).

2. Determine the relative cost-effectiveness of a swale versus other management practices.

3. For the implied State of Florida Effectiveness Criteria of 80 percent mass removal, develop using State-wide rainfall data, the runoff rates and volumes needed to meet the 80 percent criteria.

APPENDIX

APPENDIX A

DERIVATION OF EQUATIONS FOR SWALE DESIGN

Triangular Shaped Swale

Recall that the total infiltration volume (I) per unit time equals the rate of infiltration (i) times the contact area (A) between the water and the swale bottom. This infiltration volume is expressed as:

$$I = i\,A(\Delta t) \qquad \text{(A-1)}$$

Whereas the contact area can be defined as the product of the wetted perimeter (P) and the length of swale (L).

$$A = L \times P \qquad \text{(A-2)}$$

For design, the volume of infiltration equals the volume of runoff water that enters the system, or:

$$I = Q\,(\Delta t) \qquad \text{(A-3)}$$

where:

Q = flow rate of water in ft^3/sec

Substituting equations (A-2) and (A-3) into equation (A-1), we get:

$$Q = L \times P \times i$$

and solving for L, we obtain:

$$L = \frac{3600\ Q}{P \times i} \qquad \text{(A-4)}$$

where 3600 is a conversion factor from cfs to ft^3/hr.

The perimeter of the swale depends on the rate of discharge through the swale. Thus, a relationship between the two is necessary to be able to use equation A-4.

For a triangular section, the depth of flow, D, can be defined as:

$$D = \frac{P}{2\sqrt{1 + Z^2}} \tag{A-5}$$

where:

P = wetted perimeter of the swale

Z = horizontal component of side slope, Z = 1 which means Z horizontal per one vertical

So that:

$$P = 2\sqrt{1 + Z^2}\ D \tag{A-6}$$

and the cross-sectional area of the flow can be expressed as:

$$A = Z\ D^2 \tag{A-7}$$

One of the most common equations to compute flow of water in open channels is the Manning's equation. When the US customary system of units is used, Manning's equation can be expressed as:

$$Q = \frac{1.486}{n}\ A\ R^{2/3}\ S^{1/2} \tag{A-8}$$

where:

Q = average flow rate (cfs)

n = coefficient of roughness

R = hydraulic radius (ft)

S = longitudinal slope of swale

A = cross-sectional area of flow (ft^2)

For trapezoidal secions, the hydraulic radius can be defined as:

$$R = \frac{D}{2} \frac{Z}{\sqrt{1 + Z^2}} \tag{A-9}$$

Therefore, substituting equations (A-9) and (A-7) into equation (A-8), we obtain:

$$Q = \frac{1.486}{n} Z D^2 \left(\frac{DZ}{2\sqrt{1 + Z^2}}\right)^{2/3} (S)^{1/2} \tag{A-10}$$

or:

$$Q = \frac{1.486}{1.59n} \frac{Z^{5/3} S^{1/2} D^{8/3}}{Z^{2/3} (1 + Z^2)^{1/3}} \tag{A-11}$$

and:

$$Q = \frac{Z^{5/3} S^{1/2} D^{8/3}}{1.073 n (1 + Z^2)^{1/3}} \tag{A-12}$$

Solving for D, it is found:

$$D = \left[\frac{1.073 Q n (1 + Z^2)^{1/3}}{Z^{5/3} S^{1/2}}\right]^{3/8} \tag{A-13}$$

Substituting the value of D expressed in equation (A-13) into equation (A-7), the wetted perimeter can be expressed as:

$$P = 2\,(1 + Z^2)^{1/2} \left[\frac{1.073\ Q\ n\ (1 + Z^2)^{1/3}}{Z^{5/3}\ S^{1/2}}\right]^{3/8} \qquad \text{(A-14)}$$

Finally, if one substitutes the value of P into equation (A-5):

$$L = \frac{3600\ Q}{2\left[\frac{1.073\ Q\ n\ (1 + Z^2)^{1/3}}{Z^{5/3}\ S^{1/2}}\right]^{3/8} (1 + Z^2)^{1/2} \times \frac{i}{12}} \qquad \text{(A-15)}$$

Note that i is divided by 12 in order to be able to use the common units of in/hr. Then:

$$L = \frac{3600\ Q}{\frac{2.054\ Q^{3/8}\ n^{3/8}\ (1 + Z^2)^{1/8}\ (1 + Z^2)^{1/2}}{12\ \ Z^{5/8}\ S^{3/16}}} \qquad \text{(A-16)}$$

and:

$$L = \frac{21{,}032\ Q\ Z^{5/8}\ S^{3/16}}{Q^{3/8}\ (1 + Z^2)^{5/8}\ i\ n^{3/8}} \qquad \text{(A-17)}$$

Finally, we obtain:

$$L = \frac{21{,}032\ Q^{5/8}\ Z^{5/8}\ S^{3/16}}{n^{3/8}\ (1 + Z^2)^{5/8}\ i} \qquad \text{(A-18)}$$

where:

L = length of swale (ft)

Q = average flow rate to be percolated (cfs)

Z = horizontal distance per one foot of elevation change inside slope

S = longitudinal or flow slope

n = Manning's Roughness Coefficient

i = infiltration rate (in/hr)

Equation (A-18) can be used to predict the necessary length of swale to percolate certain amounts of runoff waters when U.S. Customary system of units is used.

When the International System of units is used, Manning's equation is written in the form:

$$Q = \frac{1}{n}\ A\ R^{2/3}\ S^{1/2} \qquad \text{(A-19)}$$

and equation (A-14) would be expressed as:

$$P = 2\left[\frac{1.5874\ Q\ n\ (1 + Z^2)^{1/3}}{Z^{5/3}\ S^{1/2}}\right]^{3/8} (1 + Z^2)^{1/2} \qquad \text{(A-20)}$$

Substituting this value into equation (A-5):

$$L = \frac{3600\ Q}{2\left[\frac{1.5874\ Q\ n\ (1 + Z^2)^{1/3}}{Z^{5/3}\ S^{1/2}}\right]^{3/8} (1 + Z^2)^{1/2}\ \frac{i}{100}} \qquad \text{(A-21)}$$

Note that i is divided by 100 in order to use the infiltration rate in units of cm/hr. Then:

$$L = \frac{3600\ Q}{\frac{2.378\ Q^{3/8}\ (1 + Z^2)^{5/8}\ i\ n^{3/8}}{100\ Z^{3/8}\ S^{3/16}}} \qquad \text{(A-22)}$$

which can be expressed as:

$$L = \frac{151{,}361\quad Q^{5/8}\ Z^{3/8}\ S^{3/16}}{n^{3/8}\ (1 + Z^2)^{5/8}\ i} \qquad \text{(A-23)}$$

where:

L = length of swale (m)

Q = average flow rate (m^3/s)

S = longitudinal slope

n = Manning's roughness coefficient

Z = side slope

i = infiltration rate (cm/hr)

Equation (A-23) can be used to calculate the necessary length of swale (L) to percolate the runoff (Q) when the International System of units is used.

Trapezoidal Shaped Swale

Applying Manning's equation for US Customary System of units, it is obtained:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad \text{(A-24)}$$

For a trapezoidal section, the cross-sectional area of flow is defined as:

$$A = BD + ZD^2 \qquad \text{(A-25)}$$

where:

B = bottom width of the swale (ft)

D = depth of flow (ft)

Z = side slope

and, the wetted perimeter (P) can be defined as:

$$P = B + 2D \sqrt{1 + Z^2} \qquad \text{(A-26)}$$

Substituting R = D/2 and equation (A-25) into equation (A-24), it is found:

$$Q = \frac{1.486}{n} (BD + ZD^2) \left(\frac{(B++ ZD)\ D}{B + 2D\ (1 + Z^2)^{1/2}}\right)^{2/3} \qquad \text{(A-27)}$$

which equals:

$$Q = \frac{1.486}{n} \frac{D^{5/3}\ (B + ZD)^{5/3}}{(B + 2D \sqrt{1 + Z^2})^{2/3}} S^{1/2} \qquad \text{(A-28)}$$

For design, the most efficient section should be used. For a trapezoidal section, the most efficient section is defined as:

$$\frac{B}{D} = 2\left(\sqrt{1 + Z^2} - Z\right) \tag{A-29}$$

Solving for D:

$$D = \frac{B}{2\left(\sqrt{1 + Z^2} - Z\right)} \tag{A-30}$$

Substituting

$$Q = \frac{1.486}{n} \; \frac{D^{5/3}\left[B + \dfrac{BZ}{2\left(\sqrt{1 + Z^2} - Z\right)}\right]^{5/3}}{\left[B + \dfrac{\cancel{2}B\sqrt{1 + Z^2}}{\cancel{2}\left(\sqrt{1 + Z^2} - Z\right)}\right]^{2/3}} \; S^{1/2} \tag{A-31}$$

Solving for D:

$$D = \left[\frac{Q\left[B + \dfrac{B\sqrt{1 + Z^2}}{\left(\sqrt{1 + Z^2} - Z\right)}\right]^{2/3} n}{1.486\left[B + \dfrac{BZ}{2\left(\sqrt{1 + Z^2} - Z\right)}\right]^{5/3}}\right]^{3/5} \tag{A-32}$$

$$D = \frac{Q^{3/5}\left(B + \dfrac{B\sqrt{1 + Z^2}}{\sqrt{1 + Z^2} - Z}\right)^{2/5} n^{3/5}}{1.268\left(B + \dfrac{BZ}{2\left(\sqrt{1 + Z^2} - Z\right)}\right)} \tag{A-33}$$

Substituting in Equation (A-26):

$$P = B + \dfrac{\dfrac{B\sqrt{1 + Z^2}}{2\, Q^{3/5}\, (B + \sqrt{1 + Z^2} - Z)^{2/5}\, n^{3/5}}}{\dfrac{BZ}{1.268\, (B + 2(\sqrt{1 + Z^2} - Z))}}\, (1 + Z^2)^{1/2} \tag{A-34}$$

$$D = \left[\frac{1.068\, n\, Q\, (1 + Z^2)^{1/3}}{S^{1/2}\, Z^{2/3}\, [2\, (1 + Z^2)^{1/2} - Z]}\right]^{3/8} \tag{A-35}$$

Substituting the value of D in equation (A-35) into equation (A-26):

$$P = B + 2\left[\frac{1.068\, n\, Q\, (1 + Z^2)^{1/3}}{S^{1/2}\, Z^{2/3}\, [2\, (1 + Z^2)^{1/2} - Z]}\right]^{3/8} (1 + Z^2)^{1/2} \tag{A-36}$$

Recall from the triangular section derivation that:

$$L = \frac{Q\, (3600)}{P \times i} \tag{A-37}$$

which, for units of in/hr for the infiltration rate, can be expressed as:

$$L = \frac{3600\, Q}{P \times \frac{i}{12}} \tag{A-38}$$

or:

$$L = \frac{43{,}200\ Q}{P \times i} \qquad \text{(A-39)}$$

Substituting the value of wetted perimeter defined in equation (A-36) into equation (A-39), it is obtained:

$$L = \frac{43{,}200\ Q}{\left\{B + 2\left[\dfrac{1.068\ n\ Q\ (1 + Z^2)^{1/3}}{S^{1/2}\ Z^{2/3}\ \ 2[(1 + Z^2)^{1/2} - Z]}\right]^{3/8} (1 + Z^2)^{1/2}\right\} i} \qquad \text{(A-40)}$$

where:

L = length of swale (ft)

B = bottom width of swale (ft)

Q = average flow rate (cfs)

n = Manning's roughness coefficient

Z = side slope

S = longitudinal slope

i = infiltration rate of swale (in/hr)

If the International System of units is used, the length of swale necessary to percolate the runoff (Q) can be expressed as:

$$L = \frac{360{,}000\ Q}{\{B + 2\left[\frac{n\ Q\ (1 + Z^2)^{1/3}}{S^{1/2}\ Z^{2/3}\ 2[(1 + Z^2)^{1/2} - Z]}\right]^{3/8} (1 + Z^2)^{1/2}\}\ i} \qquad \text{(A-41)}$$

where:

L = length of swale (m)

Q = average flow rate (cfs)

i = infiltration rate (cm/hr)

n = Manning's roughness coefficient

Z = side slope

S = longitudinal slope

B = bottom width of swale (m)

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